Stantec

Preliminary Foundation Investigation and Design Report North Augusta Road Underpass Replacement - Site No. 16X-0124/B0

Highway 401 Rehabilitation
Brockville, ON

G.W.P. 4003-19-00

Latitude	44.611243
Longitude	-75.685218

Geocres No. 31B-99

Prepared for:
Ministry of Transportation Ontario

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Project No. 165001160 (309)

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For
G.W.P 4003-19-00

Highway 401 Rehabilitation, Brockville, Ontario
Highway 401 North Augusta Road Underpass (Site No. 16X-0124/B0)

Brockville, Ontario

1.0 INTRODUCTION

The Ministry of Transportation, Ontario (MTO) has retained Stantec Consulting Ltd. (Stantec) to undertake an Environmental Assessment and complete the Preliminary Design for the replacement or rehabilitation of various structures along Highway 401 in the City of Brockville. The project limits extend from about 2 km west of the Highway 401 and Stewart Boulevard Interchange to 750 m east of the Highway 401 and North Augusta Road Interchange for a total length of approximately 4.5 km (G.W.P. 4003-19-00).

The foundation engineering services for the project include the preparation of preliminary foundation investigation and design reports at 4 bridge (overpass or underpass) sites and one culvert site where replacement of the existing structures is planned. This report presents the results of a preliminary foundation investigation related to the replacement of the North Augusta Road underpass structure at Site No. 16X-0124/B0. Separate Preliminary Foundation Investigation and Design Reports will be prepared for the other structure sites included in this assignment.

The purpose of the preliminary foundation investigation was to supplement existing information on the subsurface conditions at the location of the proposed bridge reconstruction by drilling two boreholes, carrying out in-situ testing and completing a laboratory testing program on selected soil samples obtained from the boreholes.

This Preliminary Foundation Investigation and Design Report (Preliminary FIDR) has been prepared specifically and solely for the proposed replacement of the North Augusta Road underpass at Highway 401 (Site No. 16-124). This preliminary report is not to be used for the detail design of this project; a Final Foundation Investigation and Design Report will need to be prepared in the future, after an additional site investigation has been carried out.

2.0 SITE DESCRIPTION

2.1 SITE LOCATION

North Augusta Road crosses over Highway 401 near Station 23+196, in the City of Brockville, Ontario. The site location is shown on the Key Plan inset on the Borehole Locations and Soil Strata Plan, Drawing No. 1 in Appendix A.

2.2 SITE DESCRIPTION

At the North Augusta Road interchange, Highway 401 is a four-lane divided freeway with two lanes in each direction that is aligned in an approximate southwest-northeast orientation. There are also two entrance speed change lanes (one in each direction) present beneath the underpass structure. At the bridge location, North Augusta Road is a four-lane undivided roadway that crosses over Highway 401 on a single-span bridge structure. For the purposes of this report, the underpass structure will be referenced as being orientated south to north.

The ground surface surrounding the underpass site consists of relatively flat terrain within an overall slope towards the south. The land within the interchange is undeveloped and contains vegetative cover consisting of grass and/or trees. The lands to the north and northwest of the interchange contain commercial developments while residential subdivisions are present to the southwest and southeast of the interchange.

The bridge deck of the North Augusta Road underpass slopes towards the south with pavement surface elevations varying from approximately 104.7 m to 104.2 m. At the bridge site, the asphalt surface on Highway 401 is at an elevation of about 97.5 m. Information contained on the structural drawings for the bridge suggest that the original ground surface elevation in the vicinity of the bridge was approximately 96 m.

The existing bridge is a single-span, rigid frame structure constructed in the 1950’s (Contract 57-12). The underpass structure has a span of approximately 37.5 m, is constructed on a skew of about 26 degrees with respect to Highway 401 and traverses over six lanes of traffic (4 through-lanes and 2 entrance speed change lanes). A photo of the bridge looking towards the west is provided below.

The bridge abutment foundations are supported on steel H-piles. Curved retaining walls, also supported on H-piles, are present adjacent to the abutments in all four quadrants of the bridge. Additional details related to the bridge foundations are included in Section 8.2.2 of this report.

2.3 SITE RECONNAISSANCE

The following items were noted during a site visit completed by a member of Stantec’s geotechnical/foundation group:

- No visible signs of settlement or deformation of the existing structure was noted. Areas of exposed rebar were visible at numerous locations on the concrete on the sides and underside of the underpass structure.
- The asphalt on the bridge surface displayed only minor cracking.
- No signs of embankment settlement or instability were observed.

2.4 SITE DRAINAGE

Regionally, surface water flow in the area of the site is typically from north to south towards the Saint Lawrence River.

Highway 401 slopes towards the west at a grade of about 2%, and pavement drainage is provided by evenly spaced catch basins, located adjacent to the median barrier.

2.5 GEOLOGICAL INFORMATION

The Physiography of Southern Ontario indicates that the site is located within a physiographic region known as the Smiths Falls Limestone Plain. The Surficial Geology Map of Southern Ontario indicates that the North Augusta Road bridge structure is located within massive to well laminated fine-textured glaciomarine deposits comprised of silt and clay with minor amounts of sand and gravel. There are also zones of stone-poor sandy silt to silty sand-textured till on Paleozoic terrain and bedrock-drift complex in Paleozoic terrain identified to the east and south of the site location.

The Paleozoic Geology Map of the Brockville Mallorytown Area indicates that the bedrock at the site is part of the March Formation consisting of interbedded sandstone, dolostone, sandy dolostone, and dolomitic sandstone.

A review of available water well records for wells located in proximity to the bridge site indicates that bedrock was encountered at depths of approximately 1 m to 10 m below ground surface.

3.0 PREVIOUS INVESTIGATIONS / AVAILABLE INFORMATION

No foundation investigation reports were available for this site in the MTO GEOCRES database/library. However, some subsurface information was available on the following drawing:

- ‘General Arrangement - Elizabethtown Township Bridge No. 15, County Road No. 18 over Highway 401 near Brockville’, Twp. Drawing No. 25-124-1-A, prepared by C.C. Parker and Associates Ltd. Consulting Engineers and dated January 15, 1955.

The drawing included the results of four (4) borings advanced at the site by Racey, MacCallum & Associates on October 6, 1954, to a maximum depth of approximately 7.8 m below ground surface. One (1) boring was advanced near each corner of the existing structure.

In the two (2) borings advanced near the south abutment of the existing bridge, the subsurface stratigraphy encountered in the test holes consisted of a layer of silty clay/clay underlain by glacial till at depths of about 4.5 m below original ground surface. In the borehole advanced near the northwest corner of the bridge, a thin (~0.3 m thick) layer of sandy and silty gravel was encountered beneath the silty clay at a depth of approximately 5.2 m. Sandstone bedrock was encountered beneath the overburden in all test holes at depths varying between approximately 5.5 m to 6.3 m below ground surface (corresponding to elevations of approximately 90.0 m to 90.5 m).

The General Arrangement drawing containing the above noted information is included in Appendix B for reference.

4.0 INVESTIGATION PROCEDURES

4.1 FIELD INVESTIGATION

The current subsurface investigation program consisted of advancing two boreholes, designated as Boreholes NA21-1 and NA21-2, at the site. One borehole was advanced on each side of the highway. Borehole NA21-1 was drilled at the highway level within the grassed area to the east of the existing north approach embankment while Borehole NA21-2 was drilled through the south approach embankment to the North Augusta Road underpass. The borehole locations are shown on the Borehole Locations and Soil Strata Plan, Drawing No. 1, in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of both private and public utilities.

The boreholes were advanced between May 3rd and May 6th, 2021, using truck and track-mounted drill rigs equipped for soil sampling and rock coring. The boreholes were advanced in the overburden using continuous hollow-stem augers. Coring methods were used to advance within bedrock after auger refusal was encountered at depths of approximately 6.3 m and 14.3 m below ground surface in Boreholes NA21-1 and NA21-2, respectively.

The subsurface stratigraphy encountered in each borehole was recorded in the field by a member of Stantec’s geotechnical staff. Standard Penetration Tests (SPTs) were carried out in the overburden and split spoon samples were collected at regular intervals. Relatively undisturbed Shelby tube samples of cohesive soil deposits were also collected at select locations. In both boreholes the bedrock was cored to the termination depth using NQ size equipment. The bedrock cores were placed in core boxes, and the boxes labelled and sealed. All of the recovered soil samples and bedrock cores were returned to our Ottawa laboratory for detailed classification and testing.

In situ shear vane testing was attempted at select locations to assess the undrained shear strengths (undisturbed and remoulded) of cohesive materials.

A monitoring well, with a well screen located in the bedrock from 7.6 m to 9.1 m below ground surface, was installed in Borehole NA21-1. The water level was measured in this well on May 6th and May 13th, 2021, and the well was subsequently decommissioned.

4.2 LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations for the boreholes were surveyed by Stantec’s geomatics division. The borehole survey data is considered accurate to 0.1 m for both coordinates and elevations.

Table 4.1 below summarizes the borehole location information with the borehole ground surface elevations, depths, and termination elevations.

Table 4.1: Borehole Coordinate and Elevation Information

Borehole	MTM Zone 11 Coordinates		Approximate Ground Surface Elevation (m)	Borehole Depth (m)	Borehole Termination Elevation (m)
	Northing	Easting			
NA21-1	4941616.9	369479.9	96.9	9.2	87.7
NA21-2	4941545.1	369488.3	104.1	17.4	86.7

4.3 LABORATORY TESTING

All samples were transported to Stantec's Ottawa laboratory where they were visually examined by a geotechnical engineer. The geotechnical laboratory testing program completed on the borehole samples is summarized in Table 4.2.

Table 4.2: Geotechnical Laboratory Testing Program

Test Description	Number of Tests
Moisture Content	27
Atterberg Limits	5
Grain Size Distribution (sieve & hydrometer)	7
Unconfined Compressive Strength (on soil samples)	1
Unconfined Compressive Strength (on bedrock cores)	4
Consolidation	2
Chemical Analysis	2

The chemical analysis was completed by Paracel Laboratories Ltd. and consisted of testing one sample from each borehole location for pH, soluble sulphate content, chloride content, and resistivity.

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

5.0 SUBSURFACE CONDITIONS

5.1 FRAMEWORK AND OVERVIEW

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are displayed on the Borehole Records included in Appendix C. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix C. The results of geotechnical laboratory testing are presented in Appendix D.

A borehole location plan and stratigraphic section of the soils encountered in the boreholes are provided on Drawing No. 1 in Appendix A. The stratigraphic boundaries on the borehole records and the strata plot are inferred from non-continuous sampling and therefore represent transitions between soil types rather than exact boundaries between geological units. The conditions will vary beyond the borehole location.

In general, the subsurface stratigraphy encountered at the borehole locations consisted of a surficial layer of topsoil in Borehole NA21-1 and an asphalt layer underlain by approximately 8 m of predominantly granular fill materials in Borehole NA21-2. The topsoil and fill materials were underlain by an approximately 4 m to 5 m thick deposit of stiff to very stiff silty clay to clay, underlain by loose to very dense silty sand till. The overburden materials were in turn underlain by Dolostone bedrock at elevations varying from 89.5 m to 90.5 m (about 7 m to 8 m below current highway level). Both boreholes were terminated within the bedrock at depths of 9.2 m and 17.4 m below existing ground surface.

The following sections provide a summary of the subsurface conditions encountered during the investigation.

5.2 OVERBURDEN

5.2.1 Topsoil

An approximately 100 mm thick surficial layer of topsoil was encountered at ground surface at Borehole NA21-1.

5.2.2 Fill

Borehole NA21-2, drilled through the easternmost lane of the south approach to the underpass, encountered 8.4 m of embankment fill, which includes an initial 200 mm thick asphalt layer. The base of the fill layer was encountered at an elevation of approximately 95.6 m.

Predominantly granular fill materials were encountered beneath the asphalt. An approximately 150 mm thick layer of gravelly sand (road base) was encountered directly beneath the asphalt. The remaining fill varies in composition from sand containing trace silt and gravel, to sandy silt, to silty sand and gravel.

The fill contains cobbles and/or boulders. Increased drilling resistance and frequent grinding of the augers were noted below a depth of about 4.5 m.

Standard Penetration Test (SPT) N-values measured within the fill material varied from 6 to 45 blows per 0.3 m of penetration but were more typically between 6 and 29 blows per 0.3 m of penetration indicating the fill is generally loose to compact.

Laboratory testing of samples of the fill materials yielded moisture contents ranging from approximately 1% to 20%, expressed as a percentage of the dry weight of the soil.

Gradation analysis was carried out on three (3) representative samples of the fill materials. The results of the tests are illustrated on the borehole records in Appendix C and on the gradation curves on Figure No. D1 in Appendix D.

An Atterberg limits test was also carried out on one of the samples referenced above. The test yielded a Plastic Limit of 13%, a Liquid Limit of 15% and a corresponding Plasticity Index of 2. The results of the tests are illustrated on the borehole records in Appendix C and on Figure No. D2 in Appendix D.

Based on the laboratory results, the Unified Soil Classification System (USCS) group symbols for the fill vary from SP to SM.

5.2.3 Silty Clay to Clay

A cohesive deposit comprised of silty clay to clay containing trace sand was encountered below the topsoil in Borehole NA21-1 and beneath the fill materials in Borehole NA21-2. This cohesive deposit was approximately 3.8 m to 5.2 m thick at the borehole locations and extended to depths of about 5.3 m and 12.2 m below ground surface, corresponding to base elevations of about 91.5 m and 91.8 m in Boreholes NA21-1 and NA21-2, respectively.

The silty clay to clay deposit was noted to be varved below a depth of 5 m in Borehole NA21-1.

SPT 'N' values varying between 11 to 21 blows per 0.3 m of penetration were measured within the cohesive deposit. An in situ shear vane test, using an N-size field vane, attempted at a depth of about 10 m in Borehole NA21-2 encountered refusal (i.e. inability to turn vane). The undrained shear strength of the cohesive deposit was determined by conducting an Unconfined Compressive Strength (UCS) test on a Shelby Tube sample recovered from Borehole NA21-1. An undrained shear strength of approximately 47 kPa was measured by this test. Based on the field and laboratory testing and examination of samples obtained, the cohesive deposit is considered to generally have a very stiff consistency with zones of firm or stiff soils present in Borehole NA21-1.

Laboratory testing of samples of the cohesive soils yielded moisture contents varying between approximately 15% to 37%.

Gradation analyses were carried out on two (2) representative samples of the silty clay/clay deposit obtained from the boreholes. The test results are illustrated on the borehole records in Appendix C and on the gradation curves on Figure No. D3 in Appendix D.

Atterberg Limits tests were carried out on portions of the samples referenced above. The tests yielded Liquid Limits of 45% and 58%, Plastic Limits of 23% and 24%, and corresponding Plasticity Indices of 21% and 35%. The results of the tests are illustrated on the borehole records in Appendix C and on Figure No. D4 in Appendix D. Based on these results, the cohesive soil is classified as silty clay of medium plasticity (CI) to clay of high plasticity (CH) in accordance with the USCS.

Two consolidation tests were carried out on relatively undisturbed Shelby Tube samples of the silty clay/clay. The test results are illustrated on the void ratio versus stress plots on Figure Nos. D5(A to D) to D6(A to D) in Appendix D. The consolidation and index property test results for these samples are summarized below in Table 5.1.

Table 5.1: Consolidation Test Results

Parameter	**Sample ID**	
	NA21-1, SH6A	**NA21-2, SH13**
Sample Depth (m below ground)	4.9	9.4
Sample Elevation (m)	92.0	94.6
Effective Vertical Stress (kPa)	57	191
Moisture Content	43%	28%
Initial Void Ratio, e_0	1.26	0.88
Initial Unit Weight, γ	17.3 kN/m^3	18.8 kN/m^3
Estimated Preconsolidation Stress, P'_c	500 kPa	650 kPa
Overconsolidation Ratio (OCR)	8.8	3.4
Recompression Index, C_r	0.08	0.015
Compression Index, C_c	0.53	0.32
Coefficient of Consolidation, C_v	0.5 mm^2/s	0.4 mm^2/s

Note: The initial void ratios presented in Table 5.1 are derived from the start of the oedometer test, at which point the sample is entirely unloaded and the degree of saturation is less than 100%.
The coefficients of consolidation identified relate to the recompression stress range.

5.2.4 Silty Sand (TILL)

A glacial till deposit comprised of silty sand containing varying amounts of clay and gravel as well as zones of clayey silt till was encountered underlying the clay/silty clay deposit in both boreholes. The till deposit was approximately 1.0 m to 2.3 m thick and extended to the depths of about 6.3 m and 14.3 m (corresponding to elevations of about 89.5 m to 90.5 m) in Boreholes NA21-1 and NA21-2, respectively.

Auger refusal was encountered on an inferred cobble or boulder in Borehole NA21-2. Cobbles and boulders are known to be present within the till deposits of Southern Ontario and are expected to be present throughout the till deposits at this site.

SPT ‘N’ values ranging from 4 to 7 blows per 0.3 m of penetration were measured in the till deposit in Borehole NA21-2 suggesting the till at that location is in a loose or firm state. SPT ‘N’ values measured in the till deposit in Borehole NA21-1 were greater than 50 blows per 0.3 m of penetration indicating the till is in a very dense state. However, the high SPT ‘N’ values are inferred to have been influenced by the presence of gravel, cobbles and/or boulders within the till.

Laboratory testing of samples of the till materials yielded moisture contents that ranged from approximately 11% to 22%.

Gradation analyses were carried out on two (2) representative samples of the till deposit obtained from the boreholes. The test results are illustrated on the borehole records in Appendix C and on the gradation curves on Figure No. D7 in Appendix D.

Atterberg Limits tests were also carried out on portions of the samples referenced above. The tests yielded Plastic Limits of 11% and 12%, Liquid Limits of 13% and 12%, and corresponding Plasticity Indices of 2% and 0% (non-plastic). The results of the tests are illustrated on the borehole records in Appendix C and on Figure No. D8 in Appendix D.

Based on the gradation and Atterberg Limit test results, the USCS group symbol for the samples of the glacial till tested is SM (silty sand).

5.3 BEDROCK

Slightly weathered to fresh, dolostone to sandy dolostone bedrock was encountered underlying the overburden described in both boreholes. The presence of bedrock was confirmed by coring. The depths that the bedrock was encountered are summarized in Table 5.2 below.

Table 5.2: Bedrock Surface Depth/Elevation

Borehole	Depth (m)	Elevation (m)
NA21-1	6.3	90.5
NA21-2	14.5	89.5

The bedrock type, depths of the coring and corresponding elevations) along with the measured total core recovery (TCR), solid core recovery (SCR) and rock quality designation (RQD) for each core run are summarized in Table 5.3 below. Photographs of the rock cores from each of the boreholes are included in Appendix C**.**

Table 5.3: Summary of Bedrock Coring Operations

Parameter	Borehole Number				
	NA21-1		NA21-2		
Run No.	9	10	17	18	19
Rock Description	Slightly weathered to fresh, light grey to grey Dolostone		Slightly weathered to fresh, light grey to light brown, Dolostone to Sandy Dolostone		
Depth (m below ground)	6.3-7.7	7.7-9.2	14.5-15.2*	15.2-16.7	16.7-17.4
Geodetic Elevation (m)	90.6-89.3	89.3-87.7	89.8-88.9	88.9-87.4	87.4-86.7
Total Core Recovery, TCR (%)	94	100	100	87	100
Solid Core Recovery, SCR (%)	89	100	100	82	100
Rock Quality Designation, RQD (%)	72	63	89	57	100
Weathering Degree	W2/W1	W2/W1	W1	W2/W1	W2/W1
Fracture Index (Fractures per 1 m)	8	7	3	17	3

*Note: Top of 0.3 m of material in Core Run 17 in Borehole NA21-2 consists of glacial till; TCR, SCR and RQD values for that core run are based on the portion of core run within bedrock only.

Based on the RQD ranges measured, the bedrock cores obtained from the boreholes can be classified as being fair to excellent in quality.

Four (4) samples of the rock cores were selected for Unconfined Compressive Strength (UCS) testing. The results of the tests are summarized in Table 5.4 below.

Table 5.4: Results of Unconfined Compressive Strength (UCS) on Samples of Rock Core

Borehole No.	Run No.	Sample Depth (m below ground)	Sample Elevation (m)	Unit Weight (g/cm3)	Unconfined Compressive Strength (UCS) (MPa)
NA21-1	9	7.1	89.8	2.645	95
	10	8.5	88.4	2.673	98.2
NA21-2	17	14.7	89.4	2.835	156.4
	19	17.2	86.9	2.835	270.7

The results of the UCS tests conducted on the rock cores ranged from 95.0 MPa to 270.7 MPa and indicate that the dolostone bedrock can be classified as strong (R4) to extremely strong (R6).

5.4 GROUNDWATER CONDITIONS

The water level in a monitoring well installed within the bedrock at Borehole NA21-1 was measured to be at depths of approximately 1.25 m and 1.3 m (corresponding to elevations of about 95.65 m and 95.6 m) on May 6th and May 13th, 2021, respectively.

Samples of the embankment fill materials from Borehole NA21-2 were observed to be wet below a depth of approximately 7.5 m (~Elev. 96.5 m) which is above the water level measured at Borehole NA21-1, suggesting perched/mounded water conditions are present within the embankment fill materials.

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

5.5 CHEMICAL ANALYSIS

Chemical analyses related to parameters associated with the potential for corrosion or sulphate attack (i.e., pH, resistivity, and chloride and sulphate content) were completed by Paracel Laboratories Inc. on one representative samples of the soils collected from each borehole. The analysis results are provided in Table 5.5.

Table 5.5: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Resistivity (Ohm-m)	Chloride (µg/g)	Sulphate (µg/g)
NA21-1	SS03	1.5-2.1	7.45	39.8	27	26
NA21-2	SS15	12.2-12.8	7.94	12.6	388	86

6.0 MISCELLANEOUS

The field work was carried out under the supervision of Karl Thom under the direction of Kevin Nelson, P.Eng.

The utility locates for the boreholes were arranged by Stantec personnel.

The drilling equipment was supplied and operated by George Downing Estate Drilling Ltd. of Grenville-sur-la-Rouge, Quebec.

The location and elevation survey of the boreholes was completed by Stantec’s Geomatics division.

Traffic control service was provided by Beacon Lite of Ottawa, Ontario.

Geotechnical laboratory testing was carried out at Stantec’s Ottawa laboratory. The chemical testing for pH, soluble sulphate and chloride contents, and soil resistivity was carried out by Paracel Laboratories Ltd. of Ottawa.

This report was prepared by Roshan Rashed, P.Eng. and reviewed by Kevin Nelson, P.Eng., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.

7.0 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 locations, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted,

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PART B - PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

For
G.W.P 4003-19-00

Highway 401 Rehabilitation, Brockville, Ontario

Highway 401 North Augusta Road Underpass (Site No. 16X-0124/B0)

Brockville, Ontario

8.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

8.1 OVERVIEW

This section of the report provides preliminary foundation design input related to the proposed replacement of the underpass structure located at the crossing of North Augusta Road over Highway 401 (Site No. 16X-0124/B0). The new underpass is being designed to facilitate the construction of a new interchange and to accommodate the ultimate 8-lane highway configuration.

The interpretation and preliminary recommendations provided in this report are intended solely to provide the designers with information to assess feasible foundation alternatives for the proposed underpass replacement. 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.

Additional subsurface investigation will be required to meet minimum MTO foundation investigation requirements for the detailed design of the replacement underpass structure. This Preliminary Report is not to be used for the detailed design of this project. 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.

8.2 PROJECT DESCRIPTION AND BACKGROUND

8.2.1 Project Description

The project involves the preliminary design for a new underpass structure at North Augusta Road and Highway 401 in the City of Brockville. This study is being completed as part of an overall study related to the rehabilitation of Highway 401 in the City of Brockville (GWP 4003-19-00).

Based on preliminary design information, the new interchange at Highway 401 and North Augusta Road will require a new underpass with four lanes of through traffic (two traffic lanes in each direction) and one turning lane on North Augusta Road. The bridge spans will accommodate an 8-lane configuration (4 lanes in each direction) on Highway 401 plus speed change lanes. These modifications will require a new underpass structure that is wider and longer than the existing bridge. The preliminary design information indicates that the new underpass structure will be located to the east of the existing structure which will permit North Augusta Road to remain open during construction of the new underpass structure.

8.2.2 Existing Underpass Structure

The following details of the existing bridge foundations are based on the information shown on the structural design drawings for the existing bridge:

- The existing underpass is an approximately 37.5 m long single-span structure constructed in the late 1950's (Contract 57-12). At the bridge site, the pavement surface elevations on the existing North Augusta Road underpass vary from approximately 104.7 m (north side) to 104.2 m (south side) while the asphalt surface on Highway 401 is at an elevation of just below 98 m.
- Curved retaining walls are present on both sides of each abutments (i.e. at the four corners of the bridge). The abutments and associated retaining walls are all supported on a series of HP10x42 piles which has the metric equivalent designation of HP250x62.
- The abutment pile caps are both approximately 19 m long by 2.1 m wide with wider portions near both ends of each abutment. The bases of the north and south abutment pile caps are at elevations of approximately 95.3 m and 94.8 m, respectively. The abutment pile caps are supported on two rows of piles, increasing to five rows at the locations of the wider portions at the ends of the pile caps, installed at approximately 1.1 m centre to centre spacings. The piles alternate from having a vertical orientation to being inclined at 1 horizontal to 3 vertical (1H:3V) away from the highway.
- The retaining wall pile caps are curved and have varying lengths. The structural drawings provide the following details for the retaining wall pile caps:
    - Pile caps for the NW and SW retaining walls (referred to as Type A walls) are approximately 3.2 wide and are founded at an elevation of approximately 95.4 m. The pile caps for these walls are supported on two rows of H-piles with pile spacings of about 1 m in the front row (closest to the highway) and 2 m in the back row.
    - The pile cap for the NE retaining wall (referred to as a Type B wall) is approximately 4.1 wide, is founded at an elevation of approximately 95.4 m and is supported by two rows of H-piles. The piles in the front and back rows are installed at approximately 0.7 m and 2.1 m spacings, respectively.
    - The pile cap for the SE retaining wall (referred to as a Type C wall) is approximately 2.9 wide, is founded at an elevation of approximately 95.4 m and is supported by two rows H-piles. The piles in the front and back rows are installed at approximately 1.3 m and 2.6 m spacings, respectively.
    - For all retaining walls, the piles in the row closest to the highway are inclined at 1H:3V towards the highway and the piles in the back row are vertical.

8.2.3 Proposed Structure Modifications and Replacement

Based on preliminary design information, we understand that the North Augusta Road underpass structure is planned to be replaced with a new underpass that will accommodate eight lanes of through-traffic on Highway 401 and the development of a revised interchange configuration with Highway 401. The new underpass is planned to consist of a new 2-span structure to be located to the east of the existing structure.

The new bridge is planned to be wider than the existing bridge in order to accommodate five lanes (four lanes of through traffic and one turning lane) on North Augusta Road. The new bridge will be a two-span structure with a central pier. The length of the new bridge will be increased to approximately 82 m.

The existing approach embankments will be widened approximately 26 m towards the east and raised slightly (by about 0.5 m or less) on both sides of the bridge. The widened approach embankments are planned to be constructed with 2H:1V sideslopes.

The new bridge will incorporate either integral abutments, if the ground conditions are conducive, or semi-integral abutments.

8.3 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 subsurface materials at 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.

8.4 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered during the current investigation at the site consisted of a surficial layer of topsoil in Borehole NA21-1 or asphalt underlain by a fill layer (pavement structure and approach embankment fill) in Borehole NA21-2. These materials are underlain by native deposits of typically very stiff silty clay/clay and then by glacial till. The till is typically comprised of silty sand with varying amounts of clay and gravel and contains cobbles and boulders. The overburden is in turn underlain by Dolostone bedrock. The groundwater level was recorded at approximately 1.3 m depth in the monitoring well screened within the bedrock at Borehole NA21-1

The soil profile identified in Table 8.1 below and on Drawing No. E1 in Appendix E can be used for the preliminary design of the bridge replacement. The geotechnical parameters identified in the soil profile were developed based on a synthesis of the borehole data, the measured penetration resistance values, and laboratory index test results (including moisture contents) of soil samples obtained in the investigation.

Table 8.1: Representative Soil Profile – North Augusta Road Underpass

Elevation (m)		**Soil Type**	**Design Parameters**				
From	**To**		**Total Unit Weight γ, (kN/m^3)**	**Drained Friction Angle[2] ϕ', (°)**	**Undrained Shear Strength, Su, (kPa)**	**Soil Modulus E, (MPa)**	**Consolidation Parameters[1]**
104.0	96.5	FILL: Loose to compact SAND (SP) to Silty SAND and GRAVEL (SM)	24.5 @ 104 m to 22.5 @ 96.5 m	30	N/A	15	N/A
96.5	91.5	Stiff to very stiff SILTY CLAY (CI) to CLAY (CH), trace sand	19	30	100	25	P_c = 500 kPa C_r = 0.015 C_c = 0.41
91.5	89.5	Loose to very dense Silty SAND (SM), some clay and gravel (TILL). Contains zones of firm CLAYEY SILT (TILL) in Borehole NA21-2. Contains cobbles and boulders.	22.5	32	N/A	25	N/A
89.5	86.6	Strong to extremely strong DOLOSTONE/Sandy DOLOSTONE. Fair to excellent quality.	27.5	N/A	100 MPa[3]	N/A	N/A

Note:

(1) Consolidation parameters: P_c = Estimated Preconsolidation Pressure, C_r = Recompression Index, C_c = Compression Index

(2) The friction angles are applicable to drained conditions only.

(3) The value in the strength column for the bedrock represents the Unconfined Compressive Strength

(4) Groundwater is assumed to be at an Elevation of 95.7 m for preliminary design purposes. Submerged unit weights (γ') should be used below the groundwater level.

8.5 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 and pile caps should be provided with a minimum of 1.4 m of soil cover or equivalent insulation for protection against frost heaving.

This depth of frost penetration should also be considered in the design of frost tapers adjacent to the bridge abutment and retaining wall backfill zones.

8.6 SEISMIC CONDITIONS

8.6.1 Site Class

The available subsurface information from previous and current investigations indicates that the new underpass site is underlain by overburden consisting of a deposit of predominantly very stiff silty clay/clay that is underlain by a thin glacial till deposit. These materials are underlain by bedrock at the depth of approximately 6 m to 7 m below ground surface. Based on these 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.

8.6.2 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 8.2 summarizes the parameters based on a 2475-year return period to be used in forced based design.

Table 8.2: Peak Ground Acceleration Data

PGA	$S_a(0.2)$	PGA_{ref}	**Site Class**	Site Adjusted PGA
0.167g	0.265g	0.134g	C	0.167g

8.6.3 Liquefaction Potential

The potential for soil liquefaction of the glacial till beneath the approach embankments 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 followed the analysis methodology suggested by Idriss and Boulanger (2008) and was based on the following input parameters:

- The SPT ‘N’ blow count values obtained from boreholes corrected for confining pressure and fines content.
- A Site Adjusted PGA of 0.167g.
- An earthquake magnitude Mw of 6.5.
- A groundwater level/elevation of 95.7 m.

Based on the results of these analyses, the factor of safety against liquefaction of these soils is greater than 1.3 under the design earthquake loading conditions, and, as such, these soils are not considered to be liquefiable.

Liquefaction of the silty clay/clay is also not considered to be a concern due to the high fines/clay content, and the stiff to very stiff and overconsolidated nature, of the deposit.

8.7 PRELIMINARY FOUNDATION ENGINEERING DESIGN INPUT

The following sections provide preliminary geotechnical engineering input related to the design of the foundations for a replacement underpass structure at this site. The input provided herein is preliminary in nature and should be reviewed, and modified as necessary during detail design, once further subsurface investigation is completed and the loading conditions for the new foundations are determined.

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2019).

8.7.1 Foundation Options

Both shallow and deep foundation options were evaluated for the proposed replacement bridge structure. Table 8.3 presents the advantages, disadvantages, relative costs, and risks/consequences for various foundation options for the North Augusta Road replacement bridge.

Table 8.3: Comparison of Foundation Options for North Augusta Road Underpass

Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
H-Piles Driven to Bedrock (Abutments and Central Pier)	• Allows for use of Integral Abutments • Reduced settlement • Reduces depth of excavations and requirements for temporary support systems. • More suitable than pipe piles for difficult driving conditions	• Piles at pier locations would be short (i.e. less than 5 m); inclined/battered piles or rock-socketed piles may be needed to resist lateral loads • Pre-drilling in till could be required if piles encounter refusal on boulders • Limited uplift capacity due to short pile lengths	Medium	• Pile damage during installation • Potential for shallow refusal of piles on cobbles and boulders requires pre-drilling
Drilled Piers / Caissons (Central Pier)	• Can transmit very large axial and lateral loads • Shorter construction time than shallow foundations	• Requires use of liners and/or drilling mud to balance water pressures; cannot be visually inspected • Difficult to drill piers/advance liners in till deposits containing boulders and cobbles • Not suitable for integral bridge abutments	High	• Liners and/or drilling mud required to mitigate groundwater issues. • Installation of liners to maintain sidewall stability may not be practical without specialized equipment.
Shallow Foundations Founded on Stiff to Very Stiff Silty Clay to Clay	• Lower foundation costs than deep foundations • Drilling through difficult deposits avoided	• Potential for overstressing silty clay/clay subgrade leading to large settlements • Not suitable for integral abutments (Semi-integral abutments possible) • Larger foundation areas required compared to integral abutments or drilled piers	Low to medium	• Potential for unacceptable total and differential settlements

For an integral abutment design, steel H-pile foundations would be a suitable foundation option. The piles would be driven to bedrock and would develop most of their load carrying capacity from the end-bearing resistance on the bedrock. Where integral abutments are adopted, the upper portion of the piles are installed within sand-filled, corrugated steel pipe (CSP) liners to provide suitable flexibility of the steel H-piles. Driven piles may “hang up”/encounter refusal within the till particularly where cobbles and/or boulders are encountered. In this regard, pre-drilling may be required to facilitate advancing the pile tips to the bedrock; assuming that the CSP liners are entirely within the embankment fills, predrilling likely will not be required, this should be confirmed in the detail design stage.

Drilled piers/caisson foundations socketed into the bedrock could be considered for support of the central pier but would require the installation of temporary liners and dewatering or the use of drilling mud to mitigate the potential risks of ground loss or collapse within the water-bearing soils present immediately above the bedrock during construction.

Due to the relatively long bridge spans and associated high structural loads, support of the abutments or the pier on shallow foundations could result in overstressing of the underlying stiff to very stiff silty clay to clay subgrade, leading to large and unacceptable settlements, and therefore is not recommended. Based on the above considerations, the preferred option from a geotechnical/foundation engineering perspective is to support the bridge abutments on driven steel H-piles and the centre pier on either driven H-piles or rock socketed caissons.

8.7.2 Driven Pile Foundations

8.7.2.1 Design Considerations

Pile foundations consisting of steel H-piles that are driven to effective refusal on the bedrock, and that derive the majority of their capacity from end-bearing, can be used to support the integral abutments and the central pier of the proposed replacement bridge. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their design orientation due to the presence of cobbles and/or boulders within till deposits and are not conducive to the use of integral abutments. Therefore, H-piles are recommended for use at this site.

Available design information suggests that the undersides of the pile caps for the abutment walls will be at approximate elevations of 98.3 m and 98.6 m for the south and north abutments, respectively, while the base of a pile cap for the central pier would be at an elevation of between 95 m and 96 m. The surface of the bedrock was encountered at elevations varying from approximately 89.5 m to 90.5 m which would result in required pile lengths of approximately 8 m to 9 m at the abutments and about 5 m at the centre pier. Effective refusal could be encountered at shallower depth within the very dense portions of the till deposits particularly if cobbles and/or boulders are encountered. For preliminary design purposes, predrilling is recommended to be carried out down to an elevation of 91 m to facilitate the piles reaching the bedrock surface and to confirm the abutment piles obtain sufficient pile embedment/satisfy the minimum pile length requirements to obtain the condition of pile fixity. The requirement for predrilling should be further reviewed during the detailed design stage.

The abutments of the new bridge are located in close proximity to the curved retaining walls present at the northeast and southeast quadrants of the existing bridge which are supported on H-Pile foundations, including battered/inclined piles. In this regard, the pile locations should be reviewed/selected at the detailed design stage to avoid conflicts with existing piles.

The driving of piles for the new underpass is not expected to adversely affect the stability of the existing approach embankments.

8.7.2.2 Geotechnical Axial Resistance

The factored geotechnical resistances at Ultimate Limit States (ULS) outlined in Table 8.4 may be used in design. These values include a resistance factor of 0.4 applied to the ultimate capacity.

Table 8.4: Recommended Factored Geotechnical Resistances (ULS) - Pile Foundations

Pile Type	Anticipated Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kN)
HP 310 x 110	89.5 to 90	1,800
HP 310 x 132	89.5 to 90	2,100

The estimated geotechnical reaction at SLS (factored) for 25 mm of vertical settlement for a HP 310x110 pile driven to effective refusal on the dolostone bedrock exceeds the factored geotechnical reaction at ULS. Therefore, the ULS (factored) resistances will govern.

8.7.2.3 Downdrag and Relaxation of Piles

The proposed replacement bridge will be constructed to the east of the existing bridge which will require that the existing highway embankments will be widened and raised approximately 0.5 m above existing site grades. The native site soils underlying the abutment locations consist of compressible clay to silty clay soils that will compress over time due to application of new loads associated with the widened embankment construction. Therefore, the piles supporting the new bridge abutments will need to be designed to resist downdrag loads that will develop as a result of soil settlement adjacent to the piles. The unfactored downdrag load that will be mobilized along the length of each abutment pile installed within the silty clay soils is calculated to be of approximately 270 kN; a load factor should be applied to these downdrag loads. Piles at the central pier, if used, would not be subject to downdrag forces.

The development of downdrag loads on piles driven to bedrock would not affect the geotechnical resistance of the piles and is primarily an issue with respect to the structural capacity of the piles. Consideration could be given to implementing a preloading/surcharging program in the approach embankment areas near the proposed bridge abutments prior to pile installation in order to reduce downdrag loads if this is a concern However, as the magnitude of the expected downdrag loads would represent only a small percentage of the overall structural capacity of the piles, this is not anticipated to be required for this site.

For H-piles driven to refusal on bedrock, post-installation relaxation and/or reduction of pile capacity will not be of concern.

8.7.2.4 Preliminary Pile Installation and Capacity Testing Considerations

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS.PROV 903 – Construction Specification for Deep Foundations.

The site soils generally consist of loose to very dense embankment fill with cobbles and/or boulders over firm to very stiff silty clay/clay that is underlain by loose to very dense glacial till with cobbles and/or boulders and then by bedrock. Obstructions to the pile driving should be anticipated due to the cobbles and boulders observed in the fill and in the till. Based on these conditions, the piles should be provided with driving shoes such as Titus “H” Bearing Pile Points (Standard Model) or equivalent.

The following pile notes should be included in the “Pile Data Table”:

- The pile driving equipment shall be appropriate to the driving conditions and capable of delivering a minimum specified hammer energy of 80 kJ.

The following “Pile Driving Note” should be included:

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

The capacity of each pile should be verified in the field by the use of either the Hiley Formula (MTO Standard Structural Drawing SS-103-11) or high-strain dynamic testing (i.e. Pile Driving Analyzer (PDA) testing) to confirm that the specified ultimate capacity is achieved.

8.7.2.5 Axial Resistance in Tension

For design against uplift, the tensile resistance provided in Table 8.5 is recommended. This value is based on a pile length of 5 m.

Table 8.5: Recommended Tensile Pile Resistance

Pile Type	Minimum Pile Length (m)	Factored Geotechnical Resistance (Tension) at ULS_f (kN)
HP310x110 or HP310x132	5	80

A resistance factor, Φ, of 0.3 has been applied to calculate the ULS_f resistance. The factored geotechnical resistance (tension) at ULS_f provided above does not include the self-weight of the piles.

8.7.3 Caisson Foundations – Center Pier

8.7.3.1 Design Considerations

Concrete caisson (drilled pier) foundations socketed within the bedrock can be considered to support the centre pier of the proposed structure. For this option, the caissons would tie into the pier columns and as such would act as partially embedded piles. No pile caps would be required at the ground surface which would reduce the depths and associated duration of excavations within the highway median. The caissons are anticipated to be 1200 mm to 1500 mm in diameter.

Rock socketed caissons can be designed on the basis of shaft resistance only, end-bearing only or a combination of shaft and end-bearing resistances (complete socket). For preliminary design purposes, the drilled pier foundations are recommended to be designed on the basis of shaft friction only due to the relatively limited depth of investigation into the bedrock and the fair quality of bedrock/close joint spacings identified in several of the core runs. Further investigation, including drilling deeper boreholes at the actual pier location, is recommended if design of the drilled pier caissons using end-bearing resistances is to be considered.

The ground surface at the central pier is not planned to be altered significantly from existing grades. Therefore, the drilled pier foundations would not be subject to downdrag loads.

8.7.3.2 Axial Resistance in Compression

The caissons are recommended to be socketed into the bedrock for a minimum length of two caisson diameters and incorporate concrete with a minimum compressive strength of 35 MPa.

The following caisson capacities may be considered for preliminary design purposes; however, additional investigation would be required to confirm the bedrock conditions and the associated design parameters prior to the detailed design stage.

Table 8.6: Caisson Capacities at ULS_f (ϕ = 0.4)

Caisson Diameter (m)	Socket Length (m)	Geotechnical Resistance at ULS_f (kN)
1.2	2.4	5,300
	3.0	6,600
	3.6*	8,000
1.5	3.0	8,300
	4.5*	12,000

*Note: These socket lengths extend below the boreholes advanced as part of the preliminary foundation investigation. Additional investigation extending below the base of the drilled pier foundations will be required prior to detailed design to verify the associated ULS design resistances.

The above geotechnical resistance reflects only the shaft resistance within the rock socket.

The parameters used for the analysis were as follows: UCS of 100 MPa; RQD of 68 for the Williams and Pells shaft resistance correction factor; empirical factor b of 0.63 as per Table 18.8 of the CFEM.

A resistance factor of 0.4 has been used to develop the factored geotechnical resistance at ULS as per the CHBDC. Settlement of a rock socketed caisson is expected to be negligible and therefore the SLS resistance is not governing the rock socketed caisson design. As per CHBDC Section 6.11.4.7, minimum caisson spacing of 2.5 B should be maintained.

8.7.3.3 Caisson Installation Considerations

The supply and installation of the caissons should be according to the OPSS.PROV 903 Construction Specification for Deep Foundations as amended by SP 109F57.

The boreholes encountered deposits of silty sand till directly above the bedrock. The presence of these wet, sandy soils will necessitate the use of liners during installation of the drilled pier/caisson foundations to minimize the potential for loss of ground into the drilled piers. Liner installation would be hindered by the presence of cobbles and boulders. The use of churn drills and possibly rock coring techniques may be required to penetrate these obstructions within the glacial till.

During the shaft construction, thorough flushing and cleaning of the rock socket wall and base should be specified and verified by inspection (e.g. CCTV and/or shaft inspection device - SID).

The drilled piers/shafts will extend below the water table and encounter permeable materials including granular till deposits and fractured bedrock. If the caisson opening cannot be made dry, concrete placement should be carried out using tremie techniques.

8.8 LATERAL EARTH PRESSURES

8.8.1 Abutment Backfill

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge/underpass abutments. The materials used as backfill behind the abutments of the replacement underpass structure 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 bridge structure 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.

8.8.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and retained soil systems (if any). 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 8.7 and Table 8.8 for horizontal and sloping (2H:1V) backfill conditions, respectively. The thrusts act at a point one third up the height of the wall. For the purposes of preliminary design, a friction angle of 30 degrees has been assumed for the existing embankment fill materials at the site; this value will need to be confirmed and/or reassessed once further subsurface investigation is completed prior to detailed design.

Table 8.7: Recommended Static Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	*Existing Embankment Fill
Bulk Unit Weight, γ (kN/m^3)	21	22	21
Effective Friction Angle	32°	35°	30°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.50
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.33
Coefficient of Passive Earth Pressure (K_p)	3.25	3.69	3.00

Table 8.8: Recommended Static Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	*Existing Embankment Fill
Bulk Unit Weight, γ (kN/m^3)	21	22	21
Effective Friction Angle	32°	35°	30°
Coefficient of Earth Pressure at Rest (K_o)	0.68	0.62	0.72
Coefficient of Active Earth Pressure (K_a)	0.47	0.39	0.54

*Note: Values for existing embankment fill materials in the above tables are presented for consideration in the design of temporary protection systems; new retaining walls should be backfilled with OPSS Granular A or B materials.

8.8.3 Seismic Lateral Earth Pressures

The following design parameters are provided for use in assessing the earth pressures induced on the bridge abutment and wingwalls under seismic loading conditions.

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

$$P_{AE} = \tfrac{1}{2} K_{AE}\, \gamma\, H^2 (1 - k_V)$$

$$P_{PE} = \tfrac{1}{2} K_{PE}\, \gamma\, H^2 (1 - k_V)$$

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 8.9. Table 8.10 and Table 8.11 provide seismic earth pressures for yielding walls with horizontal and 2H:1V backfill slopes behind the walls, respectively.

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

Table 8.9: Seismic Design Parameters to Estimate Lateral Earth Pressures

Site Adjusted PGA	Horizontal Acceleration Coefficient, k_{ho}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding $(wall\ movements\ of\ 25\ mm\ to\ 50\ mm)$
0.167g	0.167	0.084
Note: k_{ho} 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.		

As noted above, a friction angle of 30 degrees has been assumed for the existing embankment fill materials at the site for the purposes of preliminary design; this value will need to be confirmed and/or reassessed once further subsurface investigation is completed prior to detailed design.

The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 8.10: 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})	3.10	3.53
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.32	0.32
Yielding Wall		
Active Earth Pressure (K_{AE}) for Yielding Wall	0.36	0.32
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.37	0.37
Non-Yielding Wall		
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.42	0.37
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.40	0.41

Table 8.11: 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	0.68	0.52
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.416	0.398

8.9 EMBANKMENT STABILITY AND SETTLEMENTS

8.9.1 Stability Of Approach Embankments

The existing North Augusta Road Underpass approach embankments are approximately 7 m high (above original grades) have crest-to-crest widths of approximately 18 m and have sideslopes of approximately 2H:1V. The approach embankments are planned to be raised slightly (i.e. by approximately 0.5 m or less at both abutments) and widened by approximately 26 m towards the east to accommodate the new bridge structure. The new approach embankments are proposed to have sideslope inclinations of approximately 2H:1V. All embankment widening should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

Analyses were carried out to evaluate the stability of the proposed widened embankments. Stability analyses were carried out for the embankment on the north side of the Hwy 401 using the commercial program Slope/W (GeoStudio, 2020) and incorporated the stratigraphy and design parameters outlined in Section 8.4. The Factor of Safety against instability of the new embankment on the south side of Highway 401 is expected to be similar to the north side embankment based on the subsurface conditions and the height of the proposed approach embankments.

The following assumptions were made as part of the settlement analysis:

- The maximum height of the embankment is about 7.5 m.
- The widened highway embankment was assumed to be constructed using compacted Select Subgrade Material (SSM).
- The static stability analysis assumed that the slope failure would occur as a rotational slip failure. The method of analysis assumes that the potential failure surface in section may be a circular arc or non-circular curve and the depth to the failure surface is controlled by a combination of slope geometry, soil properties, and depth to the groundwater table.
- The analyses included allowance for dynamic loading due to traffic by considering a static surcharge load equivalent to 0.8 m of additional fill.

A minimum factor of safety under static conditions of about 1.4 (corresponding to a ϕ_{gu} of 0.7) is considered acceptable for permanent embankments for slip surfaces extending entirely through portions of the embankments constructed out of imported granular fill materials based on the ‘High’ degree of understanding of these materials. A minimum factor of safety under static conditions of 1.5 (corresponding to a ϕ_{gu} of 0.65) is considered acceptable against deeper-seated failure surfaces extending into the native soils based on the ‘Typical’ degree of understanding of those soils.

The results of the slope stability analyses for an embankment cross-section near the north abutment (perpendicular to North Augusta Road) under static, drained conditions and seismic conditions are provided on Figures E2 and E3, respectively, in Appendix E. The results of the stability analyses indicate that the proposed embankment configurations, which incorporate slope angles of 2H:1V, would provide a factor of safety against instability of 1.5 under static conditions for a critical failure surface extending up to the crest of the embankment. A factor of safety of greater than 1.4 was calculated under seismic conditions. Stability analyses carried out using undrained parameters provided similar or higher factors of safety.

8.9.2 Embankment Settlement

Analyses were carried out to evaluate the magnitude of settlement of the soils underlying the embankments due to the proposed widening and grade raise of the approach embankments. The evaluation of settlements for the embankment on the north side of the Hwy 401 was carried out using the commercial program Settle3D (Rocscience 2020). Settlements on the south side of Hwy 401 are expected to be similar to or less than the north side based on the subsurface conditions and planned widening/grade raise.

The following assumptions were made as part of the settlement analysis:

- The typical soil profile and associated design parameters shown in Table 8.1 were considered in the settlement analyses.
- The maximum height of the embankment grade raise is limited to about 0.5 m.

- The new embankment platform involves widening the existing approach embankments approximately 26 m towards the east and final embankment sideslopes of 2H:1V.
- The load from the bridge abutments will be transferred to the bedrock by the piles and hence will not contribute significantly to the settlement of the embankment.
- The estimated preconsolidation pressures of the silty clay/clay deposit are expected to be higher than the anticipated post-construction stresses in these deposits. Therefore, substantial consolidation settlements of the cohesive native soils are not expected to occur and only recompression settlement was considered in the analyses.

The analysis included evaluation of settlements under both the current and widened/raised embankment areas. The results of the analyses indicate that, for the conditions presented herein, the maximum incremental vertical settlement of the native soils beneath the new approach embankments leading up to the widened bridge is expected to be in the order of 60 mm due to the additional loading imposed by the proposed widening/grade raise of the approach embankments. Settlements beneath the existing roadway are calculated to be less than 10 mm. These settlements are anticipated to take place relatively rapidly and to be predominantly complete during construction of the embankments. Post-construction settlements are expected to be approximately 10 percent or less of the total values referenced above assuming a minimum 4-month construction period.

Self-weight settlement due to compression of the maximum 7.5 m of embankment fill placed during the construction process is expected to be less than 37.5 mm (approximating 0.5 % strain). The bulk of this settlement is expected to be completed almost immediately after the fill has achieved its full height.

Embankment settlements must meet the Post-Construction Settlement Criteria for New Embankments outlined in the MTO document titled ‘Embankment Settlement Criteria for Design (2010)’. Based on the analysis completed, the post-construction settlements of the new embankments are expected to be less than 25 mm. This magnitude of settlement meets the Post-Construction Settlement Criteria for New Embankments outlined in the MTO document.

8.10 CEMENT TYPE AND CORROSION POTENTIAL

Two soil samples, one from each borehole location, were submitted to Paracel Laboratories 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 analysis results are summarized in Table 5.5.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations of the samples tested were 26 and 86 µg/g. As per Canadian Standards Association (CSA) Standard A23-1.14/A23.2-14, soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil 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 of the samples tested were 7.45 and 7.94 which is within the normal pH range for soil (5.5 to 9.0). However, 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 for piles; one of the reported resistivity values was below that level.

The test results provided in Table 5.5 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

9.0 CONSTRUCTION CONSIDERATIONS

9.1 CONSTRUCTION STAGING AND DETOUR

A detour is not anticipated to be required for traffic on North Augusta Road as the existing bridge will remain in place during the construction of the new bridge.

The construction of the foundations for the new central pier of the bridge is anticipated to involve staging and/or shifting of traffic lanes away from the Highway 401 median using appropriate traffic control. The use of a temporary roadway protection system may be required near the centerline of existing Highway 401 and near the abutments to permit the foundation construction.

9.2 TEMPORARY ROADWAY PROTECTION

Temporary roadway protection may be required to protect traffic on Highway 401 or maintain traffic on North Augusta Road during excavations for the foundations of the replacement underpass structure.

The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters.

The following table compares the available roadway protection options considered for the proposed rehabilitation:

Table 9.1: Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Soldier piles with timber lagging; (struts/rakers as required)	• Simple installation process	• Additional labour required • Groundwater seepage into the excavation can occur without groundwater control • Removal of soldier piles can be difficult	Low	• Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented • Potential for minor loss of ground at rear of lagging
Steel sheet piles (SSP)	• Simple installation process • Provides cut-off to groundwater seepage from sides of excavation • Can be incorporated with groundwater cut-off system for excavation for removal of weak soils at west abutment of TMB	• Difficult to drive/install in soils where cobbles/boulders are present • May require large sections where cantilever design is adopted	Medium	• Potential for sheet piles to either be damaged, deflected or meet refusal due to obstructions (e.g. boulders within the till) during driving

Both of the above noted support systems are considered feasible for use based on the ground conditions present at this site. The temporary support systems should be supported with struts or rakers from the construction side or tie-backs/ground anchors.

Roadway protection design should meet the requirements of Performance Level 2 in accordance with OPSS.PROV 539 and should consider traffic loading. 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 bridge replacement process as described in OPSS.PROV 539.

9.3 EXCAVATION AND BACKFILLING

Excavation and backfilling for the new bridge structure should be carried out in accordance with OPSS.PROV 902 Construction Specification for Excavation and Backfilling – Structures.

Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath the proposed bridge foundations and any associated retaining/wing walls. Where deleterious materials are encountered at the 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.

Grading work should be carried out in accordance with OPSS.PROV 206 Construction Specification for Grading and SP 206S03. Where existing embankments are to be widened, the new fill materials should be benched into the existing embankments in accordance with OPSD 208.010.

All side slopes for open cut excavations should conform to the Occupational Health and Safety Act regulations for Construction Projects (OHSA). The excavations required for construction of the new pier and abutment foundations would extend to several metres depth and be developed through the existing highway and North Augusta Road approach embankment fill. These excavations are expected to encounter fill materials and the native, stiff to very stiff silty clay/clay deposits. Where space permits, these excavations may be developed using open-cut methods. The fill materials (above the water table) and the stiff to very stiff silty clay/clay deposit would be classified as Type 3 soils.

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. Granular soils (fill materials and/or native overburden) below the water table, if encountered, would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements.

9.4 TEMPORARY GROUNDWATER CONTROL

The groundwater level was observed at elevations of approximately 95.7 m to 96.5 m at the time of the investigation. Therefore, excavations required for construction of the new bridge foundations are expected to extend below the ground water level. Temporary unwatering, using conventional sump and pump techniques, is considered appropriate for shallow excavations at the site developed predominantly within the clayey silt deposits.

Increased groundwater inflow should be expected where excavations or drilled piers extend into or through the saturated glacial till deposits and the fractured bedrock. Dewatering to lower the water level within the glacial till and bedrock units could be considered to reduce the potential for encountering groundwater, and associated difficulties installing caisson foundations for the central pier. The requirements for unwatering/dewatering should be further reassessed during the detailed design stage once the preferred foundation system has been selected and additional information on the site soil and bedrock conditions at the central pier location is available.

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

9.5 EXISTING PILE FOUNDATIONS

The abutments and associated retaining walls of the existing bridge are supported on steel H-piles including battered piles inclined towards the existing highway. Based on the preliminary design information available for the new bridge, the foundation units for the new bridge will generally be located to the east of the existing abutments and walls and these piles are not expected to interfere with the construction of the south abutment foundation. However, the piles are expected to be encountered within the excavation zone for the widened highway corridor and may extend near to the asphalt-surfaced shoulder of the ultimate widened highway configuration. Piles extending into this area should either be extracted or cut-off a minimum of 1.0 m below the pavement subgrade level.

9.6 OBSTRUCTIONS

Cobbles and/or boulders are present in the fill materials and till deposits at this site. These materials could obstruct excavations and the installation of pile foundations and temporary roadway protections systems. In addition, the pile foundations of the existing bridge will also obstruct the excavation for the widening of the highway. A Non-Standard Special Provision (NSSP) should be developed during the detailed design stage for inclusion in the contract to address this issue.

10.0 FURTHER WORK FOR DETAILED DESIGN

Based on the subsurface conditions encountered in the current investigation, driven pile foundations at the abutments and driven piles or drilled pier (caisson) foundations at the central pier are the preferred foundation types to be used in the preliminary design of the underpass replacement at this site.

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 for the bridge structure. The standard minimum MTO foundation investigation for a bridge structure (i.e. two boreholes at each foundation unit advanced to 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 given the relatively uniform bedrock surface encountered at the borehole locations. If caissons or drilled pier foundations are considered for use, boreholes at the centre pier are recommended to be cored a minimum of 5 m below the bedrock surface to provide information for evaluating the end-bearing capacity of the drilled pier foundations.
- One borehole within 20 m of the new bridge abutments in the area of each approach embankment.
- Boreholes should be advanced through the existing approach embankments to determine the type, thickness and consistency/density of the existing fill materials and their potential impact on the design of the new bridge.
- Additional boreholes should also be advanced as per MTO Standards for any retaining walls or temporary roadway protection systems required for construction staging purposes.
- Piezometers/monitoring wells should be installed to confirm the water level within the existing fill embankment.
- An evaluation of the in-situ permeability of the bedrock and glacial till should be carried out to assess the dewatering efforts that would be required to support a caisson construction option for the centre pier.
- Following completion of the additional investigation and laboratory testing, the soil design parameters outlined in this report should be re-evaluated and a detailed assessment of the potential for differential settlement be undertaken if differing foundation types (i.e. shallow and deep foundations) are planned to be used.
- A Final Foundation Investigation and Design Report meeting MTO's standard requirements for foundation engineering assignments should be prepared based on the final structure configuration.

11.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 11.1: 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 3101.150	Walls, abutment, backfill – Minimum Granular Requirements
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 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates
SP517F01	Amendment to OPSS 517, July 2017
SP105S10	Construction Specification for Compaction
SP109S12	Amendment to OPSS 902, November 2010
SP 206S03	Earth Excavation, Grading

12.0 CLOSURE

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered by others 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.

This report was prepared by Roshan Rashed, P.Eng. and reviewed by Kevin Nelson, P.Eng., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.

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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Designated Principal MTO Foundations Contact

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13.0 REFERENCES

American Association of State Highway and Transportation Officials. 2012. AASHTO LFRD Bridge Design Specifications, Washington DC

ASTM. 1999. Standard Test Methods for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.

ASTM. 2000. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487). ASTM International, West Conshohocken, PA.

Canadian Standards Association. 2014. Standard A23-1.14/A23.2-14. Concrete Materials and Methods of Concrete Construction / Test Methods and Standard Practices for Concrete

C.C. Parker and Associates Ltd. Consulting Engineers. 1955. General Arrangement - Elizabethtown Township Bridge No. 15, County Road No. 18 over Highway 401 near Brockville - Twp. Drawing No. 25-124-1-A.

Chapman, L.J. and D.F. Putnam. 1984. The Physiography of Southern Ontario, Ontario Geologic Survey

CHBDC. 2019. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

NBC. 2015. National Building Code of Canada Vol.1. National Research Council of Canada, Ottawa, Ontario.

OHSA. 2015. Occupational Health and Safety Act Regulations for Construction Projects. Carswell, Toronto Ontario

Ontario Geological Survey. 1982. Paleozoic Geology of the Brockville – Mallorytown Area

Ontario Geological Survey. 2010. Surficial Geology of Southern Ontario GIS data set.

Ontario Ministry of Transportation (MTO). 2010. MTO Embankment Settlement Criteria for Design.

Ontario Ministry of Transportation (MTO). 2014. Structural Manual. Bridge Office, St. Catharines, Ontario.

APPENDIX A

A.1 DRAWING NO. 1 – BOREHOLE LOCATION PLAN AND SOIL STRATA PLOT

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

PLATE No

CONT
GWP 4003–19–00

NORTH AUGUSTA ROAD
UNDERPASS REPLACEMENT
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
–

Stantec

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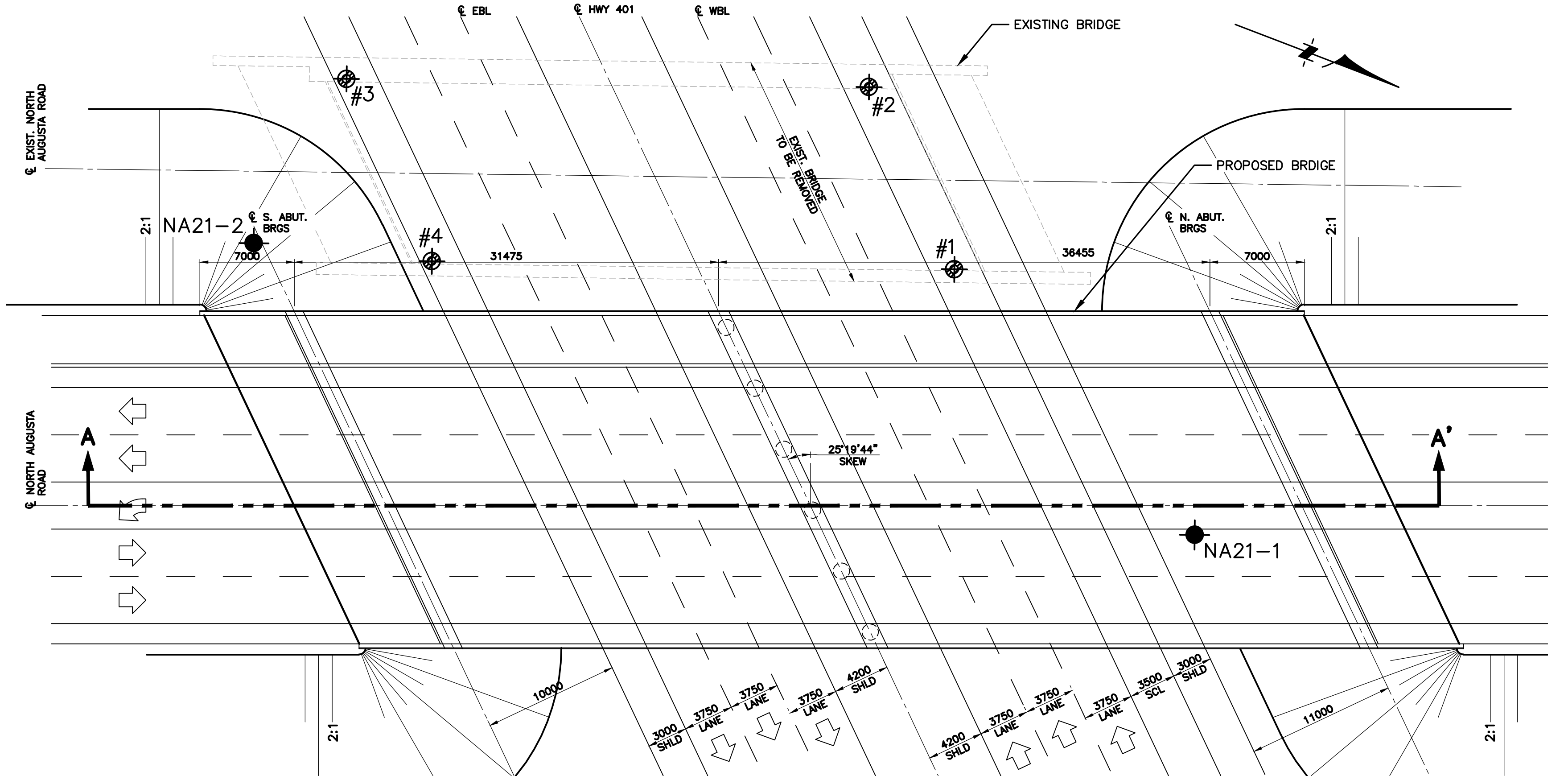
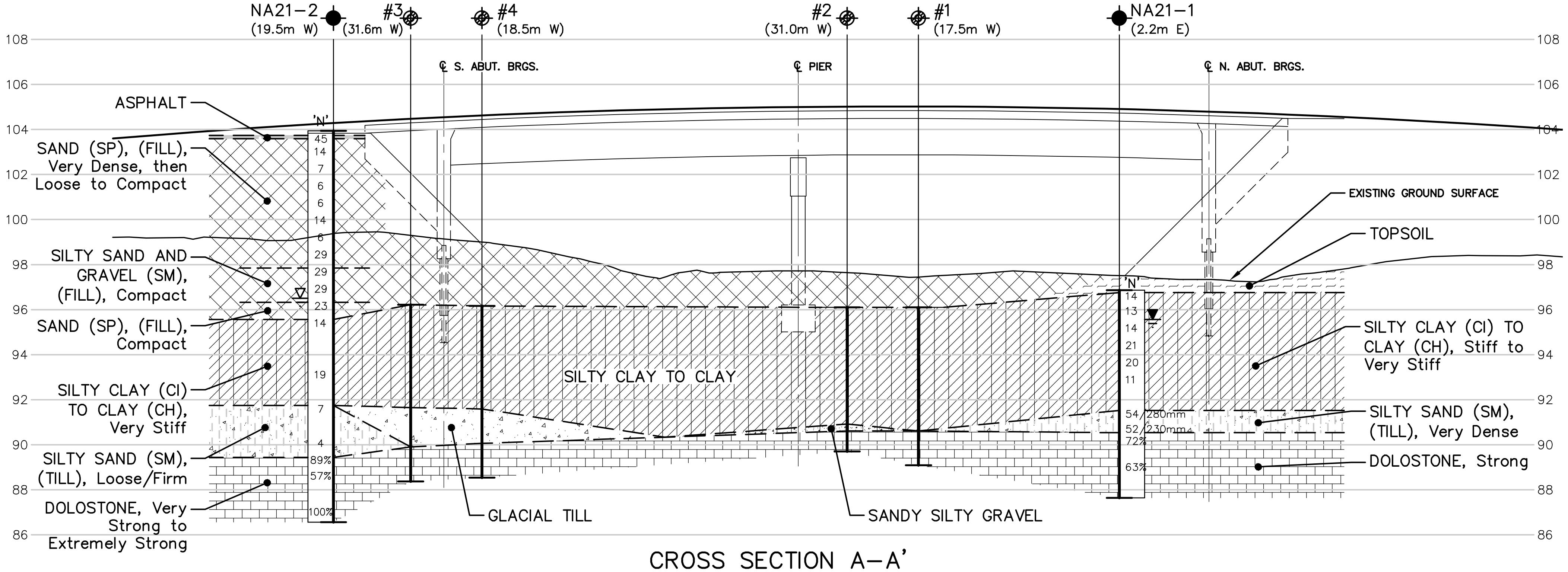
2022-03-30
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
J.G.A.R. HACHÉ
2022-03-30
PROVINCE OF ONTARIO



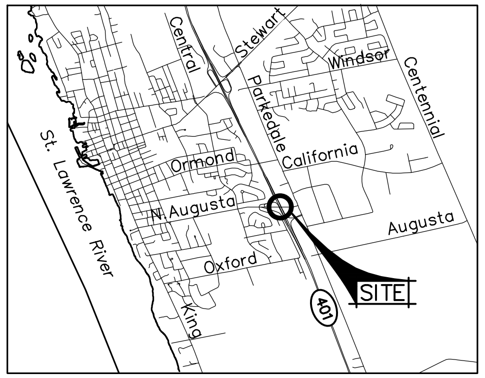
PLAN

SCALE
5m 0 5 10m



CROSS SECTION A–A'

SCALE
5m 0 5 10m HORIZONTAL
2.5m 0 2.5 5.0m VERTICAL



KEY PLAN

1km 0 1 2km

LEGEND

●	Borehole (Stantec 2021)
⊕	Test Hole (Racey McCallum 1955)
(x.xm)	Offset from Cross Section Line in meters
N	Blows/0.3m (Std Pen Test, 475 J/blow)
▽	WL at time of Investigation May 2021
▼	WL Measured on May 2021

No	ELEV	MTM ZONE 9 COORDINATES NORTH	EAST
NA21–1	96.9	4 941 616.9	369 479.9
NA21–2	104.1	4 941 545.1	369 488.3
#1	96.1	4 941 593.2	369 467.9
#2	96.1	4 941 582.5	369 457.6
#3	96.2	4 941 546.1	369 471.0
#4	92.2	4 941 556.8	369 481.2

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

GEOCRES No 31B–99

HWY No 401			DIST
SUBM'D KN	CHECKED	DATE 2022–03–30	SITE
DRAWN GBB	CHECKED	APPROVED	DWG 1

DRAWING NAME: 165001160_North Augusta.dwg
CREATED BY: GBB
MODIFIED:
C:\CAD Drawings\Acad2019 Drawings\2022\165001160\North Augusta\165001160_North Augusta.dwg (PP) Printed: Mar 30, 2022

MINISTRY OF TRANSPORTATION, ONTARIO PR–D–707 88–05

APPENDIX B

B.1 GENERAL ARRANGEMENT DRAWING - EXISTING BRIDGE

PLAN OF DECK

SCALE = 1/8" = 1'0"

WEST SIDE ELEVATION

SCALE = 1/8" = 1'0"

KEY TO ELEVATIONS

STA. 0+00 N. AUGUSTA RD. = 278+53 OF HWY 401

T.E.C. - TOP OF ESCAPE CURB.
T.S.W. - TOP OF SIDEWALK.

KEY PLAN

ELEVATIONS SHOWN ON ROADWAY ARE TO TOP OF PAVEMENT EXCEPT AS OTHERWISE NOTED

SCALE = 1/16" = 1'0"

TYPICAL CROSS SECTION

SCALE = 1/8" = 1'0"

PROPERTIES OF 64° ANGLE

SIN. - 0.898794
COS. - 0.438371
TAN. - 2.050304

TEST HOLE #1 #2 #3 #4

TEST BORING RESULTS

BORINGS MADE BY RACEY, MacCALLUM & ASSOC. LTD - OCT. 6/54. BORINGS ARE FOR GENERAL INFORMATION ONLY AND ARE NOT GUARANTEED BY THE DEPARTMENT.

FOR CONSTRUCTION JOINT DETAILS A & B - SEE DRWG. No D-3528-5

1. STRUCTURE TO BE BUILT IN ACCORDANCE WITH THE SUPPLEMENTAL SPEC. FOR STRUCTURES FORM No 9 & THE SPECIAL SPEC. ATTACHED TO THE "INFORMATION TO BIDDERS SHEET AND GENERAL SPECIFICATIONS FOR HIGHWAY BRIDGES - ONTARIO 1935.
2. CONCRETE WORK MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DIVISION ENGINEER.
3. CONCRETE TO HAVE A MINIMUM CRUSHING STRENGTH OF 3,000 LB/SQ.IN. AT 28 DAYS.
4. CONCRETE TO HAVE ¼ LB. POZZOLITH "S" PER BAG OF CEMENT.
5. CONCRETE MUST NOT BE POURED UNTIL FORMWORK, FALSEWORK AND REINFORCING STEEL HAVE BEEN CHECKED AND APPROVED BY THE ENGINEER.
6. EXCAVATIONS FOR PILE CAPS TO BE CUT AS NEAT AS POSSIBLE AND FILLED COMPLETELY WITH CONCRETE.
7. ALL PILES TO BE DRIVEN TO REFUSAL.
8. DECK SHORING MUST NOT BE STRUCK UNTIL PERMITTED BY THE ENGINEER.
9. BACKFILL BEHIND ABUTMENTS MUST BE WELL COMPACTED AS PLACED AND PLACED EVENLY AGAINST BOTH ABUTMENTS AT THE SAME TIME

LEVEL OF EXISTING GROUND TAKEN FROM D.H.O. PLAN F-3021-1 DATED JULY, 1954
FOR PROFILES OF NORTH AUGUSTA ROAD AND HIGHWAY 401 SEE D.H.O. PROFILE DM-4396 JULY 6, 1954.

H-351
C. C. PARKER

REVISIONS:		
DATE	BY	DESCRIPTION

C. C. PARKER & ASSOC. LTD
HAMILTON CONSULTING ENGINEERS ONTARIO

DEPARTMENT OF HIGHWAYS:-ONTARIO-
BRIDGE OFFICE:-TORONTO

ELIZABETHTOWN TOWNSHIP BRIDGE No 15
COUNTY ROAD No 18 OVER HIGHWAY 401
NEAR BROCKVILLE

THE KING'S HIGHWAY No. 401 DIV. No. 8
CO. LEEDS
TWP. ELIZABETHTOWN LOT 8 CON. I

GENERAL ARRANGEMENT

APPROVED

CHIEF BRIDGE ENGINEER

CHIEF ENGINEER

DESIGN	J.M.	CHECK	S.S.	CONTRACT NUMBERS	
DRAWING	G.S.N.	CHECK	S.S.		
TRACING		CHECK		LOADING	DRAWING NUMBER D-35[illegible]
DATE JA 1955 [redacted]				H20-S16	

~~16-124~~ ~~D3528~~ ~~1-60-6~~ Twp. # 25-124-1-A

PRINT RECORD

No.	FOR	DATE
2	Parker	26.5.55
30	TENDER	11.3.57
12	[illegible]	12.4.57

APPENDIX C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

C.2 BOREHOLE RECORDS

C.3 ROCK CORE PHOTOGRAPHS

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 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
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>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	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>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

▼	measured in standpipe, piezometer, or well
▽	inferred

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

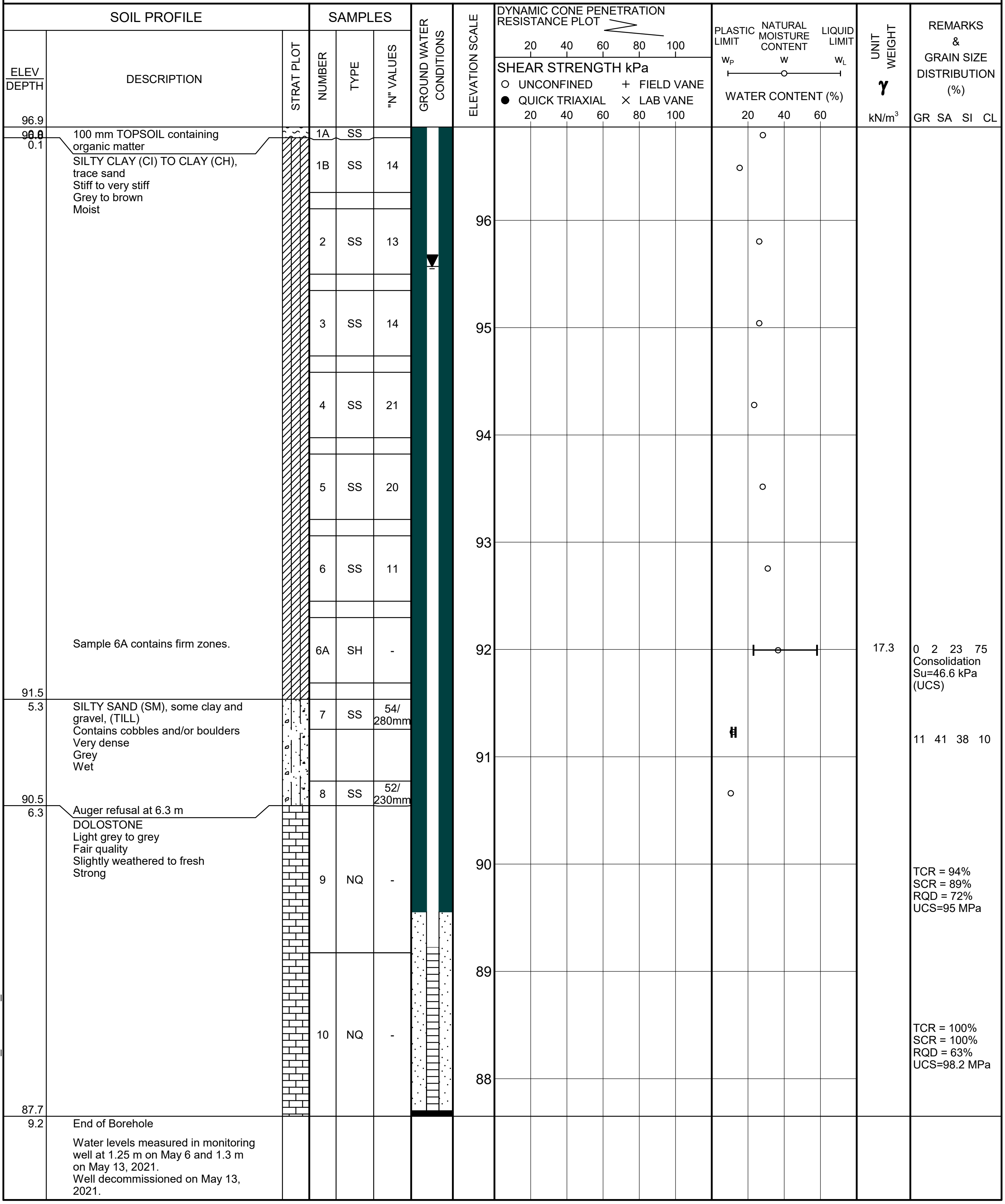
S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_U	Unconfined 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 NA21-1

1 OF 1 **METRIC**

W.P. GWP 4003-19-00 LOCATION Highway 401 - Brockville N:4941616.9 E:369479.9 ORIGINATED BY KT
DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring COMPILED BY KL
DATUM Geodetic DATE 2021.05.03 - 2021.05.03 LATITUDE 44.611652 LONGITUDE -75.685081 CHECKED BY KN



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
96.9						
96.8 0.1	100 mm TOPSOIL containing organic matter	1A	SS			
	SILTY CLAY (CI) TO CLAY (CH), trace sand Stiff to very stiff Grey to brown Moist	1B	SS	14		
		2	SS	13		
		3	SS	14		
		4	SS	21		
		5	SS	20		
		6	SS	11		
	Sample 6A contains firm zones.	6A	SH	-	17.3	0 2 23 75 Consolidation Su=46.6 kPa (UCS)
91.5 5.3	SILTY SAND (SM), some clay and gravel, (TILL) Contains cobbles and/or boulders Very dense Grey Wet	7	SS	54/ 280mm		11 41 38 10
		8	SS	52/ 230mm		
90.5 6.3	Auger refusal at 6.3 m DOLOSTONE Light grey to grey Fair quality Slightly weathered to fresh Strong	9	NQ	-		TCR = 94% SCR = 89% RQD = 72% UCS=95 MPa
		10	NQ	-		TCR = 100% SCR = 100% RQD = 63% UCS=98.2 MPa
87.7 9.2	End of Borehole Water levels measured in monitoring well at 1.25 m on May 6 and 1.3 m on May 13, 2021. Well decommissioned on May 13, 2021.					

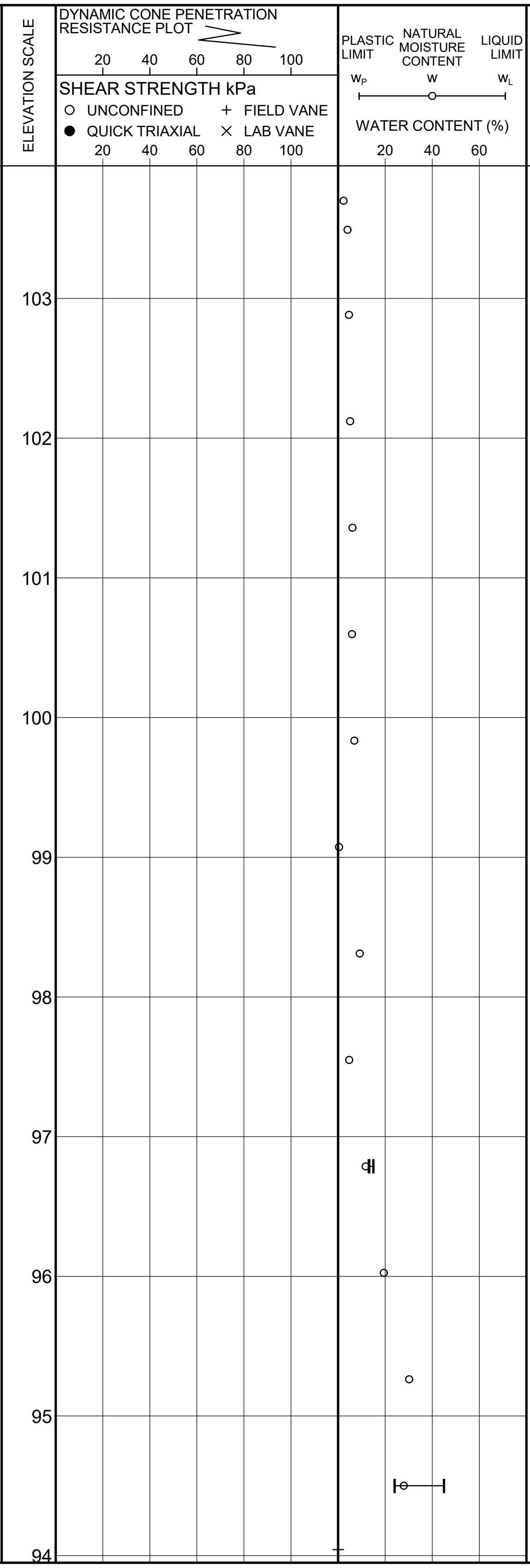
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No NA21-2

1 OF 2 **METRIC**

W.P. GWP 4003-19-00
LOCATION Highway 401 - Brockville N:4941545.1 E: 369488.3
ORIGINATED BY KT
DIST East HWY HWY 401
BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring
COMPILED BY KL
DATUM Geodetic
DATE 2021.05.06 - 2021.05.06 LATITUDE 44.610996 LONGITUDE -75.68503
CHECKED BY KN

SOIL PROFILE		SAMPLES					REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
104.0 0.0	200 mm ASPHALT									
103.8 0.2	150 mm GRAVELLY SAND, (FILL)									
103.6 0.4	SAND (SP), trace silt and gravel, (FILL) Contains cobbles and/or boulders Very dense, becoming loose to compact below 0.8 Brown Moist	1	SS	45						
		2	SS	14						
		3	SS	7			0	95	4	1
		4	SS	6						
		5	SS	6						
		6	SS	14						
	Difficult drilling and frequent auger grinding noted below 4.5 m depth Minimal recovery, rock fragments in tip of split spoon	7	SS	6						
	Rock fragments in tip of split spoon	8	SS	29						
97.9 6.1	Silty SAND and GRAVEL (SM) Contains cobbles and boulders Compact Brown Moist	9	SS	29			37	38	22	3
97.1 6.9	Silty SAND (SM), trace gravel (FILL) Contains cobbles and/or boulders Compact Brown Moist	10	SS	29	∇		9	49	37	5
96.3 7.6	SAND (SP), trace silt, (FILL) Compact Brown Wet	11	SS	23						
95.6 8.4	SILTY CLAY (CI) to CLAY (CH), trace sand Very stiff Grey Moist	12	SS	14						
		13	SH	-		18.4	0	1	51	48 Consolidation
94.0	N-vane refusal at 9.9 m depth									



Continued Next Page

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

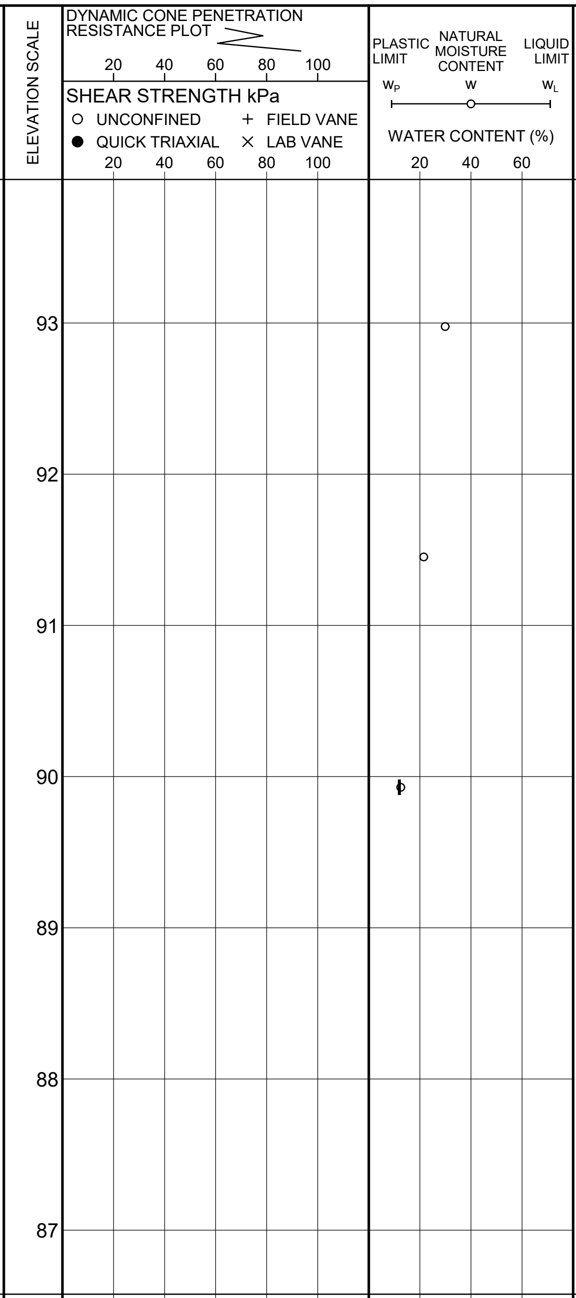
RECORD OF BOREHOLE No NA21-2

2 OF 2

METRIC

W.P. GWP 4003-19-00
LOCATION Highway 401 - Brockville N:4941545.1 E: 369488.3
ORIGINATED BY KT
DIST East HWY HWY 401
BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring
COMPILED BY KL
DATUM Geodetic
DATE 2021.05.06 - 2021.05.06 LATITUDE 44.610996 LONGITUDE -75.68503
CHECKED BY KN

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10.0	SILTY CLAY (CI) to CLAY (CH), trace sand Very stiff Grey Moist to wet				
		14	SS	19	
91.8 12.2	SILTY SAND (SM), some gravel (TILL). Contains zones of CLAYEY SILT (TILL). Contains cobbles and boulders Loose/Firm Grey Wet	15	SS	7	
		16	SS	4	13 43 36 8
	Auger Refusal at 14.2 m				
89.5 14.5	DOLOSTONE to Sandy DOLOSTONE Light grey with light brown zones Fair to good quality Slightly weathered to fresh Very strong to extremely strong	17	NQ	-	TCR = 100% SCR = 100% RQD = 89% UCS=156.4 MPa
		18	NQ	-	TCR = 87% SCR = 82% RQD = 57%
	Excellent quality below 16.8 m	19	NQ	-	TCR = 100% SCR = 100% RQD = 100% UCS=270.7 MPa
86.6 17.4	End of Borehole Groundwater observed below 7.5 m (~Elev. 96.5 m) during drilling.				



UNIT WEIGHT γ kN/m³

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

Stantec	Project No.: 165001160	Rock Core Photographs
	Site Name: Hwy 401 at North Augusta Road	

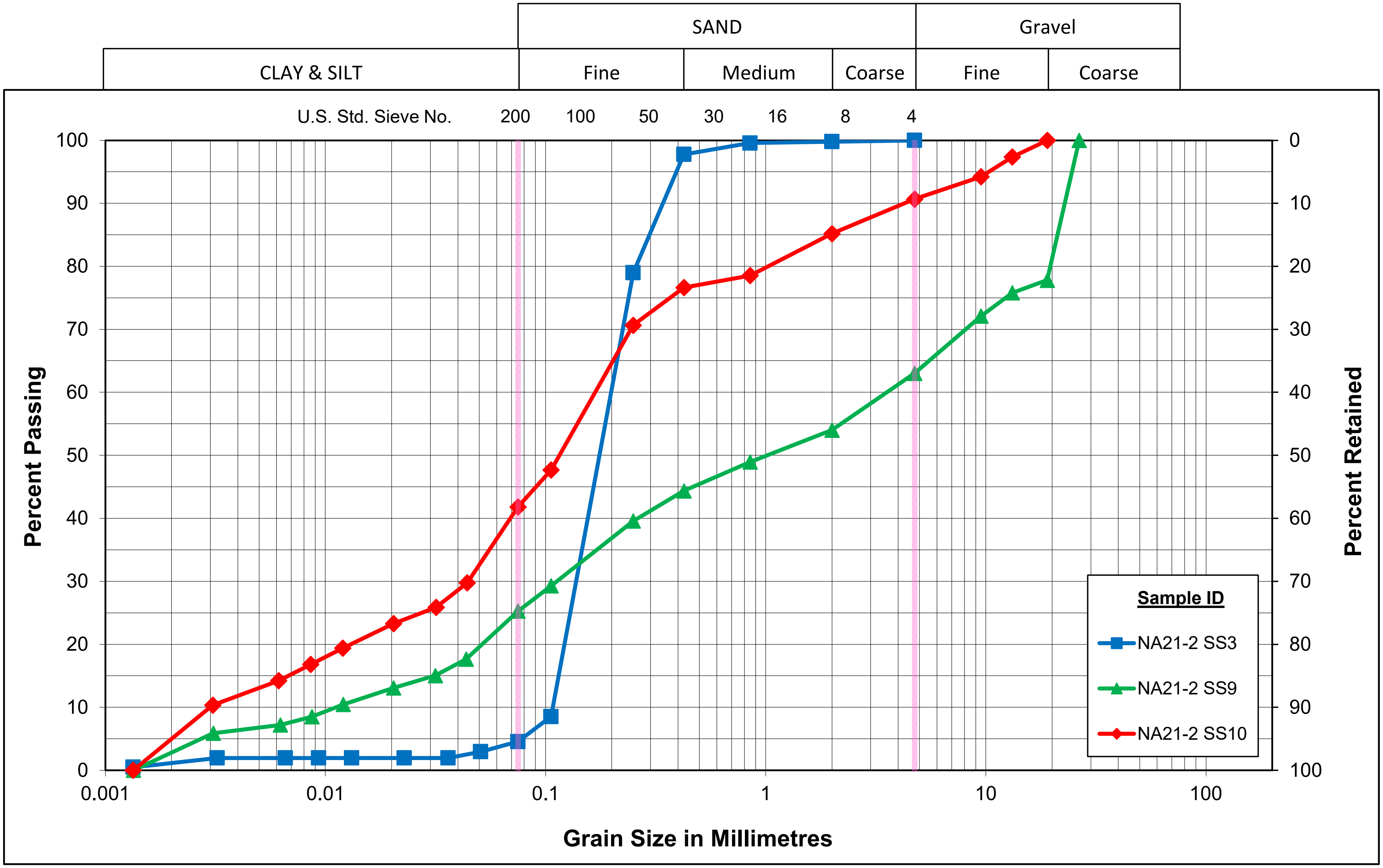
Rock Core Photo No.: 1	Borehole: NA21-1	Depth: 6.3 m to 9.2 m

Rock Core Photo No.: 2	Borehole: NA21-2	Depth: 14.2 m to 17.4 m

APPENDIX D

D.1 LABORATORY TEST RESULTS

Unified Soil Classification System



Stantec

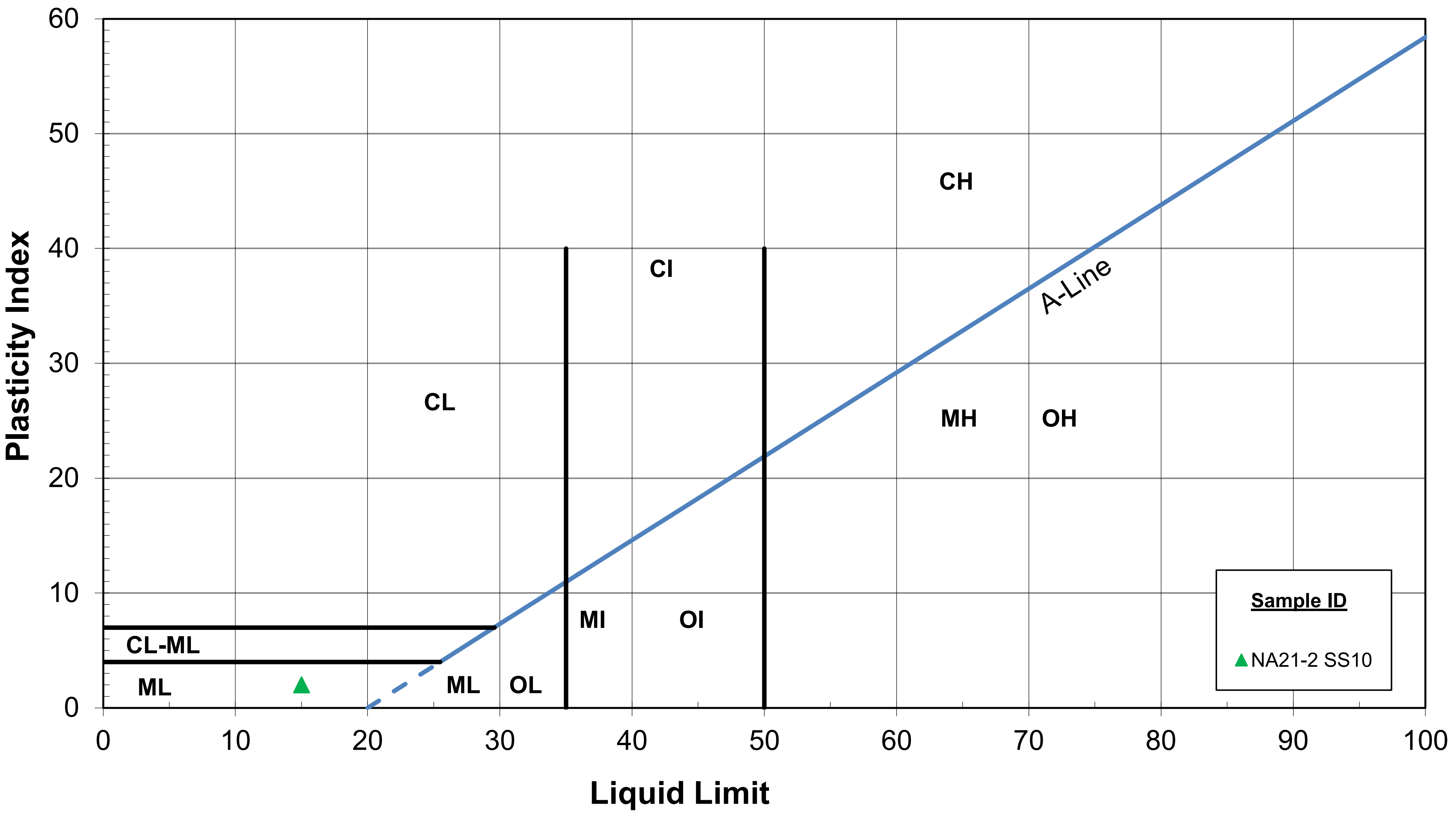
GRAIN SIZE DISTRIBUTION

FILL: SAND (SP) to Silty SAND/Silty SAND and GRAVEL (SM)

Hwy 401 - North Augusta Road Underpass

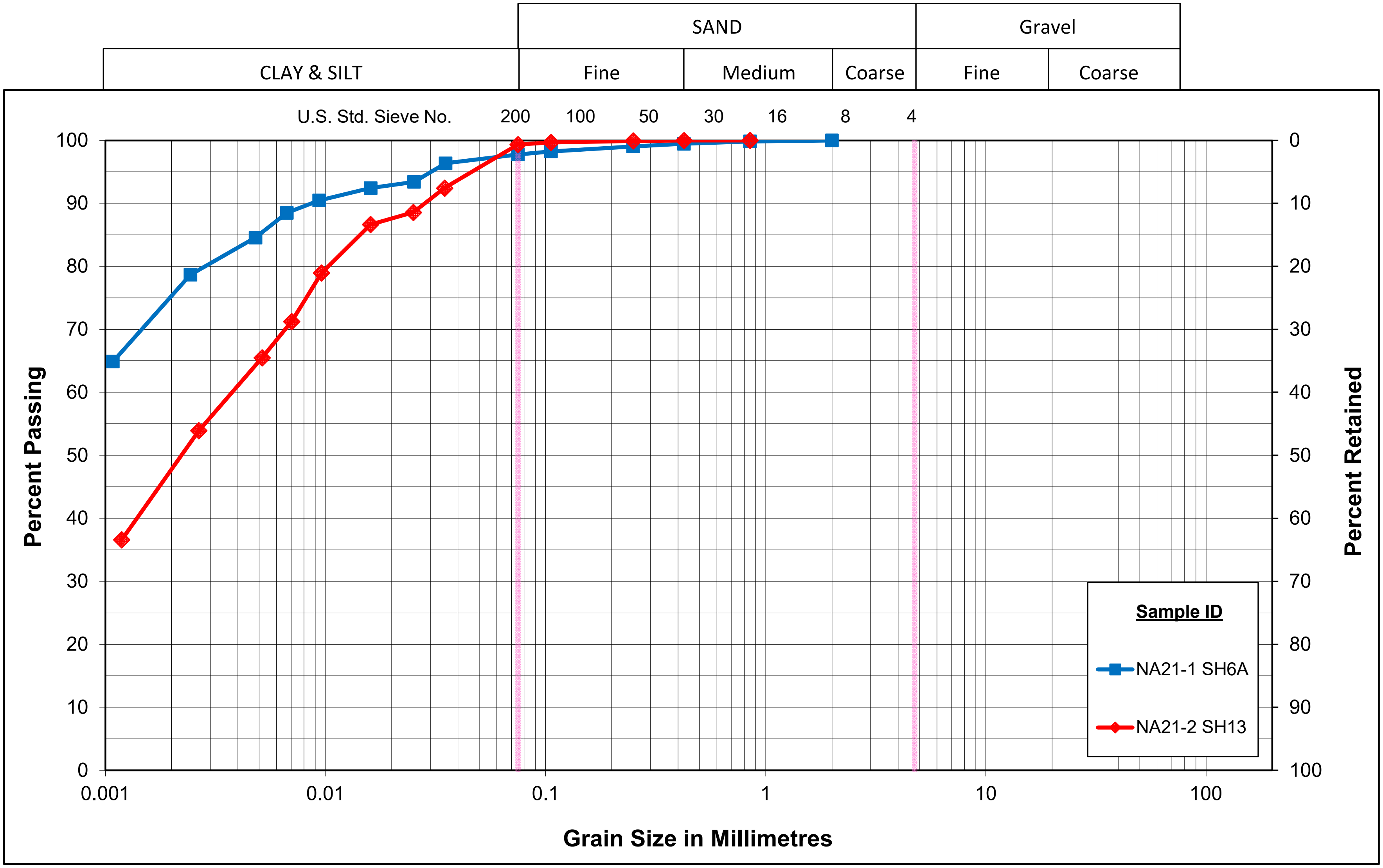
Figure No. D1

Project No. 165001160



	FILL: Silty SAND Hwy 401 - North Augusta Road Underpass PLASTICITY CHART	Figure No. D2 Project No. 165001160

Unified Soil Classification System



Stantec

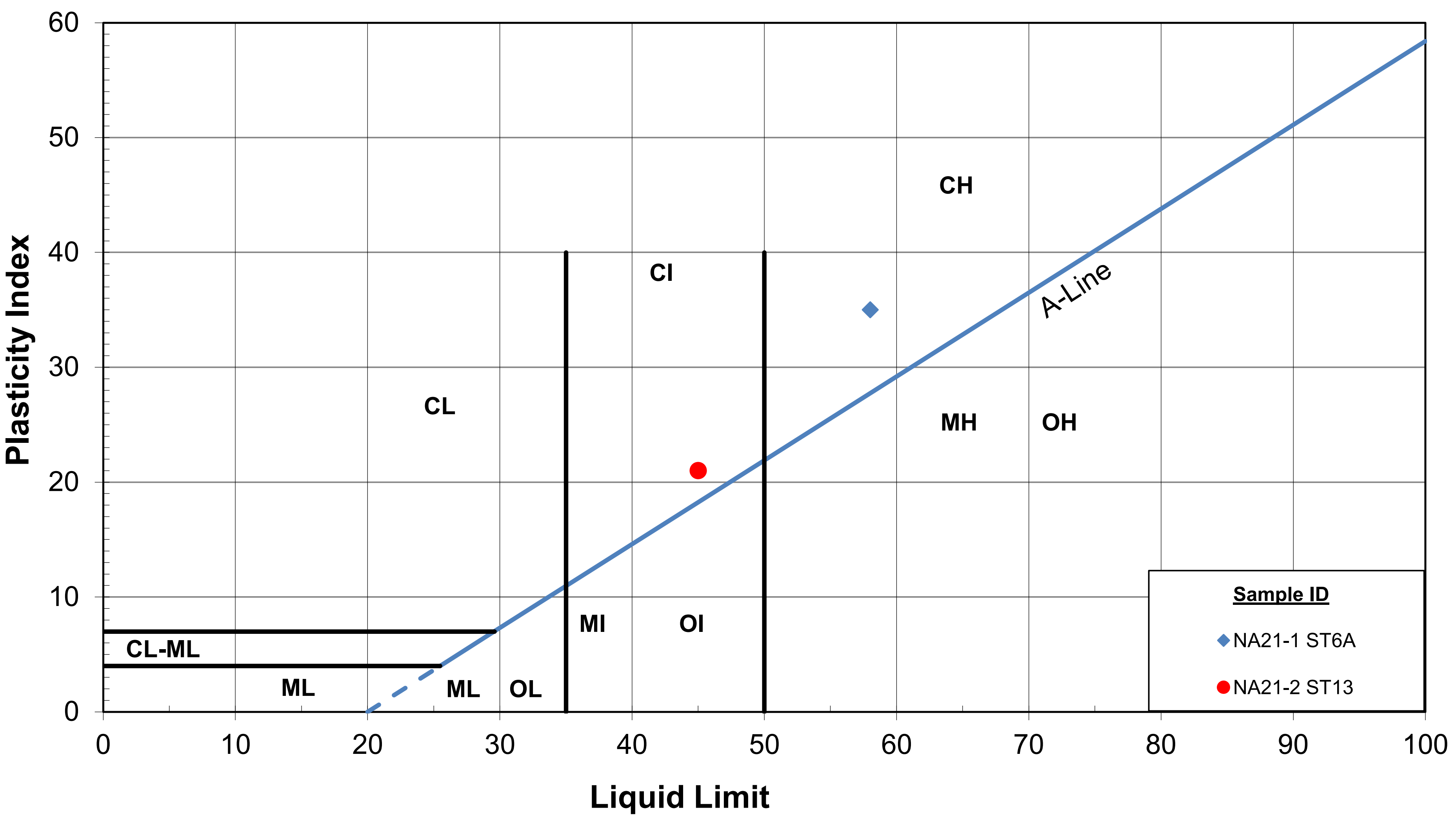
GRAIN SIZE DISTRIBUTION

SILTY CLAY (CI) to CLAY (CH)

Hwy 401 - North Augusta Road Underpass

Figure No. D3

Project No. 165001160



Stantec	SILTY CLAY (CI) to CLAY (CH) Hwy 401 - North Augusta Road Underpass PLASTICITY CHART	Figure No. D4 Project No. 165001160

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Project	**Highway 401 Brockville EA**
Project No.	**165001160.309**
Borehole No.	**NA 21-1**
Sample No.	**SA-6A**
Sample Depth	**4.57 - 5.18 m.**

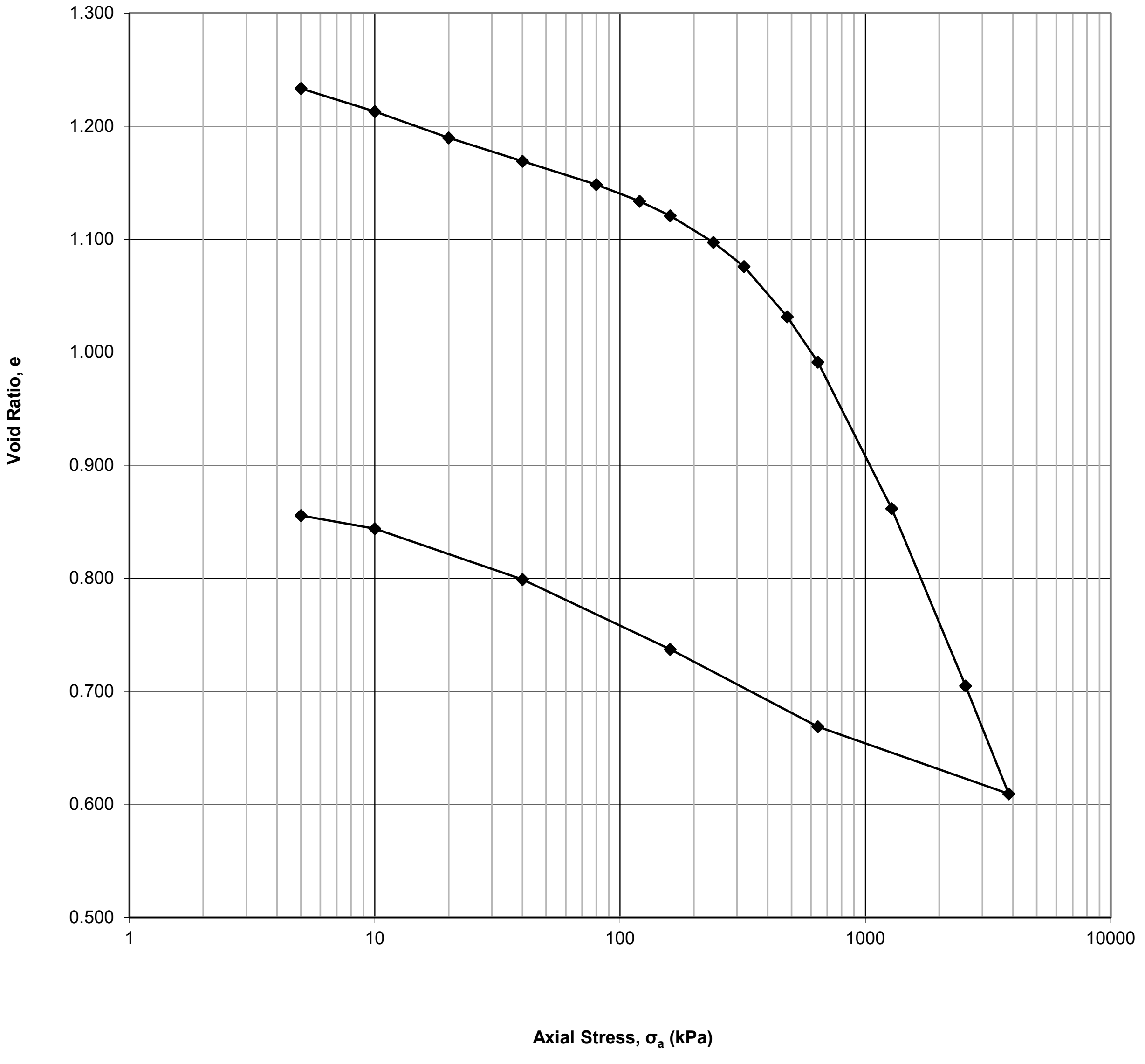


Figure D5A

Checked by: D. Boateng Date: June 15, 2021
Approved by: R. Dey Date: June 15, 2021

Stantec

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	Highway 401 Brockville EA
Project Location	Ontario, Canada
Borehole	NA 21-1
Sample No.	SA-6A
Depth	4.57 - 5.18 m.
Sample Date	May 3, 2021
Test Number	Two
Technician Name	Daniel Boateng

Soil Description & Classification

1. Silty clay, firm to stiff, brown/grey, fraible, moist (4.57-4.98 m); 2. Silty clay, firm, grey, varved, moist (4.98 - 5.18 m)

Specific Gravity of Solids		2.728
Average water content of trimmings	%	36.56

Additional Notes (information source, occurence and size of large isolated particles etc.)

Test sample taken from non-varved section

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm^2	1963
Volume	mm^3	39270
Mass	g	68.07
Dry Mass	g	47.45
Density	Mg/m^3	1.733
Dry Density	Mg/m^3	1.208
Water Content	%	43.46
Degree of Saturation	%	94.3
Height of Solids	mm	8.86
Initial Void Ratio		1.258

Final Specimen Conditions

Water Content	%	36.92
Final Void Ratio		0.856
Final Height	mm	16.44

Figure D5B

Filename: V:\01216\active\other_pc_projects\165001160\03_data\laboratory\internal\Task No. 3
Date: June 15, 2021

Checked by: D. Boateng Date: June 15, 2021
Approved by: R. Dey Date: June 15, 2021

Stantec

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	Highway 401 Brockville EA
Project Location	Ontario, Canada
Borehole	NA 21-1
Sample No.	SA-6A
Depth	4.57 - 5.18 m.
Sample Date	May 3, 2021
Test Number	Two
Technician Name	Daniel Boateng

Test Procedure

Date Started		June 8, 2021
Date Finished		June 9, 2021
Machine Number		Frame D
Cell Number		D
Ring Number		D
Trimming Procedure		Trimming turntable/cutting ring
Moisture Condition		Inundated
Axial Stress at Inundation	kPa	5
Water Used		De-aired tap water
Test Method		B
Interpretation Procedure for c_v		2

All Departures from Outlined ASTM D2435/D2435M-11 (2020) Procedure

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ε_a %	Void Ratio e
Seating	0.0	0	0.0000	20.0000	0.00	1.258
1	24.8	5	0.1993	19.8007	1.08	1.233
2	23.0	10	0.3897	19.6103	1.98	1.213
3	21.5	20	0.5893	19.4107	3.01	1.190
4	23.3	40	0.7725	19.2275	3.93	1.169
5	24.8	80	0.9518	19.0482	4.85	1.148
6	26.5	120	1.0777	18.9223	5.49	1.134
7	31.5	160	1.1902	18.8098	6.06	1.121
8	41.5	240	1.3746	18.6254	7.11	1.097
9	49.8	320	1.5595	18.4405	8.05	1.076
10	71.5	480	1.8989	18.1011	10.02	1.032
11	74.8	640	2.2439	17.7561	11.80	0.991
12	120.0	1280	3.2448	16.7552	17.54	0.862
13	142.0	2560	4.4544	15.5456	24.49	0.705
14	130.8	3840	5.4993	14.5007	28.72	0.609
15	36.8	640	5.1975	14.8025	26.09	0.669
16	73.8	160	4.6048	15.3952	23.05	0.737
17	124.5	40	4.0717	15.9283	20.32	0.799
18	152.0	10	3.9941	16.0059	18.33	0.844
19	67.5	5	3.6406	16.3594	17.81	0.856

Figure D5C

Filename: V:\01216\active\other_pc_projects\165001160\03_data\laboratory\internal\Task
Checked by: D. Boateng Date: June 15, 2021

Date: June 15, 2021
Approved by: R. Dey Date: June 15, 2021

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

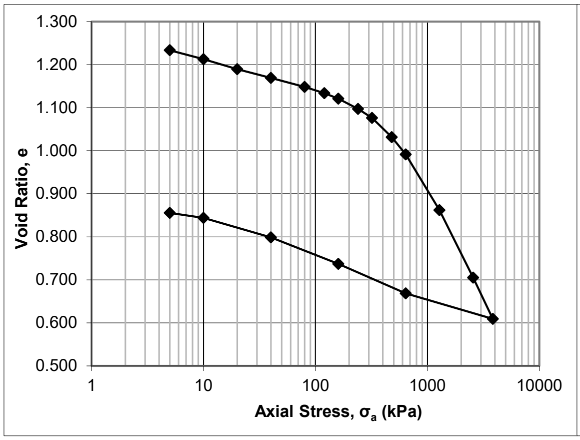
ASTM D2435/D2435M - 11(2020)

Specimen Details

Job Ref.	Highway 401 Brockville EA
Job Location	Ontario, Canada
Borehole	NA 21-1
Sample No.	SA-6A
Depth	4.57 - 5.18 m.
Sample Date	May 3, 2021
Test Number	Two
Technician Name	Daniel Boateng

Calculations

		Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
Load Increment	Axial Stress $\sigma_{a,\ average}$ kPa	Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\varepsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm^2/s	Time t_{90} sec	Coeff. Consol. c_v mm^2/s
Seating	0								
1	3	0.1557	19.8443	0.78	1.240			228	3.67E-01
2	8	0.3631	19.6369	1.82	1.217			287	2.85E-01
3	15	0.5707	19.4293	2.85	1.193			113	7.08E-01
4	30	0.7557	19.2443	3.78	1.172			166	4.73E-01
5	60	0.9306	19.0694	4.65	1.153			147	5.25E-01
6	100	1.0559	18.9441	5.28	1.139			164	4.64E-01
7	140	1.1666	18.8334	5.83	1.126			279	2.69E-01
8	200	1.3370	18.6630	6.69	1.107			169	4.38E-01
9	280	1.5171	18.4829	7.59	1.086			295	2.45E-01
10	400	1.8149	18.1851	9.07	1.053			290	2.41E-01
11	560	2.1536	17.8464	10.77	1.015			392	1.72E-01
12	960	2.9443	17.0557	14.72	0.925			267	2.31E-01
13	1920	3.9894	16.0106	19.95	0.807			239	2.28E-01
14	3200	5.2032	14.7968	26.02	0.670			533	8.71E-02
15	2240	5.4136	14.5864	27.07	0.647				
16	400	4.8867	15.1133	24.43	0.706				
17	100	4.3389	15.6611	21.69	0.768				
18	25	4.0287	15.9713	20.14	0.803				
19	8	3.6534	16.3466	18.27	0.845				



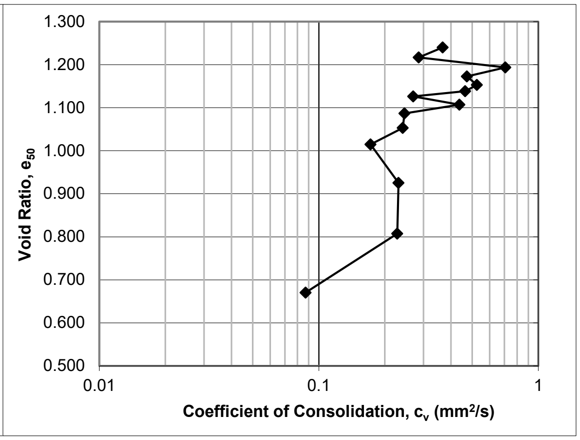


Figure D5D

One-Dimensional Consolidation Properties of Soils Using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Project	Highway 401 Brockville EA
Project No.	165001160.309
Borehole No.	NA 21-2
Sample No.	SA-13
Sample Depth	9.14 - 9.75 m.

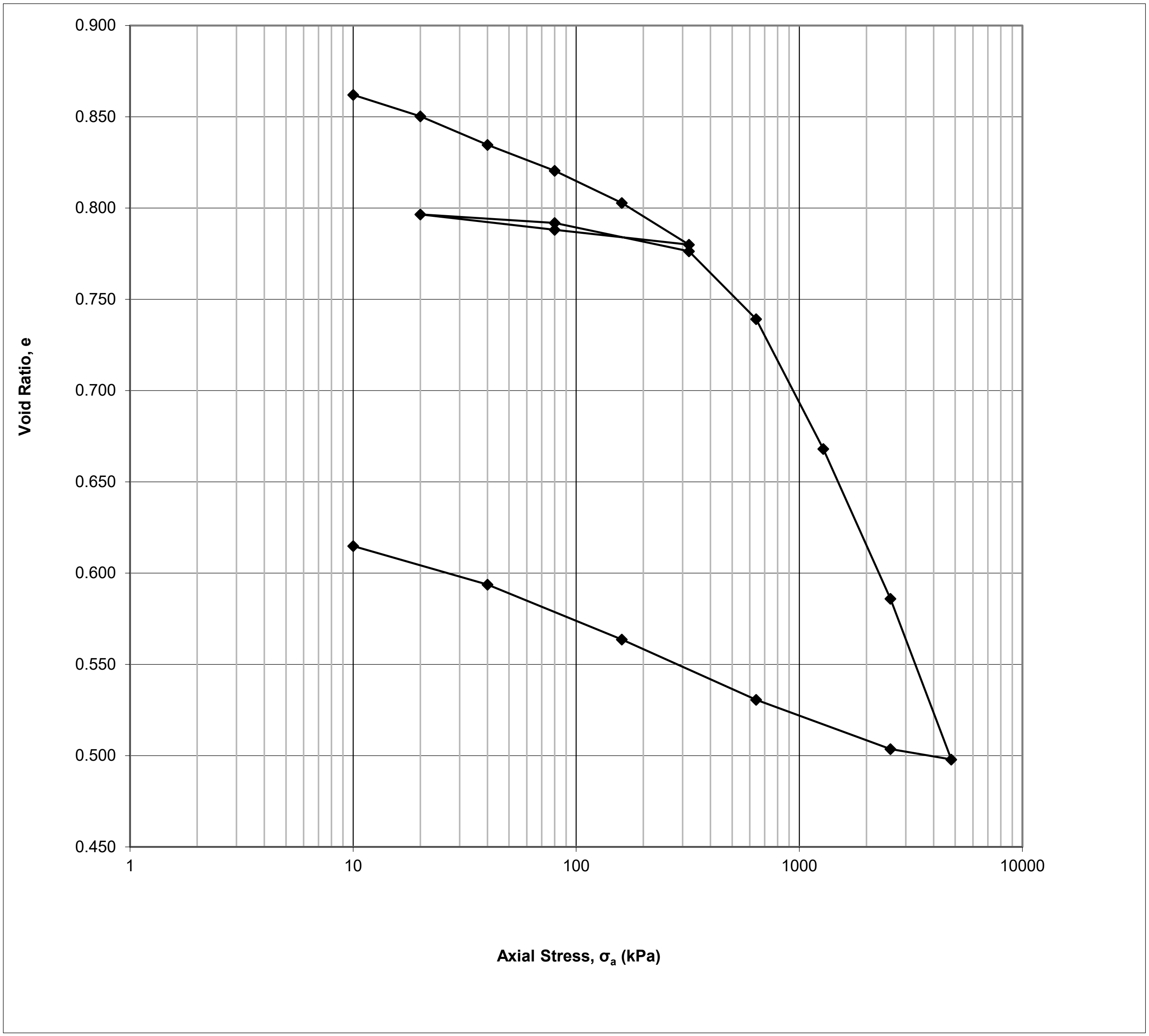


Figure D6A

Checked by: D. Boateng Date: June 15, 2021
Approved by: R. Dey Date: June 15, 2021

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	Highway 401 Brockville EA
Project Location	Ontario, Canada
Borehole	NA 21-2
Sample No.	SA-13
Depth	9.14 - 9.75 m.
Sample Date	May 6, 2021
Test Number	Three
Technician Name	Daniel Boateng

Soil Description & Classification

Silty clay, very stiff to hard, brown, friable, moist

Specific Gravity of Solids		2.760
Average water content of trimmings	%	27.86

Additional Notes (information source, occurence and size of large isolated particles etc.)

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm^2	1963
Volume	mm^3	39270
Mass	g	73.70
Dry Mass	g	57.64
Density	Mg/m^3	1.877
Dry Density	Mg/m^3	1.468
Water Content	%	27.86
Degree of Saturation	%	87.3
Height of Solids	mm	10.64
Initial Void Ratio		0.880

Final Specimen Conditions

Water Content	%	23.87
Final Void Ratio		0.615
Final Height	mm	17.18

Figure D6B

Stantec Consulting Ltd.

One-Dimensional Consolidation Test using Incremental Loading

ASTM D2435/D2435M - 11(2020)

Specimen Details

Project Name	Highway 401 Brockville EA
Project Location	Ontario, Canada
Borehole	NA 21-2
Sample No.	SA-13
Depth	9.14 - 9.75 m.
Sample Date	May 6, 2021
Test Number	Three
Technician Name	Daniel Boateng

Test Procedure

Date Started		June 8, 2021
Date Finished		June 9, 2021
Machine Number		Frame E
Cell Number		E
Ring Number		E
Trimming Procedure		Trimming turntable/cutting ring
Moisture Condition		Inundated
Axial Stress at Inundation	kPa	10
Water Used		De-aired tap water
Test Method		B
Interpretation Procedure for c_v		2

All Departures from Outlined ASTM D2435/D2435M-11 (2020) Procedure

Calculations

Load Increment	Increment Duration min	Axial Stress σ_a kPa	Corrected Deformation ΔH mm	Specimen Height H mm	Axial Strain ε_a %	Void Ratio e
Seating	0.0	0	0.0000	20.0000	0.00	0.880
1	20.0	10	0.1870	19.8130	0.98	0.862
2	21.5	20	0.3100	19.6900	1.60	0.850
3	23.3	40	0.4737	19.5263	2.43	0.835
4	26.5	80	0.6239	19.3761	3.19	0.820
5	28.0	160	0.8079	19.1921	4.12	0.803
6	34.8	320	1.0401	18.9599	5.34	0.780
7	20.0	80	0.9814	19.0186	4.91	0.788
8	20.0	20	0.8922	19.1078	4.46	0.797
9	20.0	80	0.9402	19.0598	4.71	0.792
10	23.3	320	1.0977	18.9023	5.53	0.776
11	56.5	640	1.4441	18.5559	7.51	0.739
12	69.8	1280	2.1611	17.8389	11.29	0.668
13	80.0	2560	2.9557	17.0443	15.66	0.586
14	96.8	4800	3.8275	16.1725	20.34	0.498
15	20.0	2560	4.0071	15.9929	20.04	0.504
16	36.8	640	3.7162	16.2838	18.60	0.531
17	56.8	160	3.3696	16.6304	16.85	0.564
18	82.0	40	3.0550	16.9450	15.24	0.594
19	99.0	10	3.0352	16.9648	14.12	0.615

Figure D6C

Filename: V:\01216\active\other_pc_projects\165001160\03_data\laboratory\internal\Task No. 3
Checked by: D. Boateng Date: June 15, 2021

Date: June 15, 2021
Approved by: R. Dey Date: June 15, 2021

Filename: V:\01216\active\other_pc_projects\165001160\03_data\laboratory\internal\Task
Date: June 15, 2021

Checked by: D. Boateng Date: June 15, 2021
Approved by: R. Dey Date: June 15, 2021

One-Dimensional Consolidation Test using Incremental Loading
ASTM D2435/D2435M - 11(2020)

Specimen Details

Job Ref.	Highway 401 Brockville EA
Job Location	Ontario, Canada
Borehole	NA 21-2
Sample No.	SA-13
Depth	9.14 - 9.75 m.
Sample Date	May 6, 2021
Test Number	Three
Technician Name	Daniel Boateng

Calculations

		Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
Load Increment	Axial Stress $\sigma_{a,\ average}$ kPa	Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\varepsilon_{a,50}$ %	Void Ratio e_{50}	Time t_{50} sec	Coeff. Consol. c_v mm^2/s	Time t_{90} sec	Coeff. Consol. c_v mm^2/s
Seating	0								
1	5	0.1736	19.8264	0.87	0.864			148	5.64E-01
2	15	0.2953	19.7047	1.48	0.853			118	6.98E-01
3	30	0.4572	19.5428	2.29	0.837			148	5.49E-01
4	60	0.6054	19.3946	3.03	0.823			165	4.82E-01
5	120	0.7860	19.2140	3.93	0.806			210	3.73E-01
6	240	1.0122	18.9878	5.06	0.785			221	3.45E-01
7	200	0.9889	19.0111	4.94	0.787				
8	50	0.9080	19.0920	4.54	0.795				
9	50	0.9368	19.0632	4.68	0.792			192	4.02E-01
10	200	1.0856	18.9144	5.43	0.778			262	2.89E-01
11	480	1.3983	18.6017	6.99	0.749			296	2.48E-01
12	960	2.0543	17.9457	10.27	0.687			282	2.42E-01
13	1920	2.7583	17.2417	13.79	0.621			191	3.30E-01
14	3680	3.5278	16.4722	17.64	0.549			217	2.65E-01
15	3680	4.0174	15.9826	20.09	0.503				
16	1600	3.8113	16.1887	19.06	0.522				
17	400	3.5206	16.4794	17.60	0.549				
18	100	3.2068	16.7932	16.03	0.579				
19	25	3.0396	16.9604	15.20	0.595				

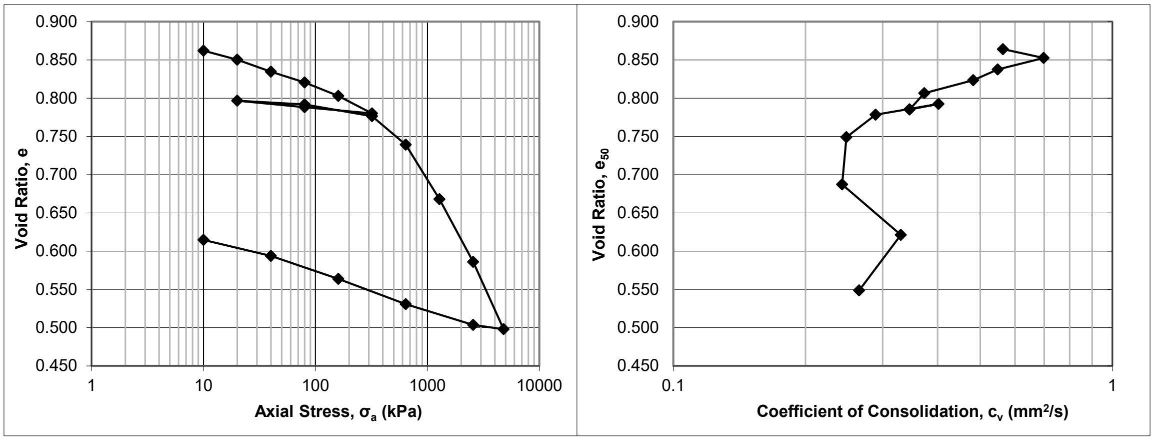
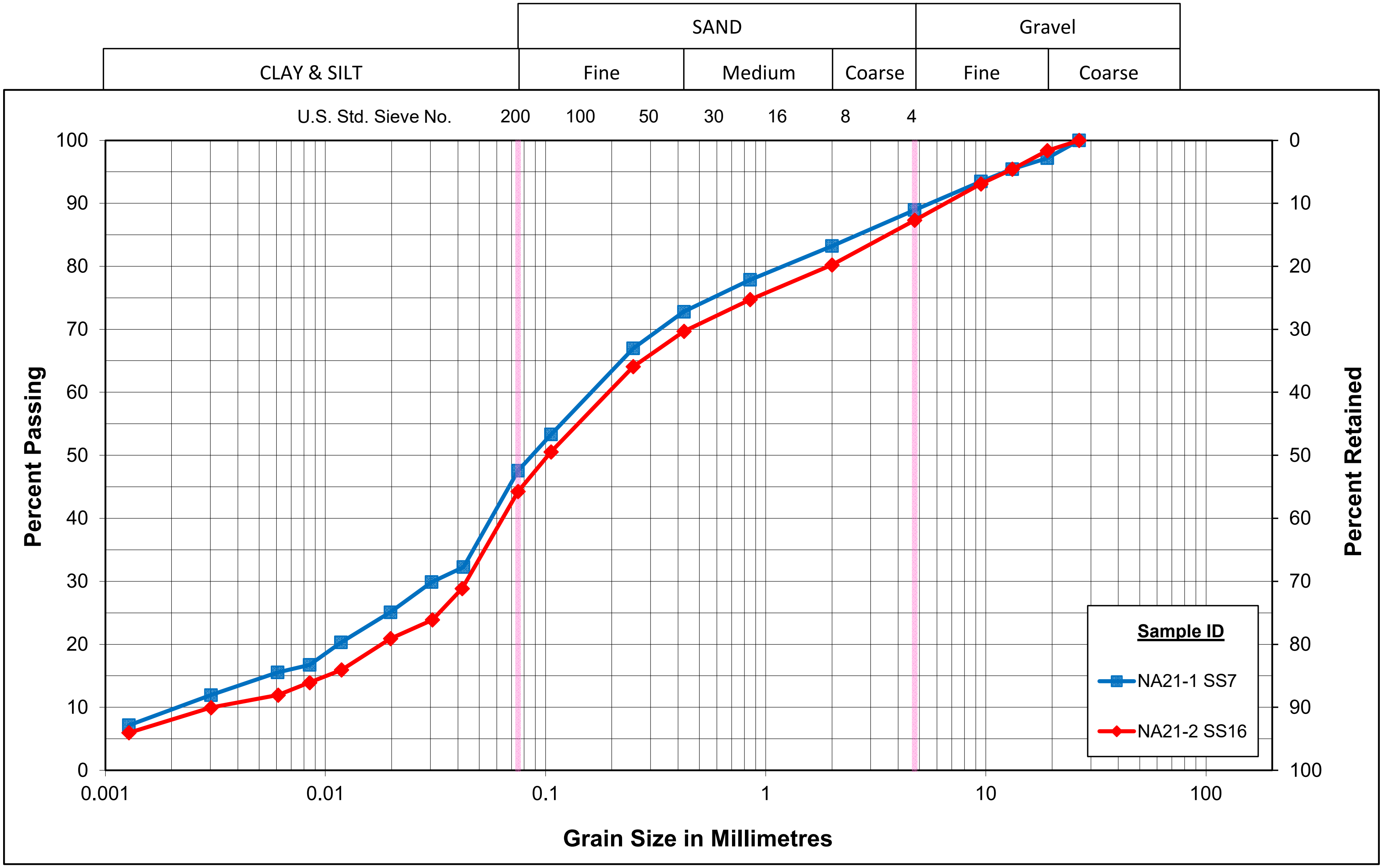


Figure D6D

Unified Soil Classification System



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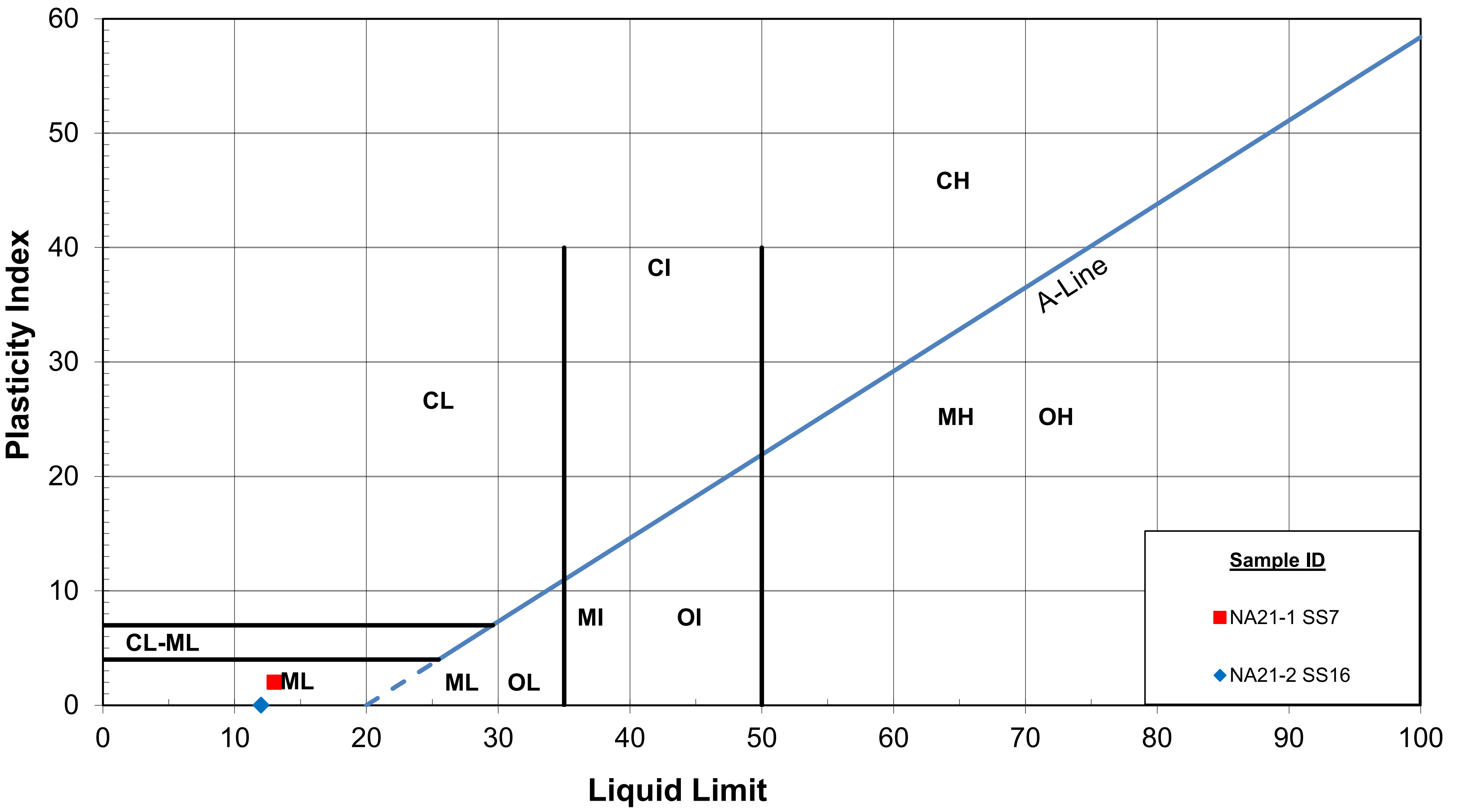
GRAIN SIZE DISTRIBUTION

TILL: Silty SAND (SM), some gravel

Hwy 401 - North Augusta Road Underpass

Figure No. D7

Project No. 165001160



TILL: Silty SAND (SM), some gravel

Hwy 401 - North Augusta Road Underpass

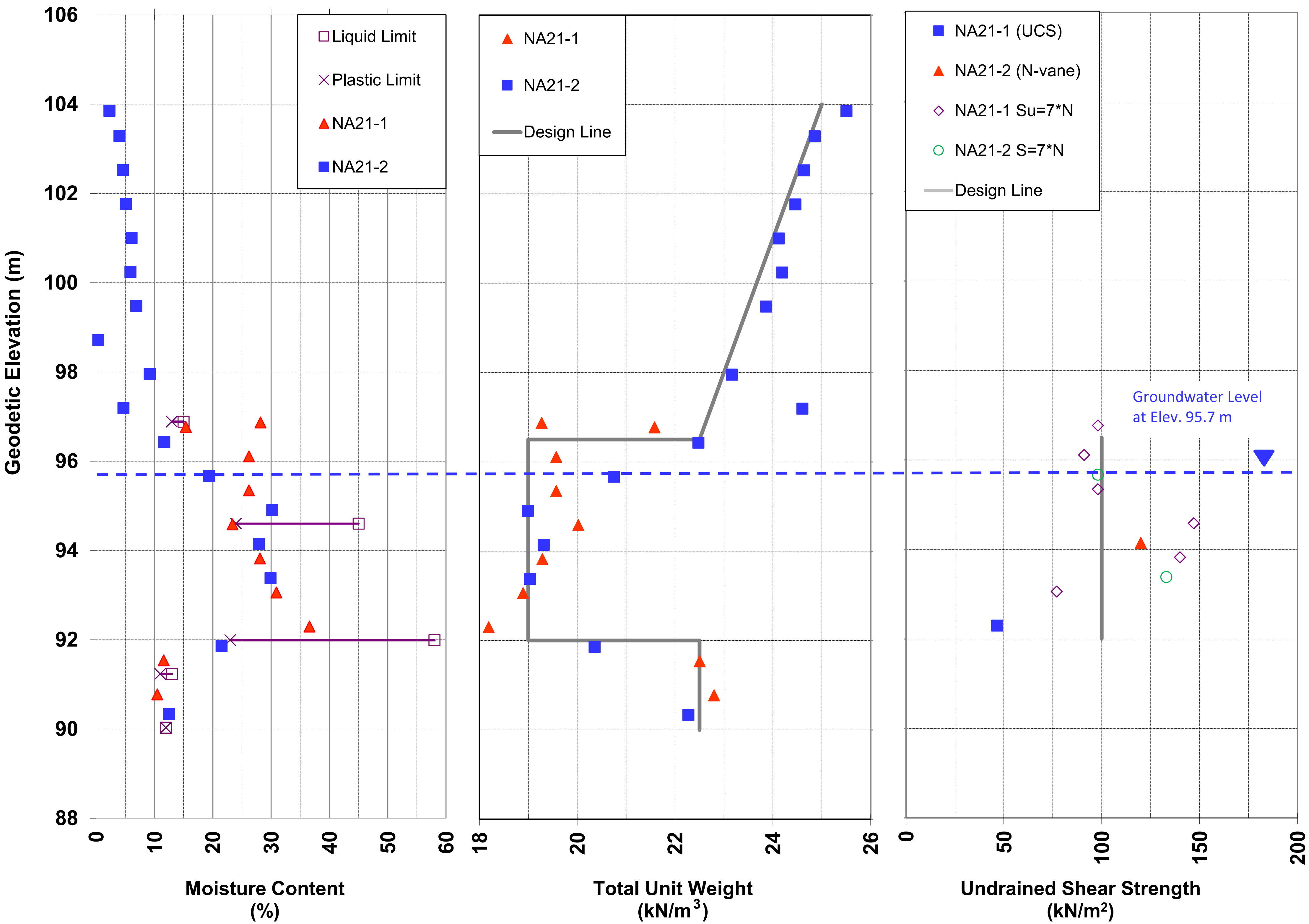
PLASTICITY CHART

Figure No. D8

Project No. 165001160

APPENDIX E

E.1 DRAWINGS E1 TO E3 - GEOTECHNICAL SOIL MODEL AND SLOPE STABILITY ANALYSES



Loose to compact, SAND (SP) to SILTY SAND and GRAVEL (SM) to SANDY SILT (ML) (FILL)

γ = 25 kN/m^3 at El. 104 m to 22.5 kN/m^3 at El. 96 m, ϕ' = 32°, E = 15 MPa

Very stiff, SILTY CLAY (CI) to CLAY (CH)

γ = 19.5 kN/m^3, ϕ' = 28°, E = 40 MPa, Su= 100 kPa

Loose, SILTY SAND (SM) (TILL)

γ = 22.5 kN/m^3, ϕ' = 32°, E = 40 MPa

DOLOSTONE to Sandy DOLOSTONE
Strong to Extremely Strong

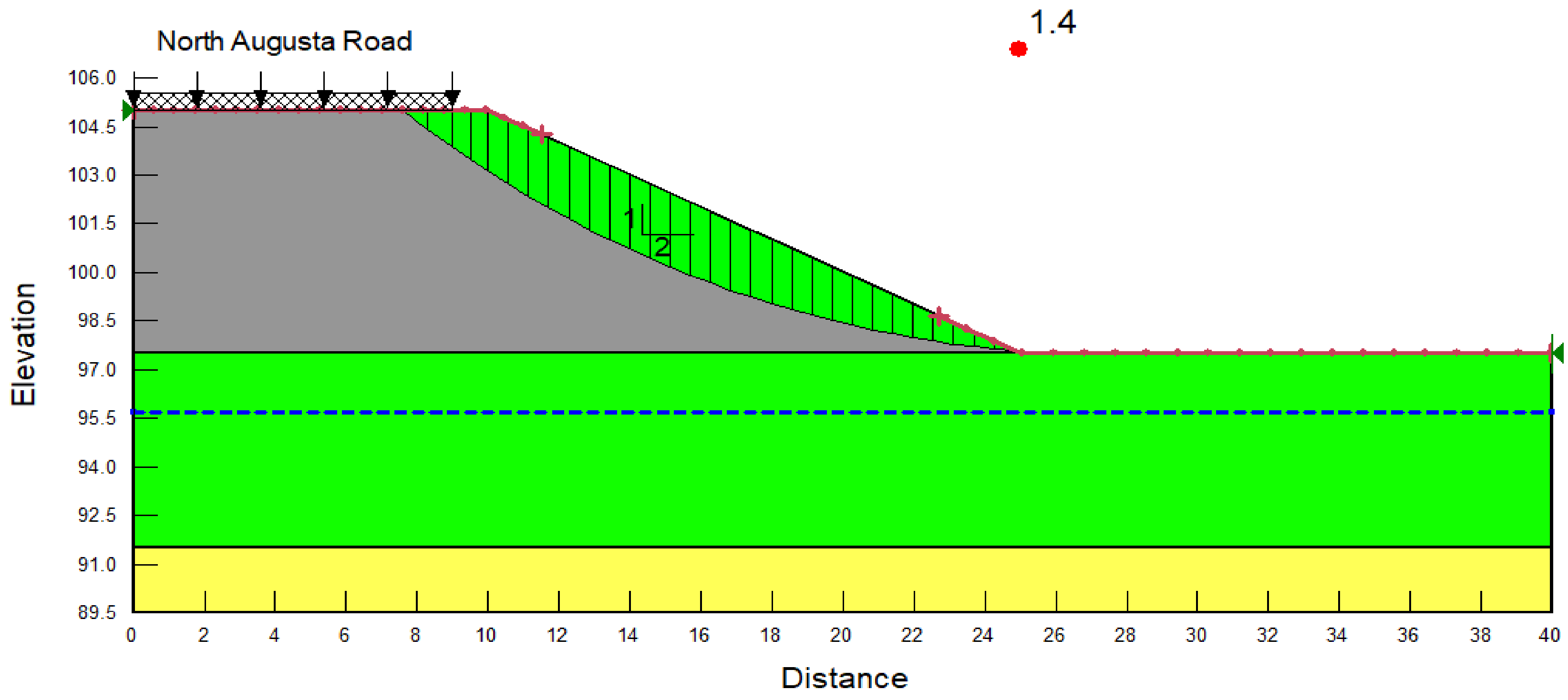
Stratigraphy and Design Parameters

Stantec **Stantec Consulting Ltd.**

Geotechnical Model
Highway 401 Reconstruction, North Augusta Underpass

Drawing No. E1

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
⬛	Select Subgrade Material (SSM)	21	0	32
🟩	SILTY CLAY (CI) to CLAY (CH)	19	0	30
🟨	Silty SAND (TILL) (SM)	22.5	0	32



Slope Stability Analysis (Static)

Highway 401 Reconstruction, North Augusta Underpass

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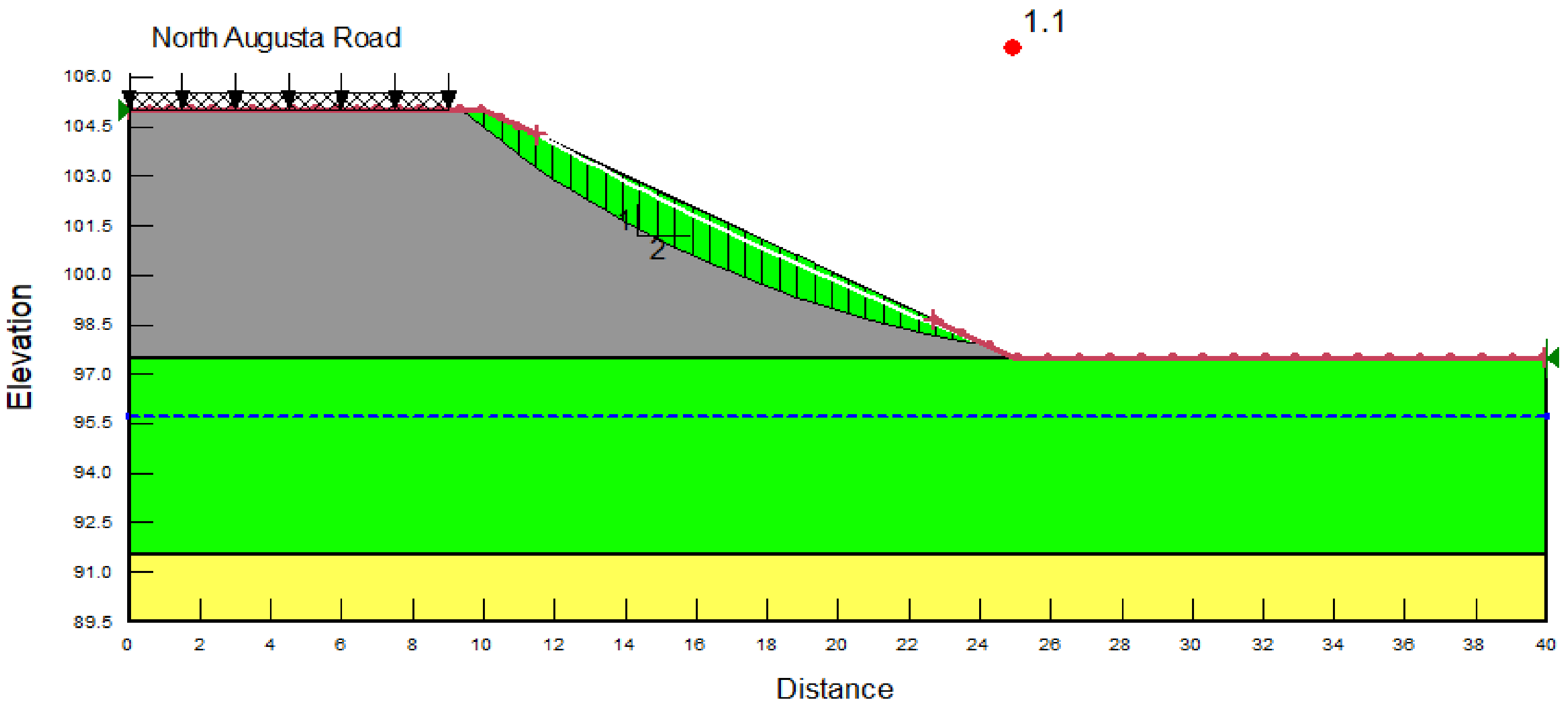
Figure E2

Project No. 165001160

GWP No. 4003-19-00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
◼	Select Subgrade Material (SSM)	21	0	32
🟩	SILTY CLAY (CI) to CLAY (CH)	19	0	30
🟨	Silty SAND (TILL) (SM)	22.5	0	32



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Slope Stability Analysis
(Pseudo-static, k_h= 0.084)

Highway 401 Reconstruction, North Augusta Underpass

Figure E3

Project No. 165001160

GWP No. 4003-19-00

APPENDIX F

F.1 2015 NATIONAL BUILDING CODE SEISMIC HAZARD CALCULATIONS

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: 44.611N 75.685W **User File Reference:** Highway 401 North Augusta Road Interchange 2021-10-04 18:37 UT

Requested by: Stantec

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)	0.240	0.147	0.095	0.031
Sa (0.1)	0.298	0.189	0.125	0.044
Sa (0.2)	0.265	0.170	0.114	0.042
Sa (0.3)	0.211	0.136	0.092	0.035
Sa (0.5)	0.159	0.102	0.069	0.026
Sa (1.0)	0.087	0.055	0.037	0.013
Sa (2.0)	0.043	0.027	0.018	0.005
Sa (5.0)	0.012	0.007	0.004	0.001
Sa (10.0)	0.004	0.003	0.002	0.001
PGA (g)	0.167	0.105	0.069	0.024
PGV (m/s)	0.134	0.082	0.052	0.017

Notes: Spectral (Sa(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