



Preliminary Foundation Investigation and Design Report

Highway 401 - Proposed Widening of Deep
Cut Adjacent to 164 Skyview Road, Grafton,
Ontario

Highway 401 Planning Study
Cobourg to Colborne, Ontario

Latitude 44.0120
Longitude - 77.9819
GWP 4060-11-00

Geocres No.: 31C-315

Prepared for:

Ministry of Transportation Ontario

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Table of Contents

1.0	INTRODUCTION	1
2.0	SITE DESCRIPTION	2
2.1	SITE LOCATION	2
2.2	SITE DESCRIPTION	2
2.2.1	Site Drainage	2
2.2.2	Geological Information	2
2.2.3	Available Subsurface Information	3
3.0	INVESTIGATION PROCEDURES	3
3.1	SITE RECONNAISSANCE	3
3.2	FIELD INVESTIGATION	6
3.3	LOCATION AND ELEVATION SURVEY	7
3.4	LABORATORY TESTING	7
4.0	SUBSURFACE CONDITIONS	8
4.1	OVERVIEW	8
4.2	OVERBURDEN	8
4.2.1	Topsoil	8
4.2.2	Fill	8
4.2.3	Sand	9
4.2.4	Silt/Sandy Silt	9
4.2.5	Clayey Silt	10
4.2.6	Lower Sand/Silty Sand	11
4.3	BEDROCK	11
4.4	GROUNDWATER	11
4.5	CHEMICAL TESTING	12
5.0	MISCELLANEOUS	12
6.0	CLOSURE	13
7.0	DISCUSSIONS AND ENGINEERING RECOMMENDATIONS	14
7.1	OVERVIEW	14
7.2	PROJECT DESCRIPTION AND BACKGROUND	14
7.2.1	Project Description	14
7.2.2	Proposed Works	15
7.3	DEGREE OF SITE AND PREDICTION MODEL UNDERSTANDING	16
7.4	GEOTECHNICAL DESIGN PARAMETERS	17
7.5	FROST PENETRATION	17
7.6	SEISMIC CONDITIONS	18
7.6.1	Site Class	18
7.6.2	Peak Ground Acceleration (PGA)	18
7.6.3	Liquefaction Potential	18



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

7.7	PRELIMINARY FOUNDATION ENGINEERING DESIGN INPUT	18
7.7.1	Slope and Retaining Wall Options	19
7.8	SLOPE STABILITY EVALUATION	21
7.8.1	Existing Slope Condition	21
7.8.2	Global Stability - Proposed Option 2 and 3 Retaining Wall Configurations	21
7.8.3	Foundation Design – Option 2 and 3 Retaining Wall Configurations.....	22
7.8.4	Geotechnical Horizontal Resistance (Sliding).....	24
7.9	LATERAL EARTH PRESSURES.....	24
7.9.1	Backfill	24
7.9.2	Static Lateral Earth Pressures.....	24
7.9.3	Seismic Lateral Earth Pressures	25
7.10	EROSION AND SCOUR PROTECTION DESIGN CONSIDERATIONS	27
7.11	CEMENT TYPE AND CORROSION PROTECTION.....	27
8.0	PRELIMINARY CONSTRUCTION CONSIDERATIONS	28
8.1	CONSTRUCTION STAGING.....	28
8.2	TEMPORARY PROTECTION SYSTEMS.....	28
8.3	EXCAVATION AND BACKFILLING.....	29
8.4	OBSTRUCTIONS.....	30
8.5	TEMPORARY GROUNDWATER CONTROL	30
9.0	FURTHER WORK FOR DETAILED DESIGN	31
10.0	SPECIFICATIONS	31
11.0	CLOSURE.....	32
12.0	REFERENCES.....	33

LIST OF TABLES

Table 3.1:	Borehole Information Summary	7
Table 3.2:	Laboratory Testing Program	7
Table 4.1:	Results of Chemical Analysis	12
Table 7.1:	Soil Profile and Design Parameters	17
Table 7.2:	Peak Ground Acceleration Data	18
Table 7.3:	Comparison of Slope and Retaining Wall Options	19
Table 7.4:	Geotechnical Vertical Resistance & Reaction – Shallow Foundations	23
Table 7.5:	Recommended Static Earth Pressure Parameters (Horizontal Backfill)	25
Table 7.6:	Recommended Static Earth Pressure Parameters (2H:1V Backfill)	25
Table 7.7:	Seismic Design Parameters to Estimate Lateral Earth Pressures.....	26
Table 7.8:	Recommended Seismic Earth Pressure Parameters (Horizontal Backfill).....	26
Table 7.9:	Recommended Seismic Earth Pressure Parameters (2H:1V Backfill).....	26
Table 8.1:	Comparison of Temporary Protection Systems.....	28
Table 10.1:	Specifications Referenced in Report.....	31



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

LIST OF APPENDICES

APPENDIX A.....	A.1
A.1 Drawing No. 1 – Borehole Locations and Soil Strata Drawing	A.1
APPENDIX B.....	B.1
B.1 Symbols and Terms Used on Stantec Borehole Records	B.1
B.2 Borehole Records.....	B.1
B.3 Available GEOCRETS Information	B.1
APPENDIX C.....	C.1
C.1 Laboratory Test Results	C.1
APPENDIX D.....	D.1
D.1 Geotechnical Model.....	D.1
D.2 Slope Stability Analyses	D.1
APPENDIX E.....	E.1
E.1 Soil Erodibility Memo	E.1
APPENDIX F	F.1
F.1 2015 National Building Code Seismic Hazard Calculation	F.1



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

PRELIMINARY FOUNDATION INVESTIGATION REPORT

For
G.W.P 4060-11-00

Highway 401 Planning Study from Cobourg to Colborne, Ontario
Proposed Widening of Deep Cut Adjacent to 164 Skyview Road

Grafton, Northumberland County, Ontario

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) has been retained by the Ministry of Transportation of Ontario (MTO) to undertake a planning, preliminary design, and Class Environmental Assessment (Class EA) study for the section of Highway 401 extending from approximately 2 km east of Nagle Road to Percy Street in Northumberland County, Ontario. The study includes the replacement or rehabilitation of structures, interchange modifications and future widening of the highway.

The foundation engineering services for the project include the preparation of foundation desktop studies, and preliminary foundation design reports at a series of bridge (overpass or underpass) and structural culvert sites where replacement or rehabilitation of the existing structure is planned. There are two sites within the study limits where it has been identified there may be insufficient space to develop new cut slopes for the future widened highway without impacting development on the adjacent properties. Foundation investigations are required in order to assess the stability of, and requirements for, deep cuts and/or retaining wall options at these sites.

The property at 164 Skyview Road was identified by the Ministry as one site where options to minimize impacts on the property would be reviewed because the conventional highway widening grading would impact a recently constructed garage structure on the property. The proposed crest of the cut slope encroaches onto the property approximately 23.4 m from the existing ROW. Through discussions with the owners, it was established that property acquisition may be possible in areas away from the garage as long as access in and out of the garage is maintained.

This report presents the results of a foundation investigation related to the widening of the existing highway cut slope on the south side of Highway 401, east of Shelter Valley Creek, and adjacent to the property at 164 Skyview Road where new retaining wall(s) are being considered in order to minimize impacts on the existing buildings/facilities present within that property. A separate Preliminary Foundation Investigation and Design Report has been prepared for the other property.

The purpose of the foundation investigation was to assess the subsurface conditions at the site by drilling two boreholes and carrying out associated in-situ and laboratory tests, and to provide preliminary foundation engineering input to the assessment of strategies/options for the proposed widening of the deep cut slope.

This Preliminary Foundation Investigation and Design Report (FIDR) has been prepared specifically and solely for the proposed widening of the cut slope adjacent to the property at 164 Skyview Road as described above.



2.0 SITE DESCRIPTION

2.1 SITE LOCATION

The subject property (164 Skyview Road) is located on the south side of Highway 401, approximately 350 km east of Shelter Valley Road, in Northumberland County. The site location is shown on the Key Plan portion of Drawing No. 1 provided in Appendix A. Widening of the existing highway cut slope will be required on the south side of the highway from approximately Station 19+600 to Station 19+900.

2.2 SITE DESCRIPTION

At the location of the site, Highway 401 is a four-lane divided freeway with two lanes in each direction that is aligned in an approximate east-west orientation. The chainage on Highway 401 increases from west to east.

The Highway 401 profile slopes up towards the east, from Elevation 147 m at Station 19+600 to 153 m at Station 19+900. The ground surface at the northern perimeter of the private property, above the highway cut slope, varies from approximately 154 m to 159 m. The existing ground surface profile along the MTO Right of Way (ROW) boundary and the Highway 401 profile in the area of the highest portion of the cut slope are shown on Drawing No. 1 provided in Appendix A.

The cut slope adjacent to the Skyview Drive property ranges from about 5 to 8 m in height and has slope inclinations varying from about 2.25 horizontal to 1 vertical (2.25H:1V) to 2.5H:1V. The current crest of the highway cut slope is typically 7 m to 10 m north of the southern extent of the MTO Right of Way (ROW) in the area of the property at 164 Skyview Drive. The existing slope is covered by vegetation including trees, brush, and grasses.

A building has recently been constructed within the property at 164 Skyview Road. The building is located near Station 19+780 and is offset approximately 20 m south of the current ROW boundary.

2.2.1 Site Drainage

A rock/rip-rap lined drainage ditch is present at the bottom of the slope.

Locally within the MTO ROW, surface water drains to the west towards the Shelter Valley Creek. Regionally, surface drainage typically flows from north to south towards Lake Ontario.

2.2.2 Geological Information

As described in The Physiography of Southern Ontario (Chapman and Putnam, 1984), the site lies within the Iroquois Plain physiographic area. The Ontario Geological Survey (OGS) Quaternary geology map also suggests the site is located within a sandy silt to silty sand till overlain to the south by coarse-grained glaciolacustrine deposits consisting of sand or gravel with minor silt and clay.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Review of available Ontario Government well records indicates that the overburden soils are quite deep in the vicinity of the site extending to depths in excess of 50 m below ground surface. Based on the geological mapping information, bedrock is expected to consist of limestone.

2.2.3 Available Subsurface Information

No site-specific foundation investigation reports were available for this site in the MTO GEOCRES database/library.

A 1957 GEOCRES report titled “Foundation Report on New Bridge at Highway 401 Crossing Shelter Valley Creek, about 2 miles North East of Grafton” provides some information on the subsurface conditions in the area of the Highway 401 crossing of the Shelter Valley Creek valley. Several boreholes and penetration tests were advanced as part of the investigation for the creek crossing, including Borehole No. 3, Borehole No. 4, and Borehole No. 10, advanced on the east side of the creek, about 300 m to 400 m west of the current study area. The subsurface conditions are described as gravel and sand to dense fine sand in Borehole No. 3, which was advanced to about 12.5 m below ground surface. Penetration resistances ranged from 38 to greater than 100 blows per 300 mm, indicating the site soils were in a dense to very dense state. Borehole No. 4 and Borehole No. 10 were unsampled, with penetration testing only, and encountered refusal on inferred boulders at about 1.2 m below ground surface.

Relevant borehole records and a site plan displaying the 1957 borehole locations are included in Appendix B for reference.

3.0 INVESTIGATION PROCEDURES

3.1 SITE RECONNAISSANCE

A site reconnaissance of the existing cut slope and adjacent private property was carried out by a Stantec geotechnical engineer on October 4, 2021. The following provides a summary of observations made during the site reconnaissance:

- No visible indications of deep-seated slope instability (e.g. headscarps/displaced blocks, tension cracks, bulging at the toe of the slope etc.) were noted. Similarly, no signs of shallow instability or slope creep (e.g. surficial sloughing, curved tree trunks etc.) were observed
- The roadside drainage ditch was lined with rip rap in most areas (see Photograph 1 below). Rip-rap, or rockfill drainage ditches are present at the base of the cut slopes on both sides of Highway 401, east and west of the Shelter Valley crossing.
- The slope was generally covered by a variety of vegetation including trees, brush and grasses. In localized areas, generally just above the drainage ditch, the vegetative and surficial topsoil coverage was sparse (for example, see Photograph 2 below) and the mineral overburden soils were visible on the slope face. No signs of significant erosion were noted in these areas.
- No signs of seepage erosion on the slope face, normally associated with high groundwater gradients, were observed.

Overall, the existing cut slope adjacent to the golf course property is considered to be performing satisfactorily.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Public utility locates for the property at 164 Skyview Road did not identify any public or MTO underground utilities within the area of the boreholes. A private utility locate company, Onsite Locates Inc., confirmed that there were no identifiable private utilities at the borehole locations. Additionally, no private underground utilities were anticipated in the vicinity of the borehole locations based on conversations with the owner of the private property. The locate process did identify the presence of a telecommunications line located parallel to, and approximately 3 m to 4 m north of, the ROW boundary in the vicinity of the boreholes.



Photo No. 1: Existing slope looking west from toe of slope.

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**



Photo No. 2: Existing slope looking east from toe of slope.



Photo No. 3: Existing slope looking northwest from crest of slope.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Photograph 4 below displays the existing, single-storey garage building present near to the MTO ROW on the property at 164 Skyview Road. The north face of the building is located approximately 22 m south of the current MTO ROW boundary and has a vehicle entrance/exit door facing the highway.



Photo No. 4: Existing building at 164 Skyview Road viewed from the north.

3.2 FIELD INVESTIGATION

The foundation investigation program consisted of advancing two boreholes, designated as MW21-1 and BH21-2, between October 18 and 21, 2021. The two boreholes were advanced within the property at 164 Skyview Road directly south of the MTO ROW boundary. Borehole BH21-2 was located immediately north of the building recently constructed at the site. The borehole locations are displayed on the Borehole Location and Soil Strata Plan, Drawing No. 1, in Appendix A.

Prior to carrying out the investigation, Stantec contacted public utility authorities and retained a private utility locating subcontractor to clear the borehole locations for drilling. Drilling was carried out with a track-mounted drill rig equipped for soil sampling. The boreholes were advanced using continuous flight hollow stem augers.

The subsurface stratigraphy encountered in the boreholes was recorded in the field by experienced Stantec field personnel. Standard Penetration Tests (SPT) (ASTM D1586) were carried out in the boreholes at regular intervals; typically, every 760 mm to approximately 6 m to 7 m depth and 1520 mm below this depth. A relatively undisturbed Shelby Tube sample was attempted to be collected at a depth of approximately 12.5 m in Borehole BH21-2; however, the tube sheared off in the borehole when trying to remove it. The borehole was abandoned, and a new borehole was advanced approximately 1.8 m east of the original borehole location to facilitate sampling and testing at greater depths.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Shear vane testing, using N-sized vane testing equipment, was carried out at various depths in BH21-2 and in a borehole located approximately 0.5 m east of MW21-1. Vane testing was carried out in accordance with ASTM D2573.

The split spoon samples recovered from the SPTs were returned to our Markham laboratory for detailed classification and testing.

A monitoring well was installed in Borehole MW21-1 with a 3 m well screen located from approximately 6 m to 9 m below ground surface; the screened section of the well was provided with a sand filter and bentonite was placed above and below the sand pack. The water level was measured in the MW21-1 well between October 19 and 21, 2021. The monitoring well was decommissioned in accordance with Ontario Regulation 903 on October 21, 2021.

Observations of the groundwater conditions in Borehole BH21-2 were made in the open borehole at the time of drilling. Bentonite was used in the backfilling of the boreholes in conjunction with the intent of Environment Regulation 903.

3.3 LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations were determined by Stantec surveying personnel.

Summary information pertaining to the Stantec boreholes included in this report is given in Table 3.1 below.

Table 3.1: Borehole Information Summary

	Borehole Number	
	MW21-1	BH21-2
MTM Zone 10 Coordinates		
Northing	4875785.2	4875798.5
Easting	426494.7	426562.0
Ground Surface Elevation, m	155.4	158.6
Total Depth Drilled, m	15.9	18.9
End of Borehole Elevation, m	139.5	139.7
Number of soil samples	16	17

3.4 LABORATORY TESTING

All samples were transported to our Markham laboratory for visual examination and laboratory testing. The geotechnical laboratory testing program completed on the borehole samples is summarized below in Table 3.2.

Table 3.2: Laboratory Testing Program

Laboratory Test Type	Number of Tests
Moisture Content	34
Gradation Analysis	9
Atterberg Limits	6

Two soil samples were also submitted to AGAT Laboratories of Mississauga for analysis of pH, soluble sulphate content, chloride content, and resistivity.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Samples remaining after testing will be placed in storage for a period of one year after issue of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

4.0 SUBSURFACE CONDITIONS

4.1 OVERVIEW

The subsurface conditions observed in the boreholes are presented in detail on the Borehole Records provided in Appendix B. Also, an explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix B. A borehole location plan and a stratigraphic profile of the soils encountered within the boreholes is provided on Drawing No. 1 in Appendix A.

The stratigraphic boundaries on the borehole records and the stratigraphic profile are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units. The subsoil conditions will vary beyond the borehole location.

In general, the subsurface stratigraphy encountered in the boreholes advanced at the site consisted of a surficial layer of topsoil, overlying fill consisting of silty sand, overlying predominantly cohesionless soils comprised of sand to silt. The sand/silt deposits are underlain by a clayey silt stratum that contains occasional sandy silt to silt seams. A deeper sand/silty sand deposit was encountered below the clayey silt at the location of Borehole MW21-1. Detailed descriptions of the subsurface conditions encountered are provided in the following subsections.

4.2 OVERBURDEN

4.2.1 Topsoil

An approximately 150 mm to 200 mm thick topsoil layer was encountered at ground surface in Boreholes MW21-1 and BH21-2.

4.2.2 Fill

Fill materials comprised of silty sand containing trace to some gravel, trace clay rootlets, and occasional cobbles were encountered below the topsoil in both boreholes. The fill materials extended to depths of about 1.5 m (~Elevation 153.9 m) and 1.7 m (~Elevation 156.9 m) below ground surface in Boreholes MW21-1 and BH21-2, respectively.

Standard Penetration Test (SPT) 'N' resistance values measured within the silty sand fill ranged between 3 and 16 blows per 0.3 m of penetration indicating the fill materials are in a very loose to compact state.

Index tests carried out on representative samples of the fill yielded the following results:

Moisture Contents:

- 7 to 16%



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Grain Size Distribution Results:

- Gravel: 0%
- Sand: 65%
- Silt: 30%
- Clay: 5%

Grain size analysis results for one sample of the fill are presented on Figure C1 in Appendix C. The Unified Soil Classification System (USCS) group symbol for the fill material is SM (Silty Sand).

4.2.3 Sand

A native sand deposit containing some silt and gravel was encountered beneath the fill in Borehole BH21-2. Occasional cobbles and boulders are inferred to present within the sand deposit based on grinding of the augers during drilling. The sand deposit was approximately 1.4 m thick and was encountered to a depth of approximately 3.1 m below ground surface corresponding to an elevation of 155.6 m.

SPT N-values measured within the sand deposit ranged from 10 to 35 blows per 0.3 m indicating these materials are in a compact to dense state.

The natural moisture content of the sand was measured to be approximately 3 percent expressed as a dry weight of the soil.

4.2.4 Silt/Sandy Silt

A silt/sandy silt deposit containing variable but generally minor amounts of gravel and clay was encountered below the fill and/or sand deposits in both boreholes. Sandy silt materials were encountered to a depth of 3.8 m below ground surface (~Elevation 151.6 m) at Borehole MW21-1. An approximately 1.5 m thick deposit of silt containing some sand that extended to a depth of 4.6 m (~Elevation 154.0 m) was encountered beneath the sand in Borehole BH21-2.

SPT N-values measured within the silt/sandy silt deposits ranged from 19 to 57 blows per 0.3 m indicating these materials are in a compact to very dense state.

Index tests carried out on representative samples from the silt/sandy silt deposit yielded the following results:

Moisture Contents:

- 16 to 20%

Grain Size Distribution Results:

- Gravel: 0%
- Sand: 14 and 33%
- Silt: 61 and 72%
- Clay: 6 and 14%

An Atterberg Limit test was completed on one sample of the silt collected from BH21-2. The test results indicate that the material tested was non-plastic.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

The results of grain size analyses and the Atterberg Limit test conducted on samples of the silt/sandy silt deposits are presented on Figures C2 and C3 in Appendix C. The USCS group symbol for these materials is ML.

4.2.5 Clayey Silt

A deposit of clayey silt containing trace sand was encountered below the silt/sandy silt deposits. The clayey silt extended to a depth of 13.7 m (~Elevation 141.7 m) at Borehole MW21-1 and to the termination depth of 18.9 m (~Elevation 139.7 m) at Borehole BH21-2. Occasional to frequent sandy silt to silt seams were encountered within the clayey silt deposit. An approximately 0.7 m thick layer/zone of silt was encountered at a depth of 8.4 m (~Elevation 147.0 m) in Borehole MW21-1 and an approximately 0.3 m thick sandy silt layer/zone was encountered at a depth of 13.0 m (~Elevation 145.6 m) in Borehole BH21-1.

SPT N-values measured within the clayey silt deposit ranged from 10 to 43 blows per 0.3 m. In-situ shear vane testing using N-vane equipment, attempted at approximate depths of 6.1 m, 7.3 m, and 9.1 m in MW21-1 and approximate depths of 9.0 m, 9.9 m, 11.7 m and 14.6 m in BH21-2, encountered refusal (i.e., inability to turn vane). Based on the field and laboratory testing, and examination of samples obtained, the deposit is considered to generally have a very stiff to hard consistency with zones of stiff soils present.

SPT N-values of 33 and 68 blows per 0.3 m were measured within the sandy silt to silt zones described above indicating those materials are in a dense to very dense state.

Index tests carried out on representative samples from the clayey silt deposit yielded the following results:

Moisture Contents:

- 13 to 23%

Grain Size Distribution Results:

- Gravel: 0%
- Sand: 0 to 3%
- Silt: 64 to 78%
- Clay: 19 to 36%

Grain Size Distribution on silt layer within clayey silt deposit:

- Gravel: 0%
- Sand: 2%
- Silt: 81%
- Clay: 17%

Atterberg Limit Testing:

- Liquid limit: 19 to 21%
- Plastic limit: 13 to 14%
- Plasticity index: 5 to 8

An Atterberg Limit test conducted on the silt layer encountered within the clayey silt deposit at a depth of approximately 8.5 m in Borehole MW21-1 indicated that the material tested was non-plastic.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

The results of grain size analyses and Atterberg limit testing conducted on samples of the clayey silt are presented on Figures C4 and C5 in Appendix C. The USCS group symbols for this deposit are considered to range from CL-ML to CL (Clayey Silt).

The grain size analysis and Atterberg limits results for testing completed on a sample of the silt interlayer (Sample 11 from Borehole MW21-1) are presented on Figures C2 and C3 in Appendix C. The USCS group symbol for this material is ML (Silt).

4.2.6 Lower Sand/Silty Sand

A lower sand/silty sand deposit containing some gravel and trace clay was encountered below the clayey silt in Borehole MW21-1. The borehole was terminated within the lower sand deposit at a depth of 15.9 m below ground surface (~Elevation 139.5).

SPT N-values measured within the sand/silty sand deposit varied from 18 to 73 blows per 0.3 m indicating this material is in a compact to very dense state.

Index tests carried out on representative samples from the lower sand deposit yielded the following results:

Moisture Contents:

- 12 and 18%

Grain Size Distribution:

- Gravel: 18%
- Sand: 61%
- Silt: 15%
- Clay: 6%

The results of the grain size analysis test conducted on a sample of the lower sand deposit are presented on Figure C6 in Appendix C. The USCS group symbol for this deposit is SM (Silty Sand).

4.3 BEDROCK

Bedrock was not encountered within the termination depths of the boreholes advanced during the current investigation.

4.4 GROUNDWATER

The water level was recorded in the monitoring well at Borehole MW21-1 at 5.8 m (~Elev. 149.6 m) below ground surface on October 19, 2021; at 3.2 m (~Elev. 152.2 m) on October 20, 2021 and at 3.4 m (~Elev. 152.0 m) on October 21, 2021.

The water level was observed below a depth of 9.1 m (~Elev. 149.5 m) in Borehole BH21-2 during drilling. It should be noted that this observed groundwater level is not a stabilized measurement and hence is deemed to be an “inferred” water level.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Groundwater levels at the site will be subject to fluctuation due to seasonal changes, precipitation, and snow melt. Accordingly, the water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation or snow melt.

4.5 CHEMICAL TESTING

Two representative samples of the native soils were submitted to AGAT Laboratories in Mississauga, Ontario, for analysis of pH, water soluble sulphates & chloride concentrations, and resistivity. The analysis results are provided in Table 4.1. The chemical testing results provided by AGAT Laboratories are also provided in Appendix C for reference.

Table 4.1: Results of Chemical Analysis

Borehole #	Sample #	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm.cm)
MW21-1	SS10	6.9 to 7.5	8.42	5	6	8330
BH21-2	SS6	3.8 to 4.4	8.63	12	5	8700

5.0 MISCELLANEOUS

The field work was carried out under the supervision of Kirby Lales, EIT, under the direction of Mr. Kevin Nelson, P.Eng.

The public utility locates for this borehole investigation were arranged by Stantec personnel and private utility locates were completed by Onsite Locates Inc.

The track-mounted drill rig was supplied and operated by Downing Drilling of Hawkesbury, Ontario.

The location and elevation survey of the boreholes was completed by Tulloch Geomatics Inc. personnel.

Geotechnical laboratory testing was carried out in Stantec's Markham laboratory. Chemical testing for pH, soluble sulphate, chloride and organic contents, and resistivity was carried out by AGAT Laboratories of Mississauga.

This report was prepared by Zach Popper, P.Eng. and reviewed by Kevin Nelson, P.Eng. and Raymond Haché, P.Eng., Designated Principal MTO Foundation Contact.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

6.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole location, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted,

STANTEC CONSULTING LTD.



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**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

For

G.W.P 4060-11-00

Highway 401 Planning Study from Cobourg to Colborne, Ontario
Proposed Retaining Wall for Highway 401 Widening at 164 Skyview Road

Grafton, Northumberland County, Ontario

7.0 DISCUSSIONS AND ENGINEERING RECOMMENDATIONS

7.1 OVERVIEW

This section of the report provides preliminary foundation design input for the proposed widening of the highway cut slope on the south side of Highway 401 adjacent to the private property at 164 Skyview Road in Northumberland County, Ontario. The proposed widening is to accommodate the ultimate 8-lane highway configuration.

The recommendations provided are based on an interpretation of the factual data obtained from the subsurface investigation carried out at the site, and on the currently proposed slope and retaining wall configurations.

The interpretations and recommendations contained in this report are intended solely to provide the designers with information to assess feasible retaining wall and cut slope alternatives for the planned highway configuration. As such, where comments are made on construction aspects of the project, they are provided only to highlight those aspects which could affect the preliminary design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

This Preliminary Report is not to be used for the detailed design of this project; additional subsurface investigation will be required to meet minimum MTO foundation investigation requirements for the detailed design of the proposed retaining walls at this site. A detailed Foundation Investigation and Design Report will need to be prepared after further field investigation is carried out. The foundation recommendations presented in this preliminary report are subject to change, if necessary, based on the findings of the future site investigation.

7.2 PROJECT DESCRIPTION AND BACKGROUND

7.2.1 Project Description

The project involves the planning, preliminary design, and Class Environmental Assessment (Class EA) study for the section of Highway 401 extending from approximately 2 km east of Nagle Road in the Town of Cobourg to Percy Street in Northumberland County, Ontario. The study includes the rehabilitation or replacement of several structures, future widening of the highway, and new retaining walls where there is insufficient space to develop new cut and fill slopes for the highway widening. The portion of Highway 401 within the study area is planned to be widened to an interim 6-lane and an ultimate 8-lane configuration.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

7.2.2 Proposed Works

Widening of the existing cut slope on the south side of Highway 401 within the study area is required to facilitate the future, wider highway configuration. The existing slope is typically about 5 to 8 m in height and has slope inclinations ranging from about 2H:1V to 3H:1V but more typically about 2.25H:1V to 2.5H:1V in the areas with the highest slopes. The distance from the crest of the existing highway cut slope to the south ROW boundary varies from about 7 m to 10 m in the vicinity of the property at 164 Skyview Drive. The slope is mostly covered by vegetation including trees, brush, and grasses with a shallow drainage ditch along the toe of slope.

A reconnaissance of the slope adjacent did not identify signs of instability or significant erosion. As such, the existing cut slope adjacent to the to the property at 164 Skyview Road is considered to be performing satisfactorily.

As part of the current study, a variety of cut slope and retaining wall options are being considered to accommodate the proposed highway widening. The sketch below depicts potential slope configurations, without a retaining wall but including a midslope bench and interceptor ditch, for permanent cut slope inclinations ranging from 2H:1V to 3H:1V in the area of the highest cut slope.

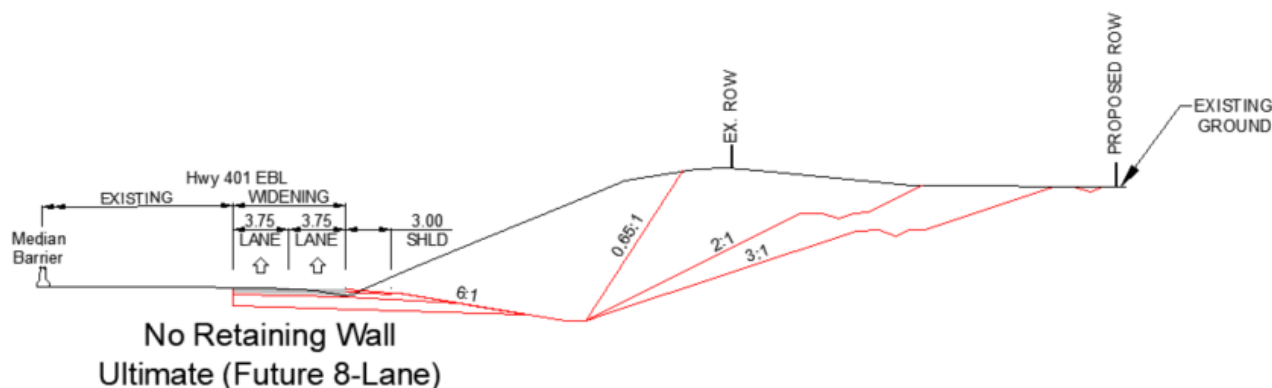


Figure 1: Preliminary Highway 401 Cut Slope Cross Section – 8-Lane Highway Configuration

These potential slope configurations would require additional property to the south of the current ROW limit of approximately 13 m (2H:1V slope) to 26 m (3H:1V slope). As such, there is insufficient space to permit widening of the existing cut slope without either the use of a retaining wall system or widening of the ROW into the adjacent property. Widening of the ROW into the adjacent property would either encroach into the footprint of the existing building or limit the ability to access the building through vehicle door in the north face of the building.

A summary of the various retaining wall options under consideration are outlined below followed by sketches of the preliminary alternatives:

- Option 1 – A retaining wall located offset approximately 6 m inside the ROW boundary. This option would require an approximately 270 m long wall with a maximum height of about 9 m near Station 19+780.
- Option 2 – A low (less than 2 m high) retaining wall located along the shoulder of the ultimate 8-lane highway configuration between about Stations 19+560 and 19+900.
- Option 3 – A RSS or cantilever retaining wall located at the clear zone for the future 8-lane highway configuration between about Stations 19+640 and 19+820 with a drainage ditch developed between the retaining wall and the



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

highway shoulder. The height of the wall for this configuration would be approximately 2.5 m to 3.7 m and the base of the wall would step down, from east to west, approximately 4 m along the length of the wall alignment. This wall would be phased out on the eastern and western extents of the site once there is sufficient space to develop a 2.5H:1V cut slope within the current ROW boundaries.

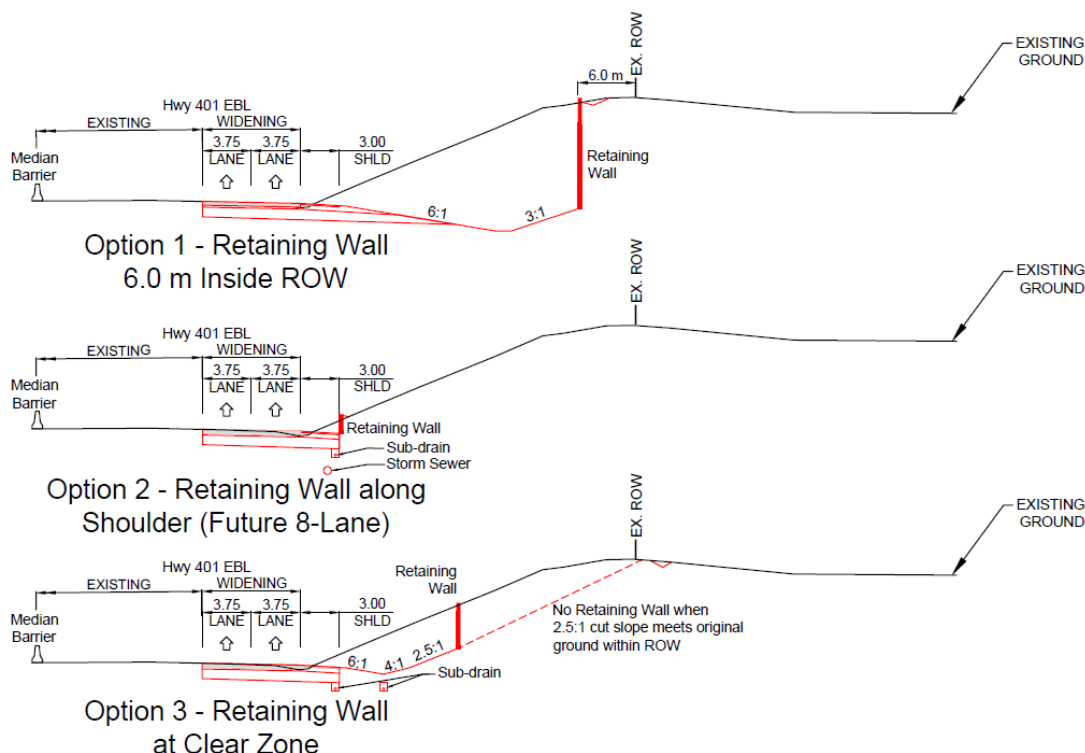


Figure 2: Preliminary Highway 401 Slope/Retaining Wall Cross Sections - 8-Lane Highway Configuration

At the site, the Highway 401 profile slopes up towards the east, from about Elevation 147 m at Station 19+600 to approximately 153 m at Station 19+900, whereas the top of the slope is typically between Elevation 155 m and 159 m. To accommodate this elevation difference, the retaining wall foundation sections would step down towards the west.

7.3 DEGREE OF SITE AND PREDICTION MODEL UNDERSTANDING

The Canadian Highway Bridge Design Code (CHBDC) [2019] requires an assessment of the “degree of site and prediction model understanding” as a component of the geotechnical engineering investigation and/or services. The site and prediction model understanding includes the geotechnical properties of the site and the accuracy and degree of confidence regarding the numerical performance prediction models to be used to estimate the geotechnical serviceability limit states reactions and ultimate limit states resistances.

Based on the scope of subsurface investigations completed and available subsurface information related to this site, a “Typical Understanding” has been adopted for foundation design assessment purposes. The consequence classification has been selected as “Typical Consequence” as per Section 6.5 of the Commentary of the CHBDC.



7.4 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at the site consist of a surficial layer of topsoil, overlying fill consisting of silty sand, overlying predominantly cohesionless soils consisting primarily of silt/sandy silt. The silt deposits are underlain by a clayey silt stratum that contains occasional sandy silt to silt seams. A deeper sand/silty sand deposit was encountered below the clayey silt at the location of Borehole MW21-1.

The soil profile identified in Table 7.1 below and Figure D1 in Appendix D can be used for the preliminary design of the planned retaining wall. This profile has been based on the soil stratigraphy encountered in Borehole BH21-2, which was advanced at/near the crest of the slope in the area of the highest portion of the existing slope and in close proximity to the building recently constructed within the property at 164 Skyview Road.

The geotechnical parameters identified in the soil profile were selected based on the synthesis of measured SPT 'N' values and laboratory index test results (including moisture contents) of soil samples obtained from the boreholes advanced at the site.

Table 7.1: Soil Profile and Design Parameters

Elevation (m)		Soil Type	Design Parameters				
From	To		Total Unit Weight γ , (kN/m ³)	Friction Angle ϕ' (°)	Cohesion c' (kPa)	Undrained Shear Strength S_u (kPa)	Soil Elastic Modulus E' , (MPa)
Surface	157.0	Very loose to compact Silty SAND (SM) (FILL)	21.0	30	-	-	15
157.0	155.5	Compact to dense SAND (SP)	21.0	32	-	-	30
155.5	154.0	Compact to dense SILT (ML)	21.0	30	-	-	30
154.0	<140.0	Stiff to hard CLAYEY SILT (CL-ML to CL) with occasional silt seams	20.0	32	2	150	30
141.5	<140.0	Compact to very dense SAND (SM) (at MW21-1 location)	20.0	32	-	-	50

Notes:

- (1) The water level was recorded in the monitoring well at Borehole MW21-1 at 5.8 m (~Elev. 149.6 m) below ground surface on October 19, 2021; at 3.2 m (~Elev. 152.2 m) on October 20, 2021, and at 3.4 m (~Elev. 152.0 m) on October 21, 2021. Groundwater was observed at ~9.1 m depth (~Elev. 149.5 m) in Borehole BH21-2 during drilling. A design groundwater elevation of 152.2 m can be used for preliminary foundation design purposes. Submerged unit weights (γ') should be used below the groundwater level.
- (2) The friction angles and cohesion values are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only.
- (3) A deeper sand deposit was encountered in Borehole MW21-1 at Elevation 141.7 m. The borehole was terminated within the deeper sand deposit at approximate Elevation 139.5 m.

7.5 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations, f , at the site is 1.4 m.

Therefore, all footings should be provided with a minimum of 1.4 m of soil cover or equivalent insulation for protection against frost heaving.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

This depth of frost penetration should also be considered in the design of frost tapers adjacent to backfill zones, if applicable.

7.6 SEISMIC CONDITIONS

7.6.1 Site Class

Based on the findings of the current borehole investigation, the site is underlain by loose to very dense silt and sand deposits overlying stiff to hard, but typically very stiff, clayey silt deposits. Based on information from the GEOCREST report for the Shelter Valley creek, road culverts located to the east of the site, and water well records in the vicinity of the site, the overburden materials are anticipated to extend to depths of more than 40 m to 60 m below the original grade.

Based on the available subsurface conditions, it is recommended that Site Class C as defined in Section 4.4.3 of the CHBDC (2019) be used for preliminary design purposes.

7.6.2 Peak Ground Acceleration (PGA)

Seismic hazard values for this site were obtained from Natural Resources Canada (2015 National Building Code). The 2015 NBC Seismic Hazard calculation sheet for this site is provided in Appendix F. Table 7.2 summarizes the parameters to be used in a force-based design corresponding to a 2475-year return period.

Table 7.2: Peak Ground Acceleration Data

<i>PGA</i>	<i>S_a(0.2)</i>	<i>PGA_{ref}</i>	Site Class	Site Adjusted <i>PGA</i>
0.11g	0.173g	0.088g	C	0.11g

7.6.3 Liquefaction Potential

The potential for soil liquefaction was evaluated by comparing the cyclic stress ratio (CSR) caused by the design earthquake with the soil resistance expressed in terms of the cyclic resistance ratio (CRR). The evaluation follows the analysis methodology suggested by Idriss and Boulanger (2008) and is based on the following:

- The SPT blow count data from boreholes.
- A Site Adjusted PGA of 0.11g.
- An earthquake magnitude *M_w* of 6.0.

Liquefaction of the cohesionless soil that underlies the site is not considered to be a significant concern due to the compact to dense nature of the cohesionless soil deposits and the relatively low seismic hazards (peak ground acceleration and earthquake magnitude) that apply for the site. A Factor of Safety against liquefaction higher than 1.5 was estimated based on the available seismic hazard values and soil parameters.

7.7 PRELIMINARY FOUNDATION ENGINEERING DESIGN INPUT

The following sections provide preliminary foundation engineering input for the new slope and/or retaining wall(s) planned as part of the highway widening. The input provided in this section of the report is preliminary in nature and should be reviewed, and modified as necessary during detail design, once the geometry, location, and loading conditions for the new foundations are determined.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

7.7.1 Slope and Retaining Wall Options

Based on preliminary information from the design team, Table 7.3 provides a summary of the alternative slope and retaining wall options (as described in Section 7.2.2) with advantages, disadvantages, risks and relative costs.

Table 7.3: Comparison of Slope and Retaining Wall Options

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Option 2 - Cantilever Type Wall adjacent to highway shoulder (no roadside drainage ditch)	<ul style="list-style-type: none"> Straight forward construction Low wall height Shallow foundations are feasible Significantly reduced excavation and associated soil disposal volumes in comparison to other wall types/slope configurations. With the base of the retaining at the level of the pavement subdrain, the retaining wall drain would effectively act as a drain for the slope 	<ul style="list-style-type: none"> May need to control groundwater / unwater for foundation construction The base of the retaining wall would need to extend down to the base of the longitudinal subdrain elevation A guardrail would be anticipated along this section of highway Not preferred from winter maintenance perspective. 	Low to Moderate	<ul style="list-style-type: none"> Low risk option Potential for erosion on slope face above wall requiring maintenance.
Option 3 - RSS Wall or Cantilever Wall at Clear Zone. Roadside drainage ditch between wall and highway.	<ul style="list-style-type: none"> Straight forward construction Low wall height Shallow foundations are feasible Reduced excavation and associated soil disposal volumes in comparison to walls constructed near property boundary. 	<ul style="list-style-type: none"> May need to control groundwater / unwater for foundation construction Seepage erosion during wet seasons and related instabilities could occur on the slope face present between the wall and drainage ditch Soils in ditch susceptible to erosion due to concentrated water flows. 	Low to Moderate	<ul style="list-style-type: none"> Low risk option Potential for erosion on slope face above wall requiring maintenance. Potential for erosion or instability of cut slope in front of wall
Widened Cut Slope (No Retaining Wall)	<ul style="list-style-type: none"> Less construction cost compared to retaining wall options Straight forward construction 	<ul style="list-style-type: none"> Insufficient space available in current right-of-way. Permanent encroachment / property acquisition required. Large volumes of excess material generated. Material may not be suitable for reuse in highway construction due to erodibility considerations. Potential for erosion on slope face. 	Low	<ul style="list-style-type: none"> Requirements for land acquisition may slow schedule Potential for erosion on slope face requiring maintenance.
Option 1 – Retaining Wall 6 m Inside ROW. Gravity or RSS Wall	<ul style="list-style-type: none"> Conventional highway/ditch geometry can be used. Very stiff soils present in slope area provide suitable subgrade conditions for wall support 	<ul style="list-style-type: none"> High and variable wall heights required. Potential for instability of slope below wall Temporary support systems, including encroachment into adjacent property for anchors, needed for wall construction due to lack of space within ROW. Large volumes of excess material generated. Material may not be suitable for reuse in highway construction due to erodibility considerations. 	High	<ul style="list-style-type: none"> Requirements for temporary encroachment into private property Potential for instability of slope below wall; increased wall embedment may be necessary



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Option 1 – Retaining Wall 6 m Inside ROW. Soldier Pile and Lagging Wall.	<ul style="list-style-type: none"> Conventional highway/ditch geometry can be used. 	<ul style="list-style-type: none"> High and variable wall heights required. Slower construction process. Large volumes of excess material generated. Material may not be suitable for reuse in highway construction due to erodibility considerations. Permanent subsurface encroachment for tie-back systems required in adjacent private property Difficulties installing soldier piles where cohesionless soils are encountered below water table 	High	<ul style="list-style-type: none"> Need for permanent subsurface encroachment into private property for ground anchors
Option 1 – Retaining Wall 6 m Inside ROW. Secant Pile Wall	<ul style="list-style-type: none"> Conventional highway/ditch geometry can be used. 	<ul style="list-style-type: none"> High and variable wall heights required. Slower construction process Potential for groundwater mounding behind wall and associated impacts to wall performance due to water pressures and frost action. Large volumes of excess material generated. Material may not be suitable for reuse in highway construction due to erodibility considerations. Permanent subsurface encroachment for tie-back systems required in adjacent private property Difficulties installing secant piles where cohesionless soils are encountered below water table 	High	<ul style="list-style-type: none"> Damage to wall due to build-up of water and frost pressures Need for permanent subsurface encroachment into private property for ground anchors

Based on the considerations provided in the table above, Option 2 would be preferable from a foundation engineering perspective. This option includes construction of a shorter, cantilever wall at the toe of the new cut slope, where the founding level of the retaining wall is at or below the proposed pavement subdrain level. With this option, the retaining wall would effectively act as a toe drain at the base of the cut slope. The use of a standard MTO toe wall (i.e. as outlined in OPSD 3120.100) is not recommended for this application due to the potential for groundwater flow from the slope below the wall and frost-related movements of the wall.

Also considered suitable is Option 3, where a slightly taller wall would be located at the limit of the future clear zone about midway into the current slope, with a drainage swale constructed between the wall and highway. Similar to Option 2, the pavement drainage would be provided by a longitudinal subdrain since a standard ditch configuration cannot be accommodated with this design. Consideration should be given to the installation of a second longitudinal subdrain with the same elevation as the pavement structure subdrain, placed directly beneath the drainage swale, in order to limit groundwater from the cut slope from accessing the pavement subdrain and to act as a toe drain to the cut slope.

In the area of the highest current cut slope, the crest of the existing cut slope is located within about 7 m to 8 m of the current right-of-way boundary which does not provide sufficient space to accommodate the proposed highway



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

widening. As identified in Figure 1 contained in Section 7.2.2 of this report, the flattest cut slope inclination that can be accommodated within the existing right-of-way would be approximately 2H:3V. Such a slope would have a factor of safety against instability of much less than 1. Therefore, a widened cut slope option without a retaining wall would only be feasible if widening of the ROW into the adjacent property is undertaken. The existing highway cut, which is performing acceptably, has slope inclinations ranging from about 2.25H:1V to 2.5H:1V. The results of slope stability analyses conducted for this project (described in the following section of this report) indicate that new cut slopes developed at an inclination of 2.5H:1V would have a Factor of Safety against instability of about 1.5; this slope inclination is recommended for portions of the slope where retaining walls will not be constructed. Widening/shifting of the ROW boundary of approximately 20 m, including allowance for the development of an interceptor drainage ditch, would be needed to provide enough room to develop a new cut slope with a similar inclination to the existing slope.

Both Options 2 and 3 have several advantages over the other slope/wall options being considered including eliminating the requirements for encroachment into adjacent private property, lowering costs/speeding up construction schedule than other wall types, and decreasing the amount of excess material generated. Further details on these preferred wall/slope configurations are provided in the following sections.

7.8 SLOPE STABILITY EVALUATION

7.8.1 Existing Slope Condition

Analyses were carried out to assess the Factor of Safety (FOS) against instability for a cross-section through the existing cut slope located at approximately Station 19+780 near the building recently constructed within the property at 164 Skyview Road. This portion of the slope had the deepest existing cut (i.e. highest cut slope) and was considered to be the most critical in terms of the potential for slope instability. The slope at this location had an overall inclination of approximately 2.25H:1V.

The evaluation was carried out using the commercial program Slope/W (Geo-Slope, 2019). The geotechnical design parameters for the native soils outlined in Table 7.1 were used in the analyses. A horizontal seismic load coefficient of 0.055g (half of the site-adjusted PGA) was used for a pseudo-static slope stability evaluation.

The results of the slope stability analyses for static and seismic loading conditions are provided in Figures D2 and D3 in Appendix D. The results of the stability analyses indicate that the factor of safety against instability for a critical, deep-seated failure surface is approximately 1.2. A factor of safety slightly above 1.0 was calculated under seismic conditions.

7.8.2 Global Stability - Proposed Option 2 and 3 Retaining Wall Configurations

Stability analyses were also completed for the proposed widened cut slope configurations incorporating a new retaining wall(s) at the shoulder or clear zone of the future highway (i.e. Options 2 and 3 described above). As the final geometry of the walls had not been determined, wall base widths equal to approximately 70% of the exposed wall heights were considered in the analyses. A friction angle of 35° and a unit weight of 21 kN/m³ was applied for the granular backfill materials behind the new walls.

The results of the stability analyses for a wall located directly adjacent to the shoulder of the widened highway (i.e. Option 2 configuration with no roadside drainage ditch) under static and seismic conditions are displayed on Figures



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

D4 and D5, respectively. Figures D6 and D7 display the results of analyses for a wall located at the Clear Zone to the south of a new highway drainage ditch (i.e. Option 3). These analyses confirm that the Factors of Safety against global instability (i.e. slip surfaces passing beneath the new walls) are greater than 1.5 for static conditions and greater than 1.4 under seismic conditions, which are considered acceptable.

The retaining wall for Option 3 is proposed to be truncated once there is sufficient room to develop a permanent cut slope with 2.5H:1V sideslopes within the current right-of-way boundaries. A stability analysis for the proposed, wider cut slope configuration near Station 19+600 (west end of proposed wall) incorporating a 2.5H:1V cut slope is displayed on Figure D8. This figure shows that a FOS against instability of 1.5 is obtained for this slope inclination, which is recommended to be used in any areas of the widened cut where retaining walls will not be constructed.

7.8.3 Foundation Design – Option 2 and 3 Retaining Wall Configurations

7.8.3.1 Foundation Subgrade Preparation and Design Considerations

The results of the foundation investigation indicate that the lower portions of the existing highway cut slope, where the foundations of the proposed wall(s) would be constructed, are formed within stiff to hard (typically very stiff) clayey silt materials. The retaining walls should be founded on the undisturbed clayey silt deposits or on compacted OPSS Granular A or B Type II materials placed above the material.

Following completion of the preparation of the founding surface, a milestone inspection should be conducted by foundation/geotechnical personnel arranged for by the Contract Administrator in accordance with SP109S12. Any soft/loose or otherwise disturbed soils and/or any deleterious fill materials should be sub-excavated and replaced with structural fill consisting of compacted Ontario Provincial Standard Specification (OPSS) Granular B Type II material, unless the replacement material thickness is to be less than 200 mm, in which case an OPSS Granular A should be used.

The subgrade soils are susceptible to disturbance, erosion and degradation on exposure to water and construction traffic. It is recommended that a minimum 200 mm thick layer of OPSS Granular A materials compacted to 100 percent of their Standard Proctor maximum dry density or a 100 mm thick working slab of lean concrete be placed over top of the subgrade soils immediately following inspection and approval of the subgrade to protect these materials from softening.

The Option 3 (Wall at Clear Zone) configuration incorporates a roadside drainage ditch with a 2.5H:1V slope present between the ditch invert and the base of the exposed face of the retaining wall. There is potential for erosion and shallow sloughing of the slope face in front of the wall which could lead to loss of support in front of the wall. To mitigate against this, the wall is recommended to be embedded such that all portions of the base of the wall are located at or below a line drawn up from the ditch invert level towards the wall at an inclination of 3H:1V. Additionally, erosion protection measures (e.g. rock protection) are recommended to be installed on the ditch/slope face below the wall.

7.8.3.2 Geotechnical Resistances and Reactions

For preliminary design purposes, the factored geotechnical resistances at ULS and geotechnical reactions at SLS provided in Table 7.4 below may be used in the design of the retaining wall foundations that are constructed on



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

properly prepared very stiff clayey silt subgrade soils at the founding depths (founding elevations will vary along the length of the walls) indicated.

Table 7.4: Geotechnical Vertical Resistance & Reaction – Shallow Foundations

Foundation Element	Minimum Founding Depth (m)	Foundation Width (m)	¹ Factored Geotechnical Resistance at ULS _r (kPa) $\phi_{gu} = 0.5$	Factored Geotechnical Reaction at SLS (kPa) $\phi_{gs} = 0.8$
Strip Footing – Flat surface in front of wall (Option 2 - Assumed cantilever wall)	1.4	2	300	200
Strip Footing – 2.5H:1V slope in front of wall (Option 3 - Assumed RSS wall)	0.8	3 to 4	150	125
Strip Footing – 2.5H:1V slope in front of wall (Option 3 - Assumed cantilever wall)	1.4	3 to 4	175	125

Notes: 1) ULS resistances and foundation settlements provided above are dependent on the footing size and depth, configuration, and applied loads; therefore, the geotechnical resistances should be reviewed if the selected footing widths or founding elevations differ from those given in the table above. The ULS_r values provided in the above table were calculated based on minimum footing widths of 2 m (Option 2) and 3 m (Option 3) and are recommended at the preliminary design stage for the stated range of footing widths. The SLS values were calculated based on a footing width of up to 4 m for Option 3 and 2 m for Option 2.

2) Minimum Founding Depths were based on assumed wall types.

In accordance with Table 6.1 in the CHBDC, the ULS and SLS Geotechnical Resistances were determined based on a consequence level of “Typical” with a consequence factor equal to 1.

In accordance with Table 6.2 of Section 6.9.1 in the CHBDC and the site and prediction model understanding classification of “Typical”, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS_r).

In accordance with Table 6.2 of Section 6.9.1 in the CHBDC and the consequence and site understanding classification of “Typical”, a resistance factor of 0.8 has been applied in calculating the geotechnical reaction at Serviceability Limit State (SLS) corresponding to a maximum settlement of 25 mm.

The structural engineer must verify that the selected footing widths are sufficient to resist overturning.

The geotechnical resistances are provided for loads applied perpendicular to the surface of the footings. Where this is not the case, eccentricity and inclination of the loads must be considered.

The preliminary geotechnical resistance and reaction values provided above must be re-evaluated and modified as necessary during detailed design based on the actual founding elevations, foundation configuration and sizes.



7.8.4 Geotechnical Horizontal Resistance (Sliding)

The unfactored horizontal resistance to sliding of the retaining wall foundations may be calculated using the following unfactored coefficient of friction:

0.55	between OPSS Granular A and cast-in-place concrete
0.40	between native clayey silt and cast-in-place concrete

In accordance with Table 6.2 of the CHBDC and the consequence and site understanding classification of “Typical”, a resistance factor against sliding of 0.8 (frictional) should be applied to obtain the resistance at ULS_f .

7.9 LATERAL EARTH PRESSURES

7.9.1 Backfill

The materials used as backfill for the proposed retaining wall structures should consist of free-draining granular fill placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this report, it is assumed that backfill materials meeting the requirements of OPSS Granular B (Type I or Type II) or Granular A materials will be used.

Excavation and backfill for the new retaining walls should be carried out in accordance with OPSS.PROV 902 Construction Specification for Excavation and Backfilling – Structures. Backfill materials should be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.

7.9.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of retaining walls and retained soil systems. Computation of earth pressures should be in accordance with Section 6.13.3 of the CHBDC (2019). For retaining walls that are designed to allow rotation, active earth pressures may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressures should be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.8 of the CHBDC.

The total at rest (P_O), active (P_A), and passive (P_P) thrusts can be calculated using the following equations:

$$P_O = \frac{1}{2} K_o \gamma H^2$$

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

where H is the height of the wall and γ is the unit weight of the backfill soil. Values for K_a , K_p , K_o and γ are provided in Table 7.5 and Table 7.6 for horizontal and sloping (2H:1V) backfill conditions, respectively. The thrusts act at a point one third up the height of the wall.



Table 7.5: Recommended Static Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m ³)	21	22
Effective Friction Angle	32°	35°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43
Coefficient of Active Earth Pressure (K_a)	0.31	0.27
Coefficient of Passive Earth Pressure (K_p)*	3.25	3.69

*Note: K_p values provided in table above are applicable when the ground surface in front of the retaining wall is flat. These values would need to be reduced where the ground in front of the wall slopes away from the wall and will need to be reviewed during the detailed design stage once wall and slope geometries are determined.

Table 7.6: Recommended Static Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m ³)	21	22
Effective Friction Angle	32°	35°
Coefficient of Earth Pressure at Rest (K_o)	0.68	0.62
Coefficient of Active Earth Pressure (K_a)	0.47	0.39

7.9.3 Seismic Lateral Earth Pressures

The following design parameters are provided for use in assessing the earth pressures induced on the retaining walls under seismic loading conditions.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where:

- K_{AE} = active earth pressure coefficient (combined static and seismic)
- K_{PE} = passive earth pressure coefficient (combined static and seismic)
- H = height of wall
- k_h = horizontal acceleration coefficient
- k_v = vertical acceleration coefficient
- γ = total unit weight

The seismic earth pressures for structures with horizontal backfill behind the walls may be calculated using the parameters provided in Table 7.7. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate. Table 7.8 and Table 7.9 provide seismic earth pressures for yielding walls with horizontal and 2H:1V backfill slopes behind the walls, respectively.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBourg TO COLBORNE, ONTARIO**

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values as per CHBDC 2019.

Table 7.7: Seismic Design Parameters to Estimate Lateral Earth Pressures

Site Adjusted PGA	Horizontal Acceleration Coefficient, k_{h0}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding (<i>wall movements of 25 mm to 50 mm</i>)
0.11g	0.11	0.055

Note: k_{h0} is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the site-adjusted PGA estimated at ground surface. The vertical acceleration coefficient (k_v) should be ignored in the calculations as per CHBDC 2019, section C6.14.7.

Table 7.8: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m ³)	21	22
Effective Friction Angle	32°	35°
Passive Earth Pressure, (K_{PE})	3.15	3.58
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.33	0.33
Yielding Wall		
Active Earth Pressure (K_{AE}) for Yielding Wall	0.34	0.30
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.36	0.36
Non-Yielding Wall		
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.37	0.33
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.38	0.38

*Note: K_{PE} values provided in table above are applicable when the ground surface in front of the retaining wall is flat. These values would need to be reduced where the ground in front of the wall slopes away from the wall and will need to be reviewed during the detailed design stage once wall and slope geometries are determined.

Table 7.9: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Granular B Type I	OPSS Granular A and Granular B Type II
Bulk Unit Weight, γ (kN/m ³)	21	22
Effective Friction Angle	32°	35°
Yielding Wall		
Active Earth Pressure (K_{AE}) for Yielding Wall	0.58	0.47
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.38	0.38



7.10 EROSION AND SCOUR PROTECTION DESIGN CONSIDERATIONS

Based on a review of the results of the subsurface investigation and laboratory test results from the current foundation investigation, Stantec's Pavement Engineering group prepared a memo assessing the susceptibility of the site soils to erosion based on k-factors that represent the potential for erodibility of the soil, which are generated by the Wishmeier nomograph. This memo, which is included in Appendix E for reference, identified that the soils encountered in Boreholes MW21-1 and BH21-2 had k-factors ranging from 0.07 to 0.55 indicative of very slight to very severe erosion severity. The silt and clay deposits encountered in the boreholes had k-factors of between 0.39 and 0.55 indicative of a moderately severe to very severe rating.

MTO personnel have informed Stantec that maintenance has been required to address several incidents of erosion on existing cut slope and highway fill embankments within the overall Highway 401 Planning Study area from Cobourg to Colborne. Based on a review of areas where this maintenance work has been required, the erosion events requiring maintenance are understood to have generally occurred in areas where concentrated overland flows could occur (e.g. within or at the outlets of drainage ditches, etc.).

A review of the existing site conditions indicates that no significant erosion (seepage or surface flow) has occurred on the face of the existing cut slope. However, a rockfill lined drainage swale or shallow ditch is present at the base of the cut slope, both east and west of Shelter Valley Creek, along Highway 401, suggesting that either surface flow erosion or drainage problems had occurred within these areas.

Based on the above, the following design measures are recommended in order to reduce the potential need for future maintenance of the slopes due to erosion:

- Drainage ditches should be installed above the crest of slope to intercept and redirect overland flow away from the new cut slope face and retaining walls.
- All proposed drainage swales or ditches are recommended to be provided erosion protection (i.e. rip rap or rock protection). For the Option 3 (Clear Zone wall) configuration, these measures are also recommended to be implemented for the entire portion of the slope face present between the invert of the roadside drainage ditch and the face of the retaining wall.
- For Option 3, it is recommended that a second longitudinal subdrain be installed below the drainage swale, at the same level as the pavement subdrain, in order to prevent groundwater from the cut slope from accessing the pavement subdrain and to act as a toe drain to the cut slope.
- Vegetation slopes should be established on the new slope faces as soon as possible after completion of the embankment construction to minimize the potential for surficial erosion.

7.11 CEMENT TYPE AND CORROSION PROTECTION

Two samples of the site soils were submitted for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The results of the analysis are summarized in Table 4.1 in Section 4 of this report.

The concentration of soluble sulphates provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater. The soluble sulphate concentration for the samples tested were 5 and 6 µg/g. As per Canadian Standards Association (CSA) Standard A23.1.14/A23.2-14, sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH values were 8.42 and 8.63, which are within what is considered the normal range for soil pH and do not indicate a highly corrosive environment. The American Association of State Highway and Transportation Officials (AASHTO) LFRD Bridge Design Specifications indicate that resistivity values of less than 20 ohm-m are indicative of a potential corrosive environment; the reported resistivity values were above that level. The additional test results provided in Table 4.1 may be used to aid in the selection of coatings and corrosion protection systems for buried infrastructure incorporating steel components.

8.0 PRELIMINARY CONSTRUCTION CONSIDERATIONS

8.1 CONSTRUCTION STAGING

The proposed cut slope widening and retaining wall construction are not anticipated to require detours or traffic control staging.

8.2 TEMPORARY PROTECTION SYSTEMS

Temporary protection systems may be required to facilitate excavations for retaining wall or sewer construction (for Option 3). The contractor will ultimately be responsible to develop and implement a roadway protection system/temporary excavation support system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters.

Based on the subsurface conditions at the site, the following table compares the available temporary protection system options considered for excavations for the proposed retaining wall foundations and/or highway widening.

Table 8.1: Comparison of Temporary Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet piles (SSP)	<ul style="list-style-type: none"> Simple installation process Provides cut-off to groundwater seepage from sides of excavation thereby reducing overall dewatering volumes 	<ul style="list-style-type: none"> Difficult to drive/install where cobbles and/or boulders are present May require large sections where cantilever design is adopted Tie-back anchors may be required 	Medium	<ul style="list-style-type: none"> Damage to sheet piles during driving
Soldier piles with timber lagging; (struts/rakers as required)	<ul style="list-style-type: none"> Simple installation process 	<ul style="list-style-type: none"> Additional labour required Groundwater control (dewatering) must be implemented prior to installation 	Low	<ul style="list-style-type: none"> Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
		where wall extends below water level. <ul style="list-style-type: none"> Removal of soldier piles can be difficult Tie-back anchors may be required 		<ul style="list-style-type: none"> Potential for minor loss of ground at rear of lagging

Temporary excavations required for construction of the retaining walls are anticipated to encounter cohesionless soils below the water table. These conditions could result in difficulties installing soldier piles (i.e. due to water ingress into the drilled holes) and/or the potential for loss of ground prior to lagging installation. Therefore, the use of sheet pile wall system, which would reduce installation difficulties and provide a cut-off to groundwater flows into the excavation areas, is considered the preferable option from a foundation engineering perspective based on the subsurface conditions at the site.

Roadway protection design should meet the requirements of Performance Level 2 in accordance with OPSS.PROV 539 and should consider loads from construction equipment and falsework. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Strut, raker or tie-back design, if and as required, must be designed not to exceed these limits.

Horizontal movement of the temporary roadway protection system should be monitored throughout the construction period as described in OPSS.PROV 539. The monitoring requirements, including the milestone inspections, are outlined in OPSS.PROV 539.

8.3 EXCAVATION AND BACKFILLING

Excavation and backfill operations for the slope regrading and retaining wall construction should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures.

Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath the new retaining walls. Where deleterious materials are encountered at foundation subgrade level, the materials should be excavated, removed and replaced with compacted granular fill materials. The lateral extent of the zone of subexcavation (and replacement) should include all deleterious material within the influence zone of the above foundation elements.

All side slopes for open cut excavations should conform to the Occupational Health and Safety Act regulations for Construction Projects (OHSA). The construction of the proposed retaining walls would require temporary excavations extending into the existing highway cut slopes, which are expected to consist of compact to very dense sandy silt/silt and stiff to hard clayey silt soils. Where space permits, these excavations may be developed using open-cut methods. The compact or stiff soils are generally classified as Type 3 soils, provided they are above the water table or dewatering of these soils is undertaken prior to excavation. Dense or very stiff portions of the soils above the water table would be classified as Type 2 soils while saturated soils would be classified as Type 4 soils if not adequately dewatered prior to excavation.

Based on OHSA requirements, the soil must be classified as the type with highest classification of the types of soils present if an excavation contains more than one soil type (e.g. if both Type 2 and Type 3 soils are present within the trench, the excavation must be sloped or supported in accordance with the requirements for Type 3 soils).



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

OHSA indicates that temporary excavations made within Type 3 soils that are above the water table and/or dewatered prior to excavation should be developed with side slopes no steeper than 1H:1V.

Notwithstanding the above, further flattening of the temporary excavation sideslopes may be required where localized instability/sloughing is noted (e.g. due to encountering changes in material types or perched groundwater etc.). To account for this, it is recommended for preliminary planning purposes that it is confirmed there is sufficient space to develop temporary excavation sideslopes using 1.5H:1V sideslopes.

Grading work should be carried out in accordance with OPSS.PROV 206 Construction Specification for Grading and SP 206S03. The retaining wall backfill materials should be benched into the cut slopes in accordance with OPSD 208.010.

8.4 OBSTRUCTIONS

Cobbles and/or boulders were encountered with the site soils during the investigation and therefore the presence of cobbles and boulders should be anticipated. These materials could obstruct excavations and the installation of temporary protections system components, if required. A Non-Standard Special Provision (NSSP) should be developed during the detailed design stage for inclusion in the contract to address this issue.

8.5 TEMPORARY GROUNDWATER CONTROL

Excavations for the cut slope widening and associated retaining wall construction are anticipated to be developed primarily within near-surface fill materials, and native deposits comprised of compact to very dense silt and sand and stiff to hard clayey silt. The piezometric level measured in the monitoring well installed in Borehole MW21-1 was above the surface of the highway at that location. Therefore, the excavations are expected to extend below the groundwater level.

Temporary unwatering using conventional ditching and/or sump and pump techniques is considered appropriate for shallow excavations at the site developed predominantly within the silt and clayey silt. More extensive dewatering systems could be required if excavations encounter sand to silty sand or other coarse-grained material. The requirements for unwatering/dewatering should be further reassessed during the detailed design stage once the preferred wall/slope configurations and foundation systems have been selected. Control of the surface water will also be necessary to allow excavation and foundation construction to be carried out in dry conditions. Surface water should be directed away from the area of the planned excavations.

All groundwater control systems required for the construction of the retaining walls should be designed and implemented in accordance with NSSP FOUN0003.



9.0 FURTHER WORK FOR DETAILED DESIGN

The current investigation focused on identifying the subsurface conditions in the area of the slope adjacent to the property at 164 Skyview Road which is located within the central portion of the overall slope requiring widening. All boreholes were advanced at the crest of the existing slope. Additional investigation is recommended to be carried out prior to finalizing the design of the slope widening and retaining walls. In this regard, the following foundation engineering related items should be completed prior to, or as part of, the detailed design to confirm and/or further assess the preliminary recommendations provided in this report:

- Additional subsurface investigation, and associated laboratory testing, should be completed in the area of the proposed walls/at the toe of the existing cut slope. The standard minimum MTO foundation investigation for retaining wall structures (i.e. Boreholes advanced at spacings of 50 m along the proposed retaining wall foundation elements that extend to depths of 10 m below the base of wall or 3 m below refusal, defined as material for which SPT 'N' values are greater than 100 blows per 0.3 m of penetration) is considered appropriate.
- Piezometers/monitoring wells should be installed to confirm the water level at the highway level.
- Following completion of the additional investigation and laboratory testing, the soil design parameters and analysis results outlined in this report should be re-evaluated.
- A Final Foundation Investigation and Design Report meeting MTO's standard requirements for foundation engineering assignments should be prepared based on the final slope/retaining wall configuration.

10.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 10.1: Specifications Referenced in Report

Document	Title
NSSP FOUN0003	Dewatering Structure Excavations
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3120.100	Walls – Retaining – Concrete Toe Wall
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS. PROV 539	Construction Specification for Temporary Protection System
OPSS.PROV 902	Construction Specification for Excavation and Backfilling – Structures
OPSS.PROV 1010	Material Specification for Aggregates
SP105S10	Construction Specification for Compaction
SP109S12	Amendment to OPSS 902, November 2010
SP 206S03	Earth Excavation, Grading



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED WIDENING OF DEEP CUT ADJACENT TO 164 SKYVIEW ROAD, GRAFTON, ONTARIO
HIGHWAY 401 PLANNING STUDY – COBOURG TO COLBORNE, ONTARIO**

11.0 CLOSURE

The recommendations made in this report were made based on Stantec's current understanding of the project. Stantec should be given the opportunity to review, and if necessary, revise, the recommendations contained herein when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

STANTEC CONSULTING LTD.



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12.0 REFERENCES

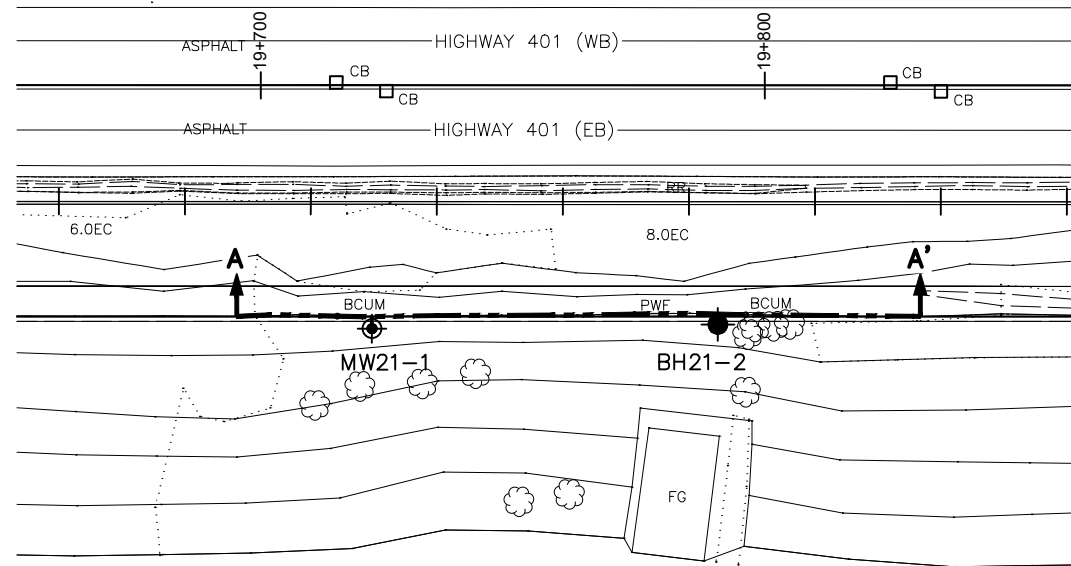
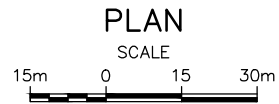
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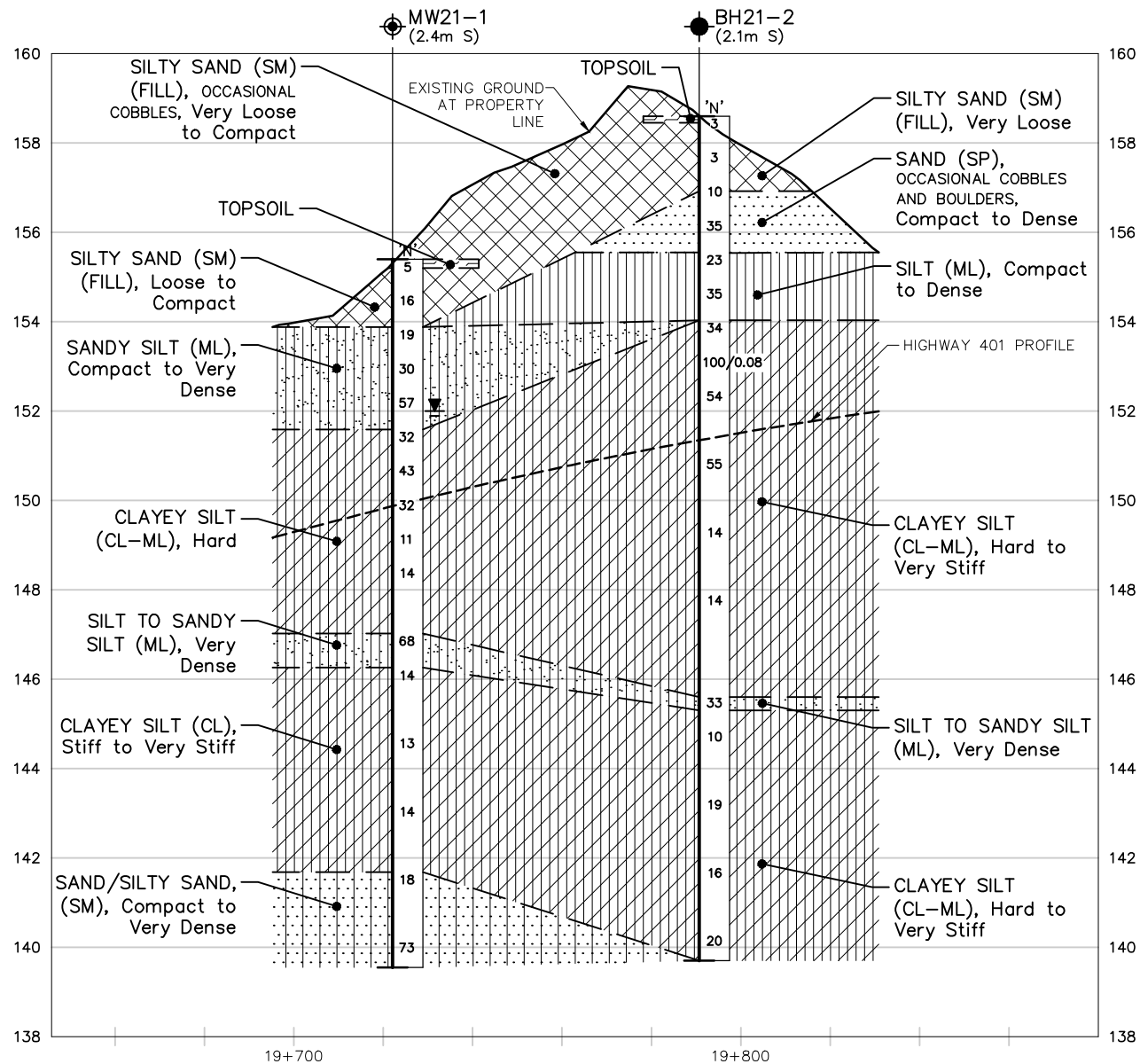
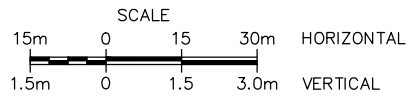
APPENDIX A

A.1 DRAWING NO. 1 – BOREHOLE LOCATIONS AND SOIL STRATA DRAWING



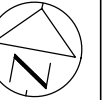


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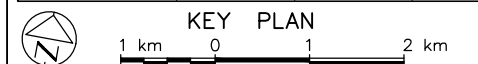
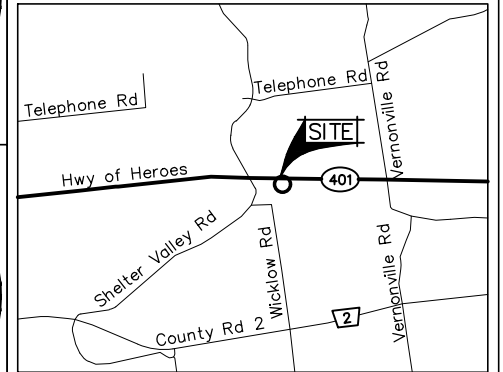
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GWP 4060-11-00



HWY 401 WIDENING
164 SKYVIEW RD., COBOURG, ON
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
—



LEGEND

- Borehole
- Monitoring Well
- (x.x m) Offset from Cross Section Line in meters
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL Measured on October 2021

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	MTM ZONE 10 COORDINATES EAST
MW21-1	155.4	4 875 785.2	426 494.7
BH21-2	158.6	4 875 798.5	426 562.0

NOTES

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

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REVISIONS	DATE	BY	DESCRIPTION

GEOGRES No 31C-315			
HWY No 401		DIST	
SUBM'D KN	CHECKED	DATE 2023-01-16	SITE
DRAWN GBB	CHECKED	APPROVED RH	DWG 1

APPENDIX B

B.1 SYMBOLS AND TERMS USED ON STANTEC BOREHOLE RECORDS

B.2 BOREHOLE RECORDS

B.3 AVAILABLE GEOCRES INFORMATION



SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Rootmat</i>	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
<i>Very Soft</i>	<0.25	<12.5	<2
<i>Soft</i>	0.25 - 0.5	12.5 - 25	2-4
<i>Firm</i>	0.5 - 1.0	25 - 50	4-8
<i>Stiff</i>	1.0 - 2.0	50 - 100	8-15
<i>Very Stiff</i>	2.0 - 4.0	100 - 200	15-30
<i>Hard</i>	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	Very Poor Quality
25-50	Poor Quality
50-75	Fair Quality
75-90	Good Quality
90-100	Excellent Quality

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	Crushed
Severely Fractured	Shattered or Very Blocky
Fractured	Blocky
Moderately Jointed	Sound
Intact	Very Sound

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	Extremely Wide	-
2000-6000	Very Wide	Very Thick
600-2000	Wide	Thick
200-600	Moderate	Medium
60-200	Close	Thin
20-60	Very Close	Very Thin
<20	Extremely Close	Laminated
<6	-	Thinly Laminated

Terminology describing rock strength:

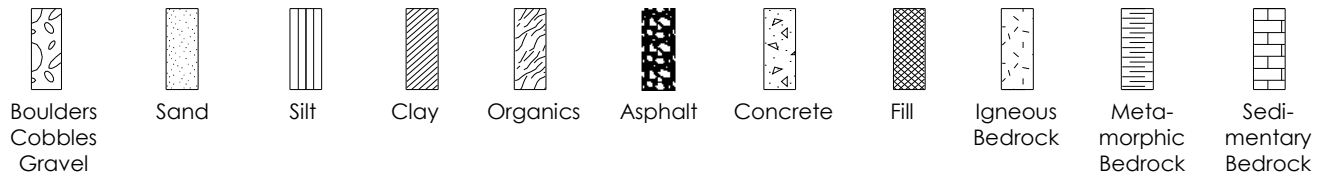
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

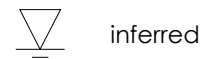
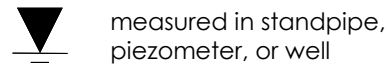
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT



RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
y	Unit weight
G _s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q _u	Unconfined compression
I _p	Point Load Index (I _p on Borehole Record equals I _p (50) in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

RECORD OF BOREHOLE No MW21-1

1 OF 2

METRIC

W.P. GWP 4060-11-00 LOCATION Highway 401 East of Shelter Valley Road N:4875785.2 E:426494.7 ORIGINATED BY KL
 DIST East HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY RR
 DATUM Geodetic DATE 2021.10.18 - 2021.10.18 LATITUDE 44.011973 LONGITUDE -77.982276 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE							w _p w w _L		
								● QUICK TRIAXIAL × LAB VANE									
						20	40	60	80	100	20	40	60				
155.4																	
150.0	200 mm TOPSOIL																
0.2	Silty SAND (SM), trace to some gravel and rootlets (FILL). Occasional cobbles. Loose to compact Brown Moist Auger grinding at 0.76 m		1	SS	5												
			2	SS	16												
153.9																	
1.5	Sandy SILT (ML), trace clay Compact to very dense Grey Moist to wet		3	SS	19												
			4	SS	30												
			5	SS	57												
151.6																	
3.8	CLAYEY SILT (CL-ML) Hard Grey Moist 50 mm wet sandy silt seam at 5.8 m Becomes very stiff below 6 m		6	SS	32												
			7	SS	43												
			8	SS	32												
			9	SS	11												
			10	SS	14												
147.0																	
8.4	SILT (ML), some clay, trace sand. Contains silty clay seams/layers. Very dense Grey Wet		11	SS	68												
146.3																	
9.1	CLAYEY SILT (CL) Stiff to very stiff Grey Moist		12	SS	14												
			13	SS	13												
			14	SS	14												
141.7																	
13.7	SAND (SM), some silt to silty, some gravel, trace clay		15	SS	18												

Continued Next Page

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

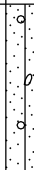
ONTARIO MTO 165001231_Hwy_401_COBOURG.GPJ ONTARIO MTO.GDT 7/18/22

RECORD OF BOREHOLE No MW21-1

2 OF 2

METRIC

W.P. GWP 4060-11-00 LOCATION Highway 401 East of Shelter Valley Road N:4875785.2 E:426494.7 ORIGINATED BY KL
 DIST East HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY RR
 DATUM Geodetic DATE 2021.10.18 - 2021.10.18 LATITUDE 44.011973 LONGITUDE -77.982276 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR	SA	SI	CL
								20	40	60	80	100					○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL				
	SAND (SM), some silt to silty, some gravel, trace clay Compact to very dense Grey Wet (<i>continued</i>)						141																
139.5			16	SS	73		140										18	61 15 6					
15.9	End of Borehole Water level recorded at 5.8 m (~Elev. 149.6 m) below ground surface on October 19, 2021; at 3.2 m (~Elev. 152.2 m) on October 20, 2021 and at 3.4 m (~Elev. 152.0 m) on October 21, 2021. Shear vane testing conducted in borehole located approximately 0.5m east of MW21-1.																						

RECORD OF BOREHOLE No BH21-2

1 OF 2

METRIC

W.P. GWP 4060-11-00 LOCATION Highway 401 East of Shelter Valley Road N:4875798.5 E:426562.0 ORIGINATED BY KL
DIST East HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY RR
DATUM Geodetic DATE 2021.10.20 - 2021.10.21 LATITUDE 44.012081 LONGITUDE -77.981434 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
158.6	150 mm TOPSOIL		1	SS	3		158							
158.6 0.2	Silty SAND (SM), trace clay and rootlets (FILL). Occasional cobbles. Very loose Brown Moist Auger grinding at 0.46 m		2	SS	3									
156.9			3	SS	10		157							
1.7	SAND (SP), some silt and gravel/rock fragments. Occasional cobbles and/or boulders. Compact to dense Brown Moist Grinding of augers noted at 2 m and 2.6 m depths. No recovery in SS4		4	SS	35		156							
155.6			5	SS	23		155							
3.1	SILT (ML), some sand and clay Compact to dense Brown to grey Moist to wet		6	SS	35									
154.0			7	SS	34		154							
4.6	CLAYEY SILT (CL-ML), trace sand Hard Brown to grey Moist SS8 dry to moist		8	SS	100/0.08		153							
	Occasional moist to wet sandy silt/silt seams below 6.1 m		9	SS	54		152							
			10	SS	55		151							
			11	SS	14		149							
	Becomes very stiff below 9 m		12	SS	14		148							
							147							
	Attempted taking a Shelby tube sample from 12.2 m to 12.8 m. Tube unable to be retrieved.						146							
145.6			13	SS	33		145							
13.0	Sandy SILT (ML) zone encountered at 13.0 m depth													
145.3														
13.3	CLAYEY SILT (CL-ML), trace sand Very stiff Brown to grey Moist		14	SS	10									
144.3														

Continued Next Page

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

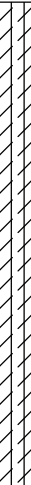
ONTARIO MTO 165001231_Hwy_401_COBOURG.GPJ ONTARIO MTO.GDT 7/18/22

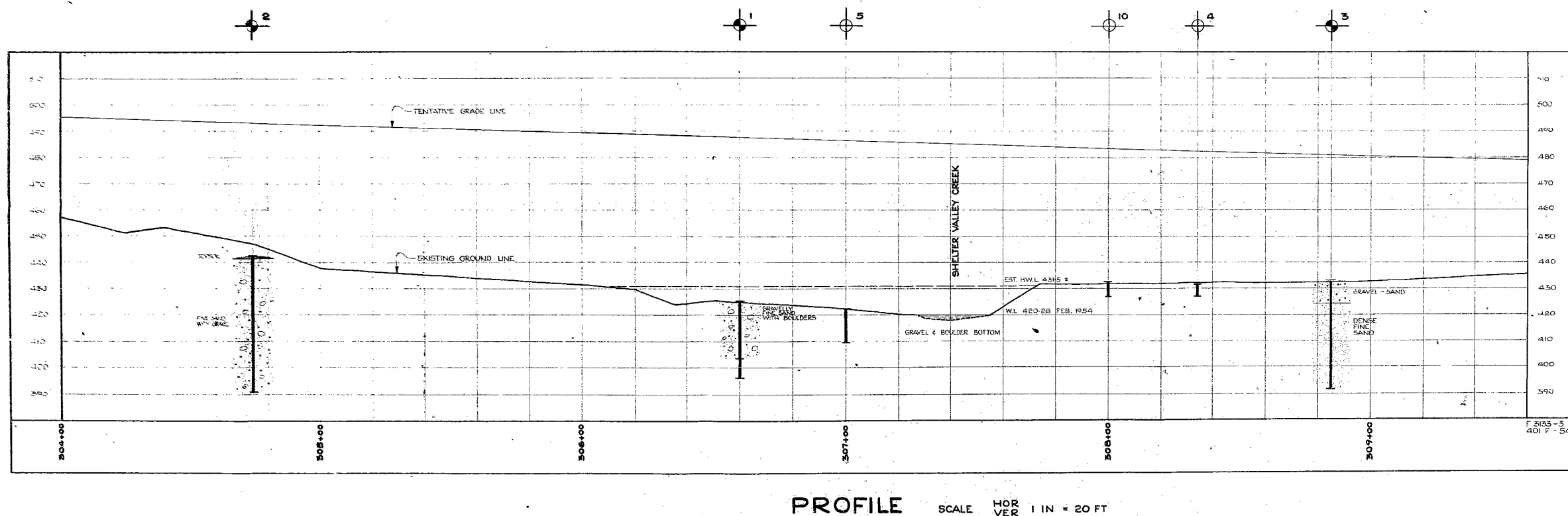
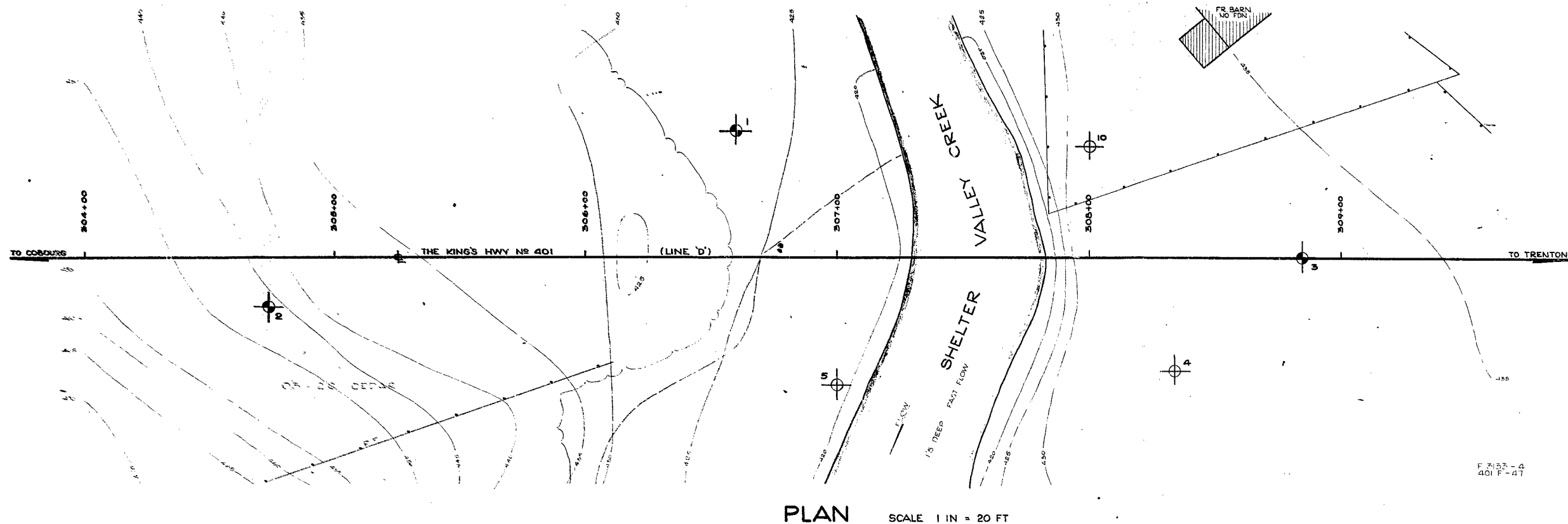
RECORD OF BOREHOLE No BH21-2

2 OF 2

METRIC

W.P. GWP 4060-11-00 LOCATION Highway 401 East of Shelter Valley Road N:4875798.5 E:426562.0 ORIGINATED BY KL
 DIST East HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY RR
 DATUM Geodetic DATE 2021.10.20 - 2021.10.21 LATITUDE 44.012081 LONGITUDE -77.981434 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
							20 40 60 80 100				W _p W W _L				
14.3	CLAYEY SILT (CL), trace sand, occasional to frequent wet silt seams Very stiff Grey Wet						144							Su > 118 kPa (N-vane refusal)	
			15	SS	19		143								
							142								
			16	SS	16		141								
							140								
139.7 18.9	End of Borehole Groundwater observed at ~9.1 m depth (~Elev. 149.5 m) during drilling. New borehole drilled approximately 1.8 m east of BH21-2. Sampling and testing below 12.2 m was conducted in adjacent borehole.		17	SS	20										



LEGEND			
BORE HOLES			
PENETRATION HOLE			
BORE & PENETRATION HOLE			
HOLE NO.	ELEVATION	STATION	DISTANCE FROM #
1	425.65	306+60'	50' LT
2	442.5	304+74'	20' RT
3	437.25	308+65'	4
4	431.4	308+34'	45 RT
5	422.5	307+00'	51' RT
10	432.6	308+00'	44' LT

NOTE

THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION - DOWNSVIEW

**SHELTER VALLEY CREEK
PROPOSED CROSSING
2 MILES N.E. OF GRAFTON**
SHOWING POSITION & ELEVATION OF HOLES

HWY. NO. 401 (LINE 'D') W.P. 55-57 DIV. NO. 7
CO. NORTHUMBERLAND
TWP. HALDIMAND LOT. 14 CON. 1

SCALE AS SHOWN	SUBMITTED BY	DATE 30 SEPT. 57
DRAWN BY R.E.F.	APPROVED BY	DRAWING NO. F-57-27A

DRILL RIG 541 OPERATION BORE & PENETIN JOB F 57-27 W.P. 55 57 BORING 3 STA. 308 +85 ON 4
CASING BA (standard samplers to fit unless noted) DATUM GEODINIC DATE REPORT SEPT. 1957
SAMPLER HAMMER WT. 350 LBS. DROP 40 INCHES COMPILED BY H.S. CHECKED BY AL DATE BORING 31 JULY 1957

SAMPLE TYPES

SAMPLE CONDITION

V - INSITU VANE SHEAR TEST	Q - TRIAXIAL QUICK	K - PERMIABILITY	C.S. - CHUNK	S.S. - SLEEVE SAMPLE
M - MECHANICAL ANALYSIS	S - TRIAXIAL SLOW	C - CONSOLIDATION	D.O. - DRIVE OPEN	P.S. - PISTON SAMPLE
U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	CA - CASING	D.F. - DRIVE FOOT VALVE	WS - WASHED SAMPLE
Q _c - TRIAXIAL CONSOLIDATED QUICK	WT - WATER TABLE IN SOIL	γ - UNIT WEIGHT	T.O. - THIN WALLED OPEN	R.C. - ROCK CORE



- DISTURBED
- FAIR
- GOOD
- LOST

SAMPLES

ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W %			O - NAT			□ - P.W.			△ - L.W.			CASING BLOW (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
					PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. ●-----●-----●																		
433.25 433.25		GROUND LEVEL																			433.25 433.25		
3.5		GRAVEL - SAND																			428.25 430.25		
5.5	424.25 424.25 9.3'																				423.25 423.25		
13.5																					418.25 420.25		
18.5																					413.25 415.25		
23.5		DENSE FINE SAND																			408.25 410.25		
28.5																					403.25 405.25		
33.5																					398.25 400.25		
38.5																					393.25 395.25		
43.5	391.75 391.75 41.5'	END OF BOREHOLE																			388.25 390.25		

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 5- OPERATION PENETRATION JOB F-37-27 WP. 55-57 BORING 4 STA. 308+34 (45' RT.)
CASING 31 (standard samplers to fit unless noted) DATUM GLIODETIC DATE REPORT SEPT. 1957
SAMPLER HAMMER WT. 25 LBS. DROP 19 INCHES COMPILED BY 45 CHECKED BY 4.L. DATE BORING 30 JULY 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST	Q - TRIAXIAL QUICK	K - PERMIABILITY
M - MECHANICAL ANALYSIS	S - TRIAXIAL SLOW	C - CONSOLIDATION
U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	CA - CASING
Q _c - TRIAXIAL CONSOLIDATED QUICK	WT - WATER TABLE IN SOIL	γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK	S.S. - SLEEVE SAMPLE
D.O. - DRIVE OPEN	P.S. - PISTON SAMPLE
D.F. - DRIVE FOOT VALVE	W.S. - WASHED SAMPLE
T.O. - THIN WALLED OPEN	R.C. - ROCK CORE

SAMPLE CONDITION



- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE

ELEVATION DEPTH		WATER CONDITIONS	DESCRIPTION	STRAT PLOT	ELEVATION SCALE	WATER CONTENT W %	0 - NAT.	□ - P.W.	△ - L.W.	CASING BLOW (ACTUAL)	OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOV. %
						PENETRATION TEST RESISTANCE BLOWS PER FOOT AT STANDARD ENERGY (4200 IN. LBS. PER BLOW) D. CONE PEN. X-----X-----X STAND. PEN. •-----•-----•										
421			GROUND LEVEL		430											
426					426	REFUSAL AT ELEV. 427' 427' HAMMER BOUNCING ON BOULDERS.										

DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL NO. 54-11 OPERATION PENETRATION JOB F-57-27 WP. 55-57 BORING 10 STA. 308+00 (44' LT)
 CASING BK (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT SEPT. 1957
 SAMPLER HAMMER WT. 350 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY AL DATE BORING 14 AUG. 1957

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
 QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

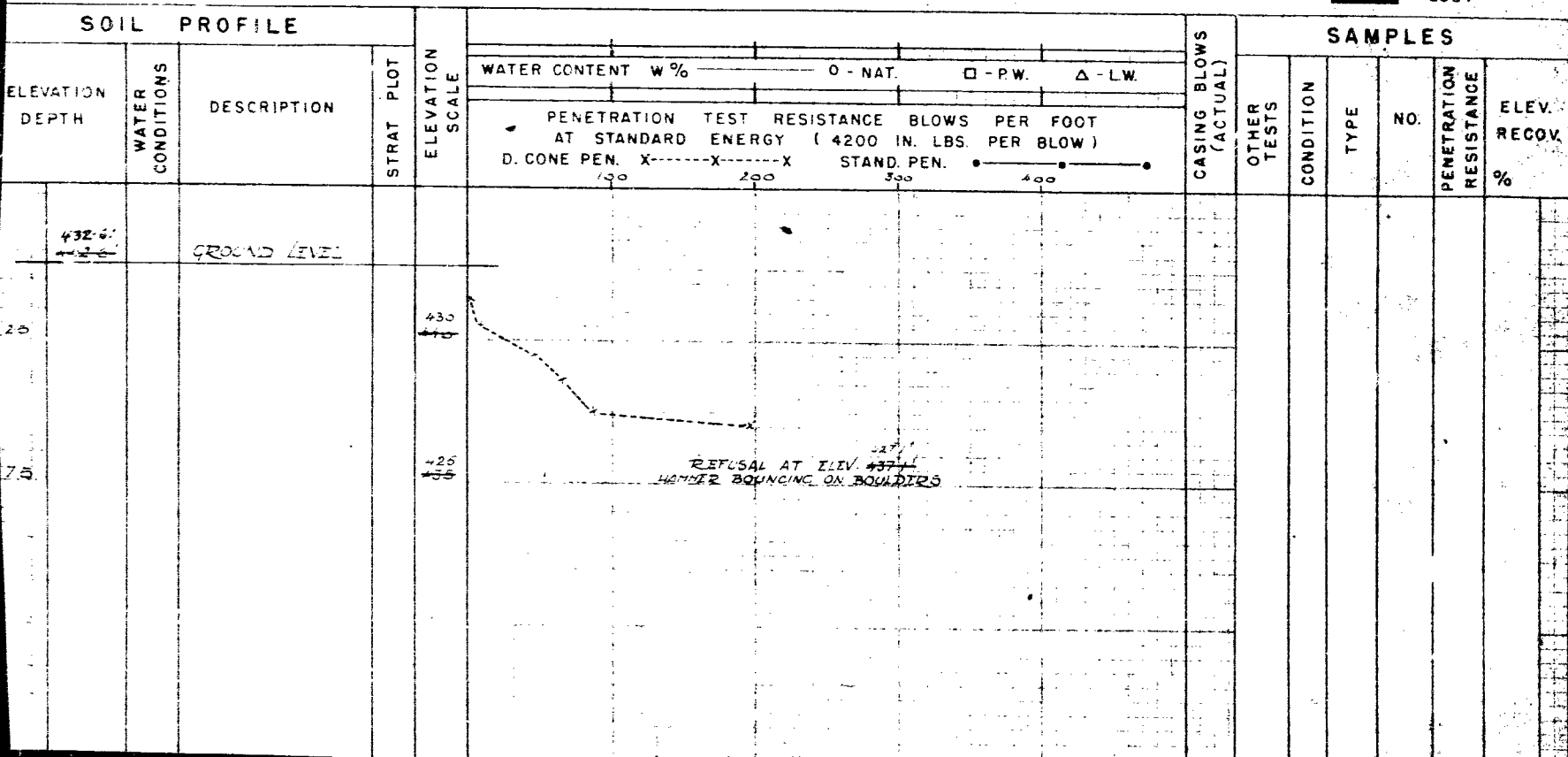
CS - CHUNK SS - SLEEVE SAMPLE
 DO - DRIVE OPEN PS - PISTON SAMPLE
 DF - DRIVE FOOT VALVE WS - WASHED SAMPLE
 TO - THIN WALLED OPEN RC - ROCK CORE

SAMPLE CONDITION



- DISTURBED
 - FAIR
 - GOOD
 - LOST

SOIL PROFILE



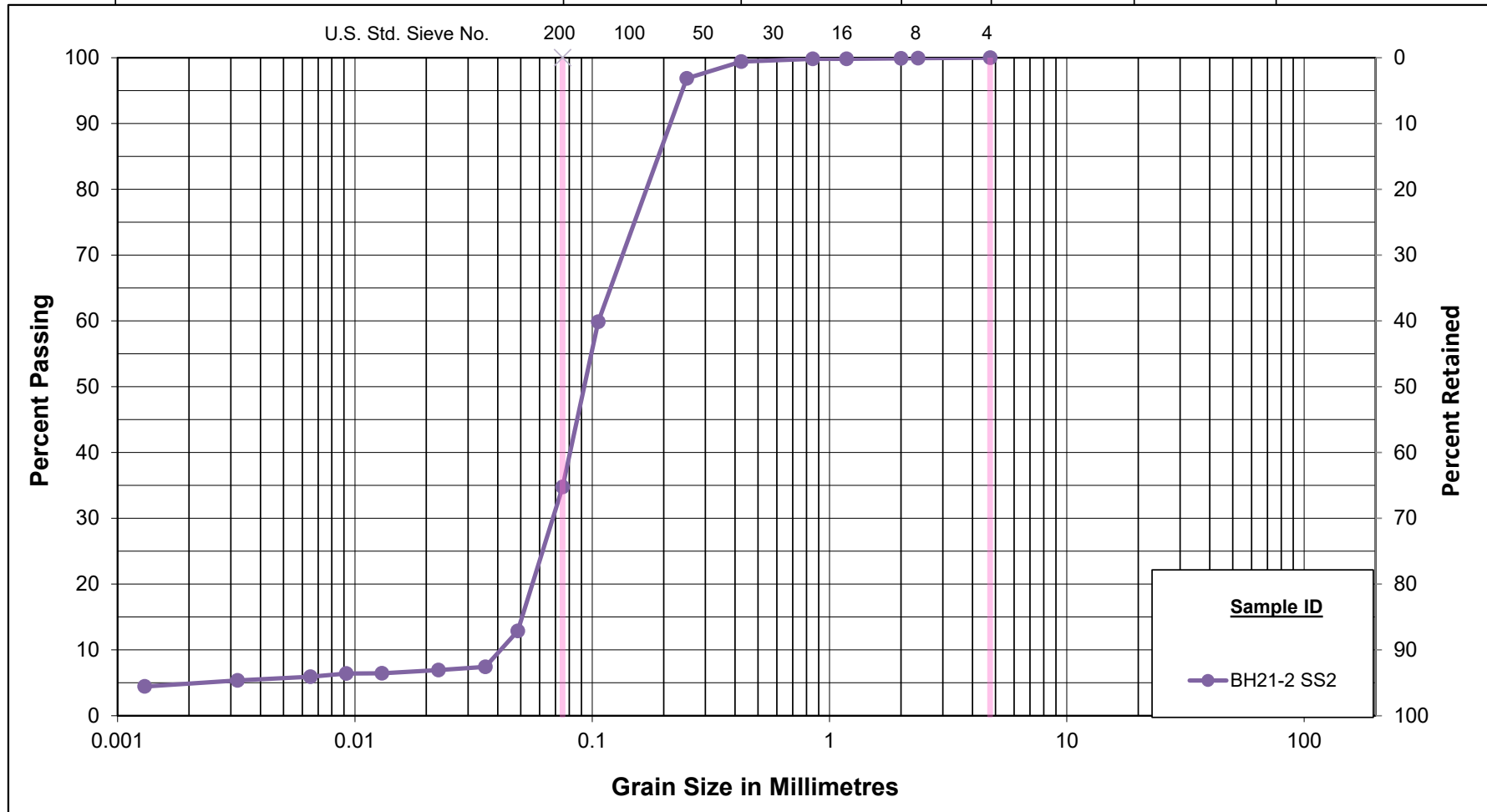
APPENDIX C

C.1 LABORATORY TEST RESULTS



Unified Soil Classification System

CLAY & SILT			SAND			Gravel	
			Fine	Medium	Coarse	Fine	Coarse



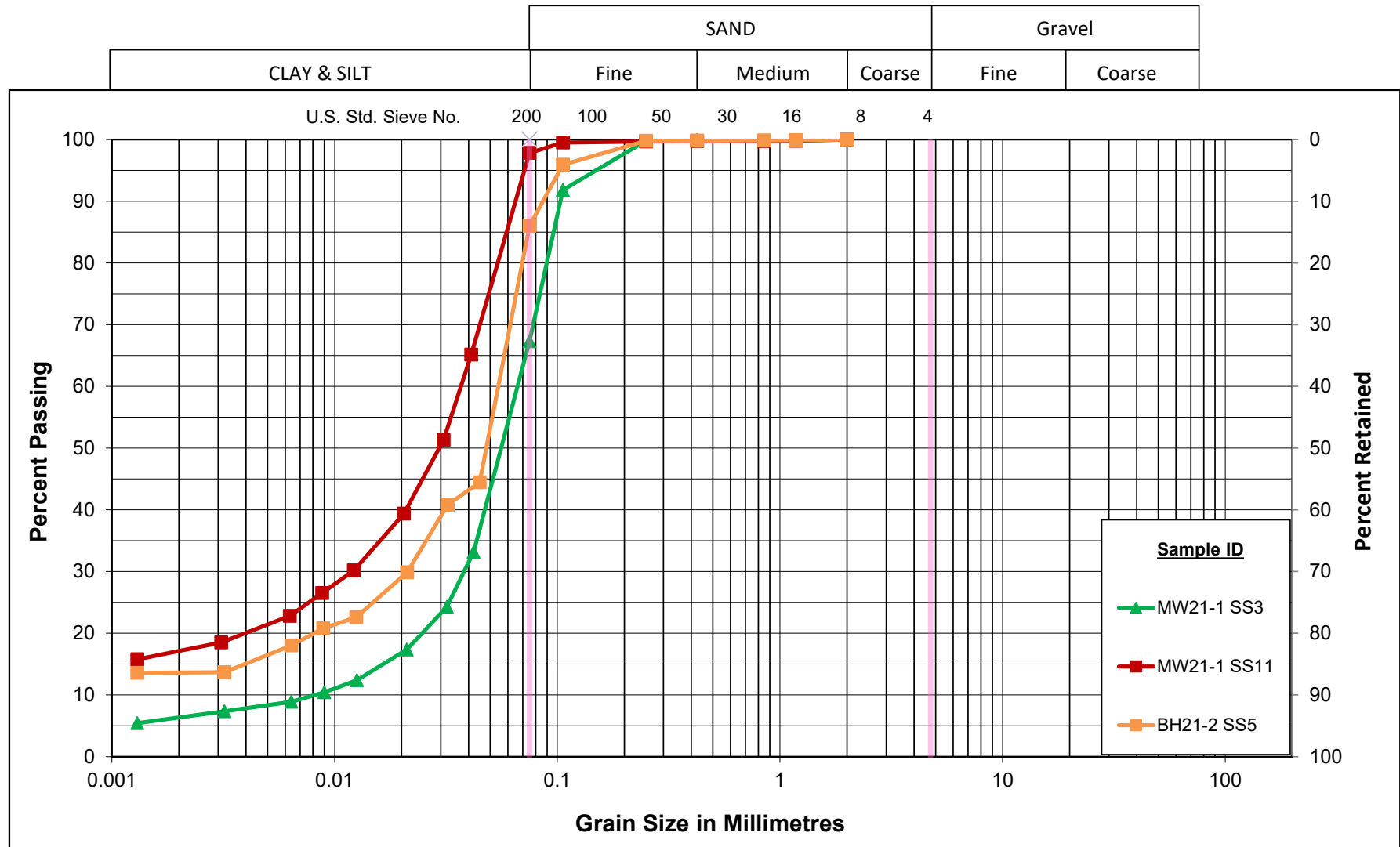
GRAIN SIZE DISTRIBUTION

FILL: Silty SAND (SM), trace clay
Highway 401 Cobourg to Colborne

Figure No. C1

Project No. 165001231

Unified Soil Classification System

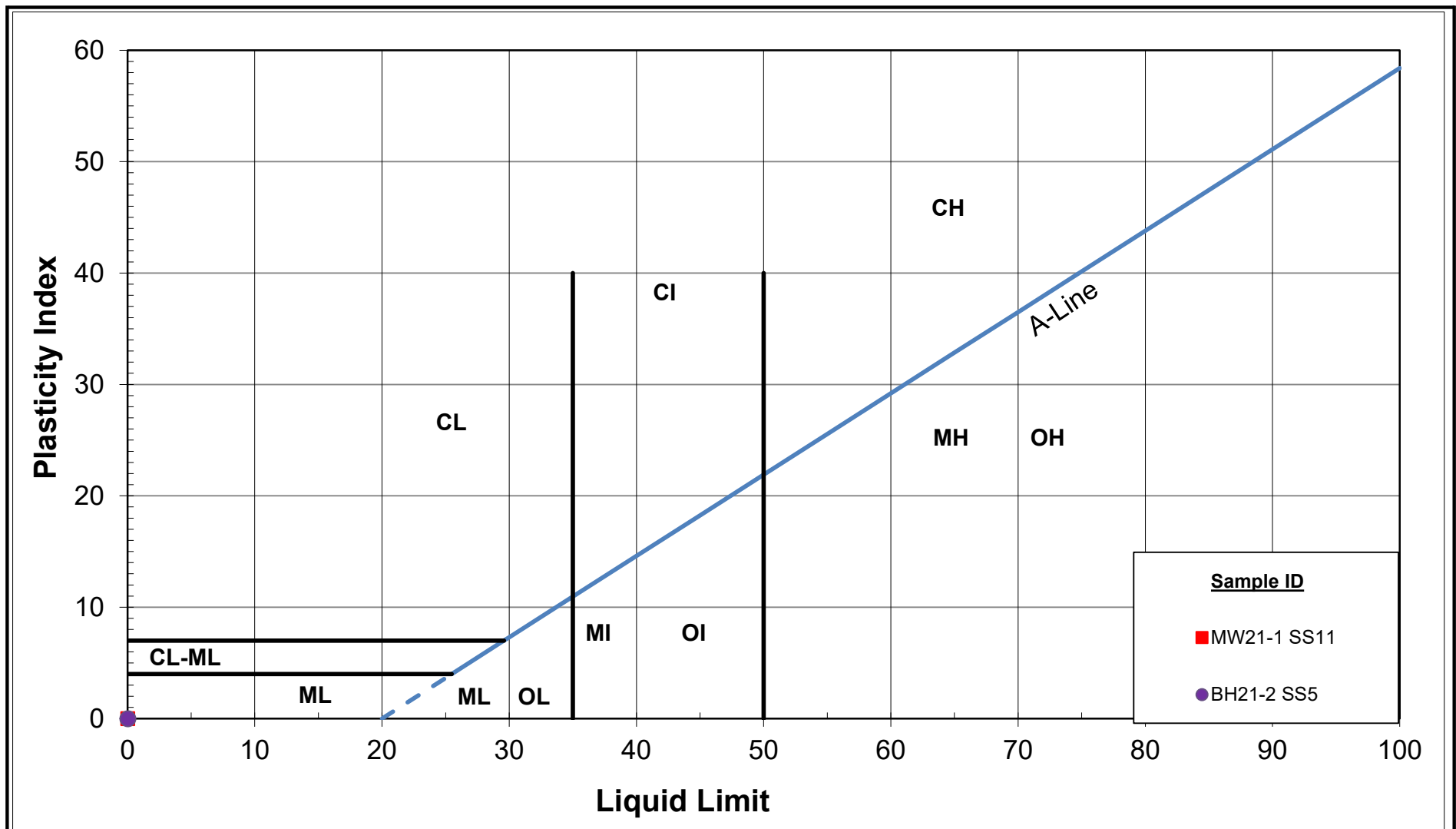


GRAIN SIZE DISTRIBUTION

SILT (ML), trace sand to sandy
Highway 401 Cobourg to Colborne

Figure No. C2

Project No. 165001231



SILT (ML), trace to some sand, some clay

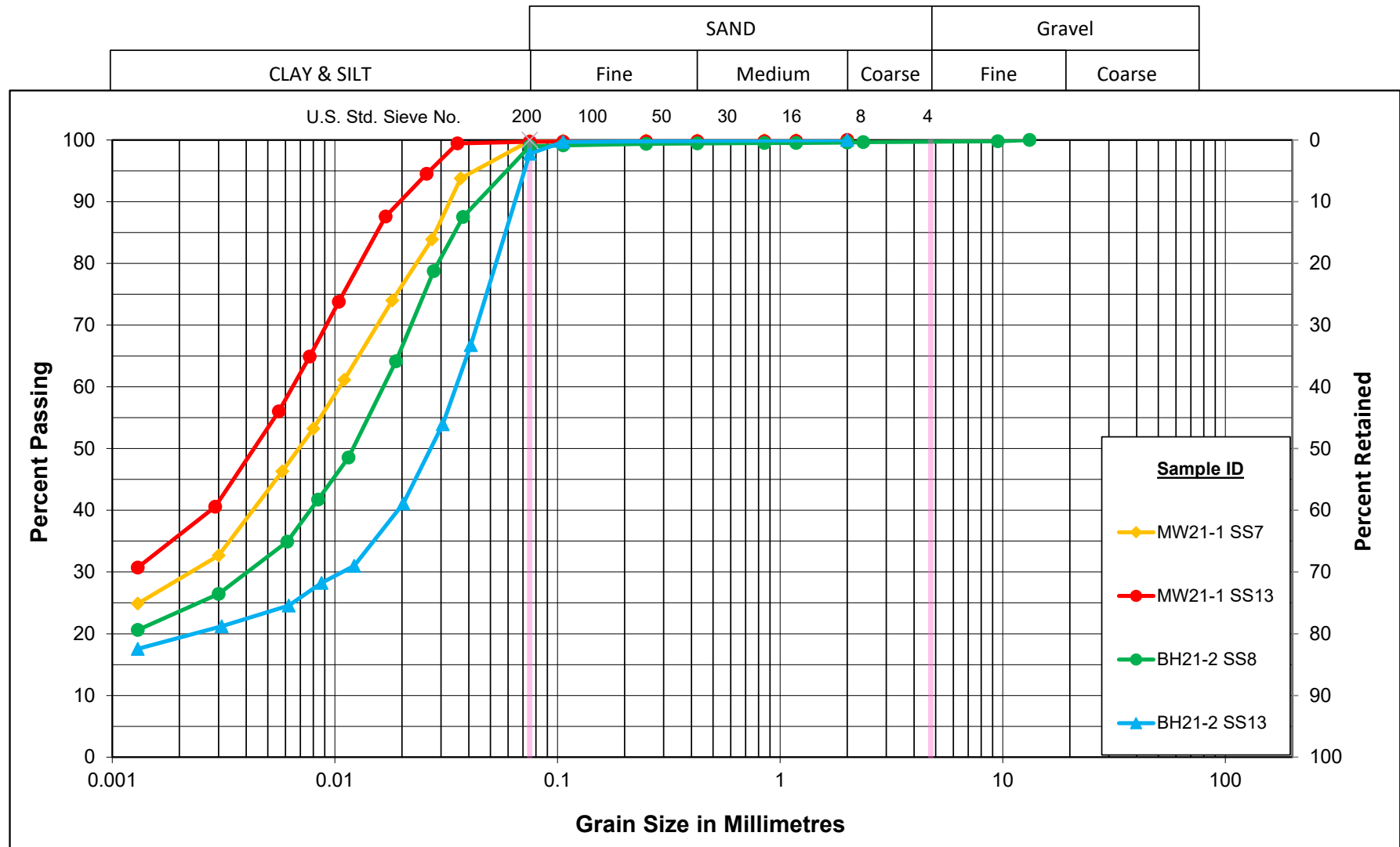
Highway 401 Cobourg to Colborne

PLASTICITY CHART

Figure No. C3

Project No. 165001231

Unified Soil Classification System

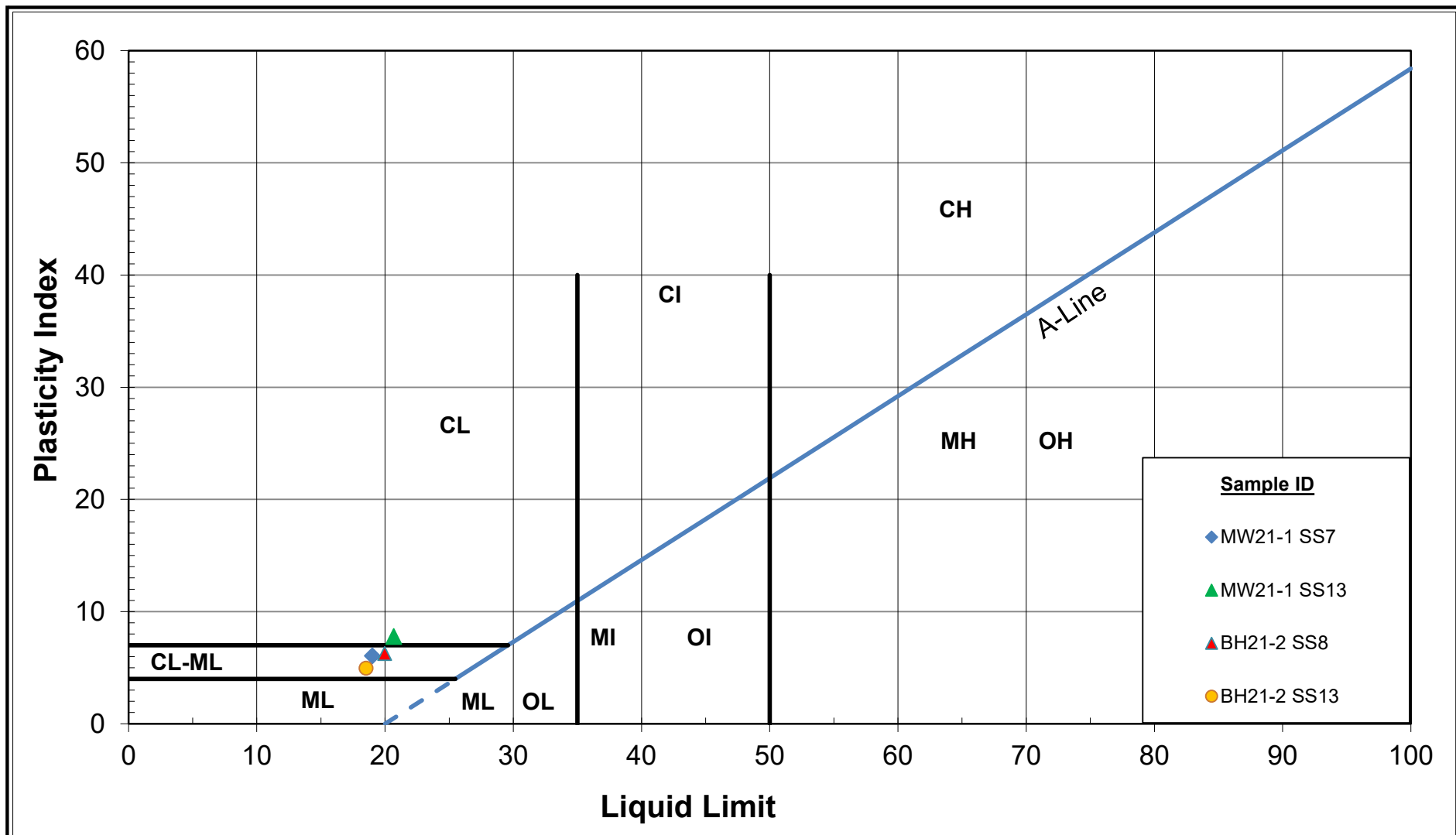


GRAIN SIZE DISTRIBUTION

CLAYEY SILT (CL to CL-ML)
Highway 401 Cobourg to Colborne

Figure No. C4

Project No. 165001231



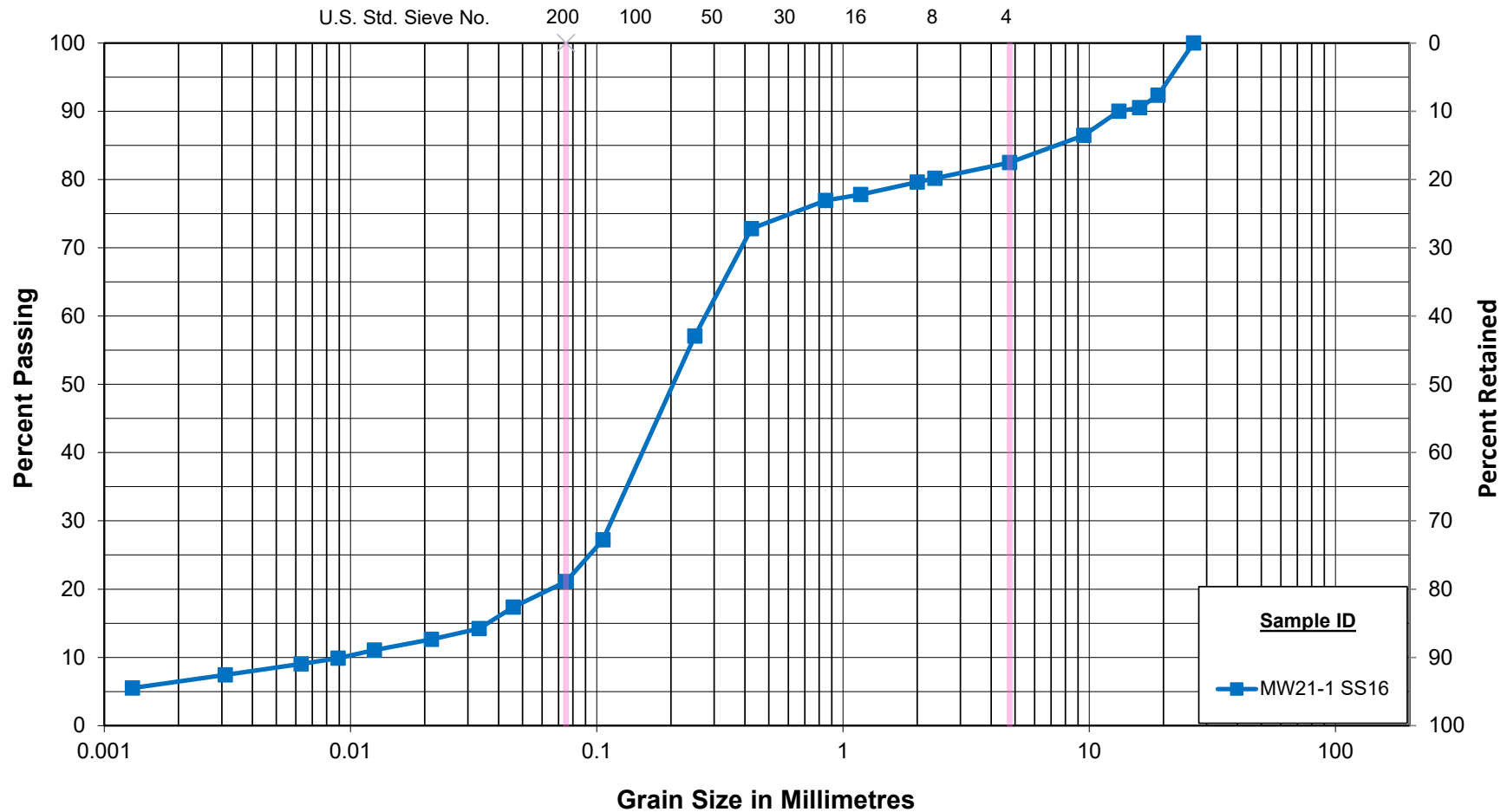
CLAYEY SILT (CL to CL-ML)
 Highway 401 Cobourg to Colborne
PLASTICITY CHART

Figure No. C5

Project No. 165001231

Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION

SAND (SM), some silt, some gravel, trace clay

Highway 401 Cobourg to Colborne

Figure No. C6

Project No. 165001231



Certificate of Analysis

AGAT WORK ORDER: 22T861910

PROJECT: 165001231.309

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: STANTEC CONSULTING LTD

SAMPLING SITE:

ATTENTION TO: Nabeel Basheer

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2022-02-09

DATE REPORTED: 2022-02-15

				MW21-1, SS-10	BH21-2, SS-6	BH21-6, SS-10	BH21-7, SS-10
SAMPLE DESCRIPTION:				22'6"-24'6"	12'6"-14'6"	25'-27'	25'-27'
SAMPLE TYPE:				Soil	Soil	Soil	Soil
DATE SAMPLED:				2021-10-18	2021-10-20	2022-02-02	2022-02-03
Parameter	Unit	G / S	RDL	3496115	3496117	3496118	3496119
Chloride (2:1)	µg/g		2	5	12	7	4
Sulphate (2:1)	µg/g		2	6	5	3	136
pH (2:1)	pH Units		NA	8.42	8.63	8.39	8.20
Electrical Conductivity (2:1)	mS/cm		0.005	0.120	0.115	0.130	0.268
Resistivity (2:1) (Calculated)	ohm.cm		1	8330	8700	7690	3730
Redox Potential 1	mV		NA	405	406	382	373
Redox Potential 2	mV		NA	406	407	384	374
Redox Potential 3	mV		NA	408	408	385	375

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

3496115-3496117 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter. Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results. Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement. Samples were received and analyzed beyond recommended hold time.

3496118-3496119 EC, pH, Chloride and Sulphate were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil). Resistivity is a calculated parameter. Redox potential measured on as received sample. Due to the potential for rapid change in sample equilibrium chemistry with exposure to oxidative/reduction conditions laboratory results may differ from field measured results. Redox potential measurement in soil is quite variable and non reproducible due in part, to the general heterogeneity of a given soil. It is also related to the introduction of increased oxygen into the sample after extraction. The interpretation of soil redox potential should be considered in terms of its general range rather than as an absolute measurement.

Analysis performed at AGAT Toronto (unless marked by *)

Certified By:

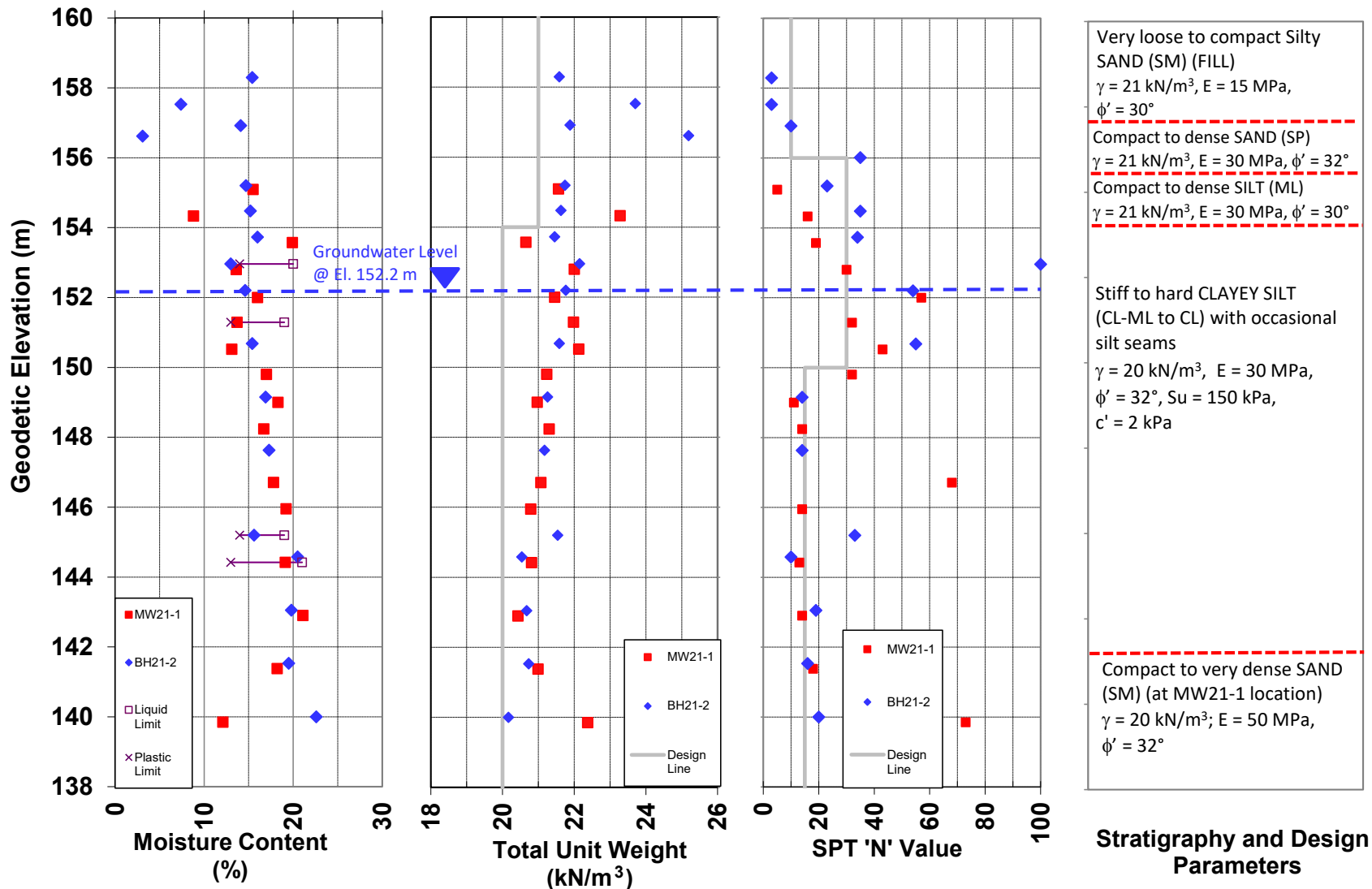
Anamjot Bhela


APPENDIX D

D.1 GEOTECHNICAL MODEL

D.2 SLOPE STABILITY ANALYSES





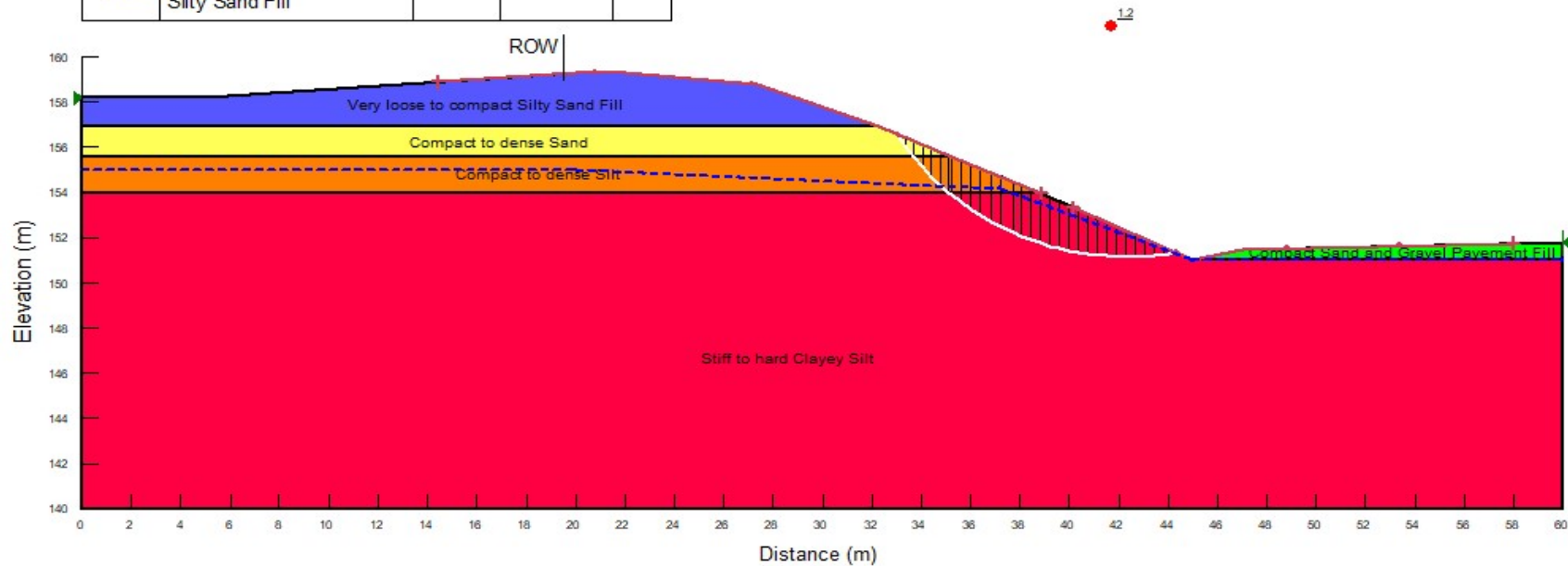
Stantec

Stantec Consulting Ltd.

Geotechnical Model
 164 Skyway Road
 Hwy 401 Planning Study - Cobourg to

Figure No. D1

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
■	Compact Sand and Gravel Pavement Fill	21	0	35
■	Compact to dense Sand	21	0	32
■	Compact to dense Silt	21	0	30
■	Stiff to hard Clayey Silt	20	2	32
■	Very loose to compact Silty Sand Fill	21	0	30



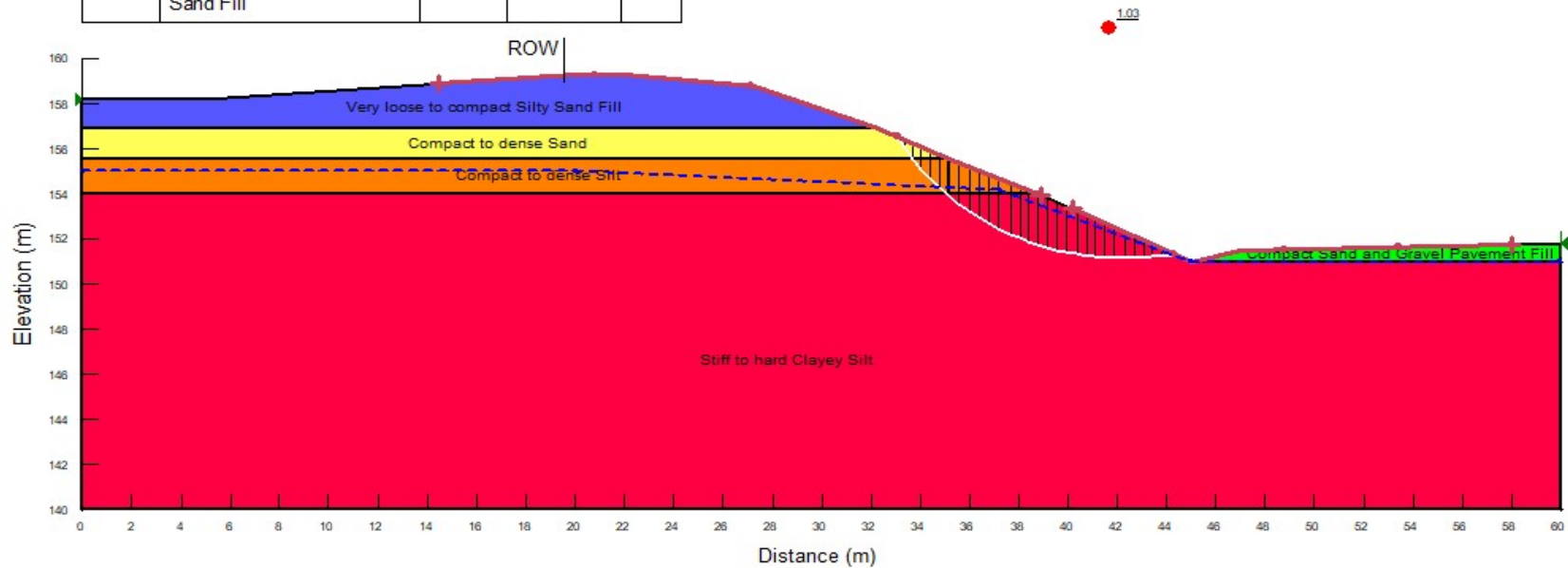
Slope Stability Analysis (Static)
Existing Slope - Station 19+780
Highway 401, East of Shelter Valley Road

Figure No. D2

Project No. 165001231

GWP No. 4060-11-00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
■	Compact Sand and Gravel Pavement Fill	21	0	35
■	Compact to dense Sand	21	0	32
■	Compact to dense Silt	21	0	30
■	Stiff to hard Clayey Silt	20	2	32
■	Very loose to compact Silty Sand Fill	21	0	30



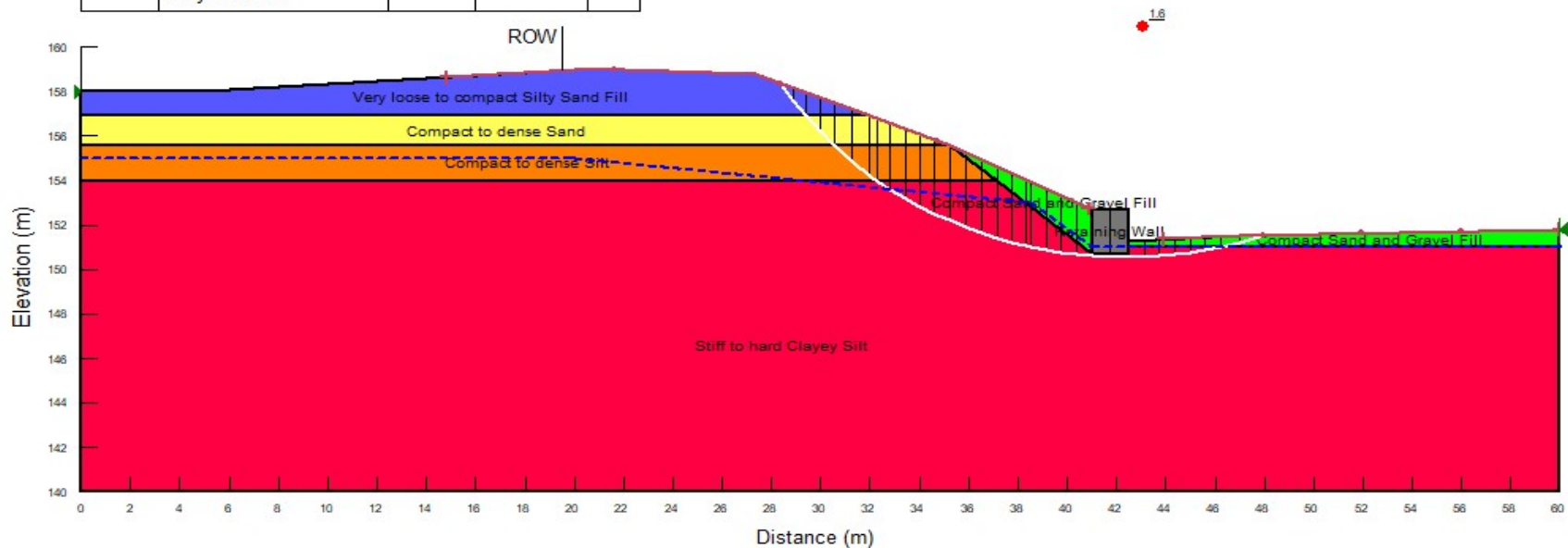
Slope Stability Analysis (Seismic)
Existing Slope - Station 19+780
Highway 401, East of Shelter Valley Road

Figure No. D3

Project No. 165001231

GWP No. 4060-11-00

Color	Name	Unit Weight (kN/m³)	Cohesion* (kPa)	Phi* (°)
■	Compact Sand and Gravel Fill	21	0	35
■	Compact to dense Sand	21	0	32
■	Compact to dense Silt	21	0	30
■	Retaining Wall	24		
■	Stiff to hard Clayey Silt	20	2	32
■	Very loose to compact Silty Sand Fill	21	0	30









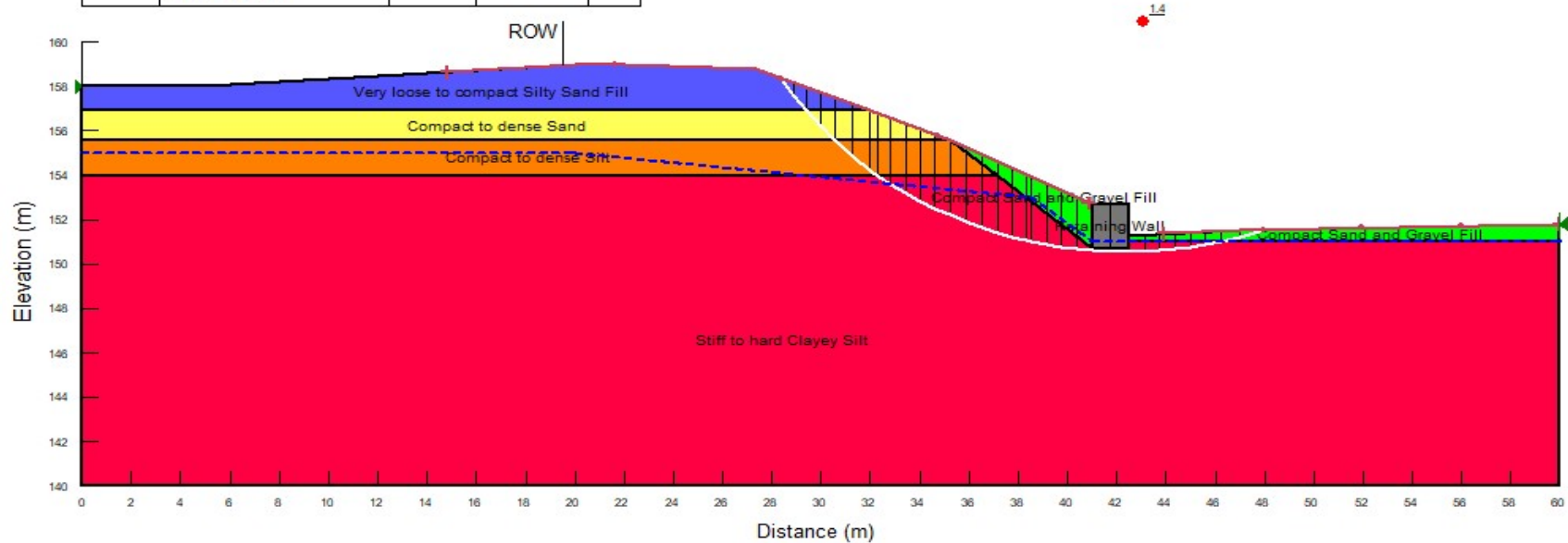
Slope Stability Analysis (Static)
Toe Wall at Highway Shoulder (Option 2)
Highway 401, East of Shelter Valley Road

Figure No. D4

Project No. 165001231

GWP No. 4060-11-00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Compact Sand and Gravel Fill	21	0	35
	Compact to dense Sand	21	0	32
	Compact to dense Silt	21	0	30
	Retaining Wall	24		
	Stiff to hard Clayey Silt	20	2	32
	Very loose to compact Silty Sand Fill	21	0	30









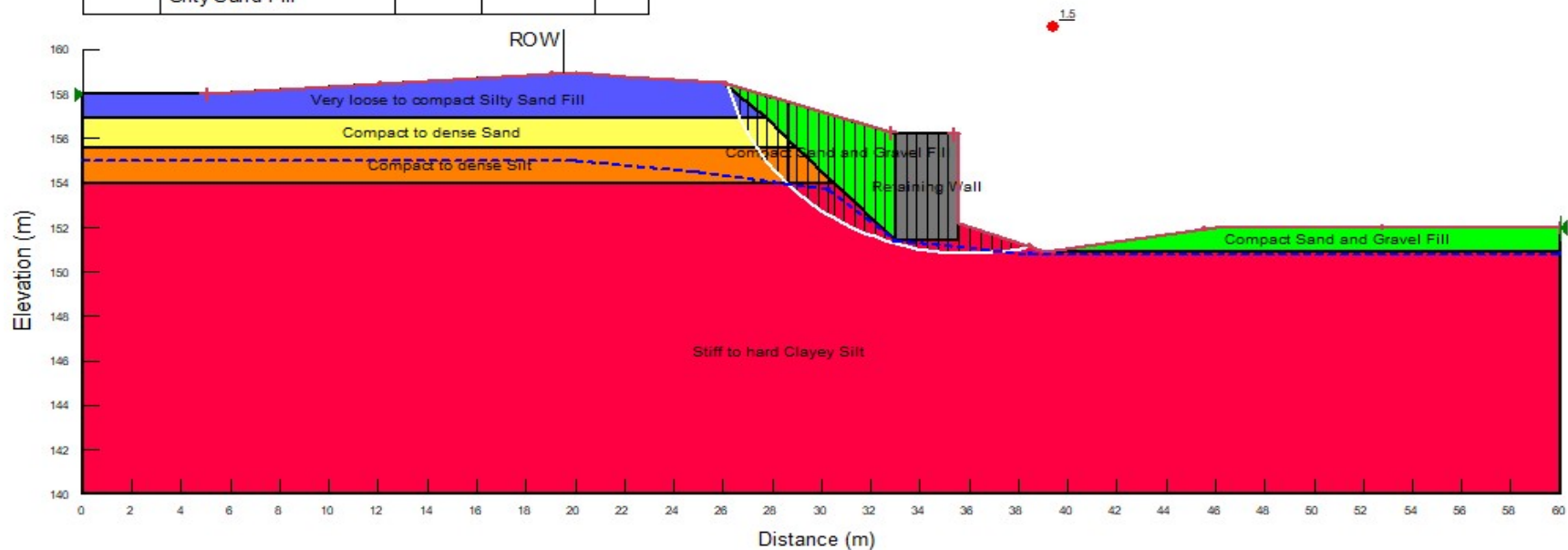
Slope Stability Analysis (Seismic)
Toe Wall at Highway Shoulder (Option 2)
Highway 401, East of Shelter Valley Road

Figure No. D5

Project No. 165001231

GWP No. 4060-11-00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Compact Sand and Gravel Fill	21	0	35
	Compact to dense Sand	21	0	32
	Compact to dense Silt	21	0	30
	Retaining Wall	24		
	Stiff to hard Clayey Silt	20	2	32
	Very loose to compact Silty Sand Fill	21	0	30









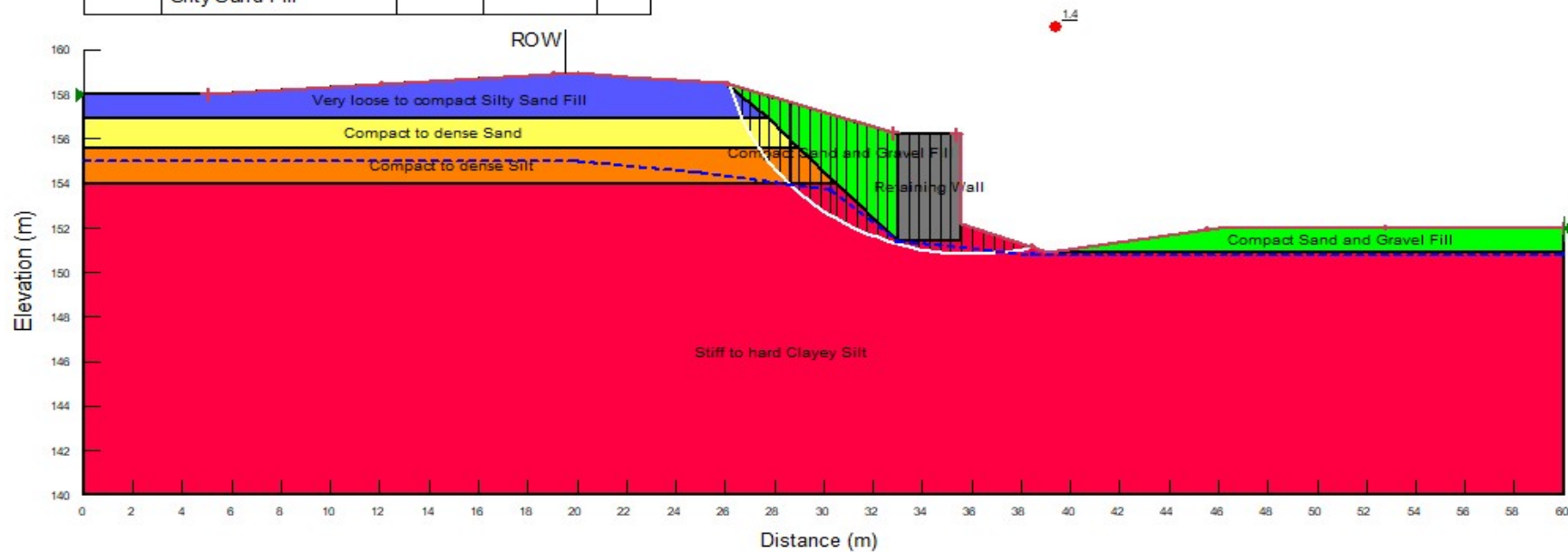
Slope Stability Analysis (Static)
RSS Wall at Clear Zone (Option 3)
Highway 401, East of Shelter Valley Road

Figure No. D6

Project No. 165001231

GWP No. 4060-11-00

Color	Name	Unit Weight (kN/m³)	Cohesion* (kPa)	Phi* (°)
	Compact Sand and Gravel Fill	21	0	35
	Compact to dense Sand	21	0	32
	Compact to dense Silt	21	0	30
	Retaining Wall	24		
	Stiff to hard Clayey Silt	20	2	32
	Very loose to compact Silty Sand Fill	21	0	30



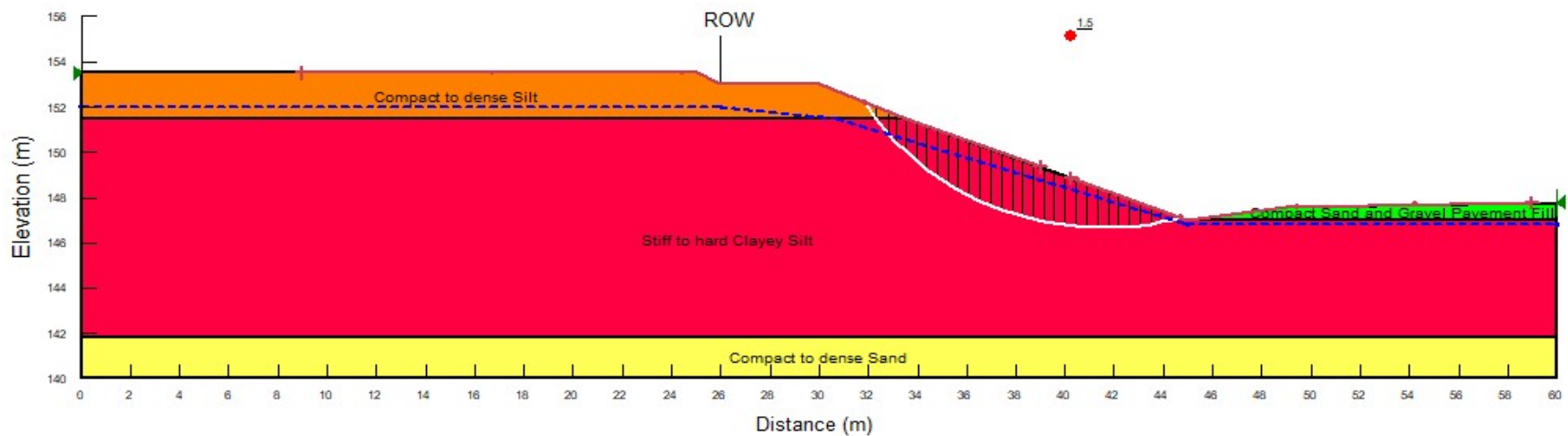
Slope Stability Analysis (Seismic)
RSS Wall at Clear Zone (Option 3)
Highway 401, East of Shelter Valley Road

Figure No. D7

Project No. 165001231

GWP No. 4060-11-00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
■	Compact Sand and Gravel Pavement Fill	21	0	35
■	Compact to dense Sand	21	0	32
■	Compact to dense Silt	21	0	30
■	Stiff to hard Clayey Silt	20	2	32



Slope Stability Analysis (Static)
Cut Slope 2.5H:1V (Option 3) Station 19+640
Highway 401, East of Shelter Valley Road

Figure No. D8

Project No. 165001231

GWP No. 4060-11-00

APPENDIX E

E.1 SOIL ERODIBILITY MEMO



To:	Jordan Broad P.Eng. 835 Paramount Drive Suite 200 Stoney Creek, ON L8J 0B4	From:	Cassandra Lewis, P.Eng 100-300 Hagey Boulevard Waterloo ON N2L 0A4
File:	165001231	Date:	August 15, 2022

Reference: Cobourg Slopes Drilling- Proposed Widening of Deep Cut Adjacent to 164 Skyview Road, Grafton, Northumberland County, Ontario, (165001231)

BACKGROUND

The Ministry of Transportation (MTO) retained Stantec Consulting Ltd. (Stantec) to complete the preliminary design for Highway 401 from Cobourg to Colborne, Ontario. Preliminary designs are required for the future widening of Highway 401 from a four-lane divided roadway to the ultimate eight-lane configuration with an interim six-lane divided roadway, and rehabilitation of the existing lanes.

As part of the project, Stantec drilled several deep foundational boreholes in the areas of proposed deep cut and deep fill locations, this memo summarizes the soil erodibility of the soil observed within those boreholes records.

SOIL ERODIBILITY

Soil erosion is the removal and movement of soil particles. Soil erosion may lead to instability of a roadway or roadway embankment. Erosion is affected by rainfall, slope geometry, vegetation, and soil type. In terms of soil type, the susceptibility of erosion is affected by the grain size distribution and the organic content of the soil.

The Wishmeier nomograph generates a k-factor which represents the erodibility of the soil. The k-factor is a value between 0 and 1.0, a higher k-factor represents a greater erodibility of the soil. The k-factor increases as the silt, very fine sand, and organic contents increase. The following Table 1 represents the severity of erosion.

Table 1: K-Factor Severity

Severity	K-Factor
Negligible	< 0.10
Very Slight	0.10 - 0.20
Slight	0.20 - 0.30
Moderately Severe	0.30 - 0.40
Severe	0.40 - 0.50
Very Severe	> 0.50

Reference: Cobourg Slopes Drilling- Proposed Widening of Deep Cut Adjacent to 164 Skyview Road, Grafton, Northumberland County, Ontario, (165001231)

RESULTS

Two boreholes were advanced and samples were taken at various depths at each of the boreholes and were submitted for analysis. The k-factor for the samples where grain size analysis was completed are summarized in the following tables.

Table 2 summarizes the average K-factor per borehole, below.

Table 2: K-Factor Severity Per Borehole

Borehole	Min	Max	Avg	Avg Severity
MW21-1	0.07	0.55	0.38	Moderately Severe
BH21-2	0.24	0.51	0.43	Severe

Table 3 summarizes the average severity of erosion per soil type.

Table 3: K-Factor Severity Per Soil Type

Soil Type	Min	Max	Avg	Avg Severity
Silt (ML)	0.50	0.50	0.50	Severe/Very Severe
Silty Clay (CL-ML)	0.39	0.55	0.47	Severe
Sandy Silt (ML)	0.35	0.49	0.41	Severe
Lean Clay (CL)	0.40	0.40	0.40	Moderately Severe /Severe
Silty, Clayey Sand (SC-SM)	0.21	0.35	0.25	Slight
Silty Sand (SM)	0.17	0.24	0.21	Slight
Silty Sand with Gravel (SM)	0.02	0.07	0.05	Negligible

The detailed k-factor for each test is presented in **Table 4**.

Table 4: K-Factor Per Test

Borehole	Soil Type	K-Factor
MW21-1 (1.8m)	Sandy Silt	0.49
MW21-1 (4.9m)	Silty Clay	0.39
MW21-1 (8.7m)	Silty Clay	0.55
MW21-1 (11.0m)	Lean Clay	0.40
MW21-1 (15.5m)	Silty Sand with Gravel	0.07
BH21-2 (1.1m)	Silty Sand	0.24
BH21-2 (3.4m)	Silt	0.50
BH21-2 (5.6m)	Silty Clay	0.45
BH21-2 (12.5m)	Silty Clay	0.51

August 15, 2022

Jordan Broad P.Eng.

Page 3 of 3

Reference: Cobourg Slopes Drilling- Proposed Widening of Deep Cut Adjacent to 164 Skyview Road, Grafton, Northumberland County, Ontario, (165001231)

Overall, the k-factors from this round of testing varied from 0.07 to 0.55 with an average of 0.40 and varied from negligible to very severe. The k-factors from the 2018/2019 pavement investigation varied from 0.04 to 0.57 with an average of 0.23.

Stantec Consulting Ltd.

Cassandra Lewis P.Eng.

Memo Author

Pavement Engineer,

Infrastructure Management &

Pavement Engineering

Phone: 519 585 7483

Fax: 519 579 6733

Cassandra.Lewis@stantec.com

APPENDIX F

F.1 2015 NATIONAL BUILDING CODE SEISMIC HAZARD CALCULATION



2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 44.012N 77.982W

User File Reference: Thomas Property

2022-03-25 12:43 UT

Requested by: Stantec Consulting Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.156	0.082	0.048	0.014
Sa (0.1)	0.197	0.110	0.067	0.022
Sa (0.2)	0.173	0.102	0.066	0.024
Sa (0.3)	0.137	0.085	0.056	0.021
Sa (0.5)	0.105	0.067	0.045	0.016
Sa (1.0)	0.059	0.038	0.026	0.008
Sa (2.0)	0.030	0.019	0.012	0.003
Sa (5.0)	0.008	0.004	0.003	0.001
Sa (10.0)	0.003	0.002	0.001	0.001
PGA (g)	0.109	0.061	0.038	0.012
PGV (m/s)	0.087	0.053	0.033	0.010

Notes: Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

References

National Building Code of Canada 2015 NRCC no. 56190; Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information



Natural Resources
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