REPORT

Foundation Investigation and Design Report

Bass Pro Mills Drive and Rutherford Road S-E/W Ramp Retaining Walls
Highway 400 Widening from Langstaff Road to Major Mackenzie Drive
Vaughan, Ontario
GWP 2836-02-00

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PART A

FOUNDATION INVESTIGATION REPORT

Bass Pro Mills Drive and Rutherford Road S-E/W Ramp Retaining Walls Highway 400 Widening Langstaff Road to Major Mackenzie Drive Vaughan, Ontario MTO GWP 2836-02-00

1.0 INTRODUCTION

WSP Canada Inc. (WSP, formerly Golder Associates Ltd., amalgamated with WSP in 2023) has been retained by Parsons Inc. (Parsons) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Highway 400 widening and rehabilitation, extending from 1.3 km south of the Langstaff Road interchange to 1.5 km north of Major Mackenzie Drive (a length of approximately 7.3 km) in the City of Vaughan, Ontario. As part of the Highway widening and rehabilitation program, two retaining walls will be constructed on the S-E/W ramps of the Bass Pro Mills Drive and Rutherford Road interchanges (referred to hereinafter as “the Bass Pro Mills Drive retaining wall” and “the Rutherford Road retaining wall”, respectively).

The purpose of these investigations is to assess the subsurface soil and groundwater conditions near the retaining wall locations through borehole drilling, in situ testing, and laboratory testing of selected soil samples.

This report summarizes the factual results of field and laboratory work (including field investigation procedures, borehole stratigraphy, and geotechnical and analytical laboratory test results) and provides a description of interpreted soil and groundwater conditions for the Bass Pro Mills Drive and Rutherford Road retaining walls.

2.0 SITE DESCRIPTION

2.1 General

The orientation (i.e., north, south, east, and west) stated in the text of this report is referenced to project north and therefore may differ from magnetic north shown on Drawings 1 and 2. For the purpose of this report, Highway 400 is considered to be oriented in a north-south direction, with the Bass Pro Mills Drive S-E/W ramp retaining wall parallel to the highway in a north-south direction, and the Rutherford Road S-E/W ramp retaining wall on a skew to Highway 400 in a west-east direction.

At the Bass Pro Mills Drive and Rutherford Road S-E/W ramps, Highway 400 has generally been constructed near the existing ground surface, with no significant sections of cut or fill; the Bass Pro Mills Drive and Rutherford Road crossing roads and associated interchange ramps have been constructed on embankment fill adjacent to the Highway 400 corridor. This section of Highway 400 is currently an eight-lane urban freeway with paved shoulders divided by a concrete median barrier. Land use surrounding the retaining wall sites is primarily commercial.

2.2 Bass Pro Mills Drive S-E/W Ramp

The existing road surface elevation of the Bass Pro Mills Drive S-E/W interchange ramp adjacent to the proposed retaining wall is about Elevation 210 m to 211 m. The ramp will be widened to the east, towards the existing roadside ditch, which is currently heavily vegetated with bulrushes and has a ditch bottom elevation ranging from about Elevation 208 m to 209 m (i.e., about 2 m below the road surface of the ramp). The roadside ditch has side slope inclinations shallower than 3H:1V. There is a guardrail along the east edge of Highway 400 that extends from the south end of the proposed wall to an overhead sign about 70 m north.

2.3 Rutherford Road S-E/W Ramp

The existing road surface elevation of the Rutherford Road S-E/W interchange ramp adjacent to the proposed retaining wall is about Elevation 223 m. The ramp will be widened to the south, beyond the limits of the existing roadside ditch, which is currently heavily vegetated with bulrushes and a few limit zones of tree cover along the fence line (which runs parallel to the interchange ramp and separates the highway right-of-way with commercial properties to the south). The existing ditch has a bottom elevation of about 221.5 m (i.e., about 1.5 m below the road surface of the ramp).

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface exploration program consisted of 7 boreholes (designated as RW-1 to RW-7) and one monitoring well (designated RW-MW); five boreholes (RW-1 to RW-5) along the Bass Pro Mills Drive S-E/W interchange ramp and two boreholes (RW-6 and RW-7) along the Rutherford Road S-E/W interchange ramp. These boreholes and the monitoring well were advanced between July 11 and July 27, 2023, at the approximate locations shown on Drawings 1 and 2.

The boreholes were advanced through the existing roadway shoulder of the interchange ramps using a truck-mounted CME 75 drill rig supplied and operated by 3D Drilling of Whitchurch-Stouffville, Ontario. The boreholes were advanced through the overburden using 159 mm outside diameter hollow stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outside diameter split spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)1. The split-spoon samplers used in the investigation limits the maximum particle size that can be sampled and tested to about 35 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions.

The groundwater conditions were noted in the boreholes during and upon completion of drilling and were backfilled in accordance with Ontario Regulation 903 (Wells, as amended), and the asphalt surface was capped with tamped cold patch asphalt. A standpipe piezometer was installed within an augered hole on the road shoulder of the Rutherford Road S-E/W ramp between Boreholes RW-6 and RW-7 to allow monitoring of the groundwater level. The installed piezometer consists of a 50 mm diameter PVC pipe, with a 3.0 m long slotted screen within a filtered sand pack. The borehole and annulus surrounding the piezometer pipe above the filter sand pack were backfilled to near ground surface with bentonite pellets. The standpipe piezometer was installed in a metal protective casing flush with the pavement surface.

The field work was observed by members of WSP’s engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled, and transported to WSP’s Mississauga laboratory where the samples underwent further visual examination. Geotechnical laboratory testing (water content, grain size distribution, and Atterberg limits) was carried out on select soil samples, in accordance with MTO and/or ASTM Standards, as appropriate. In addition, select soil samples were submitted to Bureau Veritas Laboratories of Mississauga, Ontario for analysis of select parameters to assess for the potential corrosion of buried steel and deterioration of concrete.

The as-drilled borehole and monitoring well locations and elevations were surveyed by WSP using a Trimble Geo 7x GPS unit. The locations are referenced to NAD 83(CSRS)v6 MTM Zone 10 coordinates and the ground surface elevations are referenced to CGVD28 Geodetic datum benchmark. The borehole locations, including geographic coordinates, ground surface elevations, and borehole/monitoring well depths are summarized below. All boreholes were advanced to a depth of 6.7 m below existing ground surface.

[1] ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Location	Borehole No.	MTM NAD83 Northing (Latitude, °)	MTM NAD83 Easting (Longitude, °)	Ground Surface Elevation (m)
Bass Pro Mills Drive S-E/W Ramp	RW-1	4,852,935.5 (43.816376)	301,196.7 (-79.544792)	209.3
	RW-2	4,852,987.1 (43.816841)	301,193.1 (-79.544837)	209.6
	RW-3	4,853,039.0 (43.817308)	301,193.6 (-79.544832)	209.6
	RW-4	4,853,093.1 (43.817795)	301,192.4 (-79.544847)	209.8
	RW-5	4,853,141.4 (43.818230)	301,188.5 (-79.544895)	210.1
Rutherford Road S-E/W Ramp	RW-6	4,854,121.2 (43.827049)	301,292.6 (-79.543608)	222.8
	RW-MW	4,854,142.7 (43.827252)	301,320.2 (-79.543268)	222.8
	RW-7	4,854,164.6 (43.827440)	301,345.4 (-79.542952)	222.9

4.0 SITE GEOLOGY

4.1 Regional Geology

As delineated in The Physiography of Southern Ontario (Chapman and Putnam, 1984)[2], this section of Highway 400 lies within the region known as the Peel Plain and consists of level to undulating tracts of clayey glacial till soils, which are presumed to have been derived from moraines, interspersed with non-cohesive silts and sands from interstadial stages of Wisconsinan glaciation.

Based on geological mapping by the Ministry of Northern Development and Mines (MNDM)[3], the site is underlain by bedrock from the Upper Ordovician era consisting of shale, limestone, dolostone, and siltstone.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing from the investigation are shown on the borehole records presented in Appendix A. The detailed results of the geotechnical laboratory testing are presented in Appendix B. The results of the in situ field tests (i.e., SPT 'N'-values) as presented on the borehole records and in Section 4.2 are uncorrected. The results of the analytical testing completed on select soil samples are provided in Appendix C.

The stratigraphic boundaries shown in the borehole records are inferred from non-continuous sampling and, therefore, these boundaries represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions near the proposed Bass Pro Mills Drive S-EW ramp retaining wall consist of the existing pavement structure underlain by a layer of cohesive fill consisting of clayey silt-silt to silty clay, which

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

[3] Ministry of Northern Development of Mines. Bedrock Geology of Ontario – Southern Sheet, Ontario Geological Survey - Map 2544.

extends to elevations ranging from about Elevation 207 m to 208 m. The cohesive fill is underlain by cohesive deposits of clayey silt-silt to silty clay; this deposit is generally stiff to very stiff, but the upper portion of the deposit is firm to stiff in some of the boreholes (i.e., SPT-“N” values on the order of 4 to 8 blows per 0.3 m of penetration). In Borehole RW-3, this deposit was further underlain by a non-cohesive deposit of compact sandy silt.

In general, the subsurface conditions near the proposed Rutherford Road S-EW Ramp retaining wall consist of the existing pavement structure underlain by a layer of cohesive fill consisting of clayey silt-silt to clayey silt, which extends to about Elevation 220 m to 221 m. The cohesive fill is underlain by a non-cohesive deposit of compact to dense silt, which is further underlain by a cohesive deposit of clayey silt having a hard consistency.

A more detailed description of the major stratigraphic units encountered in the boreholes is described in the sections below.

4.2.1 Bass Pro Mills Drive S-E/W Ramp

Five boreholes (Boreholes RW-1 to RW-5) were advanced through the existing road shoulder of the Bass Pro Mills Drive S-E/W ramp adjacent to the proposed retaining wall. The following subsurface conditions were encountered in Boreholes RW-1 to RW-5.

- **Asphalt:** A layer of asphalt between 200 mm and 220 mm thick was encountered at the ground surface in all boreholes.

- **Granular Fill (Pavement Structure):** A layer of granular fill between 0.6 m and 1.3 m thick was encountered underlying the asphalt in all boreholes, extending between Elevations 209.3 m to 208.1 m. The SPT “N”-values measured within the granular fill range from 15 to 37 blows per 0.3 m of penetration, indicating a compact to dense state of compactness. The results of grain size distribution testing completed on two samples of the granular fill from Boreholes RW-2 and RW-4 are presented in Figure B1. The water content measured on samples of the granular fill ranges from about 4% to 9%.

- **Clayey Silt-Silt (CL-ML) to Silty Clay (CI) Fill:** A layer of cohesive fill consisting of clayey silt-silt to silty clay was encountered underlying the granular fill in Boreholes RW-1 to RW-5. The cohesive fill was encountered at depths ranging from approximately 0.8 m to 1.4 m below ground surface (approximately Elevations 209.3 m to 208.1 m) and was about 0.8 m to 1.4 m thick, extending down to a depth of 2.2 m (approximately Elevations 207.9 m to 207.1 m). The SPT “N”-values measured within the cohesive fill range from 4 to 22 blows per 0.3 m of penetration, indicating a soft to very stiff consistency. The results of grain size distribution testing completed on two samples of the cohesive fill from Boreholes RW-3 and RW-5 are presented in Figure B2 in Appendix B. Atterberg limit testing was carried out on two samples of the cohesive fill and the results are presented on a plasticity chart in Figure B4 in Appendix B. The Atterberg limits test measured liquid limits of about 22% and 42%, plastic limits of about 13% and 19%, and a corresponding plasticity index of about 9% and 23%. Based on the grain size distribution tests, together with the results of the Atterberg limits tests, one sample of the cohesive fill is classified as clayey sand fill of low plasticity, and one sample of the cohesive fill is classified as silty clay of intermediate plasticity. The water content measured on samples of the cohesive fill ranges from about 8% to 15%.

- **Clayey Silt-Silt (CL-ML) to Silty Clay (CI) Till:** A cohesive deposit of glacial till varying in composition from clayey silt-silt to silty clay was encountered underlying the cohesive fill in all boreholes. The cohesive till deposit was encountered at a depth of 2.2 m below ground surface (approximately Elevations 207.9 m to 207.1 m) and extended to the termination depth of 6.7 m (Elevations 203.4 m to 202.6 m) in Boreholes RW-1, RW-2, RW-4, and RW-5. In Borehole RW-3, the cohesive till deposit was approximately 3.2 m thick, extending down to a depth of 5.4 m (Elevation 204.2 m). The SPT “N”-values measured within the cohesive till deposit range from 4 to 59 blows per 0.3 m of penetration; softer zones with SPT “N”-values of 4 to 8 blows per 0.3 m of penetration were generally limited to the upper 0.5 to 2 m of the deposit. The results of grain size distribution testing completed on seven samples of the cohesive till deposit are presented in Figure B4 in Appendix B. Atterberg limit testing was carried out on seven samples of the cohesive till deposit and the results are presented on a plasticity chart on Figure B5 in Appendix B. The Atterberg limits tests measured liquid limits ranging from about 17% to 47%, plastic limits ranging from about 11% to 20%, and corresponding plasticity indices ranging from 6% to 27%. The Atterberg limits tests generally indicate a clayey silt-silt to silty clay of low to intermediate plasticity, with one sample (Borehole RW-1 Sample 3) indicating a silty clay of intermediate plasticity. The water content measured on samples of the cohesive till deposit ranges from about 8% to 28%. Although not specifically encountered in the boreholes, the presence of cobbles and boulders should be expected in the cohesive till deposit.

- **Sandy Silt (ML):** A non-cohesive deposit consisting of sandy silt was encountered underlying the cohesive till deposit in Borehole RW-3. The non-cohesive deposit was encountered at a depth of 5.4 m below ground surface (Elevation 204.2 m) and extended to the termination depth of 6.7 m (Elevation 202.9 m). The SPT “N”-value measured within the non-cohesive deposit was 20 blows per 0.3 m of penetration, indicating a compact state of compactness. The results of grain size distribution testing completed on a sample of the non-cohesive deposit is presented in Figure B6 (Borehole RW-3 Sample 8) in Appendix B. Atterberg limit testing was carried out on a sample of the non-cohesive deposit and the results are presented on a plasticity chart in Figure B7 (Borehole RW-3 Sample 8) in Appendix B. The Atterberg limits test measured a liquid limit of about 15%, a plastic limit of about 12% and a corresponding plasticity index of about 3%. These results indicate that the fines portion of the sandy silt deposit has slight plasticity. The water content measured on a sample of the non-cohesive deposit was about 11%.

4.2.2 Rutherford Road S-E/W Ramp

Two boreholes (Boreholes RW-6 and RW-7) were advanced through the existing road shoulder of the Rutherford Road S-E/W ramp adjacent to the proposed retaining wall. The following subsurface conditions were encountered in Boreholes RW-6 and RW-7.

- **Asphalt:** A layer of asphalt between 100 mm and 180 mm thick was encountered at ground surface in both boreholes.
- **Granular Fill (Pavement Structure):** A layer of granular fill between 0.6 m and 0.7 m thick was encountered underlying the asphalt in both boreholes, extending to an Elevation of 222.0 m. The SPT “N”-values measured within the granular fill range from 10 to 18 blows per 0.3 m of penetration, indicating a compact state of compactness. The results of grain size distribution testing completed on a sample of the granular fill (Borehole RW-6 Sample 1) is presented in Figure B1. The water content measured on a sample of the granular fill was about 4%.
- **Clayey Silt-Silt (CL-ML) to Clayey Silt (CL) Fill:** A layer of cohesive fill consisting of clayey silt-silt to clayey silt was encountered underlying the granular fill in both boreholes. The cohesive fill was encountered at depths ranging from 0.7 m to 0.9 m below ground surface (approximately Elevation 222.0 m) and was about 0.7 m to 1.3 m thick, extending down to depths ranging from 1.5 m to 2.2 m (approximately Elevations 221.3 m to 220.7 m). The SPT-“N” values measured within the cohesive fill range from 9 to 13 blows per 0.3 m of penetration, indicating a stiff consistency. The water content measured on samples of the cohesive fill ranges from about 13% to 15%.

- **Clayey Silt-Silt (CL-ML) to Clayey Silt (CL) Till – Upper Deposit:** An upper deposit of glacial till varying in composition from clayey silt-silt to clayey silt was encountered underlying the cohesive fill in Borehole RW-7. The upper cohesive till deposit was encountered at a depth of 2.2 m below ground surface (Elevation 220.7 m) and extended to a depth of 3.0 m (Elevation 220.0 m). The SPT "N"-value measured within the upper cohesive till deposit was 25 blows per 0.3 m of penetration, indicating a very stiff consistency. The results of grain size distribution testing completed on a sample of the upper cohesive till deposit is presented in Figure B4 (Borehole RW-7 Sample 3) in Appendix B. Atterberg limit testing was carried out on a sample of the upper cohesive deposit and the results are presented on a plasticity chart in Figure B5 in Appendix B (Borehole RW-7 Sample 3). The Atterberg limits test measured a liquid limit of about 23%, a plastic limit of about 17% and a corresponding plasticity index of about 6%. The Atterberg limits test indicates a clayey silt-silt of low plasticity. Although not specifically encountered in the boreholes, the presence of cobbles and boulders should be expected in the upper cohesive till deposit.

- **Silt (ML):** A non-cohesive deposit of silt was encountered underlying the cohesive fill in Borehole RW-6 and underlying the upper cohesive deposit in Borehole RW-7. The non-cohesive deposit was encountered at depths ranging from 1.4 m to 3.0 m below ground surface (approximately Elevations 221.3 m to 220.0 m) and was about 3.6 m to 4.4 m thick, extending down to depths of 5.8 m to 6.6 m (Elevations 217.0 m to 216.4 m). The SPT "N"-values measured in the non-cohesive deposit ranges from 14 to 36 blows per 0.3 m of penetration, indicating a compact to dense state of compactness. The results of grain size distribution testing completed on three samples of the non-cohesive deposit are presented in Figure B6. Atterberg limit testing was carried out on three samples of the non-cohesive deposit; two Atterberg limits tests indicated a non-plastic silt, and the other Atterberg limits test (the results of which are presented on a plasticity chart in Figure B7 in Appendix B) measured a liquid limit of 16%, a plastic limit of 15%, and a corresponding plasticity index of 1%, which indicates a silt of slight plasticity. The water content measured on samples of the non-cohesive deposit ranges from about 17% to 25%.

- **Clayey Silt (CL) Till – Lower Deposit:** A lower cohesive deposit of glacial till consisting of clayey silt was encountered underlying the non-cohesive deposit in both boreholes. The lower cohesive till deposit was encountered at depths ranging from 5.8 m to 6.6 m below ground surface (approximately Elevations 217.0 m to 216.4 m) and extended to the termination depth of 6.7 m (Elevations 216.2 m to 216.1 m). One SPT "N"-value measured in the lower cohesive till deposit yielded 33 blows per 0.3 m of penetration, indicating a hard consistency. Atterberg limit testing was carried out on a sample of the lower cohesive till deposit and the results are presented on a plasticity chart in Figure B5 (Borehole RW-7 Sample 7B) in Appendix B. The Atterberg limits test measured a liquid limit of about 22%, a plastic limit of about 14% and a corresponding plasticity index of about 8%. The Atterberg limits test indicates a clayey silt of low plasticity. The water content measured on a sample of the lower cohesive till deposit was about 9%. Although not specifically encountered in the boreholes, the presence of cobbles and boulders should be expected in the lower cohesive till deposit.

4.3 Groundwater Conditions

The groundwater levels measured in the open boreholes at the time of the investigation are not considered representative of the stabilized hydrostatic groundwater levels at the site. All water levels recorded in the boreholes as part of this subsurface exploration program were taken shortly after drilling operations and therefore represent an unstabilized groundwater level. The unstabilized groundwater levels measured in the open boreholes upon completion of drilling are presented in the borehole records in Appendix A and are summarized below. Borehole RW-4 caved to a depth of 1.1 m (Elevation 208.7 m) and a water level was not recorded.

Location	Borehole No.	Groundwater Level in Open Borehole (Does Not Represent Stabilized Level)		Date of Reading
		Depth (m)	Elevation (m)	
Bass Pro Mills Drive S-E/W Ramp	RW-1	5.2	204.1	July 11, 2023
	RW-2	5.2	204.3	July 11, 2023
	RW-3	5.4	204.2	July 11, 2023
	RW-4	N/A	N/A	July 11, 2023
	RW-5	5.6	204.5	July 11, 2023
Rutherford Road S-E/W Ramp	RW-6	4.9	217.9	July 12, 2023
	RW-7	4.6	218.3	July 12, 2023

Based on the colour transition from brown to grey in the boreholes at the Bass Pro Drive Mills S-E/W ramp location, it is estimated that the groundwater level is between approximately Elevation 205 m to 206.5 m.

A standpipe piezometer was installed within an augered hole at the Rutherford Road S-E/W ramp between Boreholes RW-6 and RW-7. The location of this piezometer, designated RW-MW, is shown on Drawing 2. The groundwater level in the piezometer was measured at a depth of about 4.6 m (Elevation 218.2 m) on October 31, 2023.

The groundwater level and hydrostatic head at depth at this site will be subject to seasonal fluctuations and precipitation events; the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation and snow melt.

4.4 Analytical Testing of Soil

Three soil samples (two from the boreholes advanced in the vicinity of the Bass Pro Mills Drive S-E/W ramp retaining wall and one from the boreholes advanced in the vicinity of the Rutherford Road S-E/W ramp retaining wall) were submitted for laboratory analysis of parameters used to assess the potential corrosivity of the site soil to steel and concrete. Detailed analytical test results are included in Appendix C and the test results are summarized below.

Borehole No., Sample No.	pH	Resistivity (ohm-cm)	Electrical Conductivity (µmho/cm)	Soluble Chloride (µg/g)	Soluble Sulphate (µg/g)
RW-2, SS5	7.88	360	2750	1400	220
RW-4, SS3	7.72	620	1610	740	260
RW-6, SS3	7.99	640	1550	750	52

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sunduss Asghar, EIT, and Mr. Mark Henderson, P.Eng., a Geotechnical Engineer with WSP. Mr. David Staseff, P.Eng., a Senior Principal and MTO Principal Foundations Contact for WSP, and Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and an MTO Principal Foundations Contact for WSP, conducted an independent technical and quality control review of this report.

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MH/DS/LCC/ljv/al

https://golderassociates.sharepoint.com/sites/152126/project files/6 deliverables/3. foundations/2. reports/07. retaining walls/final/21490972-r-rev0_05dec2023_fidr_retaining_walls.docx

PART B

FOUNDATION DESIGN REPORT

Bass Pro Mills Drive and Rutherford Road
S-E/W Ramp Retaining Walls
Highway 400 Widening
Langstaff Road to Major Mackenzie Drive
Vaughan, Ontario
MTO GWP 2836-02-00

6.0 DISCUSSION AND FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides geotechnical/foundation design recommendations for the retaining walls to be constructed at the Bass Pro Mills Drive and Rutherford Road S-E/W ramps as part of the widening and rehabilitation works along Highway 400 from south of Langstaff Road to north of Major Mackenzie Drive in the City of Vaughan, Ontario.

These recommendations are based on interpretation of the data obtained from the boreholes advanced during the current field investigations. The discussion and recommendations presented are intended to provide the designers with information to carry out the detail design of the retaining walls. The discussion and recommendations in this Foundation Design Report are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. Contractors must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction must make their own interpretation of the data provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 Project Understanding

Based on the cross-section and plan drawings of the proposed retaining walls provided by Parsons on April 19, 2023, the proposed Bass Pro Mills Drive retaining wall has a total length of about 208 m and will be located between Stations 14+926 and 15+134 along the east (right) shoulder of the widened S-E/W ramp. Based on the cross sections, the proposed retaining wall will have a maximum wall stem height of about 2.4 m. The S-E/W ramp will be widened toward the existing highway ditch and the remainder of the ditch will be lowered to about elevation 207 m. The proposed Rutherford Road retaining wall has a total length of about 67 m and will be located between Stations 10+337 and 10+404 along the south (right) shoulder of the widened S-E/W ramp. Based on the cross sections, the proposed retaining wall will have a maximum wall stem height of about 1.7 m. The S-E/W ramp will be widened beyond the limits of the existing highway ditch and a new ditch will be constructed adjacent to the retaining wall.

6.2 Retaining Wall Options

Based on the geometries shown on the cross sections and the subsurface conditions at the site, retained soil system (RSS) walls and concrete cantilever walls are considered feasible options. Walls that utilize “top-down” construction applications (such as soldier piles with reinforced concrete facing panels or secant pile walls) were considered; however, these walls are generally not suitable for part cut and part fill scenarios (as is the case for these retaining wall sites).

It is understood that the RSS wall option is preferred as it is compatible with the proposed widening of the S-E/W ramp embankments and excavations would be located away from mainline traffic during staging of construction activities. A summary of these feasible retaining wall options is provided below.

- **Retained Soil System (RSS) Wall:** Mechanically reinforced soil retaining systems (retained soil system or RSS walls) have a front facing panel system that is supported on a concrete levelling pad placed at a shallow depth below the ground surface in front of the wall. The minimum soil cover to the base of the wall and top of the concrete levelling pad is typically at least 0.5 m below the finished grade in front of the RSS wall. An RSS wall with the front facing supported on a shallow strip footing or alignment element is feasible for the proposed retaining wall. Temporary excavations to the recommended founding stratum (below fill and softened materials) are expected to be in the order of about 1 m (relative to the ground surface in front of the wall) for both wall locations, and the excavation will extend behind the wall face for a width equal to approximately 80% of the wall height (i.e., approximately 1.4 m to 2.5 m), cutting into the existing S-E/W ramp embankment side slopes. Given the relatively modest extents of excavation and existing ramp geometries, together with potential for constructing the retaining walls during ramp closure periods, a temporary protection system will likely not be required. The MTO RSS Guideline (2008) Section 2.1.1 states that RSS walls are for embankment heights greater than 2 m. Accordingly, an RSS wall should be greater than 2 m in height to make the design feasible and preferred over a concrete cantilever wall. If the wall is between 1.5 m and 2 m, a full-height RSS wall panel could be utilized, as segmental RSS wall panels typically have dimensions of 1.5 m by 1.5 m. RSS must be selected from MTO DSM List #9.70.56 for Wall/Slope at high performance and appearance. MTO SP 599S22 and SP 599S23 should be included in the Contract Tender Documents.

- **Concrete Cantilever Wall on Shallow Foundations:** A concrete retaining wall supported on shallow foundations (concrete strip footing) is feasible for the proposed retaining wall structure. Similar to the RSS wall option, temporary excavations to allow for construction of the strip footing would be required. A concrete cantilever wall supported on a shallow foundation is preferred over an RSS wall for embankment heights less than 2 m.

A comparison table summarizing the geotechnical/foundations-related advantages, disadvantages, relative costs, and risks/consequences for both wall options is presented below. The selection of the type of walls and foundation alternatives will also depend on factors beyond geotechnical/foundation recommendations.

Wall Type	Feasibility	Advantages	Disadvantages	Relative Costs	Risks
RSS Wall	Feasible, but less advantageous compared to concrete cantilever wall when the wall height is less than 2 m	▪ Conventional construction techniques ▪ Tolerant of differential settlement ▪ Ease of construction with elimination of formwork, steel rebar placement, and curing of cast-in-place concrete ▪ Relatively rapid construction	▪ Coordination required to address obstructions through front face of wall or reinforced backfill (e.g., splaying or skewing reinforcement straps)	Lower cost than concrete cantilever wall when the wall height is greater than 2m	N/A
Concrete Cantilever Wall	Feasible, but less advantageous compared to RSS wall when the wall height is greater than 2 m	▪ Conventional construction techniques	▪ Less tolerant to settlement than RSS wall, although not anticipated to be problematic at this site given suitable foundation subgrade. ▪ Requires formwork, steel rebar placement, and curing of cast-in-place concrete.	Similar or lower cost than RSS wall when the wall height is less than 2 m	N/A

6.3 Design Considerations

6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the *Canadian Highway Bridge Design Code CAN/CSA S6-19* (CHBDC, 2019) and its Commentary, the retaining walls and their foundation systems may be classified as geotechnical systems designed for application along a transportation corridor with large traffic volumes and with potential impacts on other transportation corridors, resulting in a “typical consequence level” associated with exceeding limit states design. In addition, given the project-specific foundation investigation carried out at the retaining wall sites (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of the *CHBDC* (2019), the level of confidence for design is considered to be a “typical degree of site and prediction model understanding”. Accordingly, the appropriate corresponding ultimate limit state (ULS) and serviceability limit state (SLS) consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the CHBDC (2019) have been used for design.

6.3.2 Seismic Design

6.3.2.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation. Based on the energy-corrected average standard penetration resistance, $\overline{N}_{60}$ and average undrained shear strength, s_u within the upper 30 m of the overburden below the founding level (assumed to be existing ground surface), the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2019).

The CHBDC (2019) states that the seismic hazard values associated with the design earthquakes should be those established from the *National Building Code of Canada* (NBCC) by the Geological Survey of Canada (GSC). The 2015 seismic hazard maps (referred to as the 5th generation seismic hazard maps) have been used for preliminary design for this project, as referenced in the CHBDC (2019).

6.3.2.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the 2019 CHBDC, the peak ground acceleration (PGA), peak ground velocity (PGV) and 5% damped spectral response acceleration ($S_a(T)$) values for Site Class D are presented below. These values were obtained from the NBCC website (*earthquakescanada.nrcan.gc.ca*). Acceleration-based (F_a) and velocity-based (F_v) site coefficients have been applied to the seismic hazard values provided below to account for the Site Class D designation.

Seismic Hazard Values for Site Class D	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475-year return period)
PGA (g)	0.036	0.059	0.103
PGV (m/s)	0.029	0.046	0.076
$S_a(0.2)$ (g)	0.061	0.097	0.164
$S_a(0.5)$ (g)	0.039	0.060	0.095
$S_a(1.0)$ (g)	0.022	0.034	0.052
$S_a(2.0)$ (g)	0.010	0.016	0.026
$S_a(5.0)$ (g)	0.002	0.004	0.006
$S_a(10.0)$ (g)	0.001	0.002	0.003

6.3.2.3 Potential for Liquefaction

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil which may lead to potentially large surface deformations, and under undrained conditions generate excess pore water pressures that can lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (analogous to slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of slopes often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines. Fine-grained clayey soils which are not highly sensitive do not liquefy because surface tension holds the water-coated particles together.

In general, the soils at the Bass Pro Mills Drive retaining wall site consists of firm to hard (but generally stiff to very stiff) cohesive soils comprised of clayey silt-silt to silty clay. Considering the consistency and relatively low site-specific PGA, this retaining wall site is estimated to have a low potential for liquefaction during a seismic event based on soil type and liquefaction assessment of the soil using the simplified stress-based method (as per Section 6.14.8 of the CHBDC (2019)).

At the Rutherford Road retaining wall site, a 3.6 m to 4.4 m thick deposit of compact to dense silt was encountered, which was further underlain by hard cohesive soil. Liquefaction analyses were carried out to check if this silt deposit is considered liquefiable during the 2,475-year design earthquake. The method used to assess the liquefaction potential is consistent with that presented in the *Commentary* to the CHBDC (2019) and involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using in situ testing data collected at the borehole locations. The design groundwater level was assumed to be at about Elevation 219.5 m (i.e., the highest unstabilized groundwater level encountered in the boreholes, plus 1 m to account for seasonal fluctuations and rounded up to the nearest 0.5 m). The CRR with depth was calculated using the parameter $(N_1)_{60cs}$ that is based on the SPT “N”-value obtained in the field and corrected for the overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction analysis indicate that the silt deposit at the site is not considered liquefiable during the 2,475-year design earthquake.

6.4 Retaining Wall Foundations

6.4.1 Founding Elevations

The concrete levelling pad/reinforced soil mass (for an RSS wall) and strip footing (for a concrete cantilever wall) are recommended to be founded at or below the maximum (highest) founding elevations in the table below.

Structure	Maximum (Highest) Founding Elevation (m)	Anticipated Founding Stratum
Bass Pro Mills Drive Retaining Wall	207.1	Firm clayey silt-silt to silty clay till
Rutherford Road Retaining Wall	220.7	Very stiff clayey silt-silt till; compact to dense silt

The foundation subgrade should be inspected by qualified geotechnical personnel following removal of topsoil, fill, softened/organic soils, vegetation, and/or other unsuitable materials in accordance with OPSS.PROV 902 *(Excavating and Backfilling Structures)*. Where sub-excavation of fill or unsuitable materials is required, the sub-excavated area should be backfilled with granular material meeting OPSS.PROV 1010 Granular ‘A’, Granular

'B' Type I, Granular 'B' Type II, or SSM, placed and compacted in accordance with OPSS.PROV 501 *(Compacting)*, or the thickness of the levelling pad increased to the full sub-excavation depth.

The founding soils will be susceptible to disturbance and degradation on exposure to water, including potential for ongoing seepage associated with the highway ditches at these sites. Therefore, where RSS walls are adopted, it is recommended that the initial lift of granular fill be placed and compacted within four hours of inspection and approval of the prepared subgrade. Where concrete cantilever retaining walls are adopted, it is recommended that a concrete working slab be placed over the subgrade to protect the integrity of the foundation soils if placement of the concrete does not commence immediately following excavation for the retaining wall foundations; for this wall type, a Non-Standard Special Provision (NSSP) should be included in the Contract Documents for a working slab, a copy of which is provided in Appendix D (FOUN0001).

Where the wall is constructed by cutting into the existing slope, the back of the excavation (or reinforced soil mass for an RSS wall) should be keyed into the existing embankment by benching, as per OPSD 208.010 (Benching of Earth Slopes). Footings for a concrete cantilever wall should be founded at a minimum depth of 1.4 m below the adjacent final grade to provide adequate protection against frost penetration, in accordance with OPSD 3090.101 *(Foundation, Frost Penetration Depths for Southern Ontario).*

Depending on the final grade at the base of the RSS or concrete cantilever wall, the granular levelling pad or strip footing may need to be installed below the elevations recommended below to achieve the minimum embedment depth of 0.5 m or 1.4 m, respectively.

6.4.2 Geotechnical Resistances for RSS Wall Option

Assuming that the RSS wall acts as a unit and uses the full width of the reinforced soil mass (which has been taken as equal to 0.8 times the wall height to achieve the minimum required factor of safety for global stability), the factored ultimate and serviceability geotechnical resistances given below may be used for assessment of the reinforced mass founded on the properly prepared firm to very stiff native soils, at or below the highest founding elevations provided in Section 6.4.1.

Location	Factored Geotechnical Resistance at the Ultimate Limit State (f-ULS) (kPa)	Factored Net Geotechnical Reaction at the Serviceability Limit State (f-SLS) (kPa)	
		For 25 mm of Settlement	For 50 mm of Settlement
Bass Pro Mills Drive and Rutherford Road Retaining Walls	350	175	250

Note: The recommended minimum strip length should be taken as 80% of the wall height, which achieves the required minimum factor of safety for global stability. Longer strip lengths may be required by the proprietary designer to address internal stability of the wall, or if the geometry is modified such that there is sloping ground above the wall.

For the proposed wall height and geometry at this site, the maximum total settlement at the front of the reinforced soil mass and along the wall facing alignment is estimated to be about 15 mm.

The geotechnical resistances provided above are given for loads applied perpendicular to the subgrade surface. Where the load is not applied perpendicular to this surface, inclination of the load should be considered in accordance with Section 6.10.2 of the *CHBDC* (2019).

6.4.3 Geotechnical Resistances for Concrete Cantilever Wall Option

Strip footings constructed on the properly prepared subgrade, at or below the design elevations given in Section 6.4.1, should be designed based on the factored ultimate geotechnical resistance and the factored serviceability geotechnical resistance (for 25 and 50 mm of settlement) given below.

Location	Footing Width (m)	Factored Geotechnical Resistance at Ultimate Limit State (f-ULS) (kPa)	Factored Net Geotechnical Reaction at Serviceability Limit State (f-SLS) (kPa)	
			For 25 mm of Settlement	For 50 mm of Settlement
Bass Pro Mills Drive and Rutherford Road Retaining Walls	1	400	225	275
	2	450	150	225
	3	500	125	175

The factored ultimate and factored serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the factored geotechnical resistances should be reviewed if the footing width varies from that specified above or if the founding elevations differ from that given in Section 6.4.1. The factored ultimate geotechnical resistances provided are based on a load applied concentrically to the centreline/centroid of the footing, as shown on Figure 6.4 of the *CHBDC* (2019). Where a load is applied eccentrically from the centreline/centroid of the footing, the pressure distribution at ULS and SLS and the eccentricity limit of the footing should be taken into consideration in accordance with Section 6.10.5 of the CHDBC (2019) and its Commentary. If this option is selected, once the structural design is substantially complete, the structural engineer should verify with WSP whether the factored ultimate and serviceability geotechnical resistances provided above require revision based on any load inclination.

6.4.4 Resistance to Lateral Loads

Resistance to lateral forces / sliding between the levelling pad (for an RSS wall option) or concrete footing (for a concrete cantilever wall option) and the foundation subgrade should be calculated in accordance with Section 6.10.4 of the *CHBDC* (2019). The coefficient of friction, $\tan \emptyset'$, for interaction between various subgrade materials is presented below.

Anticipated Subgrade Material	Coefficient of Friction, $\tan \emptyset'$
Compacted Granular 'A' or cast-in-place concrete footing on firm to very stiff clayey silt-silt to silty clay till	0.60
Compacted Granular 'A' or cast-in-place concrete footing on compact to dense silt	0.62
Cast-in-place concrete footing on compacted Granular 'B' Type I pad or concrete working slab	0.70

6.5 Lateral Earth Pressures for Design

The lateral earth pressures acting on the retaining walls will depend on the type and method of placement of backfill materials, the nature of the soils behind the backfill, the magnitude of the surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II should be used as backfill behind concrete cantilever walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to subdrains and frost taper for a concrete cantilever wall should be in accordance with OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and OPSD 3190.100 (Walls, Retaining and Abutment, Wall Drain).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2019) Section 6.12.3 and Figure 6.8. Care must be taken during the compaction operation not to overstress the wall, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (Compacting). Other surcharge loadings should be accounted for in the design, as required.
- For a retaining wall constructed on shallow foundations (i.e., an unrestrained, concrete cantilever retaining wall), fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:<1V) extending up and back from the rear face of the footing or pile cap in accordance with Figure C6.31(b) of the Commentary to the CHBDC (2019).

6.5.1 Static Lateral Earth Pressures

The following guidelines and recommendations are provided regarding the lateral earth pressures for static loading conditions. The parameters below assume level backfill and ground surface behind the retaining wall. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope in accordance with the equations provided in CHBDC Section C6.12.1, Figures C6.28 (for active earth pressure), and Section C6.12.2.2 (for at-rest earth pressure).

For an unrestrained retaining wall, in the case of the cantilever wall option, the pressures are based on the properties of the granular backfill, and the following parameters (unfactored) may be used:

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

If the wall support allows for lateral yielding, active earth pressures may be used in the geotechnical design of the retaining wall. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.12 of the Commentary of the CHBDC (2019).

If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6 Global Stability

Limit equilibrium global slope stability analyses were carried out for the proposed retaining walls using the commercially available program Slide (version 9.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. The Factors of Safety of numerous potential failure surfaces were computed to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. The Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ, and the geotechnical resistance factor, ϕ_g (i.e., $FS = 1/(\Psi \cdot \phi_g)$).

Accordingly, minimum Factors of Safety of 1.5 have been used for the design of the proposed retaining walls for the long-term (permanent) conditions, as per Table 6.2 of CHBDC (2019).

The following parameters have been used in the stability analyses for the permanent (effective stress) and temporary (undrained) conditions, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):

Location	Stratigraphic Unit	γ (kN/m^3)	φ' (°)	S_u (kPa)
Bass Pro Mills Drive Retaining Wall	New granular fill behind RSS wall or wall backfill zone (Granular 'B' Type II)	21	34	-
	Existing granular fill (compact)	20	30	-
	Existing cohesive fill (firm to very stiff)	19	28	50
	Clayey silt-silt to silty clay till (firm to hard)	20	29	100
Rutherford Road Retaining Wall	New granular fill behind RSS wall or wall backfill zone (Granular 'B' Type II)	21	34	-
	Existing granular fill (compact)	20	30	-
	Existing cohesive fill (stiff)	19	28	75
	Clayey silt-silt till (very stiff)	20	29	150
	Silt (compact to dense)	20	31	-

A maximum retained wall height of 2.4 m and 1.7 m was used in the analyses for the Bass Pro Mills Drive and Rutherford Road retaining walls, respectively. The design groundwater level was considered at the ditch bottom elevation.

The stability analysis indicates that the proposed RSS wall (with a strip length assumed to be 0.8 times the height of the wall) and concrete cantilever wall options will have a Factor of Safety greater than 1.5 against global instability. The results of the stability analyses are shown on Figures 1 to 8 following the text of this report.

6.7 Settlement

Settlement analyses were carried out to estimate the magnitude of expected settlement of the widened approach embankments under the height of fill at critical sections along the proposed retaining walls. The sources of settlement at this site are considered to include the following:

- Immediate (short-term) settlement of the compact to dense silt deposits; and
- Primary and secondary (creep) time-dependent consolidation of the cohesive deposits (using Terzaghi's one-dimensional consolidation theory – long term), although the majority of these deposits are overconsolidated and contain significant sand content and settlement is expected to occur rapidly during, or shortly after, construction of the retaining walls.

Based on the cross-section and plan drawings of the proposed retaining walls provided by Parsons on April 19, 2023, the maximum height of fill to be placed is about 2 m. Accordingly, the total estimated settlement of the existing site soils under the loading imposed by the widened highway embankments is about 10 mm at both wall locations, and therefore settlement mitigation measures are not required. The analyses assume that all topsoil, loosened/softened soils, vegetation, and any other unsuitable surficial soils are stripped from the existing highway ditches prior to embankment widening and new fill consisting of OPSS.PROV 1010 *(Aggregates)* Granular ‘A’, Granular ‘B’ Type I, Granular ‘B’ Type II or SSM is placed and compacted in accordance with OPSS.PROV 501 *(Compacting)* and OPSS.PROV 206 *(Grading)*.

6.8 Construction Considerations

6.8.1 Open-Cut Excavations

The excavations for the new retaining walls will extend into the existing firm to very stiff cohesive fill material, firm to very stiff cohesive deposits of clayey silt-silt to silty clay, and compact to dense non-cohesive deposits of silt (at the location of the Rutherford Road retaining wall). Considering the widened highway embankments and retaining wall footprints will be located within the existing highway ditches, minor sub-excavation of loosened/softened soils and possibly organic soils should be expected during construction of the retaining wall. All unsuitable soils should be sub-excavated and replaced with OPSS.PROV 1010 *(Aggregates)* Granular ‘A’, Granular ‘B’ Type I, Granular ‘B’ Type II or SSM. All excavations must be carried out in accordance with Ontario Regulation 213 of the Ontario Occupational Health and Safety Act for Construction Projects (OHSA), as amended.

Based on the unstabilized groundwater levels encountered in the boreholes (ranging from about Elevation 204.1 m to 204.5 m at the location of the Bass Pro Mills Drive retaining wall and ranging from about Elevation 217.9 m to 218.3 m at the location of the Rutherford Road retaining wall), excavations are expected to be above groundwater level. Accordingly, the firm to very stiff cohesive fill, firm to very stiff cohesive deposits, and compact to dense non-cohesive deposits above the groundwater level can all be initially classified as Type 3 soils according to OHSA. Temporary excavations (i.e., those open for a relatively short time period) should be made with side slopes of 1H:1V or flatter. Although subexcavation is not anticipated to extend below groundwater level, all soils below groundwater level would be classified as Type 4 soil, which requires side slopes of 3H:1V or flatter.

Temporary excavations should be observed and reviewed during construction to confirm that the soil and groundwater conditions are as anticipated. If unexpected conditions are encountered, a qualified geotechnical engineer should review the excavation plan considering the conditions at that time.

6.8.2 Engineered and Granular Fill

The existing site soils that do not contain topsoil or organics or any other deleterious materials can generally be reused on site as engineered fill. Soils from within the project limits to be reused as engineered fill must satisfy the gradation of OPSS.PROV 1010 Select Subgrade Material (SSM). Based on the measured natural water contents, the existing site soils are generally at or slightly above their estimated optimum water contents for mechanical compaction purposes and therefore soil “wetting” will likely not be required; however, some drying may be necessary to achieve the required compaction levels. Given the protracted timeframes required for air-drying the fine-grained soils encountered at the retaining wall sites, soils that are above their estimated optimum water contents for compaction will likely need to be disposed of and imported materials meeting the required of OPSS.PROV 1010 (Aggregates) Granular ‘B’ Type I or SSM should be used for engineered fill for embankment widening behind the backfill zone, or as a replacement material where very loose to loose, soft or other deleterious soils are sub-excavated at subgrade level.

Following proof-rolling and approval of the subgrade, the engineered fill should be placed in accordance with OPSS.PROV 501 (Compacting) and compacted to 98% of the material’s Standard Proctor maximum dry density. Where sub-excavation is required below the retaining wall footing or reinforced soil mass, it is recommended that the engineered fill extend at least 1 m beyond the edges of the footings.

The final surface of the engineered fill should be protected as necessary from construction traffic and should be sloped to provide positive drainage for surface water during the construction period.

6.8.3 Control of Groundwater and Surface Water During Construction

Based on the boreholes, excavations for the retaining wall structures are generally expected to be above the groundwater table. Nevertheless, some groundwater seepage into the open excavations should be expected, especially considering their proximity to the existing highway ditches and if perched groundwater is encountered. The temporary excavations for the retaining wall and ditch lowering are anticipated to extend to about Elevation 207 m for the Bass Pro Mills Drive retaining wall and about Elevation 220.7 m for the Rutherford Road retaining wall. The design groundwater level is estimated to be at about Elevation 206.0 m at the location of the Bass Pro Mills Drive retaining wall. The stabilized groundwater level encountered in the standpipe piezometer was at about Elevation 218.2 m at the Rutherford Road retaining wall, although the water level in the non-cohesive soils at this location may be higher during wet periods of the year.

As noted in Section 6.8.1, excavations at the Bass Pro Mills Drive S-E/W Ramp retaining wall are expected to generally encounter cohesive fill or native cohesive till soils, whereas excavations at the Rutherford Road S-E/W Ramp retaining wall are expected to generally encounter cohesive fill or native non-cohesive soils (compact to dense silt). At the Rutherford Road retaining wall, excavations into the non-cohesive soils are expected to be above the design (i.e., stabilized) groundwater level unless sub-excavation depths in the highway ditch exceed 1 m. Therefore, it is expected that groundwater can be controlled by trenching or diversion ditches with sufficient sumps and pumps located within the excavations at both retaining wall locations.

Design of temporary dewatering systems is the responsibility of the Contractor, who should retain a specialist dewatering subcontractor to design and oversee dewatering operations. All dewatering operations should be carried out/managed in accordance with OPSS.PROV 902 (Excavation and Backfilling – Structures) and OPSS.PROV 517, as amended by SP 517F01 (Dewatering System, Temporary Flow Passage System). The foundation designer fill-in for Table A of SP 517F01 should indicate that the preconstruction survey distance is not applicable (“N/A”). A copy of SP 517F01 is provided in Appendix D.

Construction water takings in excess of 50,000 L/day are regulated by the Ministry of the Environment, Conservation and Parks (MECP). Certain takings of groundwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MECP’s Environmental Activity and Sector Registry (EASR), requiring a “Water Taking Plan” and a “Discharge Plan” (to be developed by the Design-Builder). A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. The contractor will be responsible for obtaining any required discharge approvals. Based on the subsurface conditions encountered, it is expected that an EASR or PTTW will not be required specific to these wall locations, although an overall EASR or PTTW may be applicable for the full project limits.

Considering the retaining walls are located directly within the existing highway ditches, consideration should be given to constructing temporary cofferdam / flow diversion structures to reduce surface water and groundwater infiltration and reduce dewatering efforts.

Surface water and stormwater should be directed away from the excavation areas to prevent ponding and/or flowing water that could result in disturbance and loosening/softening of the foundation subgrade.

6.9 Corrosion Assessment and Protection

Soil corrosivity may affect the concrete or steel elements (e.g., reinforcing steel) of foundations or related structures buried in the soil. The long-term performance and durability of the foundations are directly related to their respective corrosion resistance. Generally, the corrosivity potential to a structure can be assessed based on indicators such as soil resistivity / electrical conductivity, hydrogen ion concentration (pH), and salts (chloride and sulphate) concentrations. The analytical results of these indicators for the soil samples submitted for testing (two samples at the location of the Bass Pro Mills Drive retaining wall and one sample at the location of the Rutherford Road retaining wall) are summarized in Section 4.4 and discussed below, and the analytical laboratory test report is included in Appendix C.

6.9.1 Potential for Sulphate Attack

The analytical test results were compared to CSA Standard, CAN/CSA-A23.1-14 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* to assess potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the soil samples tested, when the designer is selecting the exposure class for foundations or buried structures, the effects of sulphates may not need to be considered. However, given the proximity of the retaining walls to de-icing salt used on the highway, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1.

6.9.2 Potential for Corrosion

According to MTO’s *Gravity Pipe Design Guidelines* (2014), the pH is not considered detrimental to steel durability as it is less than a pH of 8.5.

The resistivity measured in the tested soil samples (360 to 640 ohm-cm) indicates that the soil corrosiveness is “severe” (R < 2,000 ohm-cm) as per Table 3.2 of MTO’s *Gravity Pipe Design Guidelines* (2016) and therefore, some level of corrosion protection should be applied to the retaining walls.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Mark Henderson, P.Eng., a Geotechnical Engineer with WSP. Mr. David Staseff, P.Eng., a Senior Principal and MTO Principal Foundations Contact for WSP, and Ms. Lisa Coyne, P.Eng., Geotechnical Engineering Fellow and an MTO Principal Foundations Contact for WSP, conducted an independent technical and quality control review of this report.

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MH/DS/LCC/ljv/al

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ASTM International

ASTM D1586 Standard Test Method for Standard Penetration Test (SPT) and Split Barrel Sampling of Soils

Commercial Software:

Slide2 (Version 9.0) by Rocscience Inc.

Ontario Occupational Health and Safety Act:

Ontario Reg. 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206 Construction Specification for Grading

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 539 Construction Specification for Temporary Protection Systems

OPSS.PROV 902 Construction Specification for Excavating and Backfilling-Structures

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

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3121.150 Walls, Retaining, Backfill, Minimum Granular Requirements

OPSD 3190.100 Walls, Retaining and Abutments, Walls

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.

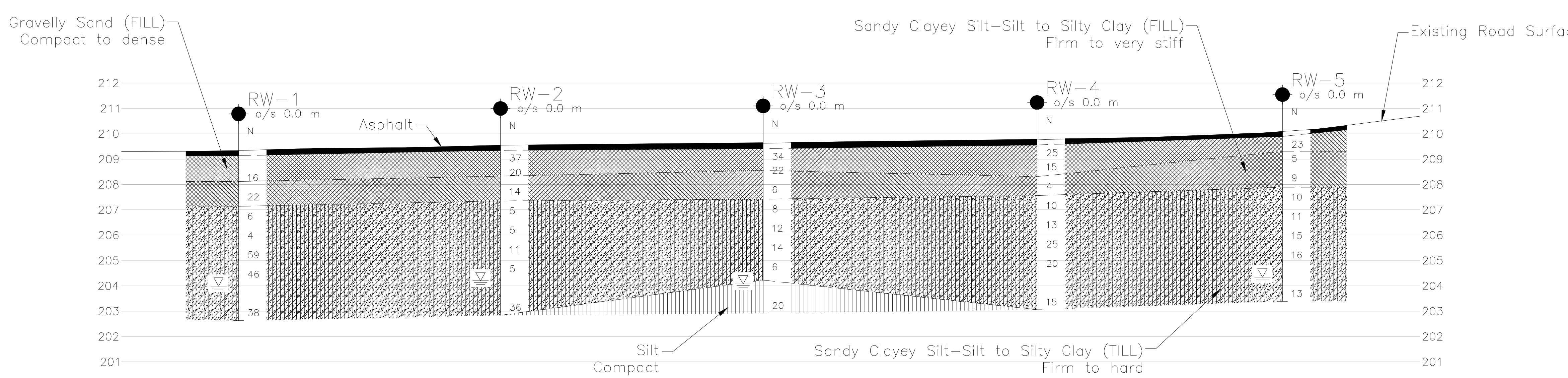
CONT No.
GWP No. 2836-02-00

SHEET

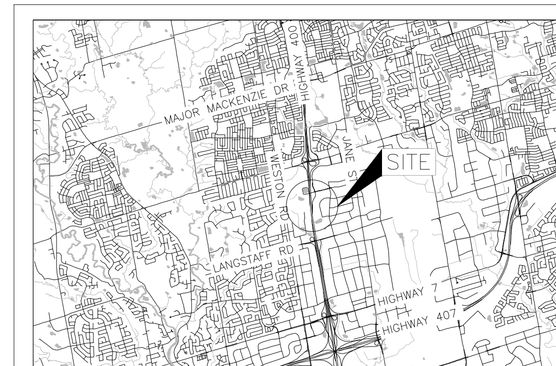
HIGHWAY 400 WIDENING
BASS PRO MILLS DRIVE S-E/W RAMP RETAINING WALL
BOREHOLE LOCATIONS AND SOIL STRATA

wsp

PLAN
SCALE
10 0 10 20 m



PROFILE A-A'
VERTICAL SCALE
2 0 2 4 m
HORIZONTAL SCALE
10 0 10 20 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
RW-1	209.3	4852935.5	301196.7
RW-2	209.5	4852987.1	301193.1
RW-3	209.6	4853039.0	301193.6
RW-4	209.8	4853093.1	301192.4
RW-5	210.1	4853141.4	301188.5

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 Parsons, drawing file nos. Hwy400_Existing Survey-Topo.dwg, H400-ROD-PLN.dwg, 73-400.xml, received June 1, 2022.
Culvert plan provided by Parsons, file no. H400-DRN-PLN.dwg, received September 29, 2022.
Design plan provided by Parsons, file no. Existing OHS Footing Locations – H400-478198-ROD-PMK.dwg, received March 8, 2023.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Dec. 5, 2023 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER M. S. HENDERSON 100523882 Dec. 5, 2023 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

Geocres No.

HWY. 400		PROJECT NO. 21490972	DIST.
SUBM'D. MH	CHKD. MH	DATE: 12/05/2023	SITE:
DRAWN: DD/SA	CHKD. MH	APPD. LCC	DWG. 1

MINISTRY OF TRANSPORTATION, ONTARIO

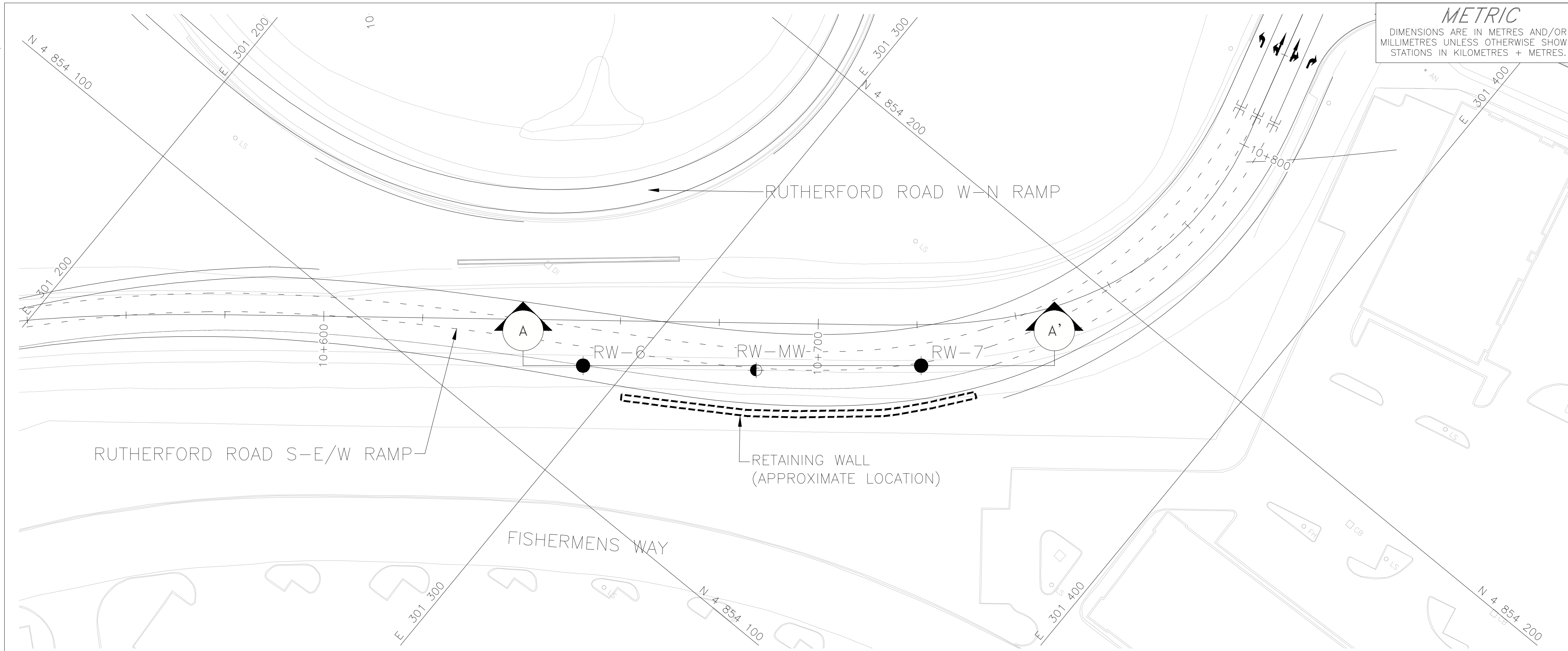
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 2836-02-00

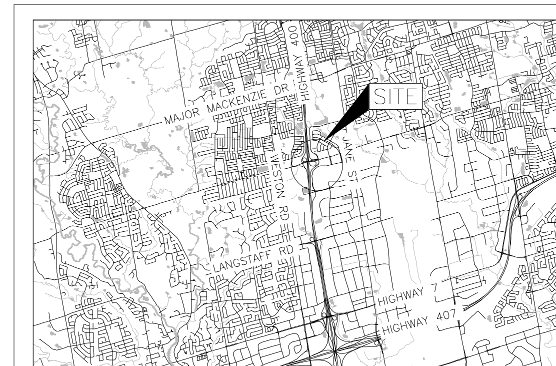
SHEET

HIGHWAY 400 WIDENING
RUTHERFORD ROAD S-E/W RAMP RETAINING WALL
BOREHOLE LOCATIONS AND SOIL STRATA

wsp



PLAN
SCALE
10 0 10 20 m



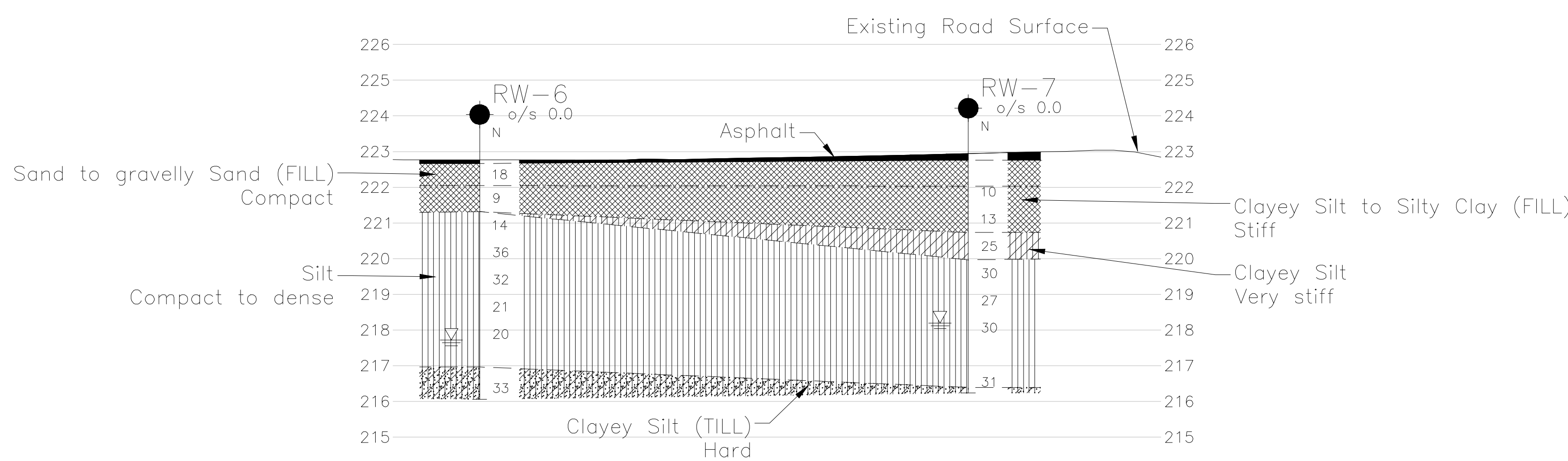
KEY PLAN
SCALE
2 0 2 4 km

LEGEND

●	Borehole – Current Investigation
◐	Monitoring Well – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▽	WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
RW-6	222.8	4854121.2	301292.6
RW-7	222.9	4854164.6	301345.4
RW-MW	222.8	4854142.7	301320.2



PROFILE A-A'
SCALE
2 0 2 4 m
HORIZONTAL SCALE
10 0 10 20 m

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.

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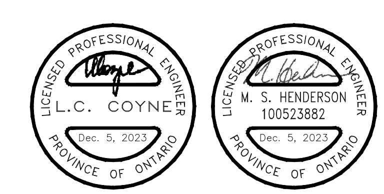


NO.	DATE	BY	REVISION

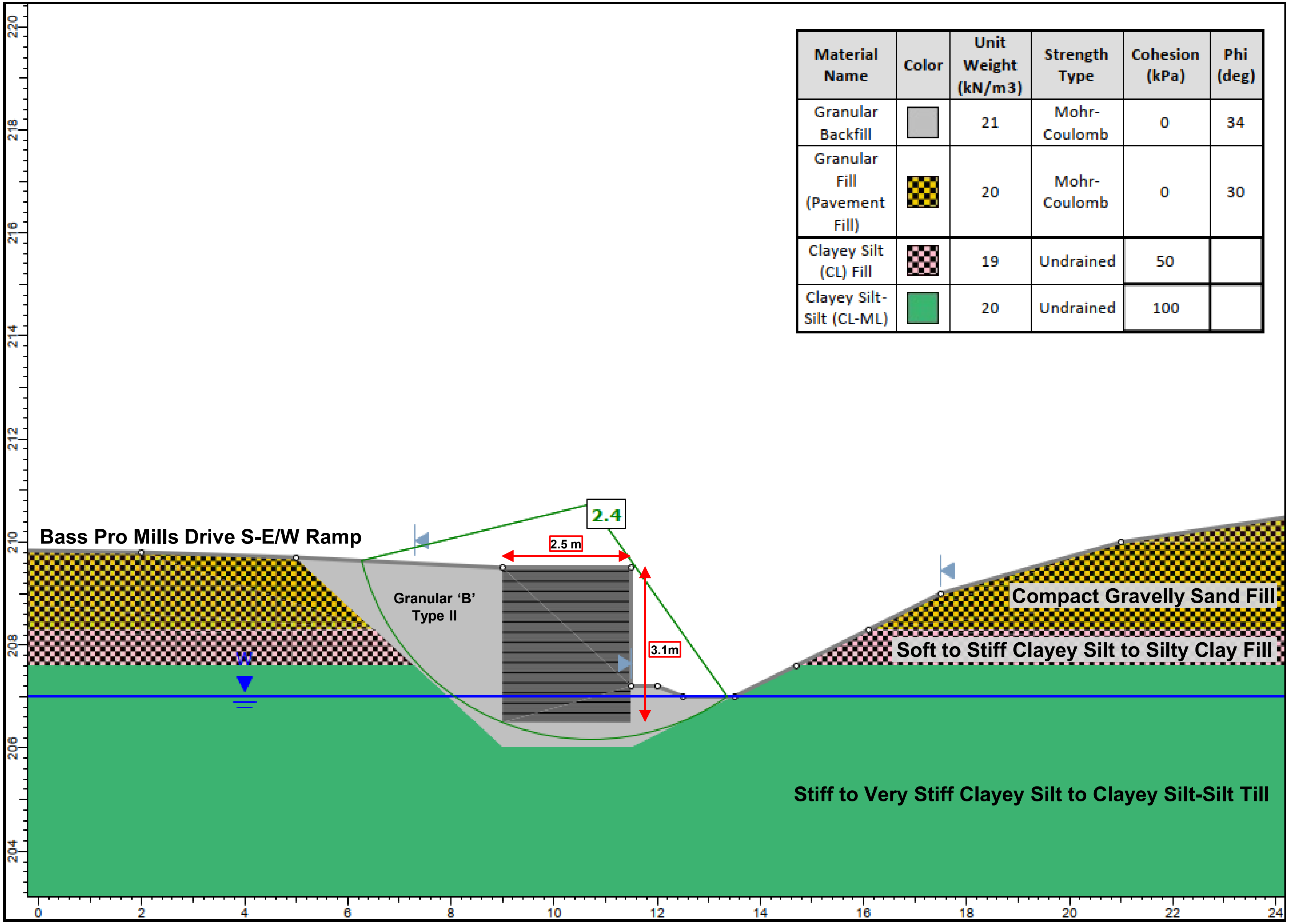
Geocres No.

HWY. 400		PROJECT NO. 21490972	DIST.
SUBM'D. MH	CHKD. MH	DATE: 12/05/2023	SITE:
DRAWN: DD/SA	CHKD. MH	APPD. LCC	DWG. 2

PLOT DATE: December 5, 2023
FILENAME: S:\Clients\MTO\Hwy_400_Langstaff_to_Major_Mackenzie\99_PRO\21490972_Parsons\40_PROD\0010_Retaining_Walls\21490972-0010-BG-0002.dwg

Global Stability Analysis Results (Undrained Condition)
Critical Case (Bass Pro Mills S-E/W Ramp – RSS Wall)

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Undrained	50	
Clayey Silt-Silt (CL-ML)		20	Undrained	100	

Date: December 2023
Project No: 21490972

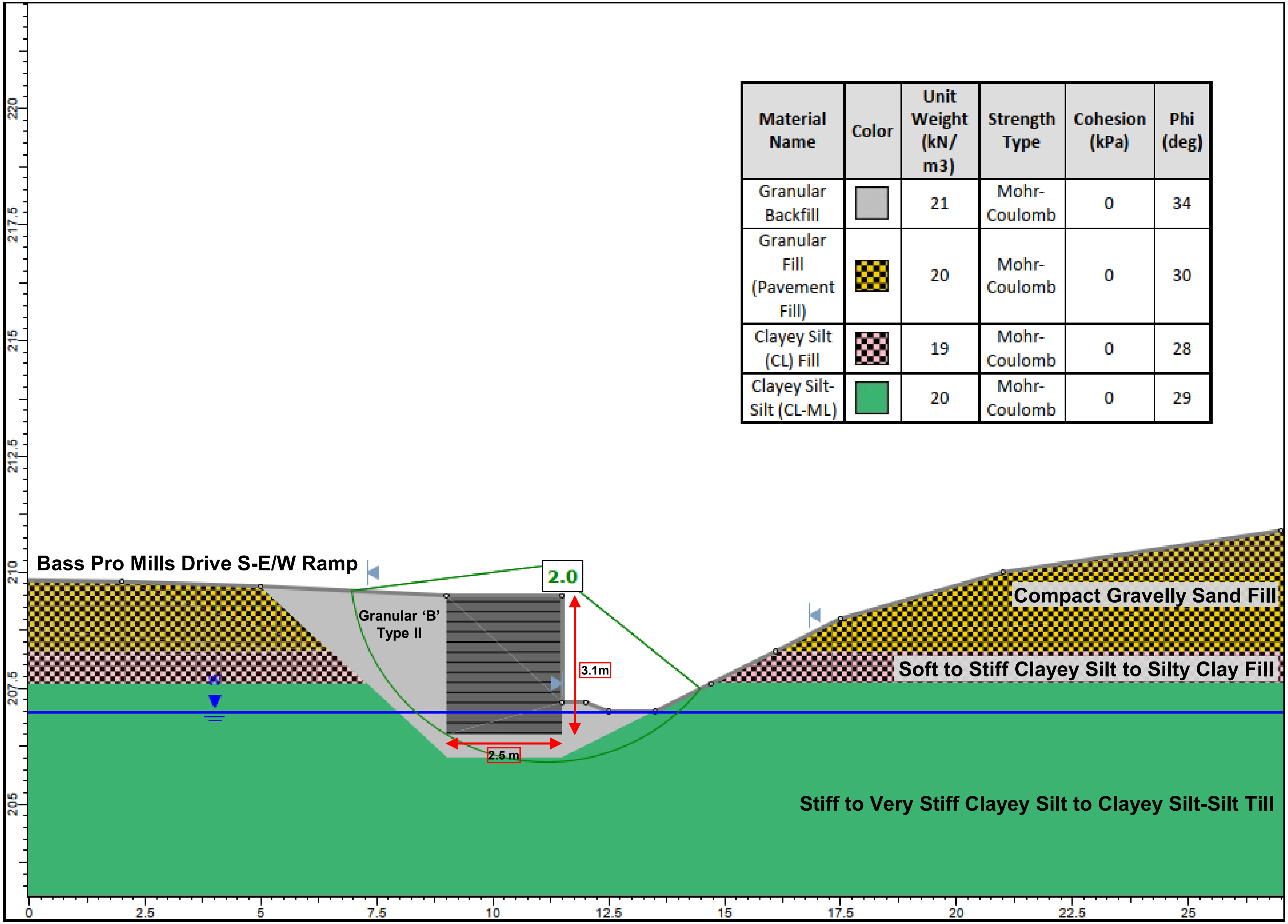
Analysis By: SA **Reviewed By:** LCC

wsp GOLDER

Global Stability Analysis Results (Drained Condition)

Critical Case (Bass Pro Mills S-E/W Ramp – RSS Wall)

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Mohr-Coulomb	0	28
Clayey Silt-Silt (CL-ML)		20	Mohr-Coulomb	0	29

Date: December 2023

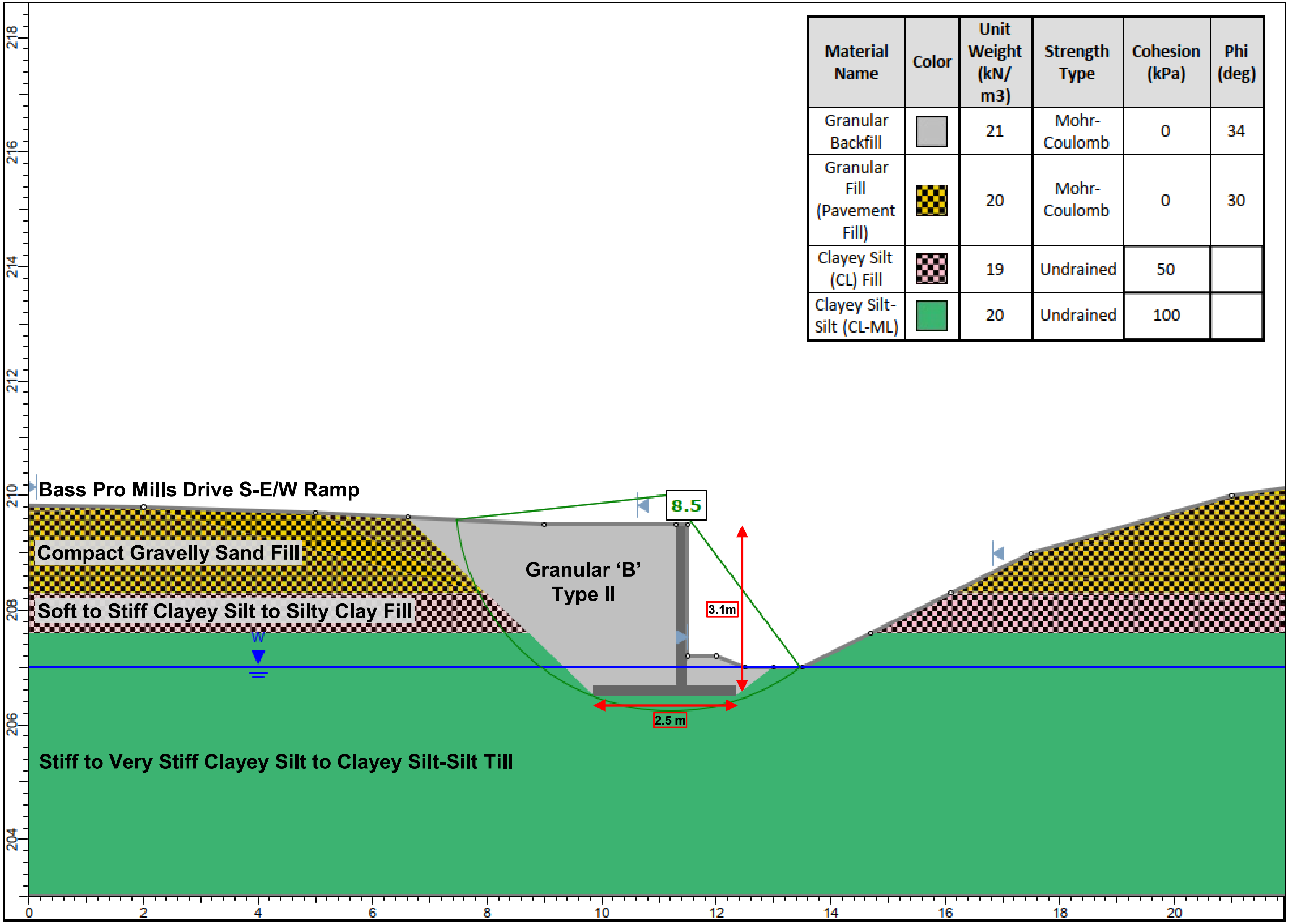
Project No: 21490972

Analysis By: SA **Reviewed By:** LCC

Global Stability Analysis Results (Undrained Condition)

Critical Case (Bass Pro Mills S-E/W Ramp – Concrete Cantilever Wall)

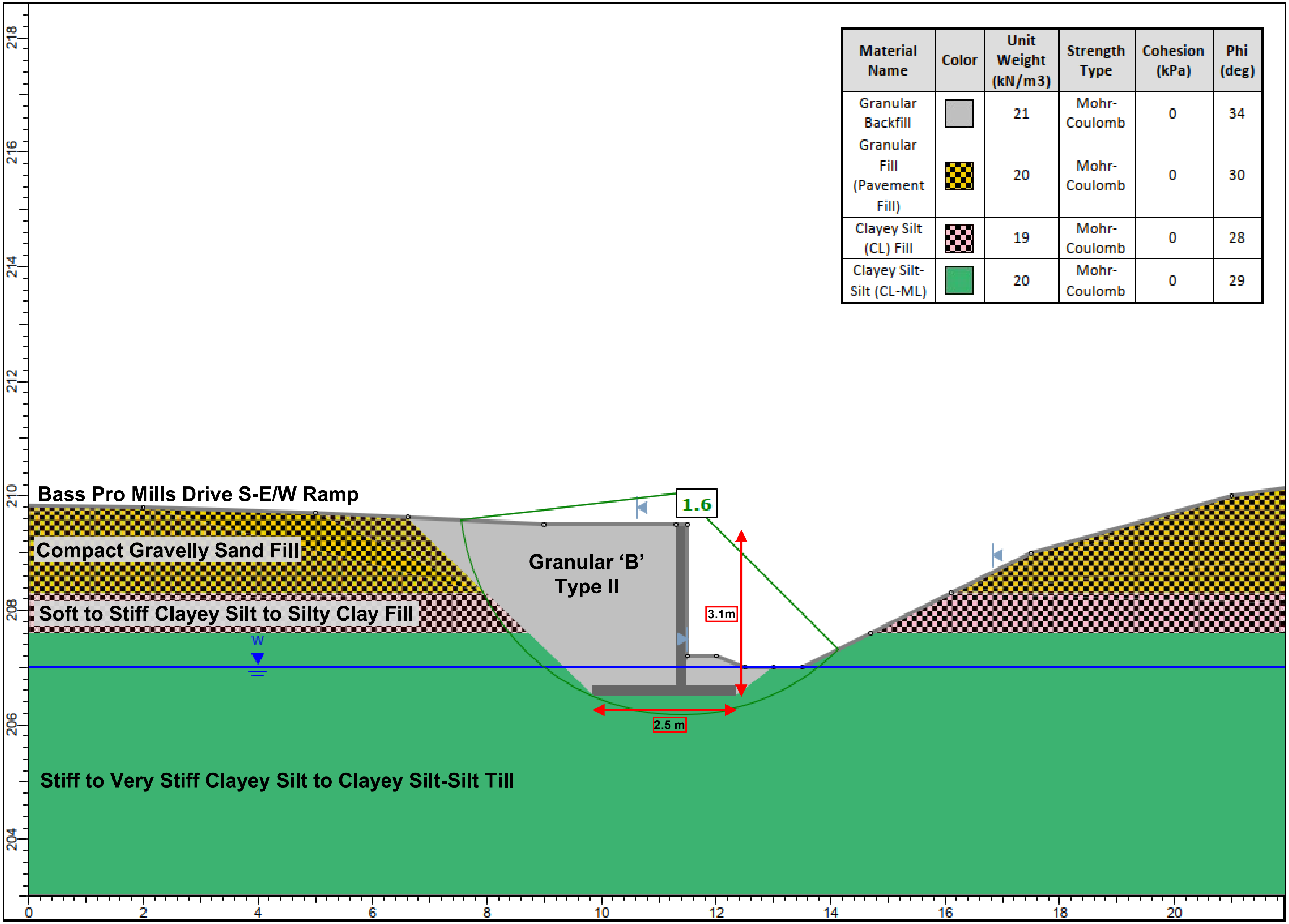
Figure 3



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Undrained	50	
Clayey Silt-Silt (CL-ML)		20	Undrained	100	

Global Stability Analysis Results (Drained Condition)
Critical Case (Bass Pro Mills S-E/W Ramp – Concrete Cantilever Wall)

Figure 4



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Mohr-Coulomb	0	28
Clayey Silt-Silt (CL-ML)		20	Mohr-Coulomb	0	29

Date: December 2023
Project No: 21490972

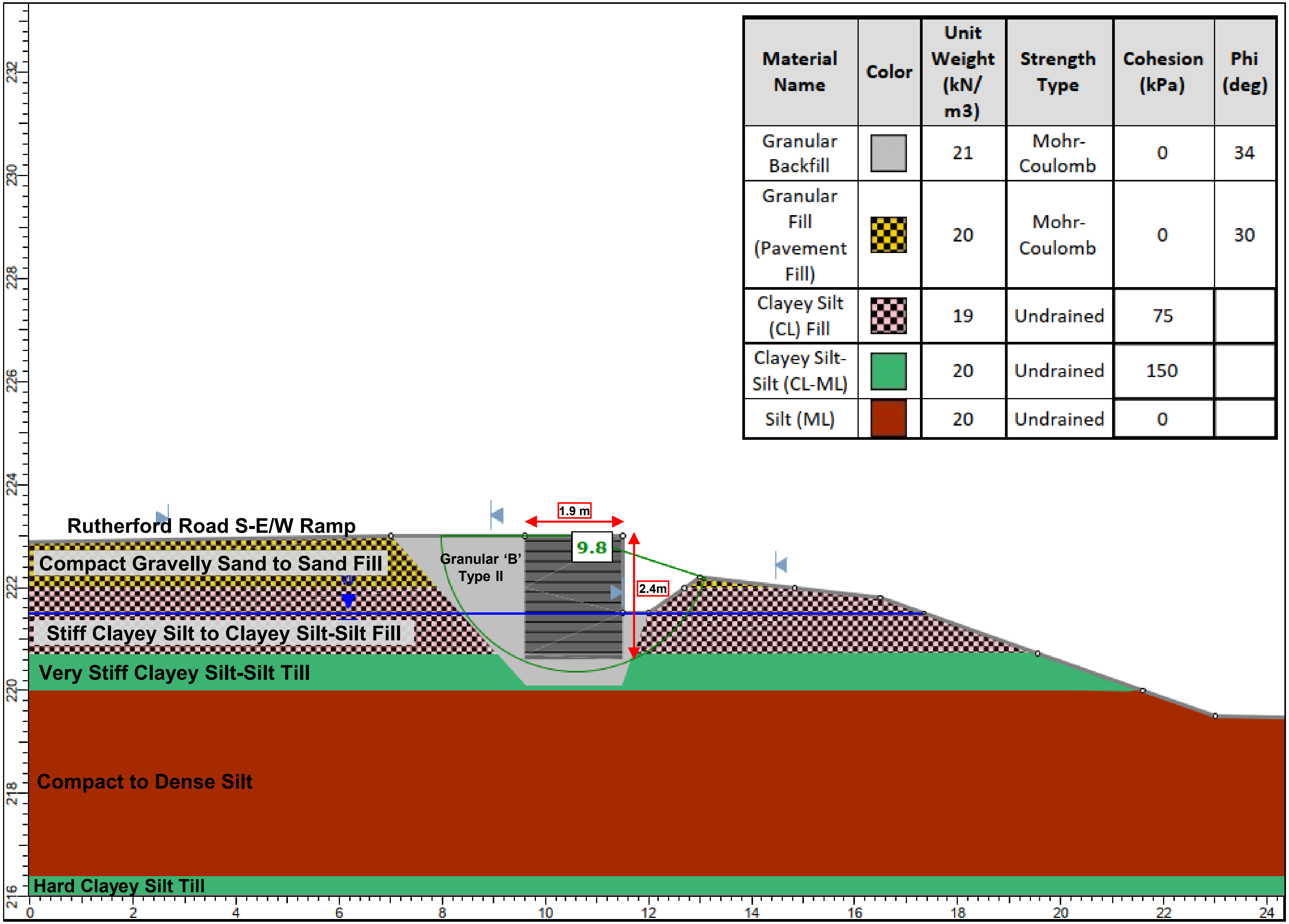
Analysis By: SA **Reviewed By:** LCC

WSP GOLDER

Global Stability Analysis Results (Undrained Condition)

Critical Case (Rutherford Road S-E/W Ramp – RSS Wall)

Figure 5



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Undrained	75	
Clayey Silt-Silt (CL-ML)		20	Undrained	150	
Silt (ML)		20	Undrained	0	

Date: December 2023

Project No: 21490972

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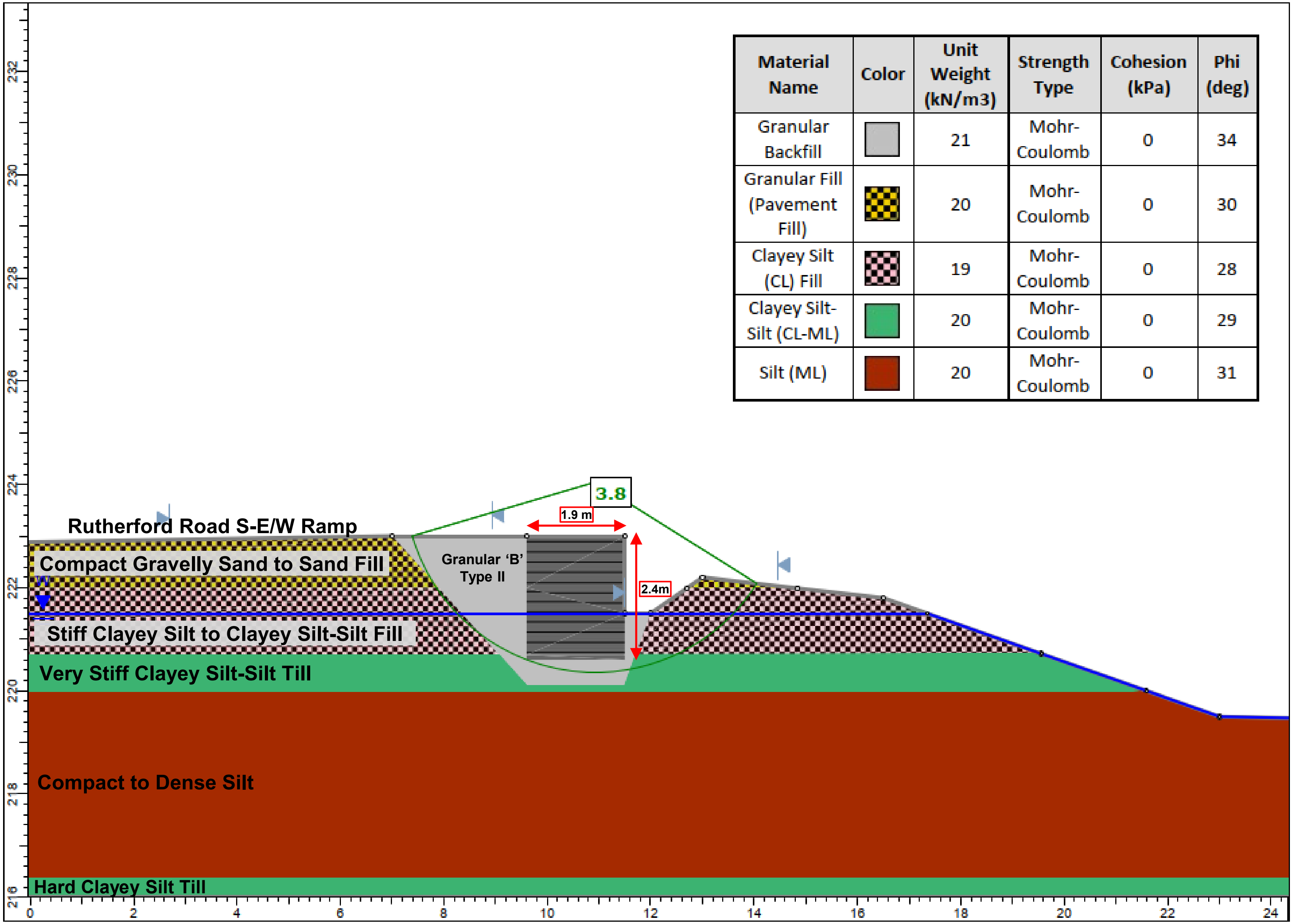
Global Stability Analysis Results (Drained Condition)
Critical Case (Rutherford Road S-E/W Ramp – RSS Wall)

Figure 6



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Mohr-Coulomb	0	28
Clayey Silt-Silt (CL-ML)		20	Mohr-Coulomb	0	29
Silt (ML)		20	Mohr-Coulomb	0	31

Date: December 2023
Project No: 21490972

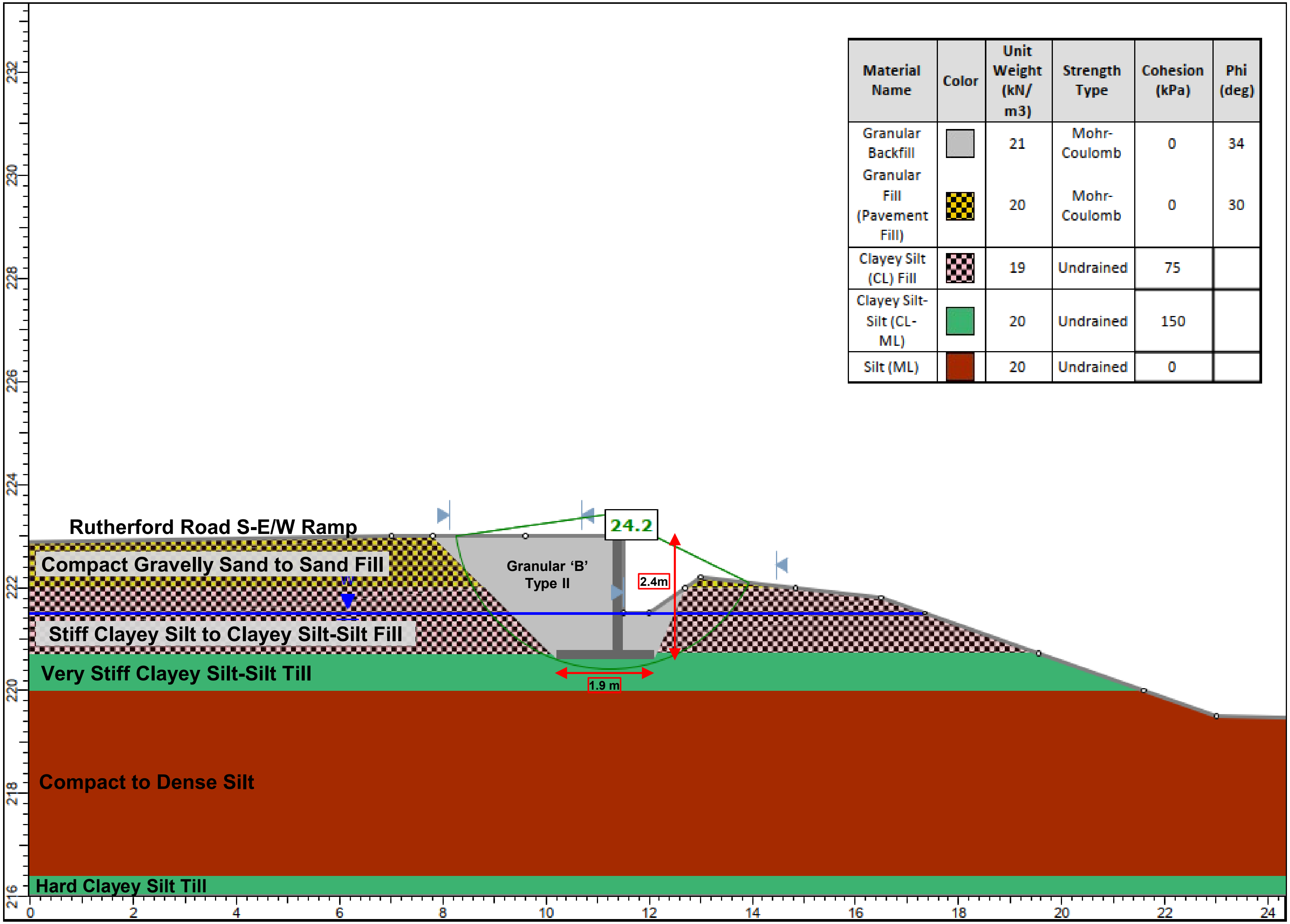
Analysis By: SA **Reviewed By:** LCC

WSP GOLDER

Global Stability Analysis Results (Undrained Condition)

Critical Case (Rutherford Road S-E/W Ramp – Concrete Cantilever Wall)

Figure 7



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Undrained	75	
Clayey Silt-Silt (CL-ML)		20	Undrained	150	
Silt (ML)		20	Undrained	0	

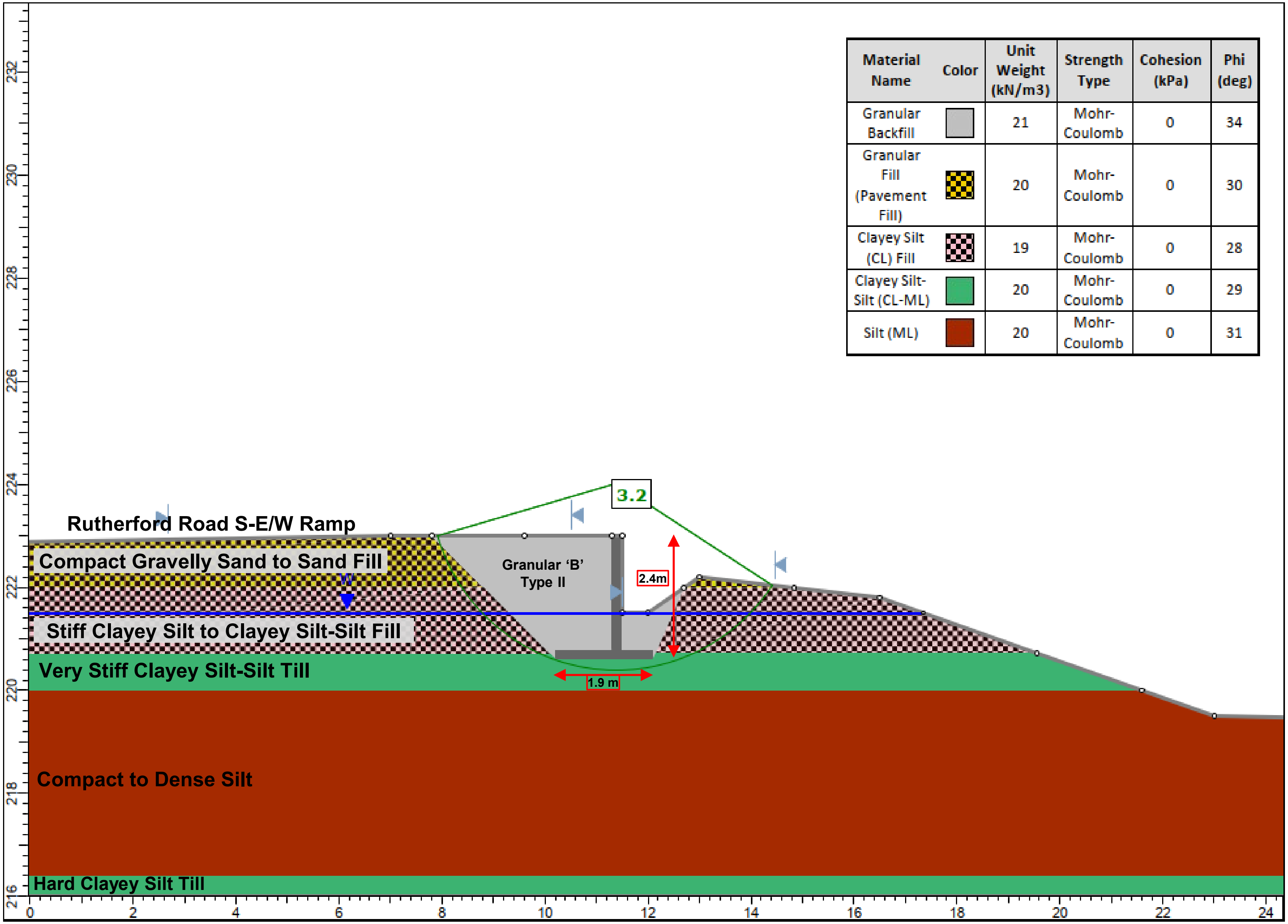
Date: December 2023

Project No: 21490972

Analysis By: SA **Reviewed By:** LCC

Global Stability Analysis Results (Drained Condition)
Critical Case (Rutherford Road S-E/W Ramp – Concrete Cantilever Wall)

Figure 8



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Granular Backfill		21	Mohr-Coulomb	0	34
Granular Fill (Pavement Fill)		20	Mohr-Coulomb	0	30
Clayey Silt (CL) Fill		19	Mohr-Coulomb	0	28
Clayey Silt-Silt (CL-ML)		20	Mohr-Coulomb	0	29
Silt (ML)		20	Mohr-Coulomb	0	31

Date: December 2023 **Analysis By:** SA **Reviewed By:** LCC
Project No: 21490972

APPENDIX A

Borehole Records

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS
MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY COMPONENTS[1,2]

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (*i.e.,* SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (*i.e.,* some sand)
≤ 10	trace (*i.e.,* 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 cm^2 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, Gs)
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)
γ	unit weight

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

COARSE-GRAINED SOILS

Compactness[1]

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

1. 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 grainsize. 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.
2. 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

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. 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.

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{\alpha(e)}$	secondary compression index
C_α	rate of secondary compression
$C_{\alpha(\varepsilon)}$	modified secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c′	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q′	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Ontario

RECORD OF BOREHOLE No. RW-1

Sheet 1 of 1 — METRIC

PROJECT 21490972
G.W.P. 2836-02-00 LOCATION N 4852935.5; E 301196.7 NAD83 / MTM Zone 10 (LAT. 43.816376; LONG. -79.544792) ORIGINATED BY T.T.
DIST CENTRAL HWY 400 BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger COMPILED BY P.T.
DATUM Geodetic Surface Elevation:209.3 m DATE Jul 11, 2023 CHECKED BY M.H.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
209.3 / 0.0	ASPHALT									
209.1 / 0.2	Gravelly SAND (SW), some silt (FILL) Compact Grey Moist	1A	SS	16						
208.1 / 1.2	CLAYEY SILT (CL), trace gravel (FILL), contains organics Very stiff Dark brown Moist	1B								
		2	SS	22						
207.1 / 2.2	SILTY CLAY (CI), some sand, contains oxidation staining (TILL) Brown and grey mottled Firm Moist	3	SS	6		0	17	45	38	
206.4 / 3.0	Sandy CLAYEY SILT - SILT (CL- ML), trace gravel (TILL) Grey Hard Moist	4	SS	4						
		5	SS	59						
		6	SS	46		1	34	51	14	
		7	SS	38						
202.6 / 6.7	End of Borehole									

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUNDWATER CONDITIONS; ELEVATION SCALE (209, 208, 207, 206, 205, 204, 203); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, × Field Vane; WATER CONTENT (%) PL W_p, NMC W, LL W_l, NP Nonplastic, 20 40 60; UNIT WEIGHT γ kN/m³

NOTES:

1. Borehole caved to a depth of 5.8 m (Elev. 203.5 m) upon completion of drilling.
2. Water measured inside borehole at a depth of 5.2 m (Elev. 204.1 m) upon completion of drilling.

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

wsp

PROJECT 21490972 **RECORD OF BOREHOLE No. RW-2** Sheet 1 of 1 **METRIC**

G.W.P. 2836-02-00 LOCATION N 4852987.1; E 301193.1 NAD83 / MTM Zone 10 (LAT. 43.816841; LONG. -79.544837) ORIGINATED BY T.T.

DIST CENTRAL HWY 400 BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger COMPILED BY P.T.

DATUM Geodetic Surface Elevation:209.5 m DATE Jul 11, 2023 CHECKED BY M.H.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL	REMARKS
209.5												
0.0	ASPHALT											
209.3 / 0.2	SAND (SW), and gravel, some silt (FILL) Compact to dense Brown to grey Moist	1	SS	37		209	o	36	51	11	2	
		2A	SS	20								
208.3 / 1.2	CLAYEY SILT - SILT (CL-ML), trace sand, trace gravel (FILL) Stiff Dark Grey Moist 2.0 - 2.1 m depth: organic sand pocket encountered . (Elev. 207.5 to 207.0 m)	2B				208						
		3A	SS	14			o					
		3B										
207.3 / 2.2	CLAYEY SILT (CL) to CLAYEY SILT - SILT (CL-ML), some sand to sandy (TILL) Firm to hard Brown; becoming grey at about 5.6 m depth (Elev. 203.9 m) Moist	4	SS	5		207	├─┤	0	11	65	24	
		5	SS	5		206	o					
		6	SS	11		205						
		7	SS	5	▼		├┤	1	22	56	21	
		8	SS	36		204 / 203	o					
202.8 / 6.7	End of Borehole											

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100. Shear Strength (kPa): X Field Vane; ⊕ Remoulded; ■ Pocket Pen; ● Quick Triaxial; ○ Unconfined. Water Content (%): PL (W_p), NMC (W), LL (W_l), NP Nonplastic; 20 40 60. Unit Weight γ kN/m³.

NOTES:

1. Borehole caved to a depth of 5.9 m (Elev. 203.6 m) upon completion of drilling.
2. Water measured inside borehole at a depth of 5.2 m (Elev. 204.3 m) upon completion of drilling.

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No. RW-3

Sheet 1 of 1

METRIC

PROJECT 21490972			
G.W.P. 2836-02-00	LOCATION N 4853039; E 301193.6 NAD83 / MTM Zone 10 (LAT. 43.817308; LONG. -79.544832)	ORIGINATED BY	T.T.
DIST CENTRAL HWY 400	BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger	COMPILED BY	P.T.
DATUM Geodetic Surface Elevation:209.6 m	DATE Jul 11, 2023	CHECKED BY	M.H.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	GR SA SI CL
209.6 / 0.0	ASPHALT					
209.4 / 0.2	SAND (SW) and gravel, some silt (FILL) Dense Brown to grey Moist	1	SS	34		
		2A / 2B	SS	22		
208.6 / 1.1	CLAYEY SAND (SC) with gravel (FILL) Loose to compact Dark Grey Moist 0.8-1.1 m depth : organic pocket encountered (Elev. 208.8 m to 208.5 m)	3	SS	6		18 35 34 13
207.4 / 2.2	CLAYEY SILT (CL) to SILTY CLAY (CI), some sand, contains oxidation stains (TILL) Firm to stiff Light brown to brown; becoming grey at about 4.4 m depth (Elev. 205.2 m) Moist	4	SS	8		
		5	SS	12		
		6	SS	14		
		7	SS	6		
204.2 / 5.4	Sandy SILT (ML), some clay, trace gravel Compact Dark grey Moist				▼ (204.2)	
		8	SS	20		5 28 55 12
202.9 / 6.7	End of Borehole					

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; ELEVATION SCALE (209, 208, 207, 206, 205, 204, 203); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) ○ Unconfined ● Quick Triaxial ■ Pocket Pen ⊕ Remoulded × Field Vane; WATER CONTENT (%) PL W_p NMC W LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; REMARKS

NOTES:

1. Borehole open upon completion of drilling.
2. Water measured inside open borehole at a depth of 5.4 m (Elev. 204.2 m) upon completion of drilling.

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

wsp

PROJECT 21490972

RECORD OF BOREHOLE No. RW-4

Sheet 1 of 1

METRIC

G.W.P. 2836-02-00 | LOCATION N 4853093.1; E 301192.4 NAD83 / MTM Zone 10 (LAT. 43.817795; LONG. -79.544847) | ORIGINATED BY T.T.

DIST CENTRAL HWY 400 | BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger | COMPILED BY P.T.

DATUM Geodetic Surface Elevation:209.8 m | DATE Jul 11, 2023 | CHECKED BY M.H.

ELEV. / DEPTH (209.8)	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
0.0	ASPHALT									
209.6 / 0.2	Gravelly SAND (SW), some silt (FILL) Compact Brown Moist	1	SS	25						
		2	SS	15	209	○	39	54	6	1
208.3 / 1.4	Sandy CLAYEY SILT (CL), trace gravel (FILL) Soft to Firm Brown and grey Moist	3	SS	4	208	○				
207.6 / 2.2	CLAYEY SILT - SILT (CL - ML) and sand to sandy, contains oxidation stains (TILL) Stiff to very stiff Brown; becoming grey at about 3.7 m depth (Elev. 206.1 m) Moist	4	SS	10	207	H	4	38	44	14
		5	SS	13						
		6	SS	25	206	○				
		7	SS	20	205	H	9	32	44	15
					204					
		8	SS	15		○				
203.1 / 6.7	End of Borehole									

NOTES:

1. Borehole caved to a depth of 1.1 m (Elev. 208.7 m) upon completion of drilling and water level could not be recorded.

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES (NUMBER, TYPE, "N" VALUES); GROUNDWATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%) PL W_p, NMC W, LL W_l, NP Nonplastic, 20 40 60; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No. RW-5

Sheet 1 of 1 — METRIC

PROJECT 21490972
G.W.P. 2836-02-00
LOCATION N 4853141.4; E 301188.5 NAD83 / MTM Zone 10 (LAT. 43.81823; LONG. -79.544895)
ORIGINATED BY T.T.
DIST CENTRAL HWY 400
BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger
COMPILED BY P.T.
DATUM Geodetic Surface Elevation:210.1 m
DATE Jul 11, 2023
CHECKED BY M.H.

ELEV. / DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
210.1 / 0.0	ASPHALT							
209.9 / 0.2	Gravelly SAND (SW), some silt (FILL) Compact Brown Moist	1	SS	23				
209.3 / 0.8	SILTY CLAY (CI), some sand (FILL) Firm to stiff Light brown to brown and grey mottled Moist	2	SS	5				
		3	SS	9	0	20	32	48
207.9 / 2.2	CLAYEY SILT (CL) to CLAYEY SILT - SILT (CL-ML), some sand to sandy, trace gravel, contains oxidation stains to about 2.2 m depth (Elev. 207.9 m), (TILL) Stiff to very stiff Brown; becoming grey at about 3.7 m depth (Elev. 206.4 m) Moist	4	SS	10				
		5	SS	11				
		6	SS	15	1	15	63	21
		7	SS	16				
		8	SS	13				
203.4 / 6.7	End of Borehole							

NOTES:

1. Borehole open upon completion of drilling.
2. Water measured inside open borehole at a depth of 5.6 m (Elev. 204.5 m) upon completion of drilling.

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUNDWATER CONDITIONS; ELEVATION SCALE (210, 209, 208, 207, 206, 205, 204); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): × Field Vane, ⊕ Remoulded, ■ Pocket Pen, ● Quick Triaxial, ○ Unconfined; WATER CONTENT (%) PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

wsp

PROJECT 21490972 **RECORD OF BOREHOLE No. RW-6** Sheet 1 of 1 **METRIC**

G.W.P. 2836-02-00 LOCATION N 4854121.2; E 301292.6 NAD83 / MTM Zone 10 (LAT. 43.827049; LONG. -79.543608) ORIGINATED BY T.T.

DIST CENTRAL HWY 400 BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger COMPILED BY P.T.

DATUM Geodetic Surface Elevation:222.8 m DATE Jul 11, 2023 CHECKED BY M.H.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
222.8											
0.0	ASPHALT										
222.7 / 0.1	Gravelly SAND (SW) Compact Brown Moist	1	SS	18			○	16	73	10	1
222.0 / 0.7	CLAYEY SILT (CL), some sand, trace gravel (FILL), contains shale fragments Stiff Brown Moist	2	SS	9		222	○				
221.3 / 1.4	SILT (ML), trace to some sand, trace gravel, trace clay, contains oxidation stains Compact to dense Brown Moist; becoming wet at about 3.7 m depth (Elev. 219.1 m)	3	SS	14		221					
		4	SS	36		220	NP	0	3	94	3
		5	SS	32							
		6	SS	21		219	○				
		7	SS	20	▼	218	▮	2	18	76	4
217.0 / 5.8	CLAYEY SILT (CL), some sand, trace gravel (TILL) Hard Grey Moist	8	SS	33		217	○				
216.1 / 6.7	End of Borehole										

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%): PL W_p, NMC W, LL W_l, 20 40 60; NP Nonplastic; UNIT WEIGHT γ kN/m³; REMARKS

NOTES:

1. Borehole caved to a depth of 5.4 m (Elev. 217.4 m) upon completion of drilling.
2. Water measured inside borehole at a depth of 4.9 m (Elev. 217.9 m) upon completion of drilling.

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

wsp

PROJECT 21490972	**RECORD OF BOREHOLE No. RW-7**	Sheet 1 of 1	**METRIC**
G.W.P. 2836-02-00	LOCATION N 4854164.6; E 301345.4 NAD83 / MTM Zone 10 (LAT. 43.82744; LONG. -79.542952)		ORIGINATED BY T.T.
DIST CENTRAL HWY 400	BOREHOLE TYPE 168 mm O.D. Hollow Stem Auger		COMPILED BY P.T.
DATUM Geodetic Surface Elevation:222.9 m	DATE Jul 11, 2023		CHECKED BY M.H.

ELEV. / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUNDWATER CONDITIONS	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
222.9 / 0.0	ASPHALT											
222.8 / 0.2	SAND (SP), some gravel (FILL) Dark brown Moist											
222.0 / 0.9	CLAYEY SILT (CL) to CLAYEY SILT - SILT (CL-ML) some sand (FILL), contains oxidation stains Stiff Light brown Moist	1A										
		1B	SS	10								
		2	SS	13								
220.7 / 2.2	CLAYEY SILT-SILT (CL-ML), some sand, contains oxidation stains (TILL) Very stiff Light brown Moist	3	SS	25				0	14	68	18	
220.0 / 3.0	SILT (ML), trace sand, trace clay Compact to dense Brown Wet	4	SS	30								
		5	SS	27								
		6	SS	30	▼	NP		0	5	92	3	
		7A	SS	31								
216.4 / 6.6	CLAYEY SILT (CL), some sand, trace gravel (TILL) Grey Moist	7B										
216.2 / 6.7	End of Borehole											

Elevation scale: 222, 221, 220, 219, 218, 217

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, × Field Vane

WATER CONTENT (%): PL W_p, NMC W, LL W_l; 20 40 60; NP Nonplastic

NOTES:

1. Borehole caved to a depth of 5.1 m (Elev. 217.8 m) upon completion of drilling.
2. Water measured inside borehole at a depth of 4.6 m (Elev. 218.3 m) upon completion of drilling.

+³, x³ : Numbers refer to Sensitivity o³% STRAIN AT FAILURE

Ontario

RECORD OF BOREHOLE No. RW-MW

Sheet 1 of 1

METRIC

PROJECT	21490972		
G.W.P.	2836-02-00	LOCATION	N 4854142.7; E 301320.2 NAD83 / MTM Zone 10 (LAT. 43.827252; LONG. -79.543268)
DIST	CENTRAL HWY 400	BOREHOLE TYPE	168 mm O.D. Hollow Stem Auger
DATUM	Geodetic Surface Elevation:222.8 m	DATE	Jul 27, 2023 - Jul 28, 2023

ORIGINATED BY	S.A.
COMPILED BY	S.A.
CHECKED BY	M.H.

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION
222.8	
0.0	ASPHALT (150 mm)
222.7 / 0.2	SAND (SP) and gravel (FILL) Brown Moist
222.0 / 0.9	CLAYEY SILT (CL) to CLAYEY SILT-SILT (CL-ML) FILL Stiff Brown Moist
221.0 / 1.9	CLAYEY SILT-SILT (CL-ML), some sand Very stiff Brown Moist
220.4 / 2.5	SILT (ML), trace to some sand, trace gravel Compact to dense Brown Moist to wet
217.1 / 5.8	CLAYEY SILT (CL), trace to some sand, trace gravel (TILL) Hard Grey Moist
216.8 / 6.1	End of Borehole

Column headings: SOIL PROFILE (ELEV. / DEPTH, DESCRIPTION, STRATA PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUNDWATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ Unconfined, ● Quick Triaxial, ■ Pocket Pen, ⊕ Remoulded, X Field Vane; WATER CONTENT (%): PL W_p, NMC W, LL W_l (20 40 60), NP Nonplastic; UNIT WEIGHT γ kN/m³; GR SA SI CL; REMARKS

Elevation scale: 222, 221, 220, 219, 218, 217

NOTES:

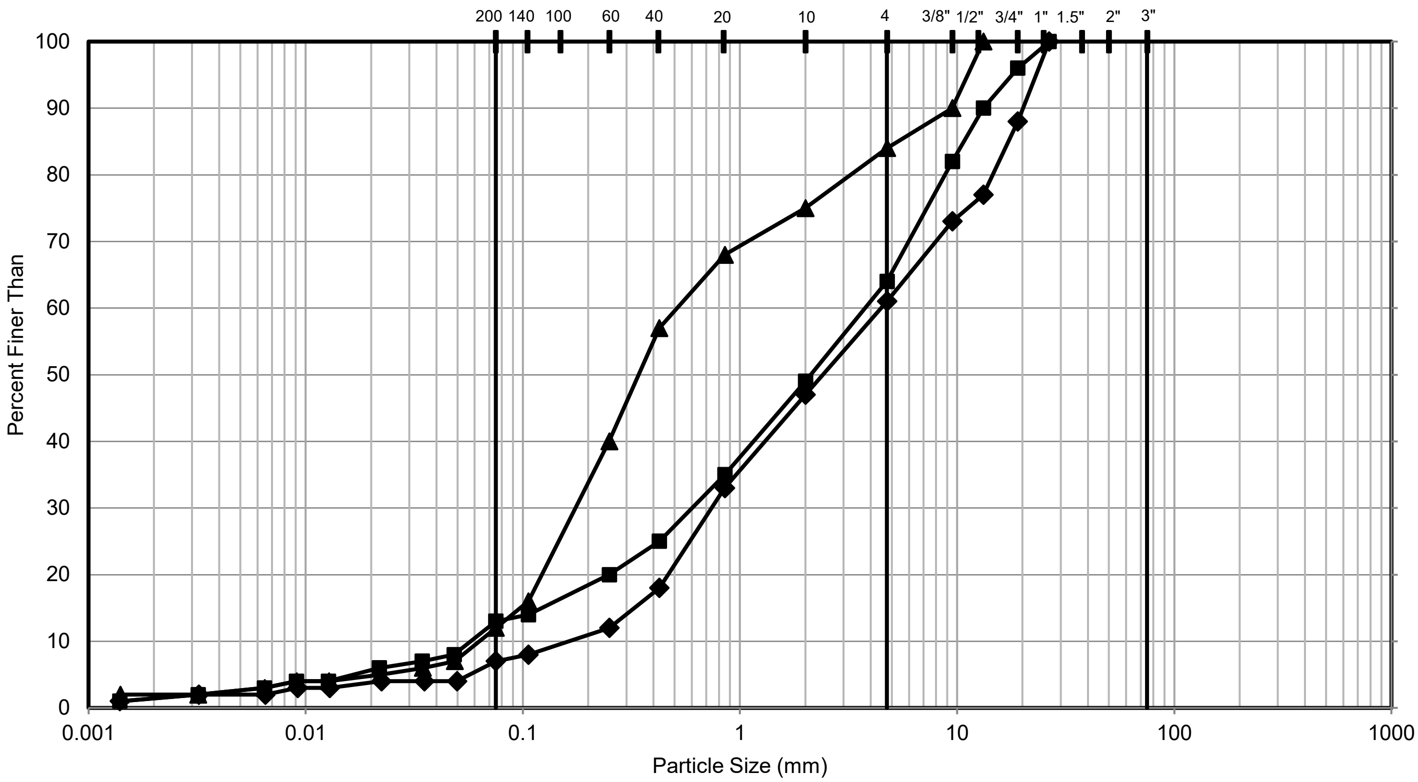
1. Borehole open and dry upon completion of drilling.
2. Soil Stratigraphy inferred from surrounding boreholes (RW-6 and RW-7).
3. Water level measured in standpipe piezometer at a depth of about 4.6 m below ground surface (Elev. 218.2 m) on October 31, 2023.

+³, x³ : Numbers refer to Sensitivity   o³% STRAIN AT FAILURE

APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	RW-2	1	0.2 - 0.8	209.3 to 208.7
◆	RW-4	2	0.8 - 1.4	209.0 to 208.4
▲	RW-6	1	0.1 - 0.7	222.7 to 222.1

CLIENT

PARSONS / MTO

PROJECT

RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

CONSULTANT

WSP GOLDER

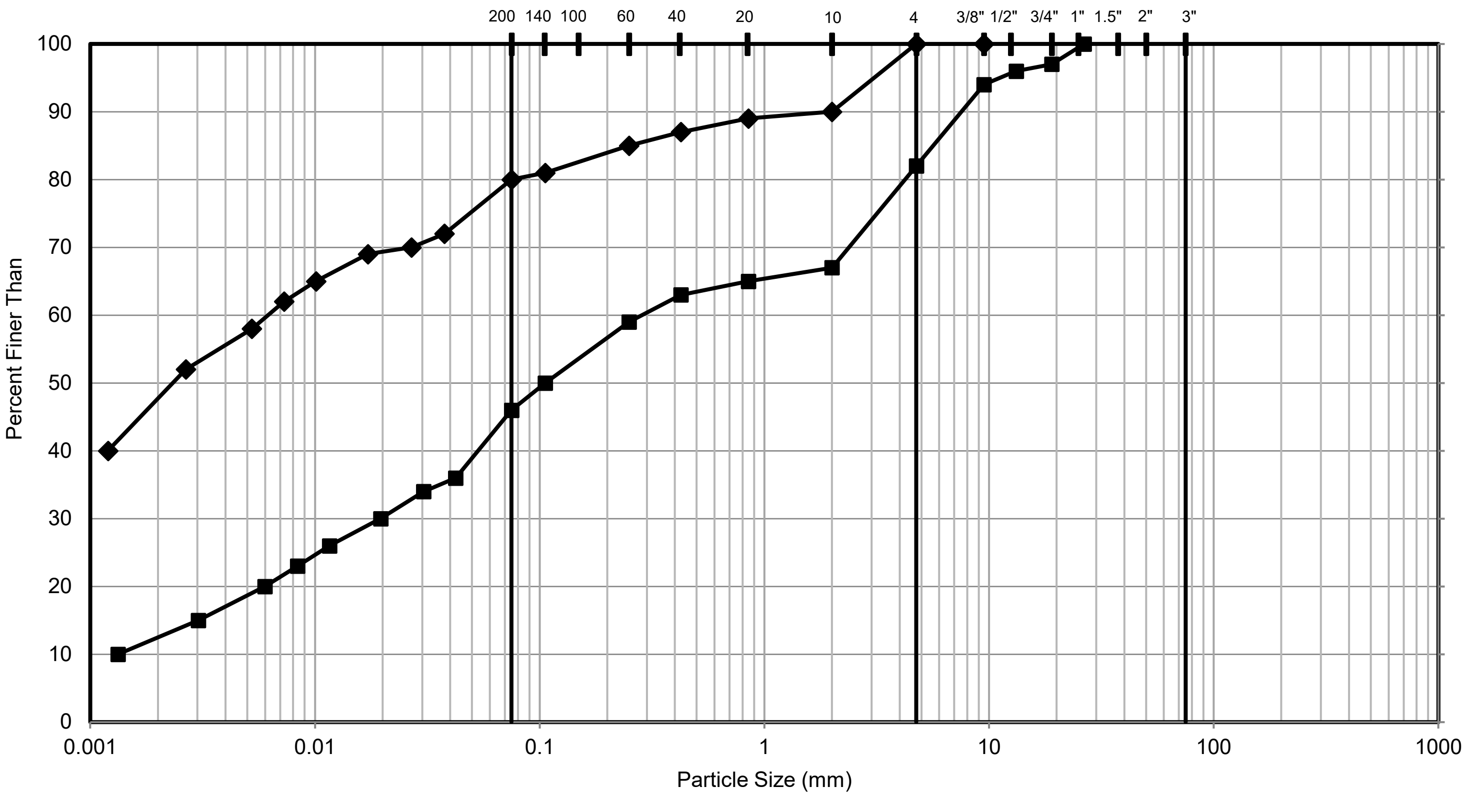
YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

TITLE

GRAIN SIZE DISTRIBUTION
SAND (SW) and gravel to gravelly (FILL)

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B1

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	RW-3	3	1.5 - 2.1	208.1 to 207.5
◆	RW-5	3	1.5 - 2.1	208.6 to 208.0

CLIENT

PARSONS / MTO

PROJECT

RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

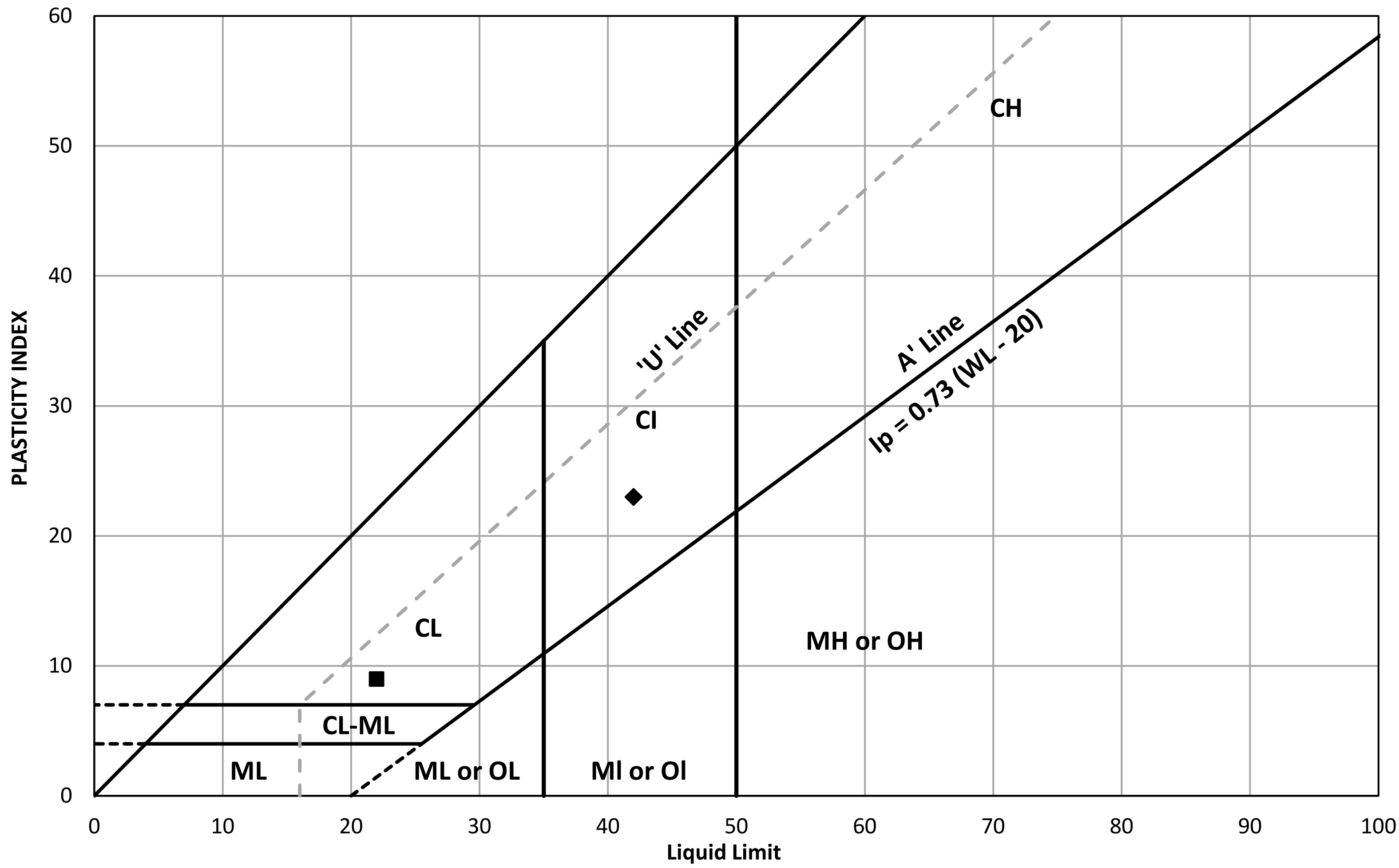
TITLE

GRAIN SIZE DISTRIBUTION
Sandy CLAYEY SILT (CL) to SILTY CLAY (CI) (FILL)

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B2

PATH: C:\Users\gld_MHenderson\AppData\Roaming\Bentley\OpenGround\Excel Extension\Temporary | FILE NAME: Laboratory Particle Size Distribution MTO.xlsm

PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Elevation (m)	Liquid Limit	Plastic Limit	Plasticity Index		
■	RW-3	3	208.10 to 207.49	22	13	9		
◆	RW-5	3	208.57 to 207.96	42	19	23		

CLIENT
PARSONS / MTO

PROJECT
RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

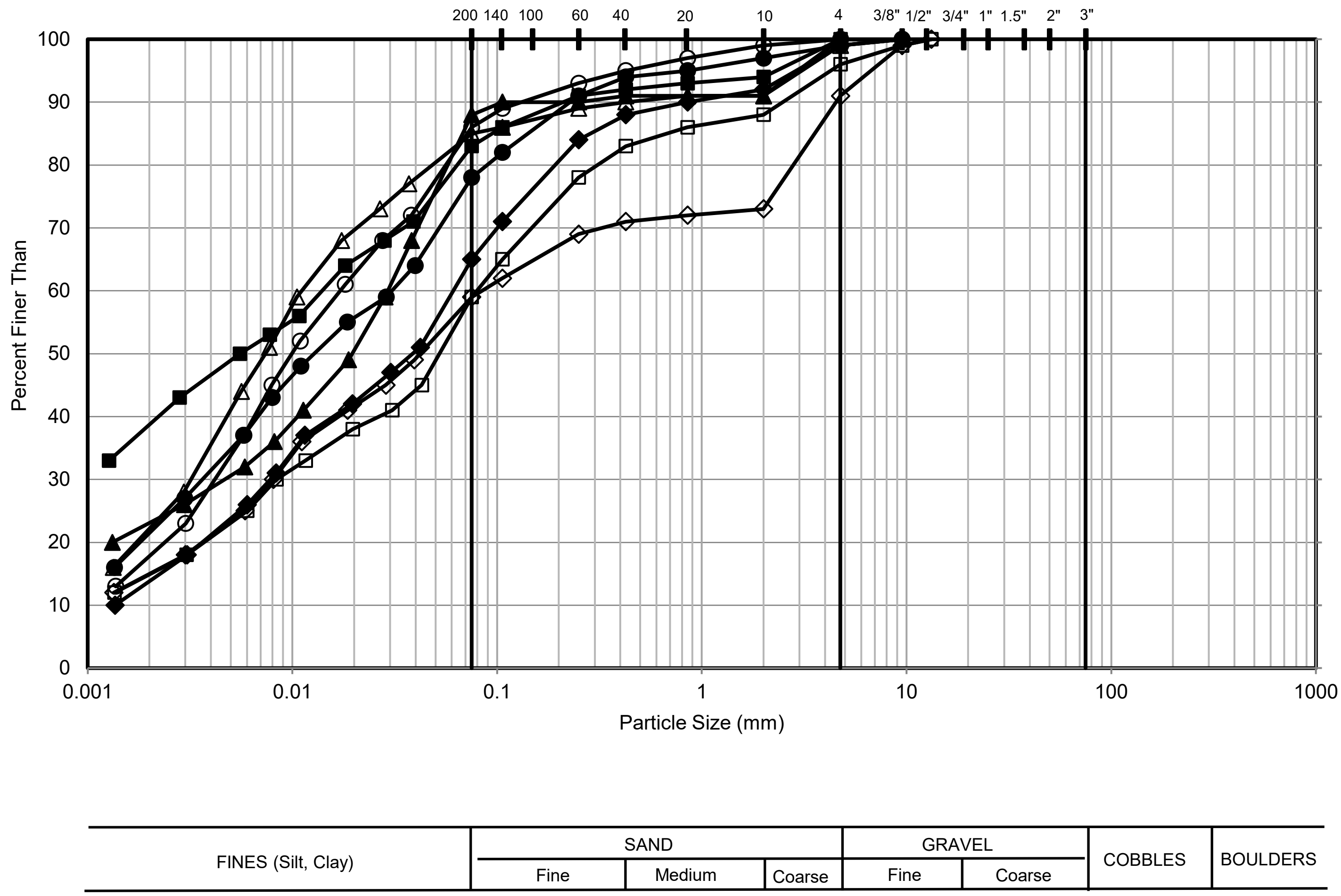
CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

TITLE
PLASTICITY CHART
Sandy CLAYEY SILT (CL) to SILTY CLAY (CI) (FILL)

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B3

GRAIN SIZE DISTRIBUTION



Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	RW-1	3	2.3 - 2.9	207.1 to 206.4
◆	RW-1	6	4.6 - 5.2	204.8 to 204.2
▲	RW-2	4	2.3 - 2.9	207.3 to 206.6
●	RW-2	7	4.6 - 5.2	205.0 to 204.4
□	RW-4	4	2.3 - 2.9	207.5 to 206.9
◇	RW-4	7	4.6 - 5.2	205.2 to 204.6
△	RW-5	6	3.8 - 4.4	206.3 to 205.7
○	RW-7	3	2.3 - 2.9	220.7 to 220.0

CLIENT

PARSONS / MTO

PROJECT

RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

CONSULTANT

WSP GOLDER

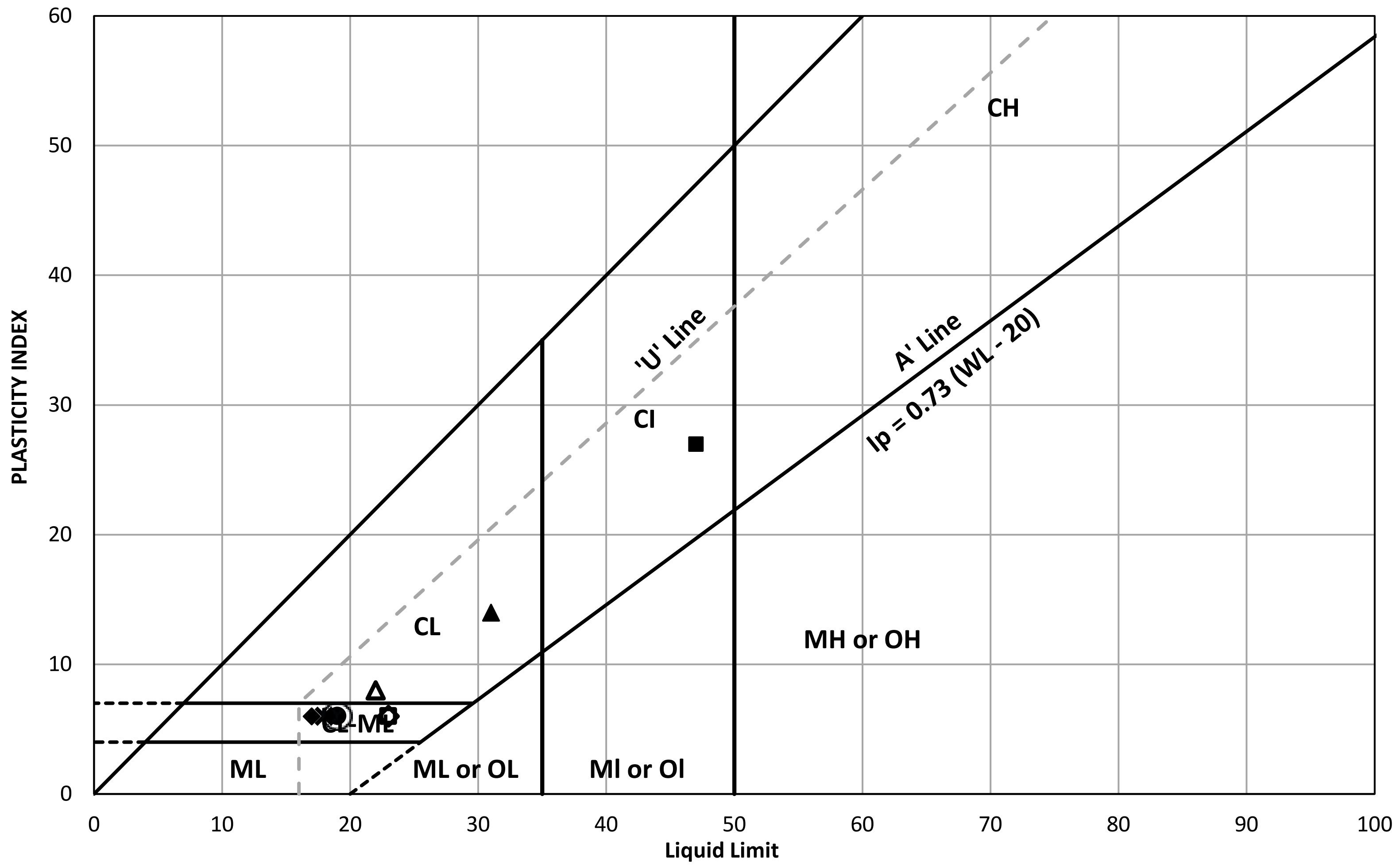
YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

TITLE

GRAIN SIZE DISTRIBUTION
CLAYEY SILT-SILT (CL-ML) to SILTY CLAY (CI) (TILL)

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B4

PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Elevation (m)	Liquid Limit	Plastic Limit	Plasticity Index		
■	RW-1	3	207.05 to 206.44	47	20	27		
◆	RW-1	6	204.76 to 204.16	17	11	6		
▲	RW-2	4	207.26 to 206.65	31	17	14		
●	RW-2	7	204.98 to 204.37	19	13	6		
∗	RW-4	4	207.47 to 206.86	18	12	6		
⊗	RW-4	7	205.19 to 204.58	19	13	6		
□	RW-5	6	206.28 to 205.67	23	17	6		
◇	RW-7	3	220.66 to 220.04	23	17	6		
△	RW-7	7B	216.39 to 216.23	8	22	14		

CLIENT
PARSONS / MTO

CONSULTANT
WSP GOLDER

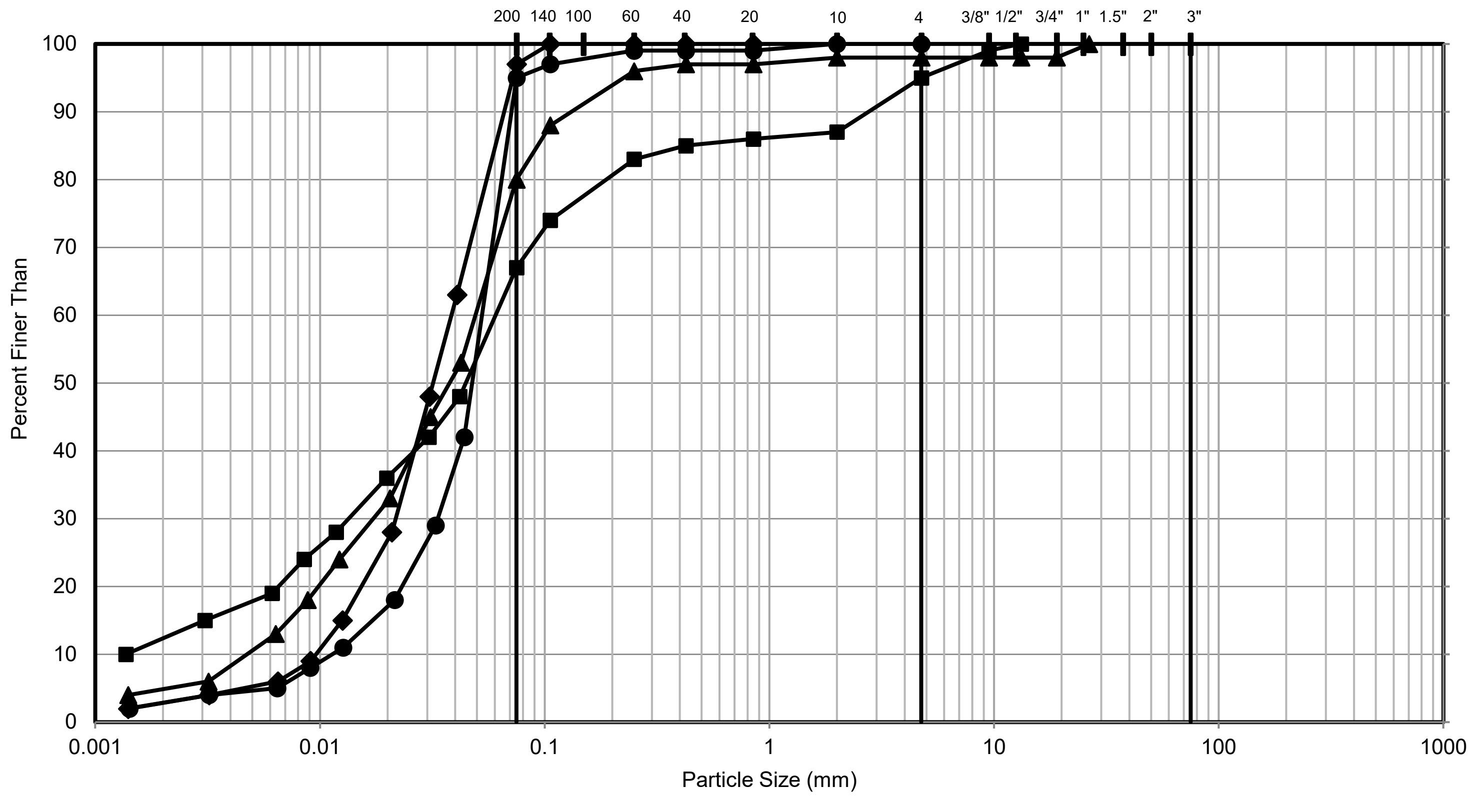
YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

PROJECT
RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

TITLE
PLASTICITY CHART
CLAYEY SILT-SILT (CL-ML) to SILTY CLAY (CI) (TILL)

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B5

GRAIN SIZE DISTRIBUTION



FINES (Silt, Clay)	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

Symbol	Sample Location	Sample Number	Depth (m)	Elevation (m)
■	RW-3	8	6.1 - 6.7	203.5 to 202.9
◆	RW-6	4	2.3 - 2.9	220.5 to 219.9
▲	RW-6	7	4.6 - 5.2	218.2 to 217.6
●	RW-7	6	4.6 - 5.2	218.4 to 217.8

CLIENT

PARSONS / MTO

PROJECT

RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

CONSULTANT

WSP GOLDER

YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

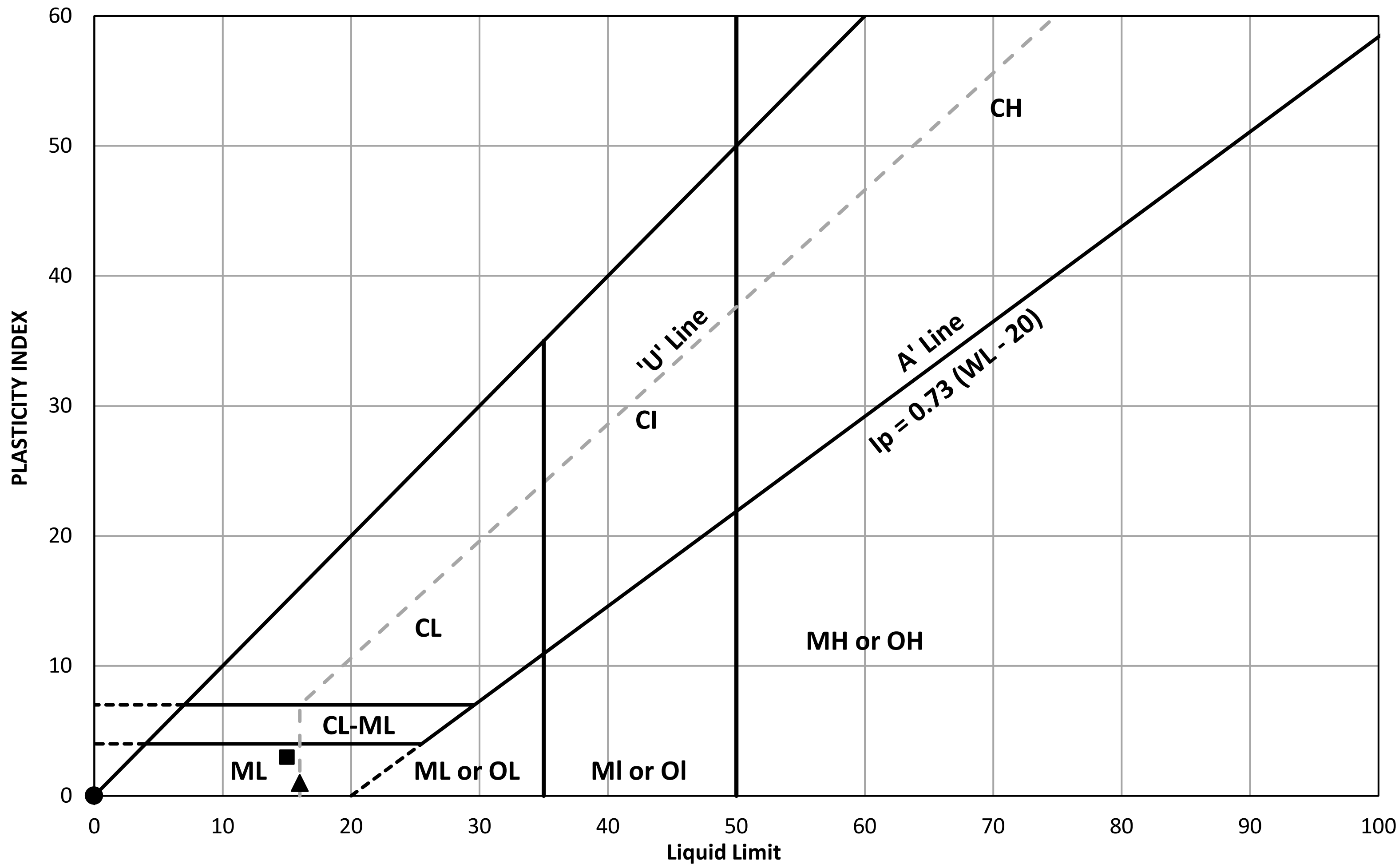
TITLE

GRAIN SIZE DISTRIBUTION
SILT (ML), trace sand to sandy

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B6

PATH: C:\Users\gld_MHenderson\AppData\Roaming\Bentley\OpenGround\Excel Extension\Temporary | FILE NAME: Laboratory Particle Size Distribution MTO.xlsm

PLASTICITY CHART



	Sample Location	Sample / Specimen Number	Elevation (m)	Natural Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	
■	RW-3	8	203.53 to 202.91	11.1	15	12	3	
◆	RW-6	4	220.48 to 219.87	-		NP		
▲	RW-6	7	218.19 to 217.59	-	16	15	1	
●	RW-7	6	218.37 to 217.76	21.5		NP		

CLIENT
PARSONS / MTO

PROJECT
RETAINING WALLS
HIGHWAY 400 WIDENING
GWP 2836-02-00

CONSULTANT
WSP GOLDER

YYYY-MM-DD	2023-08-11
DESIGNED	TT
PREPARED	TT
REVIEWED	MH
APPROVED	LCC

TITLE
PLASTICITY CHART
SILT (ML), trace sand to sandy

PROJECT NO.	CONTROL	REV.	FIGURE
21490972	0	0	B7

APPENDIX C

Analytical Laboratory Test Results

Your Project #: 21490972 (1003.5)
Site Location: HWY 400 BTWN LANGSTAFF AND MAJOR MAC
Your C.O.C. #: 947287-01-01

Attention: Maor Levy

WSP Canada Inc.
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2023/08/18
Report #: R7770150
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C3N8555
Received: 2023/08/08, 17:18

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Chloride (20:1 extract)	3	2023/08/14	2023/08/15	CAM SOP-00463	MOE E3013 m
Conductivity	3	2023/08/14	2023/08/14	CAM SOP-00414	OMOE E3530 v1 m
Moisture (Subcontracted) (1, 2)	3	N/A	2023/08/16	AB SOP-00002	CCME PHC-CWS m
Sulphide in Soil (1)	3	N/A	2023/08/15	AB SOP-00080	EPA9030B/SM4500S2-DF
pH CaCl2 EXTRACT	3	2023/08/15	2023/08/15	CAM SOP-00413	EPA 9045 D m
Redox Potential (3)	3	2023/08/16	2023/08/17	CAM SOP-00421	SM 2580 B
Resistivity of Soil	3	2023/08/09	2023/08/15	CAM SOP-00414	SM 23 2510 m
Sulphate (20:1 Extract)	3	2023/08/14	2023/08/15	CAM SOP-00464	MOE E3013 m

Remarks:

Bureau Veritas is accredited to ISO/IEC 17025 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Bureau Veritas are based upon recognized Provincial, Federal or US method compendia such as CCME, MELCCFP, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Bureau Veritas' profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Bureau Veritas in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported; unless indicated otherwise, associated sample data are not blank corrected. Where applicable, unless otherwise noted, Measurement Uncertainty has not been accounted for when stating conformity to the referenced standard.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested. When sampling is not conducted by Bureau Veritas, results relate to the supplied samples tested.

Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by Bureau Veritas Calgary (19th), 4000 19th Street NE , Calgary, AB, T2E 6P8
(2) Offsite analysis requires that subcontracted moisture be reported.

Your Project #: 21490972 (1003.5)
Site Location: HWY 400 BTWN LANGSTAFF AND MAJOR MAC
Your C.O.C. #: 947287-01-01

Attention: Maor Levy

WSP Canada Inc.
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2023/08/18
Report #: R7770150
Version: 1 - Final

CERTIFICATE OF ANALYSIS

BUREAU VERITAS JOB #: C3N8555
Received: 2023/08/08, 17:18

(3) Oxidation-Reduction Potential (ORP) values are determined using a Ag/AgCl reference electrode. The test is therefore, not SCC accredited for this matrix.

Encryption Key

Please direct all questions regarding this Certificate of Analysis to:
Ankita Bhalla, Project Manager
Email: Ankita.Bhalla@bureauveritas.com
Phone# (905) 817-5700
==
This report has been generated and distributed using a secure automated process.
Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by Rodney Major, General Manager responsible for Ontario Environmental laboratory operations.

Bureau Veritas 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.bvna.com

SOIL CORROSIVITY PACKAGE (SOIL)

Bureau Veritas ID		WQA768			WQA768			WQA769		
Sampling Date		2023/07/12			2023/07/12			2023/07/13		
COC Number		947287-01-01			947287-01-01			947287-01-01		
	UNITS	**RW-2 SS5**	**RDL**	**QC Batch**	**RW-2 SS5 Lab-Dup**	**RDL**	**QC Batch**	**RW-4 SS3**	**RDL**	**QC Batch**
Calculated Parameters										
Resistivity	ohm-cm	360		8842215				620		8842215
CONVENTIONALS										
Redox Potential	mV	310	N/A	8855362				300	N/A	8855362
Inorganics										
Soluble (20:1) Chloride (Cl-)	ug/g	1400	100	8850602	1500	100	8850602	740	20	8850602
Conductivity	umho/cm	2750	2	8851156				1610	2	8851156
Available (CaCl2) pH	pH	7.88		8852867				7.72		8852867
Soluble (20:1) Sulphate (SO4)	ug/g	220	20	8850609	230	20	8850609	260	20	8850609
Sulphide	mg/kg	3.9 (1)	0.5	8857768				1.2 (1)	0.5	8857768

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
N/A = Not Applicable
(1) Extracted past method specified hold time

Bureau Veritas ID		WQA769			WQA770		
Sampling Date		2023/07/13			2023/07/13		
COC Number		947287-01-01			947287-01-01		
	UNITS	**RW-4 SS3 Lab-Dup**	**RDL**	**QC Batch**	**RW-6 SS3**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm				640		8842215
CONVENTIONALS							
Redox Potential	mV				260	N/A	8855362
Inorganics							
Soluble (20:1) Chloride (Cl-)	ug/g				750	20	8850602
Conductivity	umho/cm	1500	2	8851156	1550	2	8851156
Available (CaCl2) pH	pH				7.99		8852867
Soluble (20:1) Sulphate (SO4)	ug/g				52	20	8850609
Sulphide	mg/kg				1.7 (1)	0.5	8857768

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate
N/A = Not Applicable
(1) Extracted past method specified hold time

RESULTS OF ANALYSES OF SOIL

Bureau Veritas ID		WQA768	WQA769	WQA770		
Sampling Date		2023/07/12	2023/07/13	2023/07/13		
COC Number		947287-01-01	947287-01-01	947287-01-01		
	UNITS	**RW-2 SS5**	**RW-4 SS3**	**RW-6 SS3**	**RDL**	**QC Batch**
Physical Testing						
Moisture-Subcontracted	%	18	13	14	0.30	8857791

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Bureau Veritas Job #: C3N8555
Report Date: 2023/08/18

WSP Canada Inc.
Client Project #: 21490972 (1003.5)
Site Location: HWY 400 BTWN LANGSTAFF AND MAJOR MAC
Sampler Initials: ML

TEST SUMMARY

Bureau Veritas ID: WQA768
Sample ID: RW-2 SS5
Matrix: Soil

Collected: 2023/07/12
Shipped:
Received: 2023/08/08

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8850602	2023/08/14	2023/08/15	Alina Dobreanu
Conductivity	AT	8851156	2023/08/14	2023/08/14	Gurparteek KAUR
Moisture (Subcontracted)	BAL	8857791	N/A	2023/08/16	Margarita Aguilera
Sulphide in Soil	SPEC	8857768	N/A	2023/08/15	Ly Vu
pH CaCl2 EXTRACT	AT	8852867	2023/08/15	2023/08/15	Surinder Rai
Redox Potential	COND	8855362	2023/08/16	2023/08/17	Gurparteek KAUR
Resistivity of Soil		8842215	2023/08/15	2023/08/15	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8850609	2023/08/14	2023/08/15	Alina Dobreanu

Bureau Veritas ID: WQA768 Dup
Sample ID: RW-2 SS5
Matrix: Soil

Collected: 2023/07/12
Shipped:
Received: 2023/08/08

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8850602	2023/08/14	2023/08/15	Alina Dobreanu
Sulphate (20:1 Extract)	KONE/EC	8850609	2023/08/14	2023/08/15	Alina Dobreanu

Bureau Veritas ID: WQA769
Sample ID: RW-4 SS3
Matrix: Soil

Collected: 2023/07/13
Shipped:
Received: 2023/08/08

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8850602	2023/08/14	2023/08/15	Alina Dobreanu
Conductivity	AT	8851156	2023/08/14	2023/08/14	Gurparteek KAUR
Moisture (Subcontracted)	BAL	8857791	N/A	2023/08/16	Margarita Aguilera
Sulphide in Soil	SPEC	8857768	N/A	2023/08/15	Ly Vu
pH CaCl2 EXTRACT	AT	8852867	2023/08/15	2023/08/15	Surinder Rai
Redox Potential	COND	8855362	2023/08/16	2023/08/17	Gurparteek KAUR
Resistivity of Soil		8842215	2023/08/15	2023/08/15	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8850609	2023/08/14	2023/08/15	Alina Dobreanu

Bureau Veritas ID: WQA769 Dup
Sample ID: RW-4 SS3
Matrix: Soil

Collected: 2023/07/13
Shipped:
Received: 2023/08/08

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	8851156	2023/08/14	2023/08/14	Gurparteek KAUR

Bureau Veritas ID: WQA770
Sample ID: RW-6 SS3
Matrix: Soil

Collected: 2023/07/13
Shipped:
Received: 2023/08/08

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	8850602	2023/08/14	2023/08/15	Alina Dobreanu
Conductivity	AT	8851156	2023/08/14	2023/08/14	Gurparteek KAUR

TEST SUMMARY

Bureau Veritas ID: WQA770
Sample ID: RW-6 SS3
Matrix: Soil

Collected: 2023/07/13
Shipped:
Received: 2023/08/08

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture (Subcontracted)	BAL	8857791	N/A	2023/08/16	Margarita Aguilera
Sulphide in Soil	SPEC	8857768	N/A	2023/08/15	Ly Vu
pH CaCl2 EXTRACT	AT	8852867	2023/08/15	2023/08/15	Surinder Rai
Redox Potential	COND	8855362	2023/08/16	2023/08/17	Gurparteek KAUR
Resistivity of Soil		8842215	2023/08/15	2023/08/15	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	8850609	2023/08/14	2023/08/15	Alina Dobreanu

GENERAL COMMENTS

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

Package 1	5.0°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8850602	Soluble (20:1) Chloride (Cl-)	2023/08/15	NC	70 - 130	96	70 - 130	<20	ug/g	2.6	35
8850609	Soluble (20:1) Sulphate (SO4)	2023/08/15	NC	70 - 130	100	70 - 130	<20	ug/g	1.2	35
8851156	Conductivity	2023/08/14			103	90 - 110	<2	umho/cm	7.3	10
8852867	Available (CaCl2) pH	2023/08/15			100	97 - 103			0.31	N/A
8855362	Redox Potential	2023/08/17			101	95 - 105			6.8	35
8857768	Sulphide	2023/08/15	27 (1)	75 - 125	87	75 - 125	<0.5	mg/kg	NC	30
8857791	Moisture-Subcontracted	2023/08/16					<0.30	%		

N/A = Not Applicable

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

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

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

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

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

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by:

Brad Newman, B.Sc., C.Chem., Scientific Service Specialist

Cristina Carriere, Senior Scientific Specialist

Veronica Falk, B.Sc., P.Chem., QP, Scientific Specialist, Organics

Suwan (Sze Yeung) Fock, B.Sc., Scientific Specialist

Bureau Veritas has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per ISO/IEC 17025, signing the reports. For Service Group specific validation, please refer to the Validation Signatures page if included, otherwise available by request. For Department specific Analyst/Supervisor validation names, please refer to the Test Summary section if included, otherwise available by request. This report is authorized by {0}, {1} responsible for {2} {3} laboratory operations.

Bureau Veritas
6740 Campobello Road, Mississauga, Ontario Canada L5N 2L8 Tel:(905) 817-5700 Toll-free:800-563-6266 Fax:(905) 817-5777 www.bvna.com

08-Aug-23 17:18
Ankita Bhalla
C3N8555
SPJ ENV-1085

Page 1 of 1

INVOICE TO:
Company Name: #1326 WSP Canada Inc.
Attention: Accounts Payable
Address: 6925 Century Ave Suite 100
Mississauga ON L5N 7K2
Tel: (905) 567-4444 Fax: (905) 567-6561
Email: CAPayablesInvoice@wsp.com

REPORT TO:
Company Name: WSP Canada Inc
Attention: Maor Levy
Address:
Tel: Fax:
Email: maor.levy@wsp.com Mark.henderson@WSP.com

PROJECT INFORMATION:
Quotation #: C31027
P.O. #:
Project: 21490972 (1003.5)
Project Name: Hwy 400 btwn Langstaff and Major Mac
Site #:
Sampled By: T.T.

Bottle Order #: 947287
COC #: C#947287-01-01
Project Manager: Ankita Bhalla

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

Regulation 153 (2011): Table 1, Table 2, Table 3, Table ___; Res/Park, Ind/Comm, Agri/Other; Medium/Fine, Coarse, For RSC

Other Regulations: CCME, Reg 558, MISA, PWQO, Other; Sanitary Sewer Bylaw, Storm Sewer Bylaw, Municipality ___, Reg 406 Table ___

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC): Field Filtered (please circle): Metals / Hg / Cr VI; Corrosivity Package

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects
Regular (Standard) TAT: ✓
(will be applied if Rush TAT is not specified):
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.
Job Specific Rush TAT (if applies to entire submission)
Date Required: ___ Time Required: ___
Rush Confirmation Number: ___ *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered	Corrosivity Package	# of Bottles	Comments
1		RW-2 SS5	2023/07/12	A.M.	Soil		✓	2	
2		RW-4 SS3	2023/07/13	A.M.	Soil		✓	2	
3		RW-6 SS3	2023/07/13	P.M.	Soil		✓	2	
4									
5									
6									
7									
8									
9									
10									

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
[signature] / Maor Levy	2023/08/08	17:15	[signature] / D. Zheng	23/08/08	17:18	

Laboratory Use Only
Time Sensitive
Temperature (°C) on Recei: 5/6/4
Custody Seal: Present / Intact — Yes / No
On ice

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO BUREAU VERITAS'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.BVNA.COM/ENVIRONMENTAL-LABORATORIES/RESOURCES/COC-TERMS-AND-CONDITIONS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT WWW.BVNA.COM/ENVIRONMENTAL-LABORATORIES/RESOURCES/CHAIN-CUSTODY-FORMS-COCS.

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

White: Bureau Veritas Yellow: Client

Bureau Veritas Canada (2019) Inc.

APPENDIX D

Non-Standard Special Provisions

<u>WORKING SLAB</u> - Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under concrete retaining wall foundations.

2.0 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 902 Excavating and Backfilling - Structures

3.0 DEFINITIONS - Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS - Not Used

5.0 MATERIALS

Concrete for working slabs shall have a minimum 28 day strength of 20 MPa.

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Excavation

Excavation for the working slab shall be according to OPSS 902.

7.02 Protection of Founding Soil

Unless the concrete footing is constructed immediately following inspection and approval of the prepared subgrade, a working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as specified in the Contract Documents.

7.03 Protection of Founding Bedrock

The surface of the footing founding rock shall be exposed, cleaned and any loose or fractured parts removed so that sound rock is exposed. The working slab shall be placed on the exposed cleaned sound founding rock surface as specified in the Contract Documents.

Thickness of the mass concrete pad shall depend on the slope and irregularities in the exposed founding rock surface. A nominal thickness and a footprint plan view area has been specified on the Contract Documents

7.04 Dewatering

Dewatering shall be carried out according to OPSS 902.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Working Slab - 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.

DEWATERING SYSTEM - Item No.
TEMPORARY FLOW PASSAGE SYSTEM - Item No.

Special Provision No. 517F01 July 2017

Amendment to OPSS 517, November 2016

Design Storm Return Period and Preconstruction Survey Distance

517.01 SCOPE

Section 517.01 of OPSS 517 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the design, operation, and removal of a dewatering or temporary flow passage system or both to control water during construction, and the control of the water prior to discharge to the natural environment and sewer systems.

517.04 DESIGN AND SUBMISSION REQUIREMENTS

517.04.01 Design Requirements

Subsection 517.04.01 of OPSS 517 is amended by deleting the first paragraph in its entirety and replacing it with the following:

A dewatering or temporary flow passage system or both shall be designed to control water at the locations specified in the Contract Documents and at any other location where a system is necessary to complete the work. The design of the system shall be sufficient to permit the work at each location to be carried out as specified in the Contract Documents.

Subsection 517.04.01 of OPSS 517 is further amended by deleting the second last paragraph in its entirety and replacing it with the following:

Temporary flow passage systems shall be designed, as a minimum, for a 2 year design storm return period and groundwater discharge, except for the work specified in Table A. For the work specified in Table A, the temporary flow passage system shall be designed, as a minimum, for the design storm return period specified in Table A and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

Intensity-Duration Factor (IDF) curve location, site specific minimum return period, return period flow estimates, and other information is provided in Table A. The IDF information can be accessed through the MTO IDF Curve Look up Tool on the Drainage and Hydrology page of MTO's website. The return period flow estimates do not include flow volumes from groundwater discharge. The Owner specifically excludes these flow estimates from the warranty in the Reliance on Contract Documents subsection of OPSS 100, MTO General Conditions of Contract.

Table A

IDF Curve Location	Latitude: *			Longitude: *		
Temporary Flow Passage Systems						
Site Name / Station Reference	Minimum Return Period (Years)	Return Period Flow Estimates (m^3/s)				Design Engineer Requirements (Note 1)
		2 Year	5 Year	10 Year	25 Year	
**	***	****	****	****	****	*****
Dewatering Systems						
Site Name / Station Reference	Preconstruction Survey Distance (Note 2) (m)					Design Engineer Requirements (Note 1)
Bass Pro Mills Drive S-E/W Ramp Retaining Wall	N/A					No
Rutherford Road S-E/W Ramp Retaining Wall	N/A					No

Note:

1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. “N/A” indicates a preconstruction survey is not required.

NOTES TO DESIGNER:

Designer Fill-in for Table A:

* Enter the latitude and longitude co-ordinates of the IDF Curve as obtained using the MTO IDF Curve Look up Tool. Create additional tables, as necessary, if more than one (1) IDF curve was used on the contract (i.e. on a very long contract there may be two IDF curves used to better represent rainfall events for two (2) different sections of the contract).

** Fill-in site name, work, and station reference as appropriate for the dewatering system and/or temporary flow passage system item locations.

*** For temporary flow passage system item locations, fill-in the minimum design storm return period for the site based on MTO Drainage Design Standard TW-1.

**** For temporary flow passage system item locations, fill-in the design flow rate estimates for the various return periods.

***** Insert “Yes” when recommended by the Foundation Engineer. Insert “No” otherwise.

****** Fill-in the required distance for preconstruction survey if recommended by the Foundation Engineer. Fill-in “N/A” if not recommended.

Table A (Sample)

<table>
<tr><td colspan="2">IDF Curve Location</td><td colspan="4">Latitude: 44.974844</td><td colspan="2">Longitude: -79.769339</td></tr>
<tr><td colspan="8">Temporary Flow Passage Systems</td></tr>
<tr><td rowspan="2">Site Name /
Station Reference</td><td rowspan="2">Minimum
Return Period
(Years)</td><td colspan="4">Return Period Flow Estimates (m^3/s)</td><td rowspan="2" colspan="2">Design Engineer
Requirements
(Note 1)</td></tr>
<tr><td>2
Year</td><td>5
Year</td><td>10
Year</td><td>25
Year</td></tr>
<tr><td>Woods Creek Culvert
Rehabilitation</td><td>2</td><td>0.7</td><td>3.5</td><td>7.5</td><td>10.9</td><td colspan="2">N/A</td></tr>
<tr><td>Site 32-145
Robbs Creek Culvert Replacement</td><td>10</td><td>1.6</td><td>7.6</td><td>17.4</td><td>25.2</td><td colspan="2">Yes</td></tr>
<tr><td colspan="8">Dewatering Systems</td></tr>
<tr><td>Site Name /
Station Reference</td><td colspan="5">Preconstruction Survey Distance (Note 2)
(m)</td><td colspan="2">Design Engineer
Requirements
(Note 1)</td></tr>
<tr><td>Site 32-145
Robbs Creek Culvert Replacement</td><td colspan="5">300</td><td colspan="2">Yes</td></tr>
<tr><td colspan="8">Note:
1. “Yes” means the design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work. “No” means a minimum experience level is not required for the design Engineer and design-checking Engineer.
2. “N/A” indicates a preconstruction survey is not required.</td></tr>
</table>

WARRANT: Always with these tender items.

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