



**Preliminary Foundation Investigation
and Design Report
Ormond Street Overpass
Replacement - Site No. 16X-0123/B0**

Highway 401 Rehabilitation
Brockville, ON

G.W.P. 4003-19-00

Latitude 44.607075
Longitude -75.690899

Geocres No. 31B-106

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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS REPLACEMENT - SITE NO. 16X-0123/B0

Introduction
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PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

For
G.W.P 4003-19-00

Highway 401 Rehabilitation, Brockville, Ontario
Highway 401 Ormond Street Overpass (Site No. 16X-0123/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 Blvd 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 four (4) bridge (overpass or underpass) sites where replacement of the existing structures is planned. This report presents the results of the preliminary foundation investigation related to the replacement of the Ormond Street overpass structure at Site No. 16X-0123/B0. An existing structural culvert (Buell's Creek Culvert - Site No. 16X-0237/C0) that currently passes beneath the Highway 401 embankment approximately 50 m west of Ormond Street is proposed to be abandoned and replaced with an open creek channel that will be located beneath the westernmost span of the new replacement bridge.

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 four 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 Ormond Street overpass structure at Highway 401 (Site No. 16X-0123/B0). 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 completed in the future after additional site investigation is completed.



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Site Description
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2.0 SITE DESCRIPTION

2.1 SITE LOCATION

Ormond Street crosses beneath Highway 401, near Station 22+551, 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 Ormond Street overpass, Highway 401 is a four-lane divided freeway with two lanes in each direction that is aligned in an approximate southwest-northeast orientation. At the bridge location, Ormond Street is a two-lane undivided roadway, with an asphalt-surface pedestrian pathway running adjacent to the southbound lane, that crosses below Highway 401 under a single-span bridge structure. For the purposes of this report, the overpass structure will be referenced as being orientated west to east.

The existing bridge is a single-span, rigid frame structure constructed in 1958 (Contract 57-165). The overpass structure has a width of approximately 29 m and carries four lanes of traffic over Ormond Street. Curved retaining walls are present adjacent to the abutments in all four quadrants of the bridge. A photo of the bridge looking towards the north is provided below.



The ground surface surrounding the overpass site is relatively flat with a gentle slope towards the Buells Creek channel to the west. The lands immediately adjacent to the highway consist of parkland and other undeveloped properties that contain vegetative cover consisting of grass, brush and/or trees.

The pavement surface elevations on Highway 401 at the overpass vary from approximately 100.4 m (west side) to 99.8 m (east side) while the asphalt surface on Ormond Street is at an elevation of about 93 m.



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Buells Creek currently crosses beneath Highway 401 about 50 m west of Ormond Street (~Station 22+503). The Buells Creek culvert (Site No. 16X-0237/C0) consists of an elliptical shaped Corrugated Steel Pipe (CSP) arch culvert with a clear span of 5.0 m, a height of 3.0 m, and a length of about 58.0 m. Based on the preliminary design plans, the existing culvert is planned to be removed and replaced with a new open channel to the west of Ormond Street below a new, longer overpass structure that will span over both Ormond Street and the realigned creek channel.

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 bridge structure were noted.
- The bridge soffit exhibits areas of delamination and spalls with exposed rebar, especially near the fascia, and there are narrow to wide vertical cracks in the abutment walls.
- The asphalt on the bridge surface displayed only minor cracking (one longitudinal crack/joint running down the centerline of the EBL and occasional transverse cracks).
- No signs of embankment settlement or significant instability were observed.

The Buells Creek culvert is in fair to poor condition with a bulge in the soffit, delamination and spalling of the grout at both ends, and areas of rusting and evidence of leakage at the water line.

2.4 SITE DRAINAGE

Regionally, surface water flow in the area is typically from north to south towards the Saint Lawrence River, located approximately 2 km from the site. Locally, surface water discharges towards Buells Creek located to the west of the overpass structure.

Highway 401 slopes down towards the east and pavement drainage is provided by a series of catch basins located adjacent to the median barrier and on the paved shoulders of the highway.

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 Ormond Street 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. The mapping also shows areas of stone-poor sandy silt to silty sand-textured till on Paleozoic terrain to the west, southeast and northeast of the bridge and bedrock-drift complex in Paleozoic terrain east of the bridge.

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



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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

Subsurface information at the site of the Ormond Street overpass at Highway 401 was obtained from the following document contained in the MTO GEOCREs database/library:

- A report titled 'Report of Foundation Investigation for Proposed Overpass Bridge – Highway 401 at Ormond Street, Brockville' for Project 55-F-9 (GEOCREs No. 31B00-015).

The report included the results of test boring and/or penetration tests at four locations advanced at the site to a maximum depth of approximately 7.5 m below ground surface in May 1955. Two boreholes, designated as Borehole No. 2 and Borehole No. 8, were advanced at the site: one near each abutment location. Penetration tests were advanced near each of the four corners of the existing structure.

The subsurface stratigraphy encountered in the boreholes consisted of a layer of medium to stiff clay that was underlain by limestone bedrock. The borehole records indicate that a 250 lb (~113.6kg) hammer, with an unspecified drop height, was used to advance 2 in. or 3 in. diameter, thin-walled open tube samplers; it is noted that this is not the standard hammer size or sampling equipment used for Standard Penetration Tests (SPTs).

The report indicated that consolidation testing was completed, and that the clay deposit was preconsolidated. The natural moisture content of the clay samples tested varied from 24 to 36 percent, expressed as a percentage of the dry weight of the soil. The shear strength of the clay ranged from about 65 kPa to 190 kPa, and the unit weight of the clay ranged from 15.7 kN/m³ to 19.2 kN/m³.

The limestone bedrock or penetration test refusal was encountered at depths varying from approximately 4.8 m to 6.4 m below ground surface, corresponding to elevations of approximately 86.6 m to 87.8 m. Approximately 2.2 m of bedrock was cored in Boring No. 2.

The General Arrangement drawing, and test hole records contained in the above noted report are included in Appendix B for reference.

4.0 INVESTIGATION PROCEDURES

4.1 FIELD INVESTIGATION

The current subsurface investigation program consisted of advancing four boreholes, identified as Boreholes BC21-1, BC21-2, OS21-1, and OS21-2 at the site. Borehole OS21-1 was drilled near the Ormond Street level within the grassed area to the east of Ormond Street and the south of the existing bridge abutment, while Borehole OS21-2 was drilled at the Highway 401 level on the westbound shoulder of the highway adjacent to the west abutment of the existing bridge. The current design includes



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abandonment of the existing culvert and realignment of the creek channel beneath a new, longer bridge structure. However, at the time of the investigation, consideration was being given to replacing the Buells Creek culvert, and Boreholes BC21-1 and BC21-2 were advanced near the ends of the existing culvert to assess culvert replacement options; these boreholes are located near to the west abutment of the planned overpass structure. The locations of these boreholes 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 utilities.

The boreholes were advanced using truck and track-mounted drill rigs equipped for soil sampling and rock coring between the dates of May 5th and May 11th, 2021. The boreholes were typically advanced in the overburden using continuous hollow-stem augers. Casing was advanced below a depth of approximately 6 m in Borehole OS21-2 due to difficult augering conditions. Coring methods were used to advance the boreholes within bedrock below depths of approximately 5.0 m, 3.7 m, 5.5 m, and 13.6 m below ground surface in Boreholes BC21-1, BC21-2, OS21-1, and OS21-2, 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. The bedrock was cored in all boreholes to the termination depths using NQ size equipment. The cores were placed in core boxes, and the boxes labelled and sealed. All 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; with the exception of one test, the soil stiffness was beyond the range of the testing equipment.

Monitoring wells were installed with well screens located in the bedrock in Boreholes BC21-1 and OS21-1; the screened sections of the wells were provided with a sand filter and bentonite was placed above the sand pack. The water level was measured in the BC21-1 well on May 5th, and in both wells on May 11th, May 13th, June 9th, and October 22nd, 2021. The monitoring wells were decommissioned on October 22nd, 2021. The other boreholes were backfilled using bentonite upon completion of drilling; gravel was used to backfill within the highway embankment portion of Borehole OS21-2.

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.



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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			
BC21-1	4941097.5	368969.0	92.3	8.3	83.9
BC21-2	4941050.0	369020.3	92.2	6.9	85.3
OS21-1	4941090.4	369043.8	93.3	8.5	84.9
OS21-2	4941100.2	369004.1	100.4	16.9	83.5

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	34
Atterberg Limits	7
Grain Size Distribution (sieve & hydrometer)	9
Unconfined Compressive Strength (on soil samples)	2
Unconfined Compressive Strength (on bedrock cores and rockfill pieces)	8
Oedometer (Consolidation) Tests	3

Four soil samples, one from each borehole location, were also tested for pH, soluble sulphate content, chloride content, and resistivity (chemical analysis) by Paracel Laboratories Ltd. of Ottawa.

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



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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 consists of a surficial layer of topsoil or fill materials. The fill materials range in composition from sand and gravel to rockfill in Borehole OS21-2 to clayey silty to silty clay in Boreholes BC21-1 and BC21-2. The surficial topsoil or fill materials are underlain by a native deposit of stiff to very stiff silty clay to clay that is in turn underlain by a deposit of glacial till ranging in composition from very stiff clayey silt to compact to very dense silty sand/silt and sand in some boreholes. The overburden materials are underlain by dolostone bedrock. The boreholes were terminated within the bedrock at depths of 6.9 m to 16.9 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 to 250 mm thick surficial layer of topsoil was encountered at the locations of Boreholes BC21-1, BC21-2, and OS21-1.

5.2.2 Fill

5.2.2.1 Granular Fill

Borehole OS21-2, which was drilled through the existing asphalt in the north shoulder of the westbound lanes of Highway 401 encountered a surficial asphalt layer that was approximately 480 mm thick.

Predominantly granular fill materials were encountered beneath the asphalt in Borehole OS21-2. An approximately 300 mm thick layer of sand and gravel (road base) was encountered directly beneath the asphalt. The fill directly beneath the road base consisted of sand, containing trace to some gravel and pockets/zones of sandy silt, that extended to a depth of 3.8 m below ground surface. Below the sand fill, rockfill materials comprised of cobbles and boulders in a matrix of sand were encountered to a depth of about 8.4 m below ground surface (~Elev. 92.0 m). Increased drilling resistance and frequent grinding of the augers was noted during drilling through the fill below a depth of about 3.8 m. Casing was required to be advanced below 6 m due to the frequent cobbles and boulders encountered in the rockfill. A photograph of the pieces of rock retrieved during casing advancement from a depth of 6.2 m to 7.6 m in Borehole OS21-2 is provided below.



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Standard Penetration Test (SPT) N-values measured within the granular fill material varied from 2 to more than 50 blows per 0.3 m of penetration but were typically between 4 and 20 blows per 0.3 m of penetration indicating the fill is generally in a loose to compact state. Below 6 m depth, N-values of greater than 50 blows per 0.3 m of penetration were recorded in the rockfill materials; the higher N-values are considered to have been influenced by the presence of cobbles and boulders.

Two samples of pieces of rock retrieved within the embankment fill during casing advancement in Borehole OS21-2 were selected for testing to determine their Unconfined Compressive Strengths (UCS). The UCS test results were 116.3 MPa at a depth of 6.7 m and 176.0 MPa at a depth of 7.0 m below ground surface and indicate that the cobbles and/or boulders in the rockfill are classified as very strong (R5).

Laboratory testing of samples of the granular fill materials yielded moisture contents varying between approximately 2% to 7%, expressed as a percentage of the dry weight of the soil.

Gradation analyses were carried out on two representative samples of the sand fill materials. The results of the tests are illustrated on the borehole records in Appendix C and on the gradation plots on Figure No. D1 in Appendix D.

Based on the laboratory results, the USCS group symbols for the sand fill varies from SP to SM.

The granular fill was approximately 7.9 m thick with the base of the layer encountered at an elevation of approximately 92.0 m.

5.2.2.2 Cohesive Fill

Cohesive fill materials were encountered beneath the surficial topsoil in Boreholes BC21-1 and BC21-2. The cohesive fill is comprised of clayey silt to silty clay and contains trace to some sand and trace gravel and organic matter. The cohesive fill extended to depths of about 1.4 m below ground surface in Borehole BH21-1 and approximately 0.8 m below ground surface in Borehole BH21-2.



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Standard Penetration Test (SPT) N-values recorded within the cohesive fill materials varied from 6 to 8 blows per 0.3 m of penetration. In-situ shear vane tests conducted at in Borehole BC21-1 using N-vane equipment measured an undrained shear strength of 118 kPa at a depth of about 0.8 m and encountered refusal (i.e. inability to turn vane) at a depth of 1.0 m. Based on the field and laboratory testing and examination of samples obtained, the cohesive fill is considered to generally have a firm to very stiff consistency.

Laboratory testing of samples of the cohesive fill materials yielded moisture contents varying between approximately 11% to 28%, expressed as a percentage of the dry weight of the soil.

The cohesive fill was approximately 0.6 m to 1.3 m thick with the base of the layer at elevations of approximately 90.9 m and 91.4 m in Boreholes BC21-1 and BC21-2, respectively.

5.2.3 Silty Clay/Clay

A cohesive deposit comprised of silty clay/clay containing trace sand was encountered below the topsoil in Borehole OS21-1 and beneath the fill materials in Boreholes BC21-1, BC21-2, and OS21-2. The cohesive deposit was approximately 1.6 m to 5.2 m thick.

The deposit was noted to be varved below a depth of 2.0 m and 2.5 m in Boreholes BC21-1 and BC21-2, respectively.

SPT 'N' values varying between 6 to 23 blows per 0.3 m of penetration were measured within the cohesive deposit. In-situ shear vane testing using N-vane equipment attempted at depths of 1.4 m to 3.7 m in Boreholes BC21-1, BC21-2, and OS21-1 encountered refusal (i.e. inability to turn vane). The undrained shear strength of the cohesive deposit was also determined by conducting Unconfined Compressive Strength (UCS) tests on Shelby Tube samples recovered from Boreholes BC21-1 and OS21-2. Undrained shear strengths of approximately 143 kPa and 140 kPa were measured by this testing. 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 stiff soils present in Borehole BC21-2.

Laboratory testing of samples of the cohesive soils yielded moisture contents varying between approximately 23% to 40%.

Gradation analyses were carried out on five (5) 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. D2 in Appendix D.

Atterberg Limits tests were carried out on portions of the samples referenced above. The tests yielded Liquid Limits ranging from 39% to 62%, Plastic Limits ranging from 21% to 25%, and Plasticity Indices ranging from 18% to 37%. Based on these results, the cohesive soil is classified as silty clay of medium plasticity (CI) to clay of high plasticity (CH). The results of the tests are illustrated on the borehole records in Appendix C and on Figure No. D3 in Appendix D.



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Three (3) consolidation tests were carried out on relatively undisturbed Shelby Tube samples recovered from the boreholes. The test results are illustrated on Figure Nos. D4(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		
	BC21-1, SH3A	OS21-1, SH5	OS21-2, SH13
Sample Depth (m below ground)	2.5	3.3	9.5
Sample Elevation (m)	89.8	90.0	90.9
Effective Vertical Stress (kPa)	25.8	32.0	171.8
Moisture Content	33%	40%	23%
Initial Void Ratio, e_0	0.960	1.127	0.698
Initial Unit Weight, γ	18.1 kN/m ³	17.8 kN/m ³	19.6 kN/m ³
Estimated Preconsolidation Stress, P'_c	580 kPa	900 kPa	650 kPa
Overconsolidation Ratio (OCR)	22.5	28.2	3.8
Recompression Index, C_r	0.06	0.02	0.02
Compression Index, C_c	0.33	0.49	0.23
Coefficient of Consolidation, C_v	0.3 mm ² /s	0.5 mm ² /s	0.5 mm ² /s

Notes: 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.

The silty clay/clay deposit extended to depths of about 3.0 m, 3.7 m, 5.5 m and 13.0 m below ground surface in Boreholes BC21-1, BC21-2, OS21-1, and OS21-2, respectively, corresponding to base of deposit elevations ranging from about 87.4 m to 89.2 m.

5.2.4 Glacial TILL

A glacial till deposit varying in composition from clayey silt containing some sand and trace gravel to silty sand/silt and sand containing some gravel and trace to some clay was encountered underlying the clay/silty clay deposit in Boreholes BC21-1 and OS21-2. The till deposit was approximately 2.0 m and 0.6 m thick and extended to the depths of about 5.0 m and 13.6 m, (corresponding to elevations of about 87.3 m and 86.8 m) in Boreholes BC21-1 and OS21-2, respectively.

Auger grinding on inferred cobbles and boulders was encountered within the till below a depth of about 3.8 m in Borehole BC21-1. 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.

An SPT 'N' value measured in the clayey silt portion of the till deposit in Borehole BC21-1 was 12 blows per 0.3 m of penetration suggesting the till at that location had a stiff consistency. SPT 'N' values ranging



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from 18 to greater than 50 blows per 0.3 m of penetration were measured in the silty sand till deposit suggesting the coarser portions of the till are in a compact to very dense state. The highest SPT 'N' values may 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 7% to 12%.

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 test results indicated that one sample was non-plastic, and the other test yielded a Plastic Limit of 12%, a Liquid Limit of 17%, and a corresponding Plasticity Index of 5%. 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 varies from SM (silty sand till) to CL-ML (silt and sand till).

5.3 BEDROCK

Slightly weathered to fresh, dolostone bedrock was encountered underlying the overburden described in the preceding sections in all boreholes. The depth to bedrock is summarized in Table 5.2 below.

Table 5.2: Depth to Bedrock and Bedrock Surface Elevation

Borehole	Depth (m)	Elevation (m)
BC21-1	5.0	87.3
BC21-2	3.7	88.4
OS21-1	5.5	87.9
OS21-2	13.6	86.8

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.



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Table 5.3: Summary of Bedrock Coring Operations

Borehole No.	Run No.	Rock Description	Depth (m below ground)	Geodetic Elevation (m)	Total Core Recovery, TCR (%)	Solid Core Recovery, SCR (%)	Rock Quality Designation, RQD (%)	Weathering Degree (W1=Fresh, W2=Slightly Weathered)	Fracture Index (No. of fractures per m)
BC21-1	7	Slightly weathered to fresh, light grey to grey Dolostone	5.0-6.1	87.3-86.2	100	95	71	W2/W1	8
	8		6.1-7.7	86.2-84.6	98	95	72	W1	7
	9		7.7-8.3	84.6-83.9	100	100	100	W1	2
BC21-2	6	Slightly weathered to fresh, light grey to grey Dolostone	3.7-4.7	88.5-87.5	100	96	96	W2/W1	4
	7		4.7-6.0	87.5-86.2	96	96	82	W1	8
	8		6.0-6.9	86.2-85.3	100	100	72	W1	7
OS21-1	9	Slightly weathered to fresh, light grey to grey Dolostone	5.5-6.2	87.8-87.1	100	100	100	W2/W1	2
	10		6.2-7.6	87.1-85.7	100	96	67	W1	11
	11		7.6-8.5	85.7-84.8	94	94	85	W1	4
OS21-2	16	Slightly weathered to fresh, light grey Dolostone	13.6-14.8	86.8-85.6	100	58	21	W2/W1	13
	17		14.8-16.3	85.6-84.1	96	95	78	W1	7
	18		16.3-16.9	84.1-83.5	100	100	100	W1	0

Based on the RQD range indicated in the table, the bedrock cores obtained from the boreholes can be classified as fair to excellent in quality with the exception of the first core run in Borehole OS21-2 where very poor quality rock was encountered.

Six (6) 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 Rock Core Samples

Borehole No.	Run No.	Sample Depth (m below ground)	Sample Elevation (m)	Unconfined Compressive Strength (MPa)
BC21-1	7	5.3	87.0	113.9
BC21-2	6	4.3	87.9	137.8
OS21-1	9	6.0	87.3	144.9
	11	8.1	85.2	124.5
OS21-2	17	15.6	84.8	60.4
	17	16.1	84.3	159.8

The UCS test results of the rock cores ranged from 60.4 MPa to 159.8 MPa and indicate that the dolostone bedrock can be classified as strong (R4) to very strong (R5).



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5.4 GROUNDWATER CONDITIONS

The water levels recorded in the monitoring wells installed within the bedrock in Boreholes BC21-1 and OS21-1 are summarized in Table 5.5 below.

Table 5.5: Water Level Measurements in Monitoring Wells Sealed in Bedrock

Borehole No.	Date	Measured Groundwater Depth (m)	Groundwater Elevation (m)
BC21-1	May 5, 2021	0	92.30
	May 11, 2021	0	92.30
	May 13, 2021	1.09 above ground	93.39
	June 9, 2021	0.50 above ground	92.80
	October 22, 2021	0.05 below ground	92.25
OS21-1	May 11, 2021	0.45 below ground	92.85
	May 13, 2021	0.21 above ground	93.51
	June 9, 2021	0.23 above ground	93.53
	October 22, 2021	0.89 below ground	92.41

The above-ground water level readings indicate that artesian conditions were measured and should be anticipated at the site. As the water levels within the bedrock were originally at or slightly below ground surface, rose to above ground surface, and then returned to levels below ground surface, the artesian conditions appear to be transitory in nature.

Perched water conditions may also develop within and above finer-grained portions of the embankment fill materials, and also within the natural silty clay to clay cohesive deposit.

Groundwater levels at the site will be subject to fluctuations due to seasonal changes, snowmelt and/or precipitation events and the water level in Buells Creek. 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 sample of the soils collected from each borehole. The analysis results are provided in Appendix D and are summarized in Table 5.5.

Table 5.6: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Resistivity (Ohm-m)	Chloride (µg/g)	Sulphate (µg/g)
BC21-1	SS3	1.5-2.1	7.48	16.3	244	72
BC21-2	SS3	1.5-2.1	7.81	15.1	264	49
OS21-1	SS1	0.3-0.6	7.62	44.3	22	8
OS21-2	SS3B	1.8-2.1	7.91	30.2	118	16



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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 Zach Popper, P.Eng. and reviewed by Kevin Nelson, P.Eng., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.



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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,

STANTEC CONSULTING LTD.

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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 Ormond Street Overpass (Site No. 16X-0123/B0)

Brockville, Ontario

8.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

8.1 OVERVIEW

This section of the report provides preliminary foundation engineering design input related to the proposed replacement of the overpass structure located at the crossing of Ormond Street below Highway 401 (Site No. 16X-0123/B0), including the abandonment of the existing Buells Creek culvert that is to be replaced with an open creek channel below the new overpass. The new overpass will be longer than the existing bridge to accommodate the new open channel and wider to accommodate the ultimate 8-lane highway configuration on Highway 401.

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 overpass 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 overpass structure. This Preliminary Report is not to be used for the detail 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.



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8.2 PROJECT DESCRIPTION AND BACKGROUND

8.2.1 Project Description

The project involves the preliminary design for a new overpass structure at the Highway 401 crossing of Ormond Street in the City of Brockville. As part of the project, the existing Buells Creek culvert that is located about 50 m west of Ormond Street is planned to be abandoned and replaced with an open creek channel located below the new overpass. This study is being completed as part of the overall study related to the rehabilitation of Highway 401 in the City of Brockville (G.W.P. 4003-19-00).

Based on preliminary design information, the new overpass at Highway 401 and Ormond Street will include a new wider overpass with two (2) spans that will accommodate a 8-lane highway configuration (4 lanes in each direction), as well as a new open channel below the overpass to replace the existing Buells Creek culvert. This will require a new overpass structure that is wider and longer than the existing bridge.

8.2.2 Existing Overpass Structure and Culvert

The Ormond Street Overpass is a single-span structure constructed in 1958 (Contract 57-165). At the bridge site, the pavement surface elevations on Highway 401 at the overpass vary from approximately 100.4 m (west side) to 99.8 m (east side) while the asphalt surface on Ormond Street is at an elevation of about 93 m.

The following details on the existing bridge are provided based on the information shown on the available structural design drawings for the overpass structure:

- The existing overpass consists of a cast-in-place concrete rigid frame structure with a clear span of 16.1 m. The bridge, and Ormond Street underneath the structure, are skewed at about 15.3° relative to Highway 401. The roadway width in each direction of Highway 401 is about 13.4 m, and the overall deck width is 29.0 m.
- Curved retaining walls are present on both sides of each abutment (i.e. at the four corners of the bridge).
- The bridge abutments are founded on 318 mm diameter concrete filled steel tube pile foundations. The abutment pile caps are both approximately 29 m long and 1.8 m wide, with wider portions near the ends of each abutment. The bases of the abutment pile caps are at elevations of approximately 91.5 m. The abutment pile caps are supported on two rows of 318 mm diameter piles installed at approximately 1.1 m center to center spacing. The outside rows of piles (i.e. piles furthest away from Ormond Street) are battered at 1 horizontal to 10 vertical (1H:10V) away from Ormond Street. The design drawings indicate that the piles used for the abutment foundations are approximately 4.9 m long.
- The curved retaining walls are supported on 2.1 m wide, shallow strip/spread foundations that are founded at the same elevation as the base of the abutment pile caps.

The Buells Creek Culvert consists of an elliptical-shaped, corrugated steel pipe (CSP) culvert with a clear span of 5.0 m, a height of 3.0 m, and a length of about 58.0 m. The highway above the culvert carries



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two lanes of traffic in each direction. The culvert is oriented perpendicular to the highway and there is approximately 8 m of fill over the culvert obvert.

8.2.3 Proposed Structure Modifications and Replacement

The Ormond Street overpass structure is planned to be replaced with a wider and longer structure that will accommodate both the future 8-lane highway configuration and the relocation of the Buells Creek channel beneath the bridge. The new overpass is planned to consist of a 2-span structure with a length of 45 m (two 22.5 m spans) that will have a 15°5' skew to Ormond Street and 7 m long wingwalls on each side of the bridge. The new bridge will incorporate either integral abutments, if the ground conditions are conducive, or semi-integral abutments.

Ormond Street will have two 4.6 m wide lanes, a 3 m wide multi-purpose pathway on the west side and a 1 m clear zone on the east side at the overpass location. Ormond Street will have a road surface elevation of approximately 93.6 m.

The preliminary design information indicates that the new overpass structure will be situated at the location of the existing structure. In this respect, the new structure will need to be constructed/the existing bridge replaced in stages in order maintain traffic in both directions on Highway 401 during construction. Initially, the north and south portions of the existing bridge would be removed and replaced with sections of the new bridge while highway traffic is maintained in the central portion of the right-of-way. Once complete, highway traffic would be shifted to these new structures and the central/median portion of the existing bridge would be demolished and replaced.

The realigned Buells Creek open channel will be located below the western span of the overpass structure and Ormond Street will pass beneath the eastern span. The realigned Buells Creek channel will be located approximately 28.3 m east of the existing culvert, will have 2H:1V sideslopes, and the base of the channel will be at an approximate elevation of 91.8 m.

The approach embankments will need to be widened to accommodate the wider bridge. The approach embankments will be approximately 7.5 m high and are planned to be constructed with 2H:1V sideslopes.

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.



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8.4 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered during the current investigation at the site consisted of a surficial layer of topsoil or asphalt underlain by a fill layer (pavement structure and road/embankment fill in Borehole OS21-2 and clayey silt to silty clay fill in Boreholes BC21-1 and BC21-2) underlain by a deposit of silty clay to clay. The clay/silty clay overlies a thin layer of glacial till at some borehole locations and dolostone bedrock was encountered beneath the overburden at all borehole locations. The till typically consists of clayey silt to silty sand with varying amounts of clay and gravel and contains cobbles and boulders. Transient artesian conditions were encountered in the monitoring wells installed within the bedrock in Boreholes BC21-1 and OS21-1.

The soil profile identified in Table 8.1 below and 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 – Ormond Street Overpass

Elevation (m)		Soil Type	Design Parameters				
From	To		Total Unit Weight γ , (kN/m ³)	Drained Friction Angle ² ϕ' , (°)	Undrained Shear Strength, S_u , (kPa)	Soil Modulus E , (MPa)	Consolidation Parameters ¹
100.0	96.6	FILL: SAND, very loose to compact	21	32	N/A	15	N/A
96.6	92.0	ROCKFILL: Cobbles and boulders in a matrix of SAND, very loose to compact	21	35	N/A	15	N/A
92.0	88.0	Very stiff SILTY CLAY (CI) to CLAY (CH), trace sand	19	30, $c' = 5$ kPa	125	30	$P_c = 580$ kPa $C_r = 0.02$ $C_c = 0.33$
88.0	87.0	³ Compact to very dense Silty SAND (SM), some clay and gravel (TILL). Contains zones of stiff CLAYEY SILT (TILL), cobbles and boulders.	22.5	32	N/A	50	N/A
87.0	83.5	Very strong DOLOSTONE. Very poor to excellent quality.	26	N/A	⁴ 115 MPa	N/A	N/A

Notes:

- (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 TILL unit was only encountered in boreholes on the north side of the site.
- (4) The value in the strength column for the bedrock represents the Unconfined Compressive Strength
- (5) Groundwater is assumed to be at an Elevation of 93.5 m for preliminary design purposes. Submerged unit weights (γ) should be used below the groundwater level.



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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 overpass 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 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

<i>PGA</i>	$S_a(0.2)$	PGA_{ref}	Site Class	Site Adjusted <i>PGA</i>
0.166g	0.263g	0.133g	C	0.166g

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 0.166g.
- An earthquake magnitude M_w of 6.5.
- A groundwater level/elevation of 93.5 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.



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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 the replacement of the overpass structure. 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 Ormond Street replacement bridge.

Table 8.3: Comparison of Foundation Options for Ormond Street Overpass

Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
H-Piles Driven to Bedrock (Integral Abutments)	<ul style="list-style-type: none"> 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 	<ul style="list-style-type: none"> Piles at pier locations would be short (i.e. less than 5 m) Pre-drilling in rockfill and/or till could be required if piles encounter refusal on boulders Limited uplift capacity due to short pile lengths 	Medium	<ul style="list-style-type: none"> Pile damage during installation Potential for shallow refusal of piles on cobbles and boulders requires pre-drilling Based on preliminary general arrangement, new piles may conflict with existing bridge foundations and piles
Drilled Piers / Caissons (Central Pier)	<ul style="list-style-type: none"> Can transmit very large axial and lateral loads Shorter construction time than shallow foundations 	<ul style="list-style-type: none"> Construction method most affected by artesian groundwater conditions. Requires dewatering or the use of liners and/or drilling mud to balance water pressures; cannot be visually inspected Difficult to drill piers/advance liners in/through rockfill or till deposits containing boulders and cobbles Not suitable for integral bridge abutments 	High	<ul style="list-style-type: none"> Liners and/or drilling mud required to mitigate groundwater issues. Installation of liners to maintain sidewall stability may not be practical without specialized equipment. Based on preliminary general arrangement, caissons may conflict with existing bridge foundations and piles
Shallow Foundations Founded on Very Stiff Silty Clay to Clay	<ul style="list-style-type: none"> Lower foundation costs than deep foundations Pile driving/drilling through difficult deposits avoided 	<ul style="list-style-type: none"> 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	<ul style="list-style-type: none"> Potential for unacceptable total and differential settlements



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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 rockfill materials. In this regard, pre-drilling is recommended to facilitate advancing the piles through the highway embankment materials.

Drilled piers/caisson foundations socketed into the bedrock could be considered for support of the central pier but would require dewatering to depressurize the bedrock and the use of temporary liners and/or drilling mud to mitigate the potential risks of ground loss or collapse within the water-bearing soils present immediately above the bedrock during construction. Liner installation could be hindered by the presence of cobbles and boulders. The use of special construction procedures including down-the-hole hammers, churn drills and/or possibly rock coring techniques may be required to penetrate these obstructions in the rockfill and within the glacial till. Furthermore, the use of “wet” installation methods would preclude the ability to review/confirm the removal of loose materials present at the base of the caissons increasing the risk of unsuitable foundation performance.

Driving steel H-pile foundations to support the center pier is also considered feasible, however, it is anticipated that the pile lengths below the pile cap could be as little as 4 m.

Foundation installation methods will need to account for potential artesian groundwater conditions (e.g. temporary dewatering of the bedrock would likely be needed to facilitate drilled pier installation).

Support of the abutments and/or pier on shallow foundations could result in overstressing of the underlying stiff to very stiff silty clay/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 drilled 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 elevations of approximately 94 m to 96 m and the base of the pile cap for the central pier would be located at about 91.5 m. The surface of the bedrock was encountered at elevations varying from approximately 86.8 m to 88.4 m which would result in required pile lengths of approximately 3 m to 5 m at the center pier. Effective refusal could be encountered at shallower depth within the very dense portions



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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 92 m to facilitate the piles being able to be installed through the rockfill embankment materials; this will allow the abutment piles to obtain sufficient pile embedment to satisfy the minimum pile length requirements to obtain the condition of pile fixity. Predrilling may also be required through the till to permit the piles to reach the bedrock surface; the requirement for such predrilling should be further reviewed during the detailed design stage.

The east abutment and center pier of the new bridge are located in close proximity to the existing bridge abutments which are supported on 318 mm diameter concrete filled steel tube piles, 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. Where practical, existing piles and pile caps are recommended to be removed in areas where they could interfere with the installation of new piles.

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

Depending on the time of year that the construction is carried out, it is possible that the artesian condition results in water flowing along the shaft of some of the driven piles. Since the piles for this project would obtain their axial resistance from their pile tip resistance, this possible occurrence would not impact the geotechnical resistance of the piles.

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	86.8 m to 88.4	2,000
HP 310 x 132	86.8 m to 88.4	2,400

The estimated geotechnical reaction at SLS (factored) for 25 mm of vertical settlement for piles 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 wider and longer than the existing bridge which will require widening of existing highway embankments. The native site soils underlying the abutment locations consist of compressible clay to silty clay soils that will consolidate over time due to application of new loads associated with the widened embankment construction. Therefore, the piles supporting the new bridge abutments (particularly those located within the widened embankment areas) will need to be designed to resist downdrag loads that will develop as a result of soil settlement adjacent to the piles. An



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estimated unfactored downdrag load of approximately 250 kN will be mobilized along the length of each abutment pile installed within the silty clay soils; a load factor should be applied to these downdrag loads but it is noted that downdrag loads should not be combined with live 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 widened portions of the approach embankment areas near the proposed bridge abutments 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 highway embankment fill materials including rockfill materials, and stiff to very stiff silty clay to clay that is underlain by compact to very dense glacial till with cobbles and/or boulders and then by bedrock. Obstructions to pile driving should be anticipated due to the cobbles and boulders observed in the fill and in the till. Based on these conditions, pre-drilling is recommended through the existing fill materials and the piles should be provided with driving shoes such as Titus “H” Bearing Pile Point (Standard Model) or equivalent. The piles should be driven to bedrock in accordance with the requirements of OPSS.PROV 903.

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.PROV 903. .

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.



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Table 8.5: Recommended Tensile Pile Resistance

Pile Type	Minimum Pile Length(m)	Factored Geotechnical Resistance (Tension) at ULS _r (kN)
HP310x110 or HP310x132	5	75

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

8.7.3 Caissons – Centre 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. Pile caps would not necessarily be required at the ground surface, which would reduce the depths and associated durations of excavations required at the centre pier location. 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 very poor to 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 soils conditions and the associated design parameters prior to the detailed design stage.



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Table 8.6: Caisson Capacities at ULS_f ($\phi = 0.4$)

Caisson Diameter (m)	Socket Length (m)	Geotechnical Resistance at ULS _f (kN)
1.2	2.4	5,500
	3.0	6,800
	3.6*	8,200
1.5	3.0	8,600
	4.5*	12,900

*Notes: 1) These socket lengths extend below the boreholes advanced as part of the preliminary foundation investigation. Additional investigation extending to below the base of the drilled pier foundations will be required prior to detailed design to verify the associated ULS design resistances.
2) The above geotechnical resistance reflects only the shaft resistance within the rock socket.

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, a 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 results of the preliminary foundation investigation identified the presence of transitory artesian groundwater conditions. The installation of a temporary dewatering system to lower the water level/depressurize the bedrock prior is recommended to facilitate caisson construction if this foundation option is selected.

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.



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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 abutments. The materials used as backfill behind the abutments of the replacement overpass 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 pressure 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.



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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:

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

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

where:

K_{AE} = active earth pressure coefficient (combined static and seismic)

K_{PE} = passive earth pressure coefficient (combined static and seismic)

H = height of wall

k_h = horizontal acceleration coefficient

k_v = vertical acceleration coefficient

γ = total unit weight

The seismic earth pressures for structures with horizontal backfill behind the walls may be calculated using the parameters provided in Table 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.



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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_{h0}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding (<i>wall movements of 25 mm to 50 mm</i>)
0.166g	0.166	0.083
Note: k_{h0} is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the site-adjusted PGA estimated at ground surface. The vertical acceleration coefficient (k_v) should be ignored in the calculations as per CHBDC 2019, section C6.14.7.		

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	0.41	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 SETTLEMENT

8.9.1 Stability of Approach Embankments

The existing approach embankments are approximately 7 m high (above original grades), have crest-to-crest widths of approximately 29 m and have sideslopes of approximately 2H:1V.

The existing approach embankments will be required to be widened by approximately 10 m to the north and 7 m to the south 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 embankments. The evaluation of stability for the widened highway embankment on the west side of Ormond Street was carried out 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 widened highway embankment on the east side of Ormond Street is expected to be similar to the west side based on the subsurface conditions and the height of the proposed approach embankments.

The following assumptions were made as part of the stability 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 analysis 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 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 on the south side of Highway 401 under static, drained conditions and seismic conditions are provided on Figures E2 and E3 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 1.4 under static conditions for a critical failure surface extending up to the crest of the embankment; stability analyses carried out using undrained parameters provided similar or higher factors of safety. A factor of safety of greater than 1.1 was calculated under seismic conditions.



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Slope stability analyses were also carried out to assess the FOS against instability for a failure surface extending beneath the west abutment of the new bridge towards the relocated creek channel. The results of the stability analyses indicate that the proposed bridge geometry and embankment configuration outlined on the preliminary General Arrangement drawing would provide a factor of safety against instability of approximately 1.5 under static conditions (refer to Figure E4 in Appendix E). The factor of safety against instability should be re-evaluated during the detailed design stage of the project following completion of additional investigation and based on the final proposed bridge and creek channel configuration.

8.9.2 Embankment Settlement

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

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

- The typical soil profile and associated design parameters for the north side of the bridge shown in Table 8.1 were considered in the settlement analyses. Settlements on the south side of the bridge are expected to be similar or less based on the subsurface conditions.
- The new embankment platform involves widening the existing approach embankments approximately 10 m towards the north and 7 m towards the south with final embankment sideslopes of 2H:1V.
- The structural loads from the bridge abutments will be transferred to the bedrock by the deep foundations and hence will not contribute significantly to the settlement of the embankment.
- The estimated preconsolidation pressures of the silty clay/clay deposits 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 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 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 of the approach embankments. Settlements beneath the existing travelled surface of the highway 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.



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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 (approximately 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

Four 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 ranged from 8 to 72 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 ranged from 7.48 to 7.91, 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 and two of the reported resistivity values were 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 local detour may be required for the construction of the new overpass structure as Ormond Street may be closed to traffic during the construction of the new bridge.

The new bridge is planned to be erected in stages to permit traffic to be maintained on Highway 401 in both directions during construction. Initially, the north and south portions of the existing bridge would be removed and replaced with sections of the new bridge while highway traffic is maintained in the central portion of the right-of-way. Once complete, highway traffic would be shifted to these new structures and the central/median portion of the existing bridge would be demolished and replaced. The use of a temporary roadway protection systems will be required to facilitate this staged construction approach.

9.2 TEMPORARY ROADWAY PROTECTION

Temporary roadway protection will be required to protect traffic on Highway 401 during replacement of the overpass 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)	<ul style="list-style-type: none">• Simple installation process	<ul style="list-style-type: none">• Additional labour required• Groundwater seepage into the excavation can occur without groundwater control• Removal of soldier piles can be difficult• Predrilling may be needed to facilitate soldier pile installation in rockfill	Low	<ul style="list-style-type: none">• 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)	<ul style="list-style-type: none">• Simple installation process• Provides cut-off to groundwater seepage from sides of excavation	<ul style="list-style-type: none">• Difficult to drive/install in rockfill present in embankment fills and soils where cobbles/boulders are present• May require large sections where cantilever design is adopted	Medium	<ul style="list-style-type: none">• Potential for sheet piles to either be damaged, deflected or meet refusal due to obstructions (e.g. rockfill in embankment fill and boulders within the till) during driving



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Due to the potential difficulties installing sheet piles through the rockfill (i.e. very dense fill containing frequent cobbles and boulders encountered in borehole OS21-2) and into the native ground conditions (i.e. very dense glacial till containing cobbles and boulders) at the site, the use of a soldier pile and lagging protection system is considered to be more viable than sheet piles at this site. As noted above, difficulties may be encountered installing conventional H-pile soldier piles due to cobbles and boulders in the rockfill. As an alternative, consideration could be given to the use of drilled pipe piles installed using a down-the-hole hammer drilling system as the soldier piles.

Cobbles and boulders encountered at the face of the temporary support system will need to be removed to permit lagging installation. The removal of these materials will cause the formation of gaps/voids in the soils immediately behind the lagging which could propagate into settlement of the highway surface adjacent to the roadway protection system. Therefore, provision should be made to locate concrete barriers back from the face of temporary protection system as much as possible to limit the potential for settlements on the travelled surface of the highway.

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 meters depth and be developed through the existing highway 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



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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.

OSHA 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

9.4.1 Hydrostatic Uplift

Artesian groundwater conditions, with water levels 0.2 m to 1.1 m above ground surface (corresponding to elevations of about 93.4 m to 93.5 m), were measured in the wells installed into the bedrock in Boreholes OS21-1 and BC21-1 on June 9th, 2021. The excavations for the central pier pile caps are expected to be at elevation 91.5 m or 2 m below the artesian pressure head. The bottom of the cohesive layer at the pier location is anticipated to be near elevation 88.0 m and therefore, hydrostatic uplift at the base of the excavation is not anticipated an issue. Although not currently anticipated as an issue, as part of the detail design, the risk of hydrostatic uplift at the base of the construction excavations should be re-evaluated as a function of the final proposed configuration.

9.4.2 Shallow Excavations

Temporary unwatering, using conventional sump and pump techniques, is considered appropriate for shallow excavations at the site developed predominantly within the silty clay/clay deposits.

9.4.3 Caisson Construction

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 should be considered to reduce the potential for encountering groundwater and associated difficulties installing caisson foundations (if used to support the central pier).

9.4.4 General

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 is available.

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



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS REPLACEMENT - SITE NO. 16X-0123/B0

Construction Considerations
February 2023

9.5 CONSIDERATION OF EXISTING BRIDGE FOUNDATIONS

The existing bridge abutments and wingwalls are founded on 318 mm diameter concrete filled steel tube pile foundations including piles battered/inclined away from Ormond Street. The retaining walls present at each end of the existing abutments are understood to be founded on spread footings.

The existing abutment and wingwall pile caps are located in close proximity to, and in some cases overlap with, the proposed footprint of foundations for the centre pier and east abutment of the replacement bridge. The back line of the existing piles in the abutment pile caps are battered away from Ormond Street (i.e. towards the locations of the proposed new center pier and east abutment) and could be encountered within the excavation zone for drilled caissons or during driving for H-piles. Accordingly, the design of piles for central pier and east abutment of the new bridge will need to be selected to minimize the potential for encountering the existing pile foundations (i.e. new piles/caissons will need to be located to avoid conflicts with existing piles).

The deviation of the existing piles from the intended/design locations depends on several factors including quality of construction, potential that obstructions were encountered during installation, and pile length/stiffness (e.g. long/slender or flexible piles have a greater potential for deviation than shorter or stiffer piles). There is currently no information available on the installation of the existing piles (e.g. pile driving records, as-built pile locations, or similar) and the actual lengths of the existing piles were not identified on the available structural drawings; however, they are expected to have been driven to the surface of the bedrock which is about 4 m to 5 m below the pile cap level. The subsurface conditions through which the piles were installed consisted predominantly of very stiff silty clay/clay with a thin zone of till present over top of the bedrock in some areas. The short length of the piles and the very stiff nature of the cohesive soils would reduce the potential for pile deviation.

The shallow foundations for the existing retaining walls are recommended to be removed prior to installation of the new bridge foundations.

9.6 OBSTRUCTIONS

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



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS REPLACEMENT - SITE NO. 16X-0123/B0

Further Work For Detailed Design
February 2023

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 overpass 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 caisson/drilled pier foundations are considered for use, boreholes at the center 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.
- Triaxial testing of the native silty clay/clay is recommended to be completed to verify strength parameters used in the stability analyses.
- Following completion of the additional investigation and laboratory testing, the soil design parameters outlined and analysis results outlined in this report should be re-evaluated.
- An evaluation of the rock permeability should be carried out to assess the dewatering efforts that would be required to support a caisson construction option for the centre pier.
- An evaluation of the hydrostatic uplift potential should excavations extending below elevation 91.5 m be proposed.
- 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.



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS
REPLACEMENT - SITE NO. 16X-0123/B0**

Specifications
February 2023

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



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS
REPLACEMENT - SITE NO. 16X-0123/B0**

Closure
February 2023

12.0 CLOSURE

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information 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 Zach Popper, 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.

Zach Popper, P.Eng.
Geotechnical Engineer



Kevin Nelson, P. Eng.
Principal, Senior Geotechnical Engineer



Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundations Contact



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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS REPLACEMENT - SITE NO. 16X-0123/B0

References
February 2023

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
- 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.
- Duncan Hopper and Associates. 1956. Bridge Design Drawings. Elizabeth Twp. Bridge No. 13, Ormond Street - Drawing No. 5638-2 to 5638-9.
- 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 – Department of Highways. 1958. Report of Foundation Investigation for Proposed Overpass Bridge, Highway 401 at Ormond St., Brockville, 1958 (GEOCRETS Reference No. 31B00-015).
- 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.



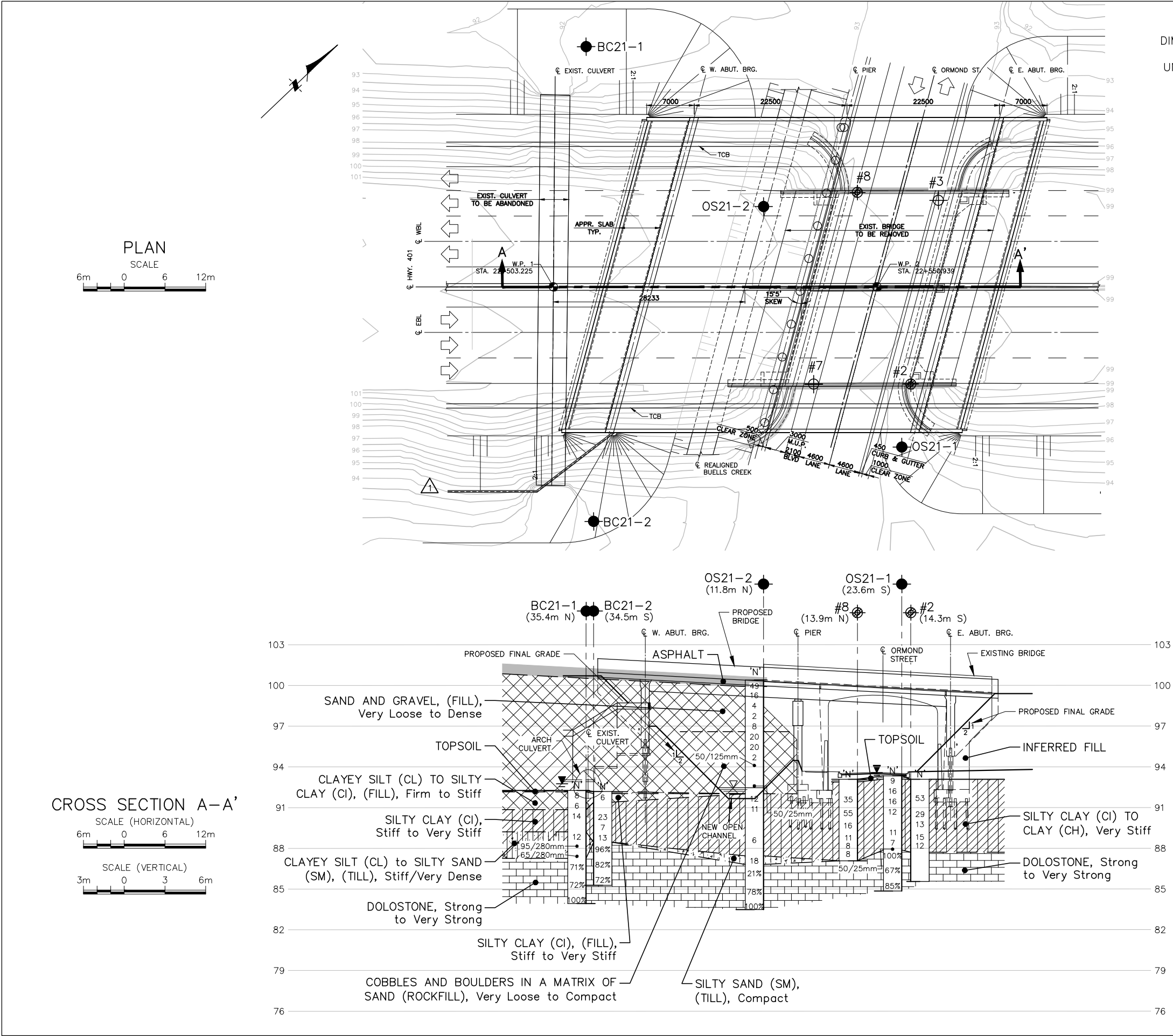
**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - ORMOND STREET OVERPASS
REPLACEMENT - SITE NO. 16X-0123/B0**

February 2023

APPENDIX A

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



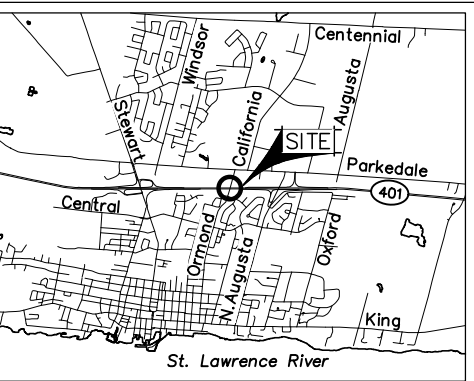


METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



PLATE No
CONT
GWP 4003-19-00

HIGHWAY 401
ORMOND ST., BROCKVILLE
BOREHOLE LOCATIONS & SOIL STRATA



KEY PLAN
1 km 0 1 2 km

LEGEND

- Borehole (Stantec 2021)
- Penetration Test & Borehole (MTO 1955)
- Penetration Test Hole (MTO 1955)
- (x.xm) Offset from Cross Section Line in meters
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL Measured on July 2021

No	ELEV	MTM ZONE 9 COORDINATES NORTH	COORDINATES EAST
BC21-1	92.2	4 941 097.5	368 969.0
BC21-2	92.2	4 941 050.0	369 020.3
OS21-1	93.3	4 941 090.4	369 043.8
OS21-2	100.4	4 941 100.2	369 004.1
#2	93.1	4 941 097.8	369 038.2
#3	93.1	4 941 119.4	369 021.3
#7	93.0	4 941 087.4	369 028.1
#8	93.1	4 941 111.6	369 012.2

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-106	DIST	
HWY No 401		SITE	16-123
SUBM'D KN	CHECKED	DATE 2023-02-15	DWG 1
DRAWN GBB	CHECKED	APPROVED RH	

February 2023

APPENDIX B

B.1 AVAILABLE GEOCRETS INFORMATION INCLUDING SOIL STRATA PLOT AND BOREHOLE RECORDS



MATERIALS LABORATORY-DEPARTMENT OF HIGHWAYS - ONTARIO
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG - CORE DRILL # 4
CASING - BX (STANDARD SAMPLERS TO FIT UNLESS NOTED)
SAMPLER HAMMER WT 250 - * DROP - INCHES

JOE F-55-D BROCKVILLE BORING NO. 2
 DATUM STA 257+60.2 R.L. 48@ 105'15" DATE REPORT
 COMPILED BY B.H. CHECKED BY W. Wong BORING DATE 10 ± 1" May 1955.

SAMPLE CONDITION



DISTURBED
GOOD
LOST

SAMPLE TYPES

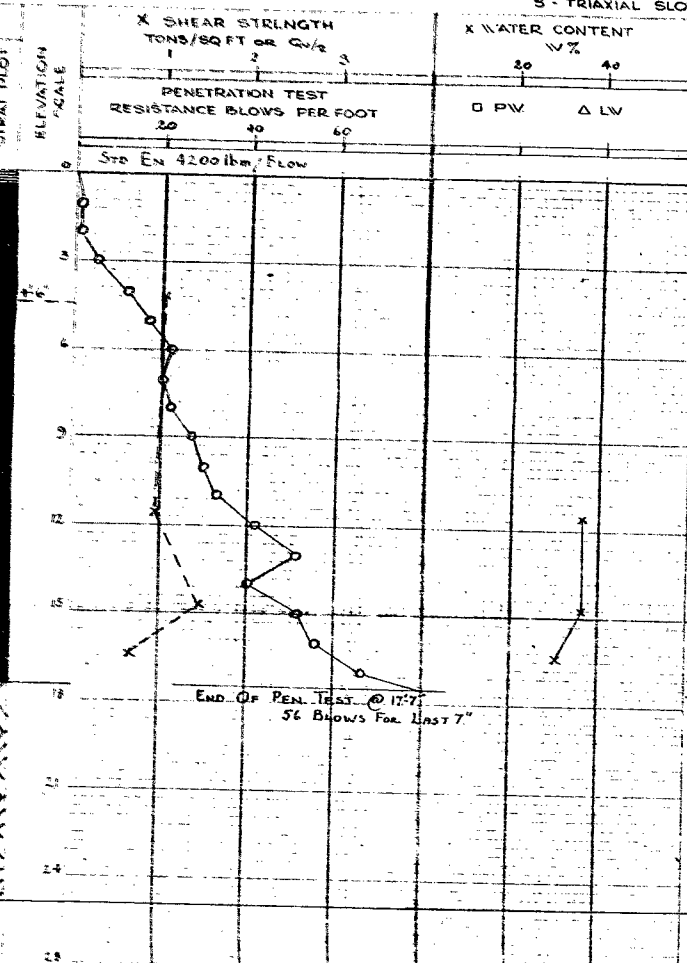
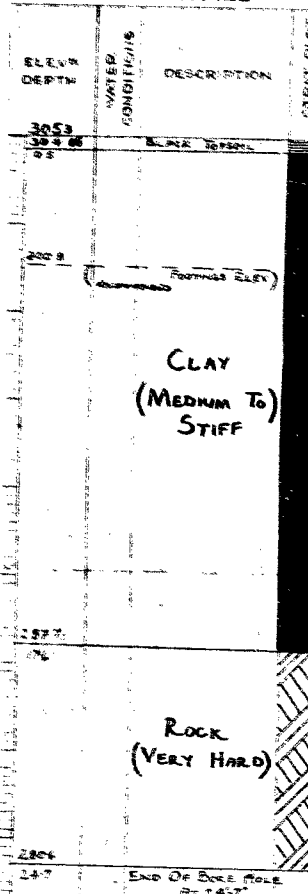
CS - CRUNK
DO - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN

ABBREVIATIONS







ABBREVIATIONS

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

SOIL PROFILE



SAMPLES

OTHER TESTS	CONDITION	TYPE	N ₂	PENETRATION RESISTANCE	ELEV. RECOVER.
					27 30.3 No. Of Blows Per Foot 5th. EN. 4200 lb in
I = 103		T.O. 3"	1	53	301.3 100
a = 12.2		T.O. 3"	2	29	297.3 100
a = 116		T.O. 2"	3	13	295.3 79
J = 116		T.O. 2"	4	15	292.3 109
J = 123		T.O. 2"	5	12	290.3 100
					287.7 100
		RC	6		

TL 115
54-90

MATERIALS LABORATORY - DEPARTMENT OF HIGHWAYS - ONTARIO
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG CORE DRILL #4
CASING DB (STANDARD SAMPLERS TO FIT UNLESS NOTED)
SAMPLER HAMMER WT. 250 # DROP INCHES

JOB F-55-9 BROCKVILLE BORING NO 8
 DATUM STA 257+18, L1 4.8 @ 76' 75" DATE REPORT
 COMPILED BY D.H. CHECKED BY W. Wong BORING DATE 12TH MAY 1955

SAMPLE CONDITION



DISTURBED
GOOD
LOST

SAMPLE TYPES

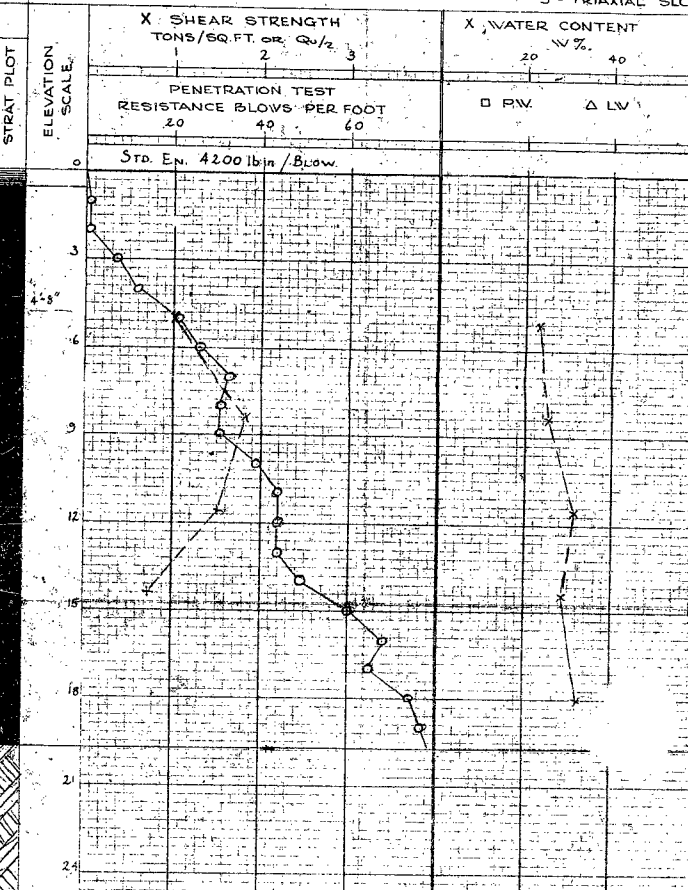
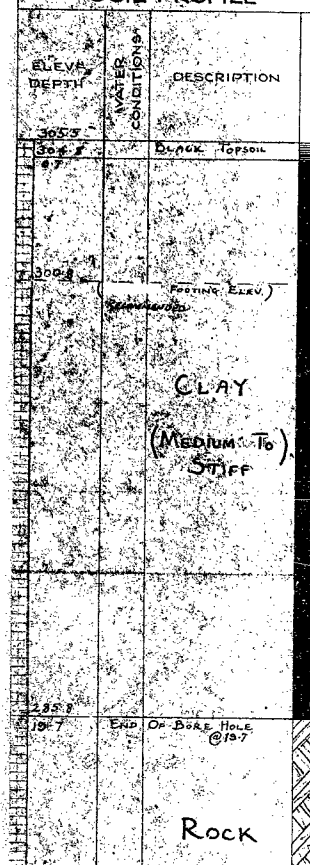
CS - CHUNK
DO - DRIVE OPEN
DF - DRIVE FOOT VALVE
TO - THIN WALLED OPEN

ABBREVIATIONS







Abbreviations

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

SOIL PROFILE



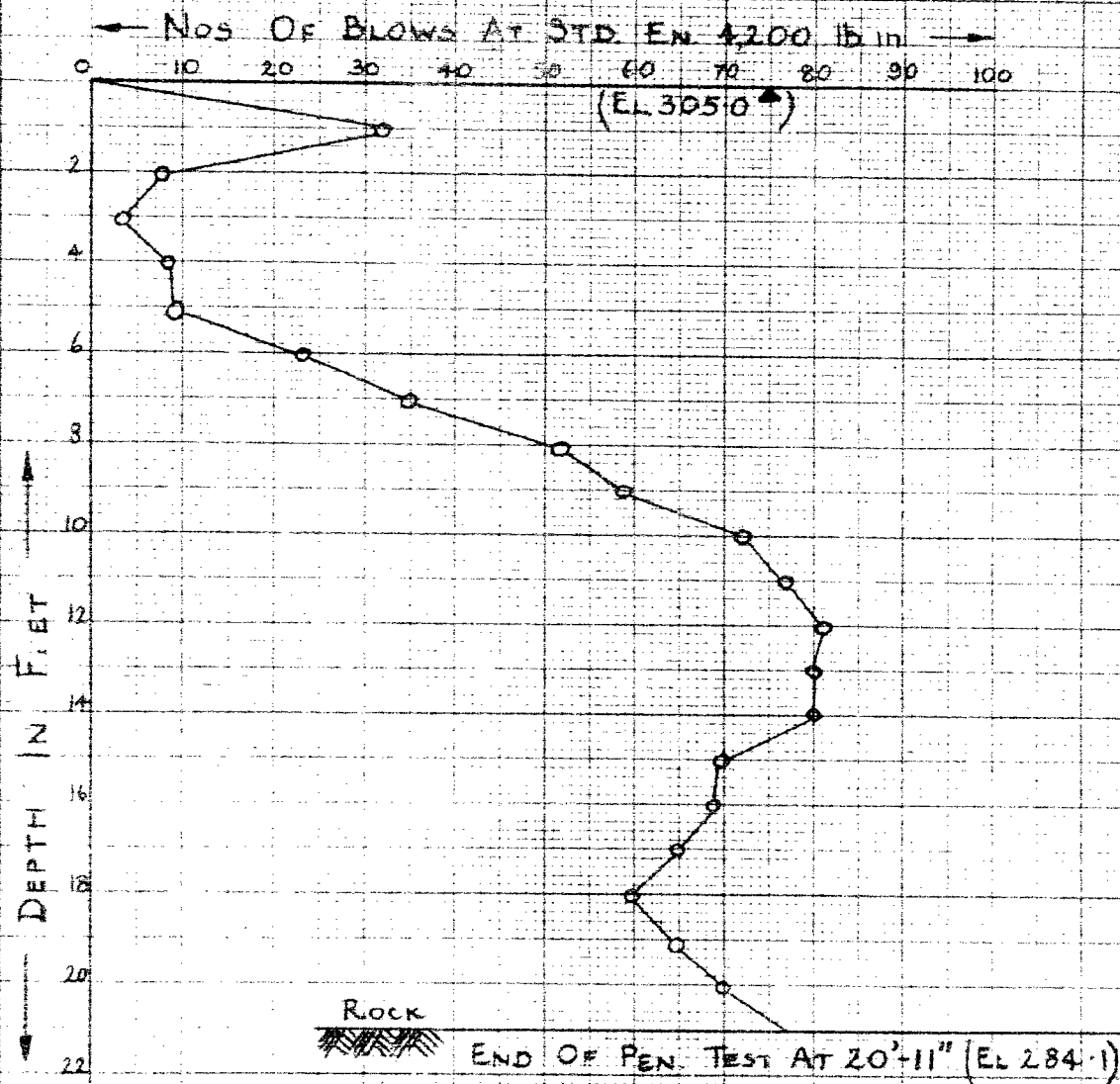
SAMPLES

OTHER TESTS	CONDITION	TYPE	NS	PENETRATION RESISTANCE	ELEV. RECOV.
				TO EN 4200 lb in per Blow	302.5
					299.5 100
4 = 111		T.O. 3"	1	35	296.5 46
4 = 117		T.O. 3"	2	55	293.5 71
4 = 116		T.O. 2"	3	16	290.5 75
		T.O. 2"	4	11	288.5 100
		T.O. 2"	5	8	286.5 100
		T.O. 2"	6	8	

GRAPH OF CONE PENETRATION TEST

PEN. HOLE N°3 JOB F-55-9

LOCATION STA. 257+58.2; LT. 44' @ 76° 75'



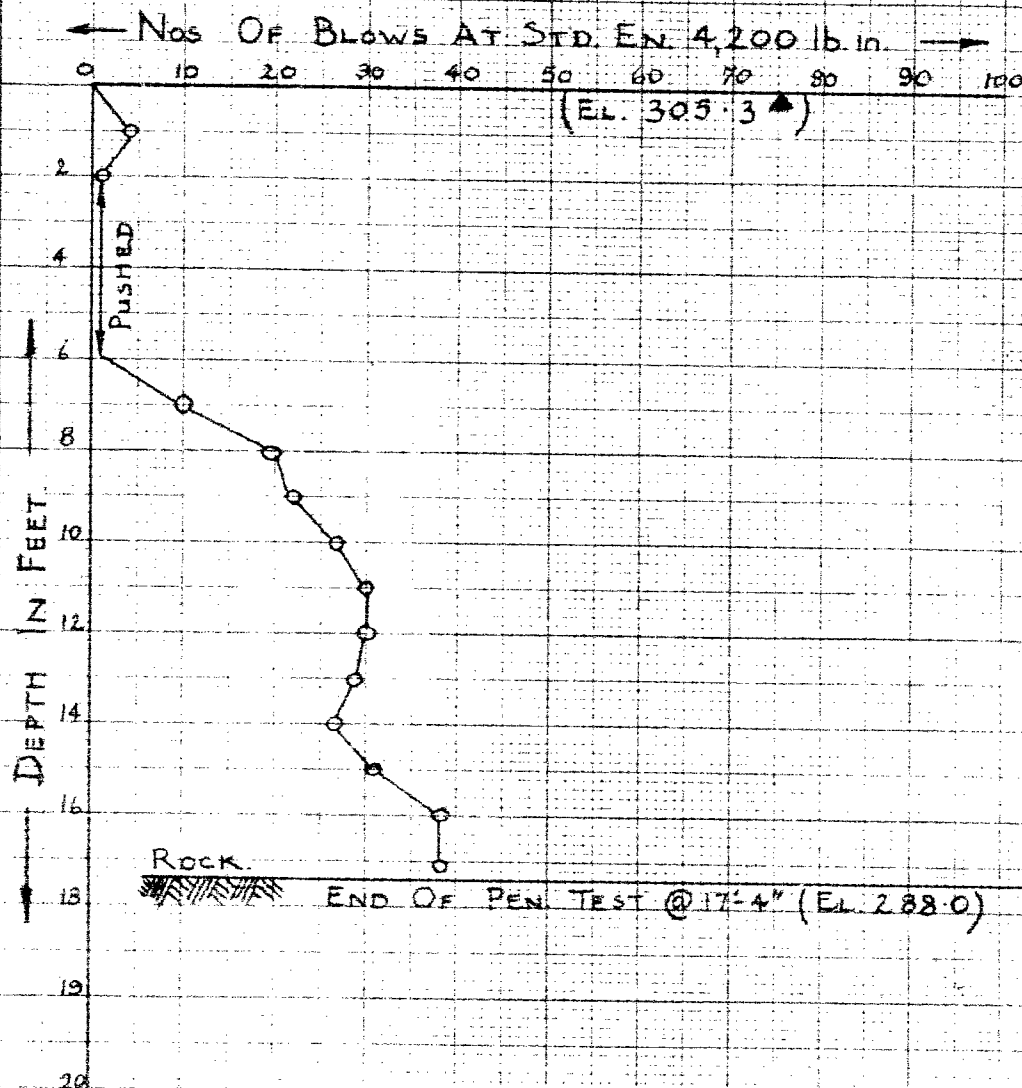
CHECKED W. WONG

GRAPH OF CONE PENETRATION TEST

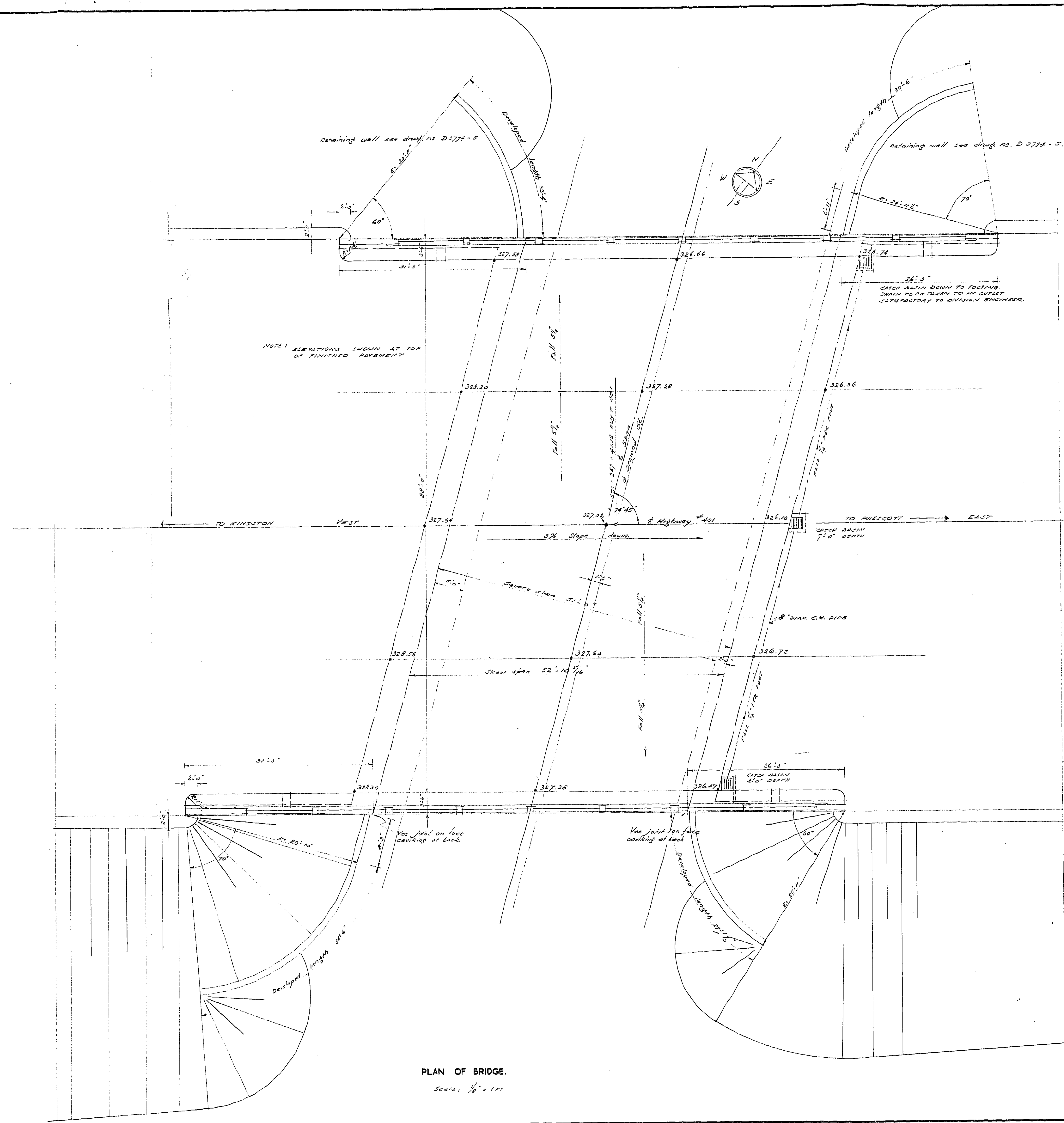
PEN. HOLE N° 7

JOB F-55-9

LOCATION STA 257+22; RT 48° @ 105° 15'

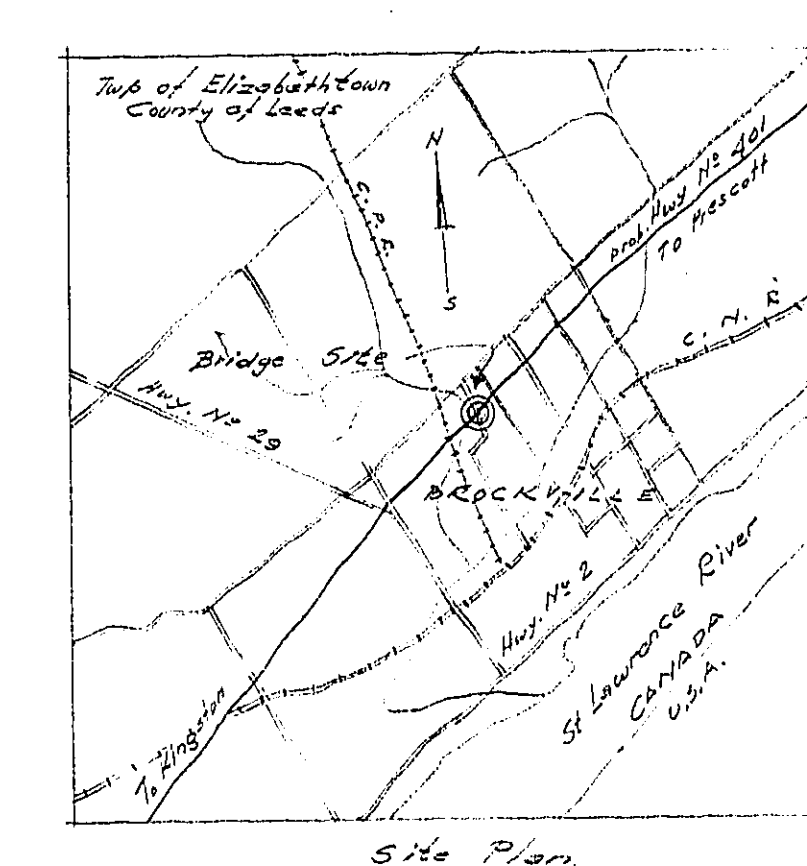
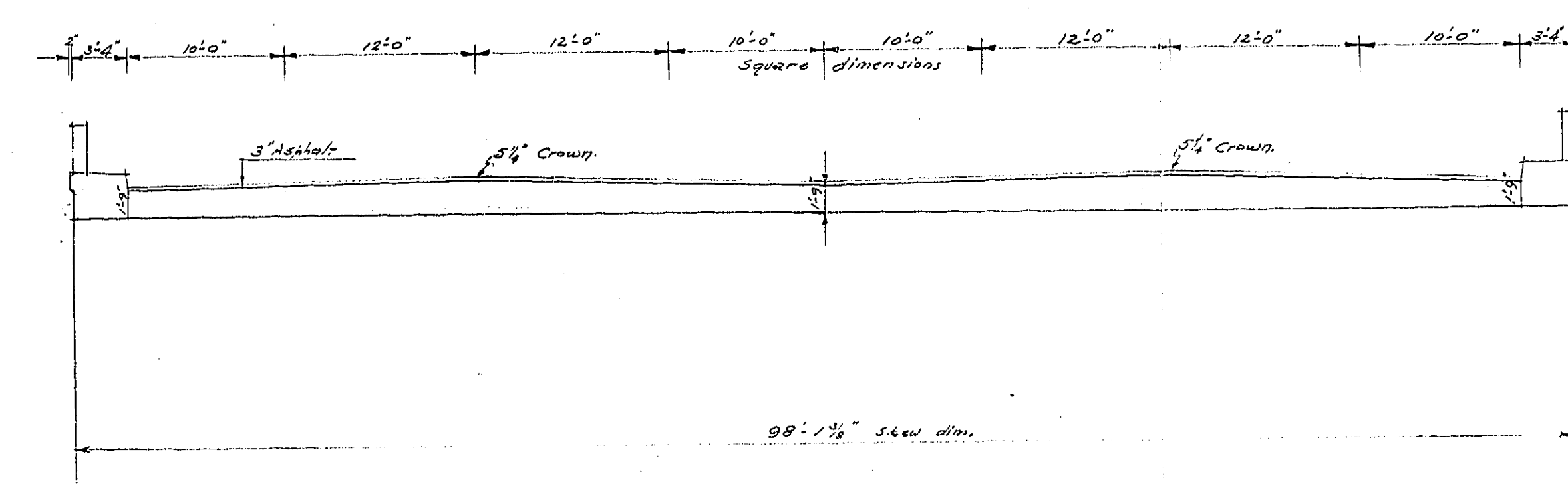
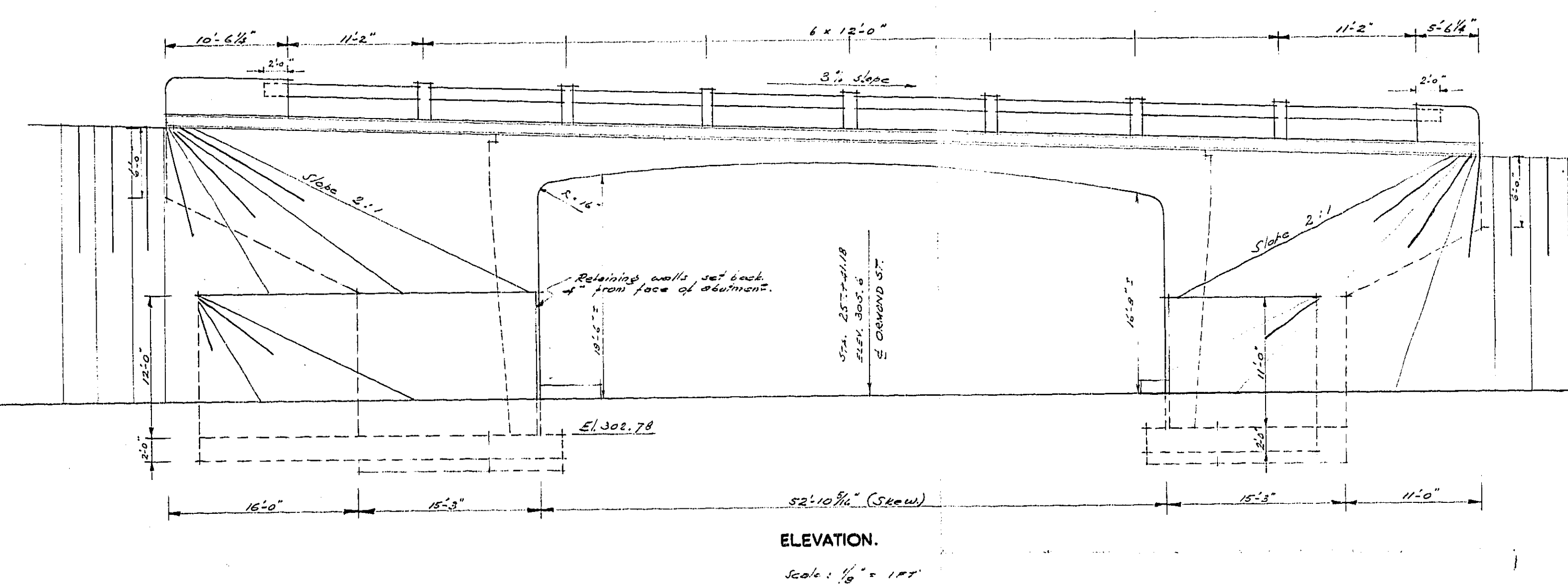


CHECKED W. WONG



NOTES :
FOR DIVISION ENGINEER :
CONCRETE WORK MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX LINES AND GRADES HAVE BEEN ERECTED BY THE DIVISION ENGINEER.

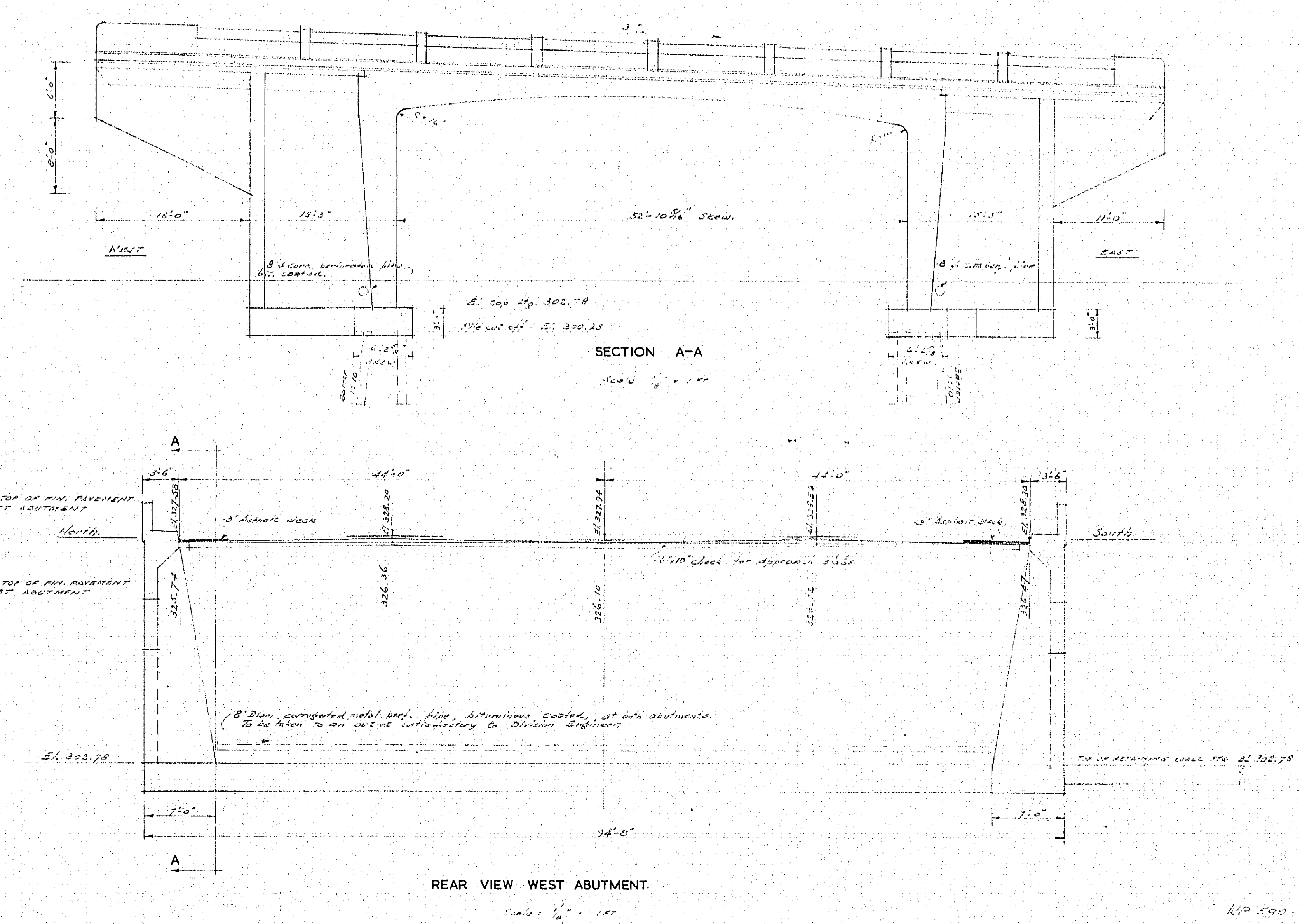
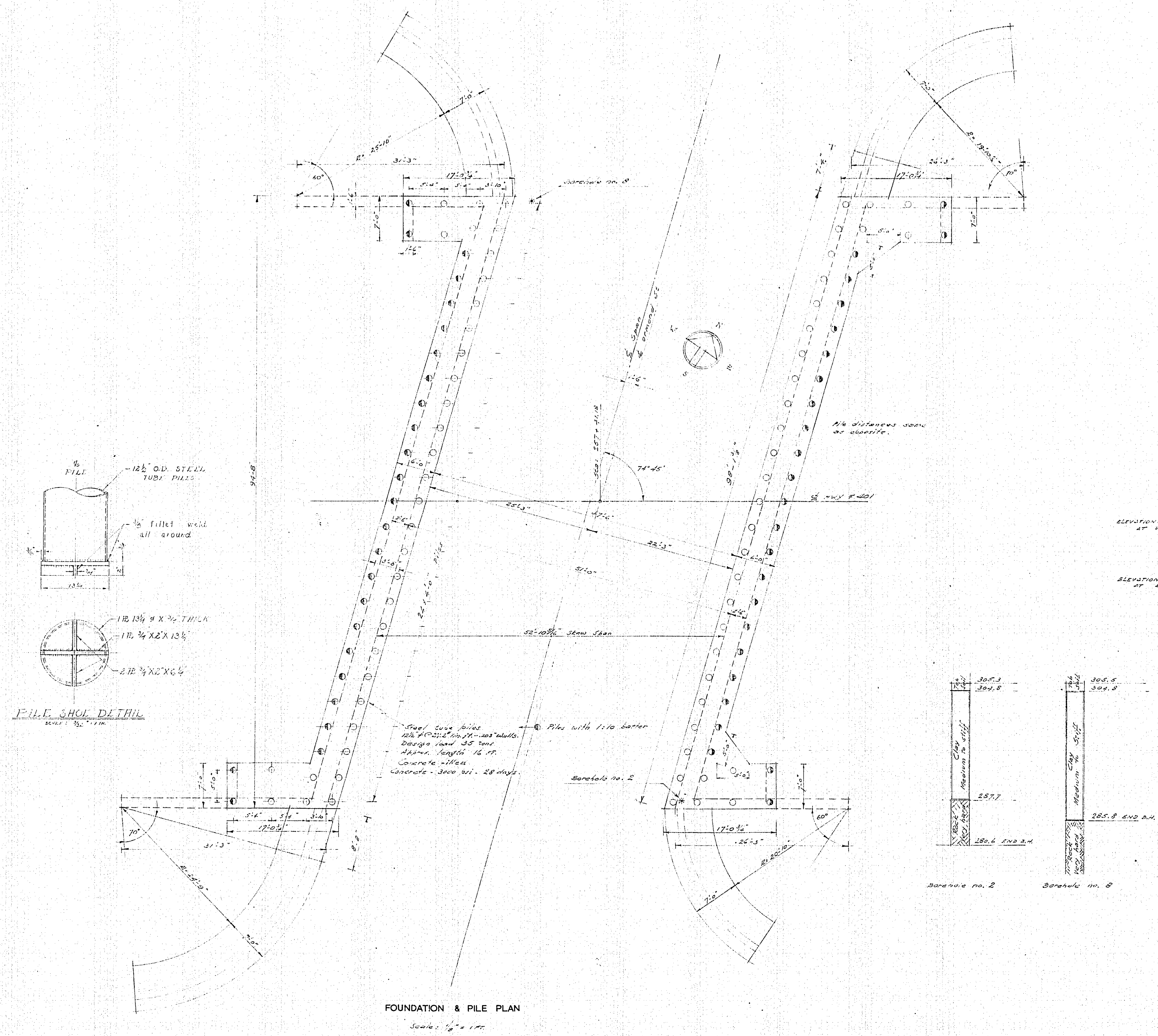
FOR CONTRACTOR:
STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM No. 9
REVISED MARCH 1957 AND THE SPECIAL PROVISIONS, EXTERA
COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT
ENGINEER.
ALL CONSTRUCTION JOINTS TO BE STRAIGHT AND TRUE.
CONSTRUCTION JOINTS IN LOCATIONS NOT SHOWN ON THE PLANS ARE TO BE PROVIDED
BY THE SCOPE OF DESIGN.
ALL JOINTS FOR CURBS AND STRUCTURE TO BE 3000 P.S.I. @ 28 DAYS -
AND 1/4" AIR-LOCKING "S" PER GALS OF CEMENT, MAXIMUM AGRESTA SIZE TO BE 1/2"
ALL EXPOSED CORNERS TO BE CHAMFERED 1" UNLESS OTHERWISE NOTED.
REINFORCEMENT SHALL BE "41-BOND" MEDIUM HARD GRADE STEEL TO
CONFORM TO A.C.I. SPECIFICATIONS UNLESS OTHERWISE NOTED.
MINIMUM CLEAR COVER ON REINFORCEMENT TO BE 2" AT EXPOSED
CONCRETE SURFACES, OTHERWISE "3"



# BRIDGE	4120 590-56. REC. 14. H.P.E. INSURANCE BOARD Jan. 4	
	DUNCAN ROOPER & ASSOCIATES CONSULTING ENGINEERS 1393 WILSON AVENUE, DOWNSVIEW ONTARIO	Drawing No 5638 - 2
<u>DEPARTMENT OF HIGHWAYS:ONTARIO-</u> BRIDGE OFFICE-TORONTO		
ELIZABETHTOWN TWP BRIDGE NO. 13. ORMOND STREET		
THE KING'S HIGHWAY No. 401 _____		DIV. No. 8 _____
CO. LEEDS _____		
TWP. ELIZABETHTOWN _____	LOT 10 _____	CON. I _____
PLAN, ELEVATION, & DECK SECTION.		
<div style="display: flex; justify-content: space-between;"> APPROVED <i>[Signature]</i> </div> <hr style="border-top: 1px dashed black;"/> <div style="display: flex; justify-content: space-between;"> CHIEF ENGINEER GENERAL </div>		
DESIGN DRAWING TRACING DATE	<i>J.L.H.</i> CHECK <i>J.L.H.</i> CHECK <i>K.C.</i> CHECK Nov. 20, 1956.	CONTRACT NUMBERS LOADINGS DRAWING NUMBER
		57-165 <i>D-5748</i>

REVISIONS:					REFERENCE PLANS	BRIDGE EXISTING				GENERAL	
					DESIGN	J. C. H.	CHECK	CONTRACT		57-165	
					DRAWING	J. C. H.	CHECK	NUMBERS			
					TRACING	H.	CHECK	LOADING			
					DATE	Nov. 20, 1956.		420.514	DRAWING	57-165	
	DATE	BY	DESCRIPTION							NUMBER	

PRINT RECORD		
No.	FOR	DATE
30	FORWARD	11/10/56
13	REVISION	11/10/56



DUNCAN HOPPER & ASSOCIATES CONSULTING ENGINEERS 130 WILSON AVENUE, DOWNSVIEW, ONTARIO		Drawing No. 57-163
DEPARTMENT OF HIGHWAYS-ONTARIO BRIDGE OFFICE-TORONTO		
ELIZABETHTOWN TWP. BRIDGE NO. 13. ORMOND STREET		
THE KING'S HIGHWAY No. 401		DIV. No. 8
CO. LEEDS		CON. 1
TWP. ELIZABETHTOWN		LOT 10
FOUNDATION & PILE PLAN- ELEVATION & SECTION		
APPROVED <i>Bill Long</i> BRIDGE ENGINEER		
CHIEF ENGINEER [Signature]		
DESIGN CHECK TRACING DATE Nov. 20 1956	CHECK CHECK CHECK Nov. 16	CONTRACT NO. 516 DRAWING NO. 57-163

Twp# 25-123-2-A

February 2023

APPENDIX C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

C.2 BOREHOLE RECORDS (CURRENT INVESTIGATION)

C.3 BEDROCK CORE PHOTOS



SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

Terminology describing soil types:

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

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

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

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

Terminology describing compactness of cohesionless soils:

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

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

Terminology describing consistency of cohesive soils:

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

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

ROCK DESCRIPTION

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

Terminology describing rock quality:

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

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

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

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

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

Terminology describing rock with respect to discontinuity and bedding spacing:

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

Terminology describing rock strength:

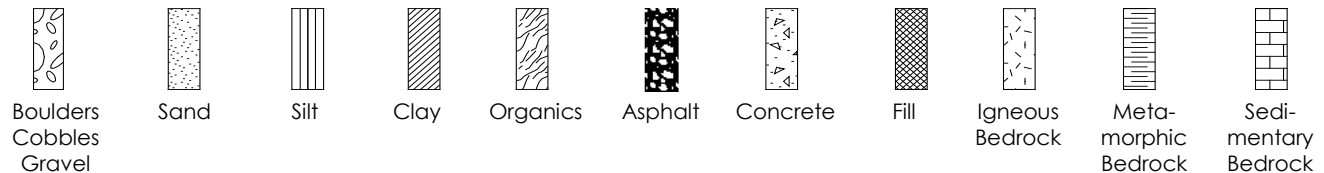
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

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

STRATA PLOT

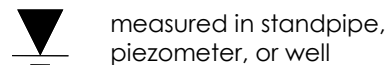
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

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

WATER LEVEL MEASUREMENT



RECOVERY

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

N-VALUE

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

DYNAMIC CONE PENETRATION TEST (DCPT)

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

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
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 BC21-1

1 OF 1

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 - Brockville N:4941097.5 E:368969.0 ORIGINATED BY KT
 DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring COMPILED BY KL
 DATUM Geodetic DATE 2021.05.05 - 2021.05.05 LATITUDE 44.607024 LONGITUDE -75.691582 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE								
						● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)									
						20	40	60	80	100	20	40	60				
92.3																	
90.0	100 mm TOPSOIL																
0.1	CLAYEY SILT (CL) to SILTY CLAY (CI), some sand, trace gravel and organic matter, (FILL) Firm to very stiff Brown Moist		1	SS	8												
			2	SS	6												
90.9																	
1.4	SILTY CLAY (CI) Very stiff Grey Moist Varved below 2 m		3	SS	14												
			3A	SH	-												
89.2																	
3.0	CLAYEY SILT (CL), some sand, trace gravel, (TILL) Contains cobbles and boulders Stiff Grey Wet Auger grinding below 3.8m Transitions to SILTY SAND (SM), some gravel, trace clay, (TILL) Very dense Grey Moist		4	SS	12												
			5	SS	95/ 280mm												
			6	SS	65/ 280mm												
87.3																	
5.0	DOLOSTONE Light grey to grey Fair quality Fresh to slightly weathered Very strong		7	NQ	-												
			8	NQ	-												
	Excellent quality below 7.7 m		9	NQ	-												
83.9																	
8.3	End of Borehole																
	Water levels measured in monitoring well: At ground surface on May 5, 2021 At ground surface on May 11, 2021 1.09 m above ground on May 13, 2021 0.50 m above ground on June 9, 2021 0.05 m below ground on October 22, 2021																

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

ONTARIO MTO 165001160_Hwy 401_BROCKVILLE.GPJ ONTARIO MTO.GDT 12/1/21

RECORD OF BOREHOLE No BC21-2

1 OF 1

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 - Brockville N:4941050.0 E:369020.3 ORIGINATED BY KT
DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring COMPILED BY KL
DATUM Geodetic DATE 2021.05.05 - 2021.05.05 LATITUDE 44.606592 LONGITUDE -75.690941 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED	+	FIELD VANE	×						LAB VANE	
92.2	150 mm TOPSOIL		1A	SS			20	40	60	80	100					GR SA SI CL		
92.0	SILTY CLAY (CL), trace sand and organic matter, (FILL) Stiff Brown Moist SH2 contains 70 mm organic matter on top		1B	SS	6													
91.4	SILTY CLAY (CL), trace sand Brown Stiff to very stiff Moist		2	SH	-											0 2 58 40		
0.8																Su > 118 kPa Vane Refusal		
			3	SS	23													
	Contains clayey silt layers/varves below 2.5 m Trace gravel		4	SS	7													
			5	SS	13													
88.4	Auger Refusal at 3.7 m DOLOSTONE Light grey to grey Fair to excellent quality Fresh to slightly weathered Very strong		6	NQ	-											TCR = 100% RQD = 96% UCS=137.8 MPa		
3.7																		
			7	NQ	-											TCR = 96% RQD = 82%		
			8	NQ	-											TCR = 100% RQD = 72%		
85.3	End of Borehole																	
6.9	Groundwater not observed in borehole prior to initiation of bedrock coring.																	

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

ONTARIO MTO 165001160_HWY 401_BROCKVILLE.GPJ ONTARIO MTO.GDT 12/1/21

RECORD OF BOREHOLE No OS21-1

1 OF 1

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 - Brockville N: 4941090.4 E: 369043.8 ORIGINATED BY KT
 DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring COMPILED BY KL
 DATUM Geodetic DATE 2021.05.07 - 2021.05.07 LATITUDE 44.606953 LONGITUDE -75.690641 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
93.3							20	40	60	80	100					
0.0	250 mm TOPSOIL: Sandy Silt, containing organic matter															
93.1	Dark brown															
0.3	Moist		1	SS	9		93									
	SILTY CLAY (CI) to CLAY (CH), trace sand															
	Very stiff															
	Grey-Brown		2	SS	16		92									
	Moist															
			3	SS	16											
			4	SS	12		91									
	Stiff to very stiff and grey below 3 m															
			5	SH	-		90									
			6	SS	11		89									
			7	SS	7											
87.9	Auger grinding at 5.2 m depth SS8 contains wood pieces		8	SS	50/ 25		88									
5.5	DOLOSTONE Light grey to grey Fair to excellent quality Slightly weathered to fresh Very strong Auger Refusal at 5.5 m		9	NQ	-		87									
			10	NQ	-		86									
			11	NQ	-		85									
84.9	End of Borehole															
8.5	Water levels in monitoring well: 0.45 m below ground on May 11, 2021 0.21 m above ground on May 13, 2021 0.23 m above ground on June 9, 2021 0.89 m below ground on October 22, 2021															

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

ONTARIO MTO 165001160_Hwy 401_Brockville.GPJ ONTARIO MTO.GDT 12/1/21

RECORD OF BOREHOLE No OS21-2

1 OF 2

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 - Brockville N:4941100.2 E:369004.1 ORIGINATED BY KT
DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger (0 to 6m), NW casing below 6 m, NQ Rock Coring COMPILED BY KL
DATUM Geodetic DATE 2021.05.11 - 2021.05.11 LATITUDE 44.607044 LONGITUDE -75.691139 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE							
						● QUICK TRIAXIAL	× LAB VANE									
100.4							20 40 60 80 100							GR SA SI CL		
0.0	480 mm ASPHALT															
99.9			1	SS	49		100									
0.5	SAND and GRAVEL, trace silt (FILL)															
99.6	Dense															
0.8	Brown															
	Moist															
	SAND, trace to some gravel, (FILL)		2	SS	16									13 65 19 3		
	Contains pockets/zones of sandy silt															
	Very loose to compact															
	Brown															
	Moist															
			3	SS	4											
			4	SS	2											
	Rock in tip of the split spoon		5	SS	8									2 93 4 1		
96.6																
3.8	Cobbles and boulders in a matrix of SAND, trace gravel (ROCKFILL)		6	SS	20									No Recovery		
	Very loose to compact															
	Brown															
	Moist															
	Auger grinding noted below 4.5 m															
			7	SS	20											
			8	SS	2											
	NW casing advanced below 6 m															
	Pieces of rock retrieved during casing advancement		9	SS	50/ 125mm									No Recovery		
														UCS= 116.3 MPa		
														UCS= 176 MPa		
			10	SS	50/ 25mm									No Recovery		
92.0																
8.4	SILTY CLAY (CI) to CLAY (CH), trace sand		11	SS	12											
	Very stiff															
	Grey															
	Moist															
	Contains stiff zones below 9 m															
			12	SS	11											

Continued Next Page

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

ONTARIO MTO 165001160 HWY 401 BROCKVILLE GPJ ONTARIO MTO.GDT 12/1/21

2 OF 2

METRIC

DATUM	Geodetic	DATE	2021.05.11 - 2021.05.11	LATITUDE	44.607044	LONGITUDE	-75.691139	CHECKED BY	KN
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[illegible]

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



Project No.: 165001160

Project Name: Hwy 401-Ormond St. Overpass

Rock Core
Photographs



Rock Core Photo No.: 1

Borehole: BC21-1

Depth: 5.0 m to 8.3 m



Rock Core Photo No.: 2

Borehole: BC21-2

Depth: 3.7 m to 6.9 m



Project No.: 165001160

Project Name: Hwy 401-Ormond St. Overpass

Rock Core
Photographs



Rock Core Photo No.: 1

Borehole: OS21-1

Depth: 5.2 m to 8.5 m



Rock Core Photo No.: 2

Borehole: OS21-2

Depth: 13.6 m to 16.9 m

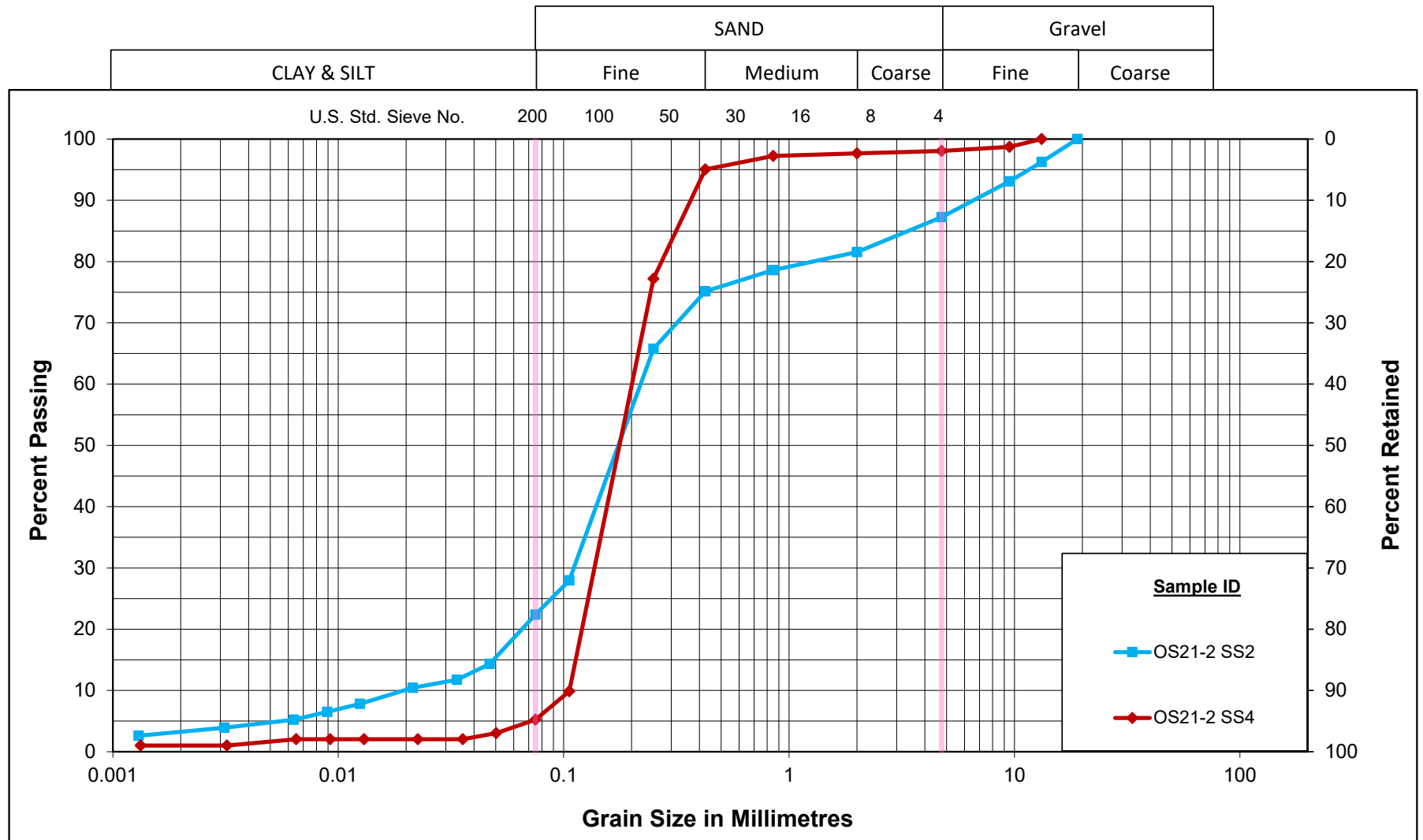
February 2023

APPENDIX D

D.1 LABORATORY TEST RESULTS



Unified Soil Classification System



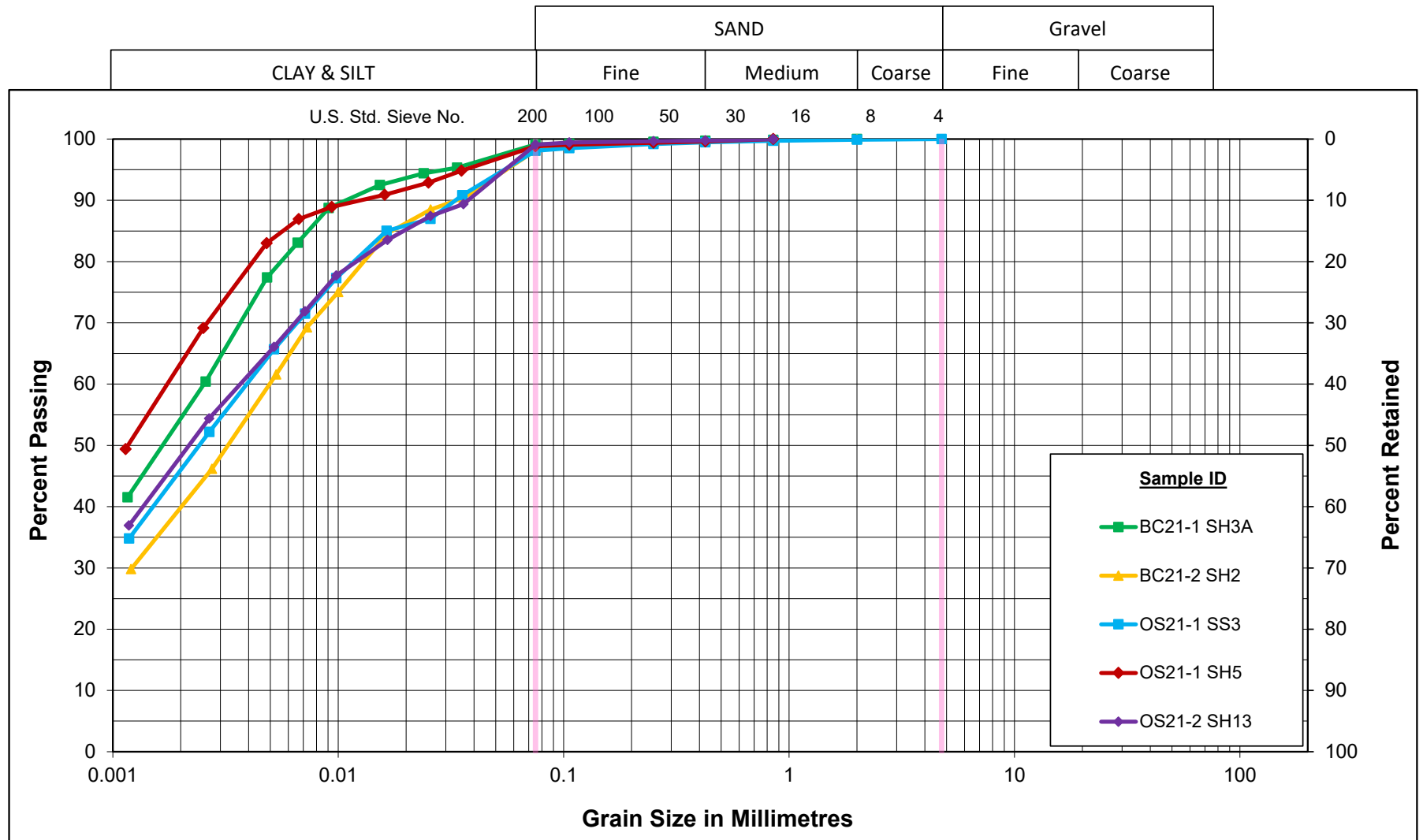
GRAIN SIZE DISTRIBUTION

FILL: SAND to Silty SAND (SP to SM)
Hwy 401 Brockville - Ormond Street Overpass

Figure No. D1

Project No. 165001160

Unified Soil Classification System



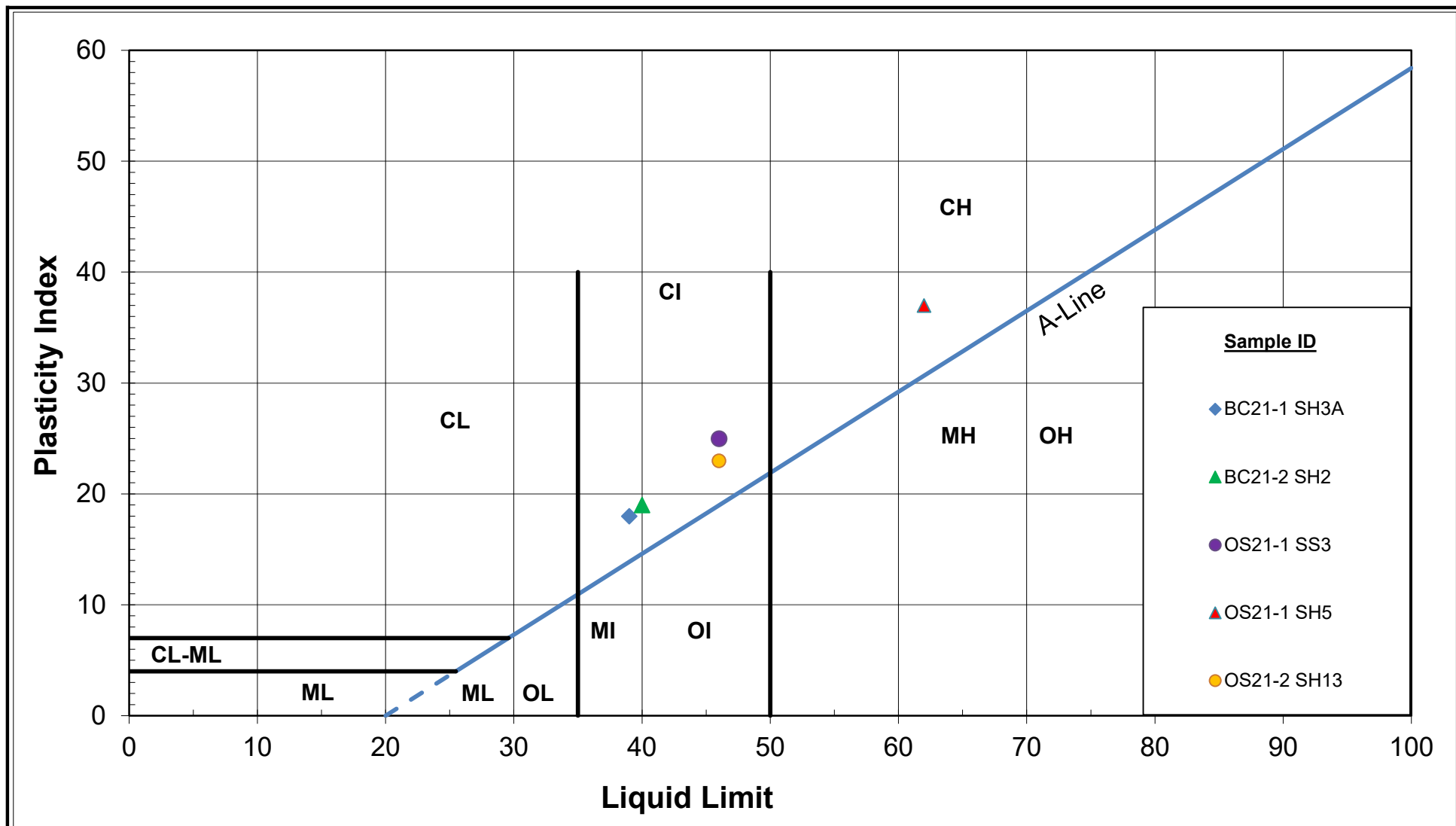
GRAIN SIZE DISTRIBUTION

SILTY CLAY (CI) to CLAY (CH)

Hwy 401 Brockville - Ormond Street Overpass

Figure No. D2

Project No. 165001160



SILTY CLAY (CI) to CLAY (CH)
Hwy 401 Brockville - Ormond Street Overpass
PLASTICITY CHART

Figure No. D3

Project No. 165001160

Project
Project No.
Borehole No.
Sample No.
Sample Depth

Hwy 401 Brockville - Ormond Street Overpass
165001160(309)
BC 21-1
SH-3A
2.13 - 2.74 m.

Figure No. D4

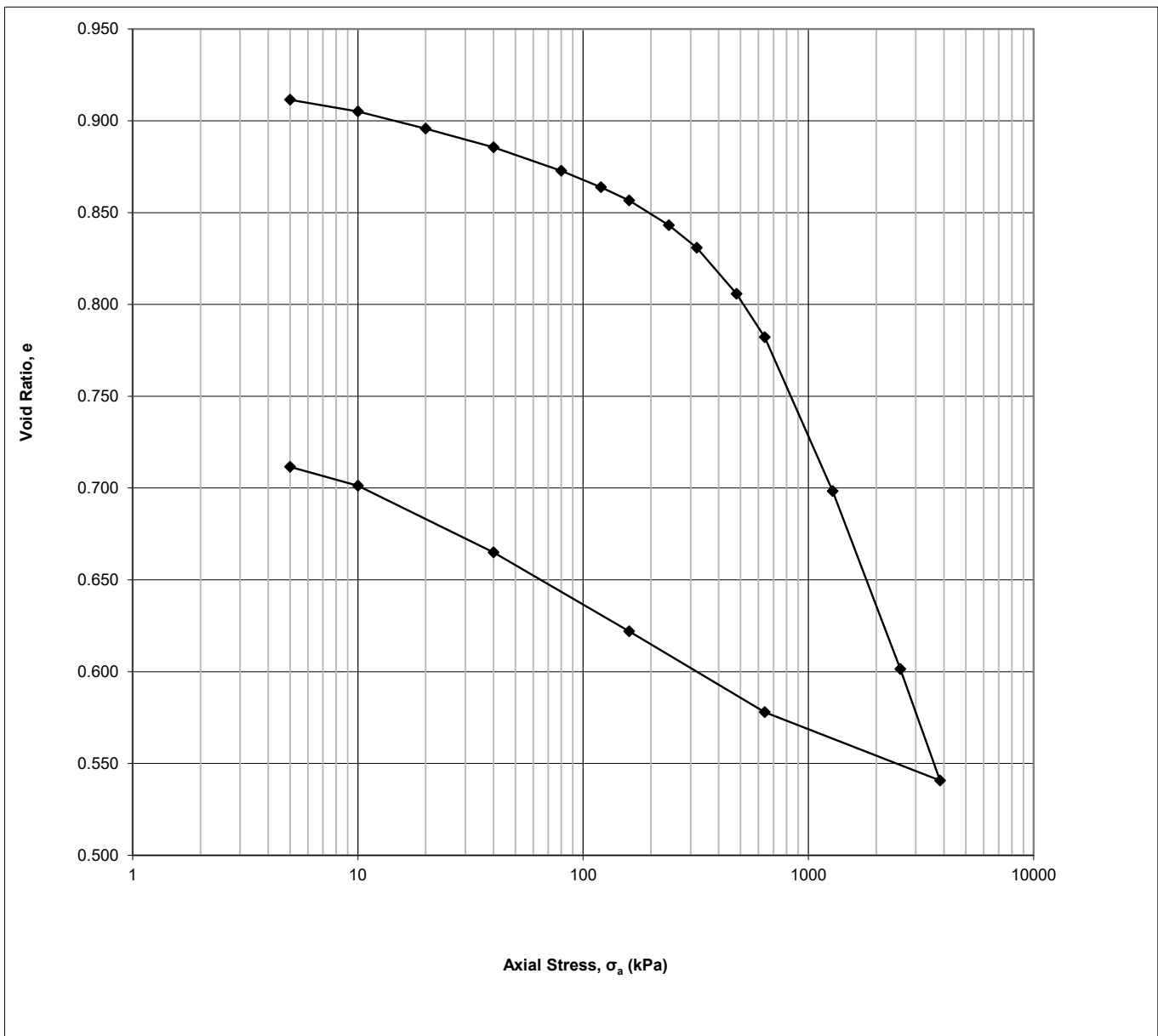


Figure D4A



Stantec Consulting Ltd.

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

Specimen Details

Project Name	Hwy 401 Brockville - Ormond Street Overpass
Project Location	Brockville, Ontario
Borehole	BC 21-1
Sample No.	SH-3A
Depth	2.13 - 2.74 m.
Sample Date	May 5, 2021
Test Number	One
Technician Name	Daniel Boateng

Soil Description & Classification

<i>Silty clay, firm to stiff, dark grey, varved/desiccated, moist</i>	
Specific Gravity of Solids	2.728
Average water content of trimmings %	32.93
Additional Notes (information source, occurrence 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	72.67
Dry Mass	g	54.67
Density	Mg/m ³	1.851
Dry Density	Mg/m ³	1.392
Water Content	%	32.92
Degree of Saturation	%	93.6
Height of Solids	mm	10.21
Initial Void Ratio		0.960

Final Specimen Conditions

Water Content	%	29.49
Final Void Ratio		0.712
Final Height	mm	17.47

Figure D4B

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

Specimen Details

Project Name	Hwy 401 Brockville - Ormond Street Overpass
Project Location	Brockville, Ontario
Borehole	BC 21-1
Sample No.	SH-3A
Depth	2.13 - 2.74 m.
Sample Date	May 5, 2021
Test Number	One
Technician Name	Daniel Boateng

Test Procedure

Date Started	June 8, 2021
Date Finished	June 9, 2021
Machine Number	Frame C
Cell Number	C
Ring Number	C
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

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Calculations

Load	Increment	Axial	Corrected	Specimen	Axial	Void
Increment	Duration	Stress	Deformation	Height	Strain	Ratio
	min	σ_a kPa	ΔH mm	H mm	ϵ_a %	e
Seating	0.0	0	0.0000	20.0000	0.00	0.960
1	20.0	5	0.4860	19.5140	2.46	0.911
2	20.0	10	0.5528	19.4472	2.78	0.905
3	20.0	20	0.6485	19.3515	3.26	0.896
4	20.0	40	0.7489	19.2511	3.77	0.886
5	21.5	80	0.8796	19.1204	4.43	0.873
6	24.8	120	0.9698	19.0302	4.89	0.864
7	24.8	160	1.0429	18.9571	5.26	0.857
8	28.3	240	1.1776	18.8224	5.94	0.843
9	34.8	320	1.2903	18.7097	6.57	0.831
10	43.3	480	1.5384	18.4616	7.84	0.806
11	56.5	640	1.7601	18.2399	9.06	0.782
12	83.3	1280	2.5136	17.4864	13.33	0.698
13	91.3	2560	3.4279	16.5721	18.27	0.601
14	96.5	3840	4.0705	15.9295	21.37	0.541
15	28.3	640	3.8786	16.1214	19.47	0.578
16	48.0	160	3.4346	16.5654	17.22	0.622
17	73.0	40	3.0135	16.9865	15.03	0.665
18	109.5	10	2.6413	17.3587	13.18	0.701
19	54.8	5	2.6093	17.3907	12.66	0.712

Figure D4C

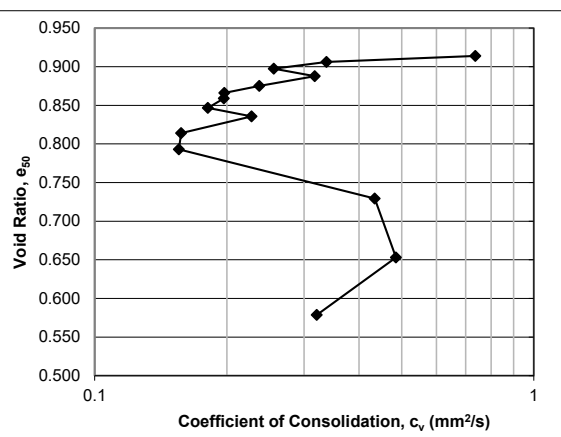
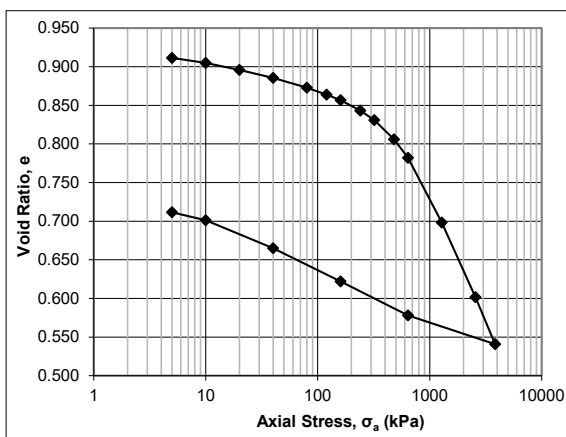
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Job Ref.	Hwy 401 Brockville - Ormond Street Overpass
Job Location	Brockville, Ontario
Borehole	BC 21-1
Sample No.	SH-3A
Depth	2.13 - 2.74 m.
Sample Date	May 5, 2021
Test Number	One
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress σ_a , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	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	0.4653	19.5347	2.33	0.914			110	7.36E-01
2	8	0.5437	19.4563	2.72	0.906			238	3.37E-01
3	15	0.6355	19.3645	3.18	0.897			311	2.56E-01
4	30	0.7326	19.2674	3.66	0.888			248	3.17E-01
5	60	0.8622	19.1378	4.31	0.875			327	2.37E-01
6	100	0.9529	19.0471	4.76	0.866			389	1.98E-01
7	140	1.0253	18.9747	5.13	0.859			388	1.97E-01
8	200	1.1521	18.8479	5.76	0.847			416	1.81E-01
9	280	1.2636	18.7364	6.32	0.836			327	2.28E-01
10	400	1.4866	18.5134	7.43	0.814			462	1.57E-01
11	560	1.7000	18.3000	8.50	0.793			457	1.55E-01
12	960	2.3493	17.6507	11.75	0.729			152	4.34E-01
13	1920	3.1301	16.8699	15.65	0.653			124	4.85E-01
14	3200	3.8877	16.1123	19.44	0.579			172	3.20E-01
15	2240	4.0156	15.9844	20.08	0.566				
16	400	3.6349	16.3651	18.17	0.603				
17	100	3.2230	16.7770	16.11	0.644				
18	25	2.8217	17.1783	14.11	0.683				
19	8	2.6228	17.3772	13.11	0.703				


Figure D4D

Project
Project No.
Borehole No.
Sample No.
Sample Depth

Hwy 401 Brockville - Ormond Street Overpass
165001160(309)
OS 21-1
SH-5
3.05 - 3.66 m.

Figure No. D5

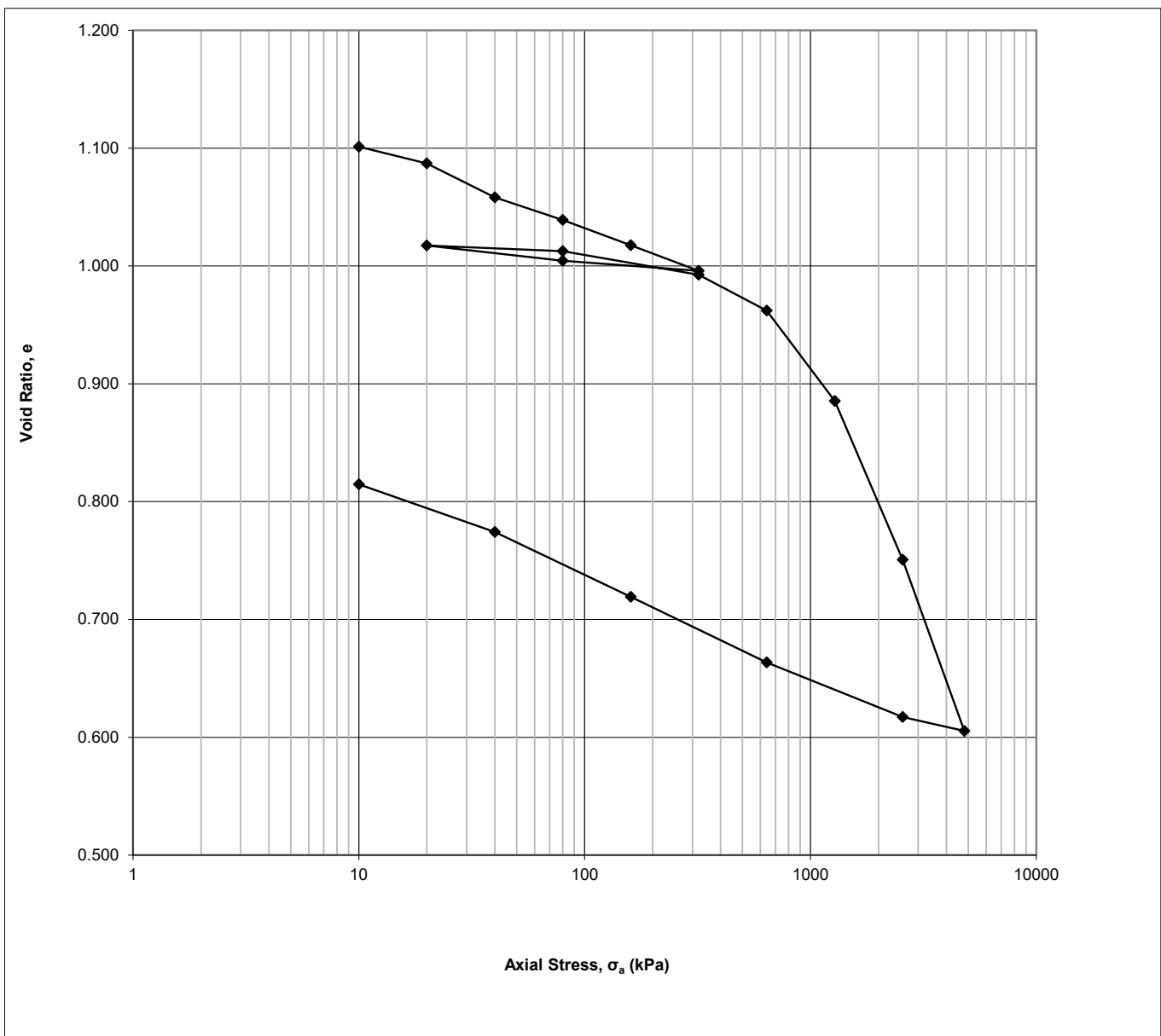


Figure D5A

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

Specimen Details

Project Name	Hwy 401 Brockville - Ormond Street Overpass
Project Location	Brockville, Ontario
Borehole	OS 21-1
Sample No.	SH-5
Depth	3.05 - 3.66 m.
Sample Date	May 7, 2021
Test Number	Four
Technician Name	Daniel Boateng

Soil Description & Classification

<i>Silty clay, very stiff to hard, brown, friable, moist</i>	
Specific Gravity of Solids	2.760
Average water content of trimmings %	39.61
Additional Notes (information source, occurrence and size of large isolated particles etc.)	
<i>Test sample taken from lower half of tube</i>	

Initial Specimen Conditions

Height	mm	20.00
Diameter	mm	50.00
Area	mm ²	1963
Volume	mm ³	39270
Mass	g	71.14
Dry Mass	g	50.96
Density	Mg/m ³	1.812
Dry Density	Mg/m ³	1.298
Water Content	%	39.60
Degree of Saturation	%	97.0
Height of Solids	mm	9.40
Initial Void Ratio		1.127

Final Specimen Conditions

Water Content	%	33.95
Final Void Ratio		0.815
Final Height	mm	17.06

Figure D5B

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

Specimen Details

Project Name	Hwy 401 Brockville - Ormond Street Overpass
Project Location	Brockville, Ontario
Borehole	OS 21-1
Sample No.	SH-5
Depth	3.05 - 3.66 m.
Sample Date	May 7, 2021
Test Number	Four
Technician Name	Daniel Boateng

Test Procedure

Date Started	June 9, 2021
Date Finished	June 10, 2021
Machine Number	Frame C
Cell Number	C
Ring Number	C
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	Axial	Corrected	Specimen	Axial	Void
Increment	Duration	Stress	Deformation	Height	Strain	Ratio
	min	σ_a kPa	ΔH mm	H mm	ϵ_a %	e
Seating	0.0	0	0.0000	20.0000	0.00	1.127
1	20.0	10	0.2416	19.7584	1.21	1.101
2	20.0	20	0.3717	19.6283	1.87	1.087
3	20.0	40	0.6364	19.3636	3.23	1.058
4	20.0	80	0.8151	19.1849	4.13	1.039
5	20.0	160	1.0156	18.9844	5.14	1.018
6	28.3	320	1.2123	18.7877	6.16	0.996
7	20.0	80	1.1503	18.8497	5.76	1.004
8	21.5	20	1.0311	18.9689	5.15	1.017
9	20.0	80	1.0719	18.9281	5.37	1.013
10	21.5	320	1.2562	18.7438	6.32	0.993
11	43.3	640	1.4968	18.5032	7.75	0.962
12	93.3	1280	2.0628	17.9372	11.36	0.885
13	135.3	2560	3.1831	16.8169	17.69	0.751
14	130.8	4800	4.6088	15.3912	24.52	0.605
15	21.8	2560	4.7897	15.2103	23.96	0.617
16	43.5	640	4.3470	15.6530	21.79	0.663
17	82.3	160	3.8352	16.1648	19.17	0.719
18	128.0	40	3.3215	16.6785	16.58	0.774
19	157.0	10	3.2935	16.7065	14.68	0.815

Figure D5C

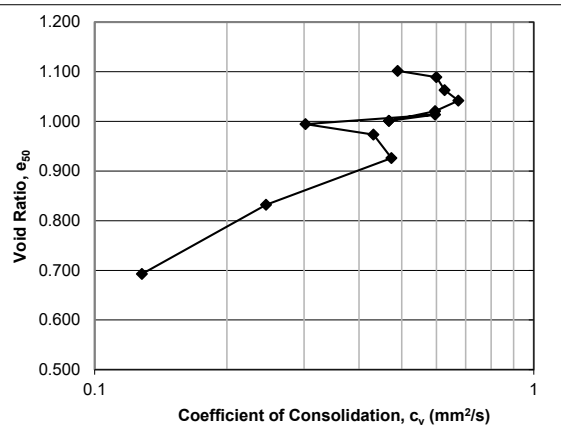
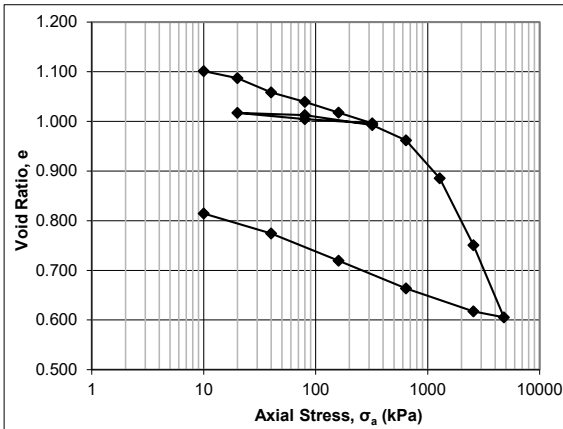
One-Dimensional Consolidation Test using Incremental Loading ASTM D2435/D2435M - 11(2020)

Specimen Details

Job Ref.	Hwy 401 Brockville - Ormond Street Overpass
Job Location	Brockville, Ontario
Borehole	OS 21-1
Sample No.	SH-5
Depth	3.05 - 3.66 m.
Sample Date	May 7, 2021
Test Number	Four
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress σ_a , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	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	0.2357	19.7643	1.18	1.102			169	4.90E-01
2	15	0.3532	19.6468	1.77	1.089			136	6.00E-01
3	30	0.5994	19.4006	3.00	1.063			127	6.27E-01
4	60	0.8017	19.1983	4.01	1.042			116	6.74E-01
5	120	0.9967	19.0033	4.98	1.021			128	5.96E-01
6	240	1.1845	18.8155	5.92	1.001			160	4.68E-01
7	200	1.1661	18.8339	5.83	1.003				
8	50	1.0567	18.9433	5.28	1.014				
9	50	1.0666	18.9334	5.33	1.013			127	5.96E-01
10	200	1.2440	18.7560	6.22	0.995			247	3.02E-01
11	480	1.4419	18.5581	7.21	0.974			169	4.31E-01
12	960	1.8881	18.1119	9.44	0.926			147	4.74E-01
13	1920	2.7718	17.2282	13.86	0.832			256	2.46E-01
14	3680	4.0806	15.9194	20.40	0.693			419	1.28E-01
15	3680	4.8211	15.1789	24.11	0.614				
16	1600	4.5249	15.4751	22.62	0.646				
17	400	4.0890	15.9110	20.44	0.692				
18	100	3.5768	16.4232	17.88	0.746				
19	25	3.3000	16.7000	16.50	0.776				


Figure D5D

Project
Project No.
Borehole No.
Sample No.
Sample Depth

Hwy 401 Brockville - Ormond Street Overpass
165001160(309)
OS 21-2
SH-13
9.91 - 10.52 m.

Figure No. D6

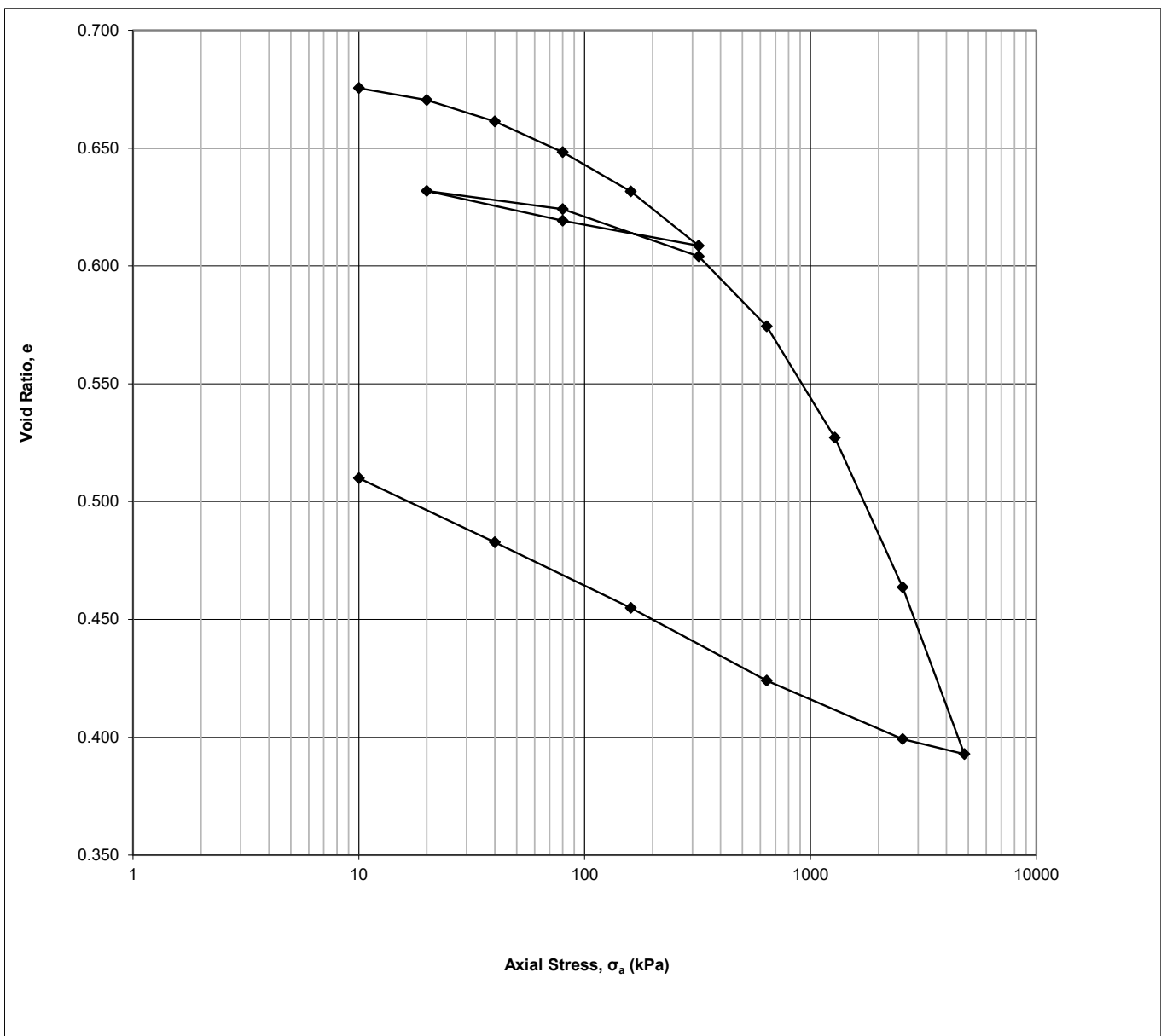


Figure D6A

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

Specimen Details

Project Name	Hwy 401 Brockville - Ormond Street Overpass
Project Location	Brockville, Ontario
Borehole	OS 21-2
Sample No.	SH-13
Depth	9.91 - 10.52 m.
Sample Date	May 11, 2021
Test Number	Five
Technician Name	Daniel Boateng

Soil Description & Classification

<i>Silty clay, very stiff to hard, brown, friable, moist</i>	
Specific Gravity of Solids	2.760
Average water content of trimmings %	22.87
Additional Notes (information source, occurrence 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	78.44
Dry Mass	g	63.84
Density	Mg/m ³	1.997
Dry Density	Mg/m ³	1.626
Water Content	%	22.87
Degree of Saturation	%	90.5
Height of Solids	mm	11.78
Initial Void Ratio		0.698

Final Specimen Conditions

Water Content	%	21.22
Final Void Ratio		0.510
Final Height	mm	17.79

Figure D6B

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

Specimen Details

Project Name	Hwy 401 Brockville - Ormond Street Overpass
Project Location	Brockville, Ontario
Borehole	OS 21-2
Sample No.	SH-13
Depth	9.91 - 10.52 m.
Sample Date	May 11, 2021
Test Number	Five
Technician Name	Daniel Boateng

Test Procedure

Date Started	June 9, 2021
Date Finished	June 10, 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	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

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Calculations

Load	Increment	Axial	Corrected	Specimen	Axial	Void
Increment	Duration	Stress	Deformation	Height	Strain	Ratio
	min	σ_a kPa	ΔH mm	H mm	ϵ_a %	e
Seating	0.0	0	0.0000	20.0000	0.00	0.698
1	20.0	10	0.2641	19.7359	1.31	0.675
2	20.0	20	0.3135	19.6865	1.61	0.670
3	23.0	40	0.4166	19.5834	2.15	0.661
4	24.8	80	0.5655	19.4345	2.91	0.648
5	26.5	160	0.7600	19.2400	3.89	0.632
6	33.0	320	1.0226	18.9774	5.26	0.609
7	20.0	80	0.9256	19.0744	4.63	0.619
8	24.8	20	0.7771	19.2229	3.89	0.632
9	20.0	80	0.8656	19.1344	4.34	0.624
10	24.8	320	1.0925	18.9075	5.52	0.604
11	39.8	640	1.4113	18.5887	7.27	0.574
12	54.8	1280	1.9277	18.0723	10.05	0.527
13	68.3	2560	2.6245	17.3755	13.79	0.464
14	70.0	4800	3.4502	16.5498	17.96	0.393
15	20.0	2560	3.5171	16.4829	17.59	0.399
16	25.0	640	3.2203	16.7797	16.12	0.424
17	43.3	160	2.8571	17.1429	14.31	0.455
18	61.8	40	2.5406	17.4594	12.66	0.483
19	102.0	10	2.2076	17.7924	11.06	0.510

Figure D6C

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

Specimen Details

Job Ref.	Hwy 401 Brockville - Ormond Street Overpass
Job Location	Brockville, Ontario
Borehole	OS 21-2
Sample No.	SH-13
Depth	9.91 - 10.52 m.
Sample Date	May 11, 2021
Test Number	Five
Technician Name	Daniel Boateng

Calculations

Load Increment	Axial Stress σ_a , average kPa	Calculated using Interpretation Procedure 2				Interpretation Procedure 1		Interpretation Procedure 2	
		Corrected Deformation ΔH_{50} mm	Specimen Height H_{50} mm	Axial Strain $\epsilon_{a,50}$ %	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	0.2507	19.7493	1.25	0.676			250	3.31E-01
2	15	0.3031	19.6969	1.52	0.672			121	6.81E-01
3	30	0.3983	19.6017	1.99	0.664			138	5.90E-01
4	60	0.5426	19.4574	2.71	0.652			121	6.62E-01
5	120	0.7343	19.2657	3.67	0.635			171	4.59E-01
6	240	0.9892	19.0108	4.95	0.614			202	3.79E-01
7	200	0.9435	19.0565	4.72	0.618				
8	50	0.8176	19.1824	4.09	0.628				
9	50	0.8567	19.1433	4.28	0.625			253	3.08E-01
10	200	1.0760	18.9240	5.38	0.606			270	2.81E-01
11	480	1.3614	18.6386	6.81	0.582			190	3.87E-01
12	960	1.8472	18.1528	9.24	0.541			185	3.77E-01
13	1920	2.4695	17.5305	12.35	0.488			109	5.96E-01
14	3680	3.2457	16.7543	16.23	0.422			138	4.32E-01
15	3680	3.5246	16.4754	17.62	0.399				
16	1600	3.2832	16.7168	16.42	0.419				
17	400	2.9863	17.0137	14.93	0.444				
18	100	2.6882	17.3118	13.44	0.470				
19	25	2.3677	17.6323	11.84	0.497				

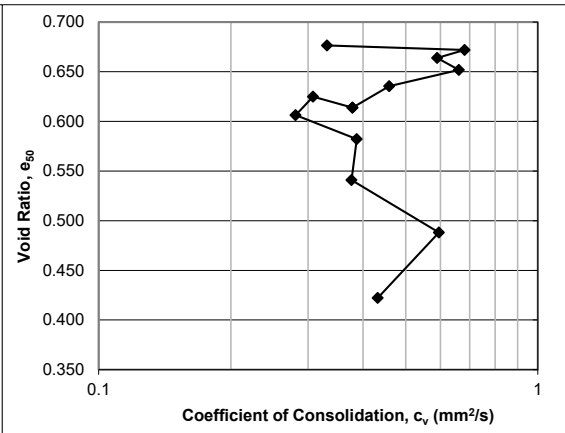
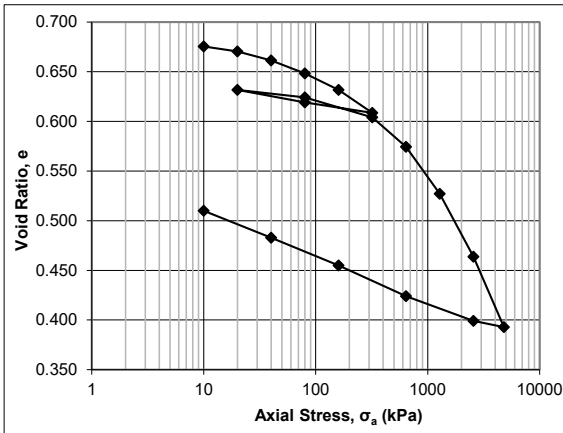
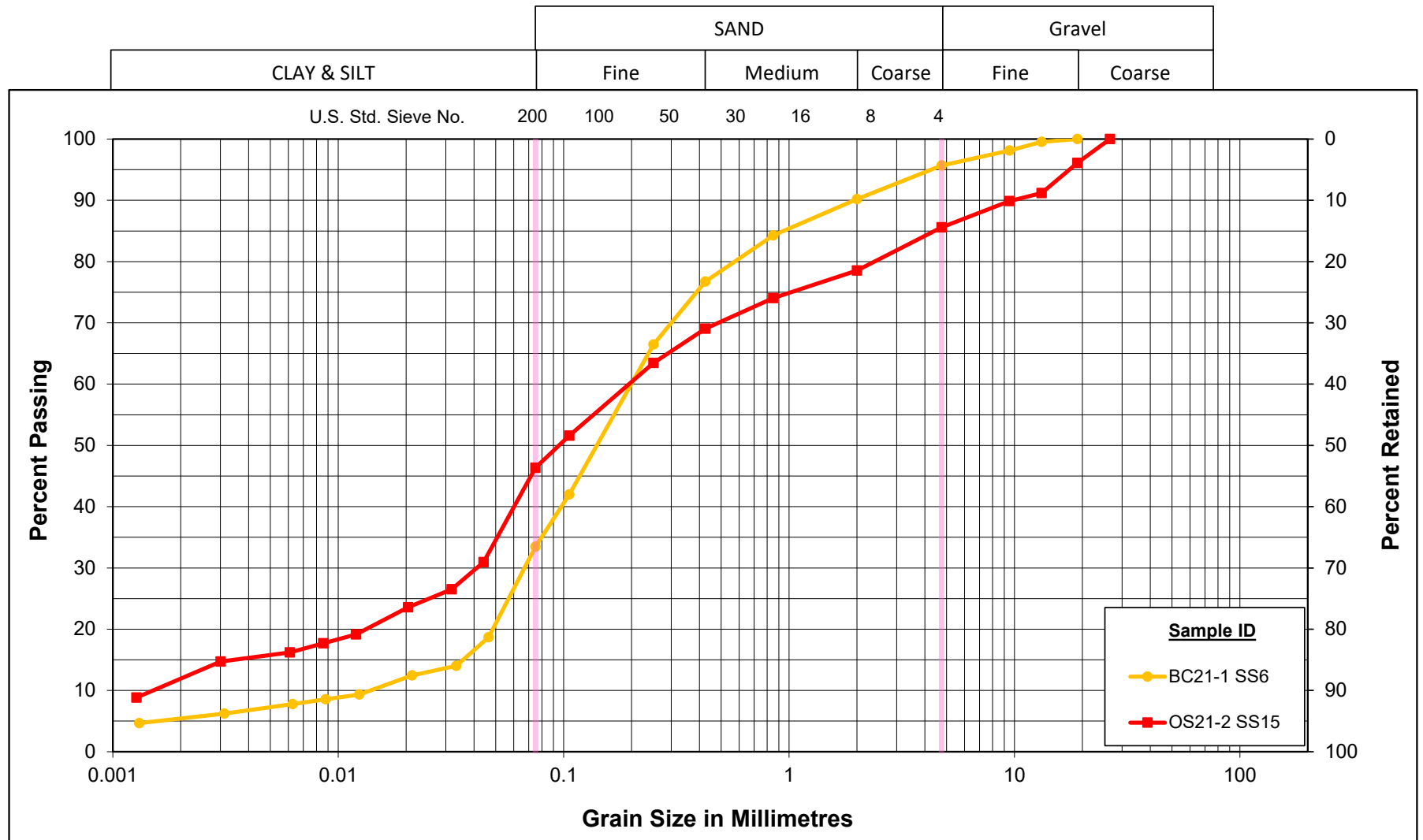


Figure D6D

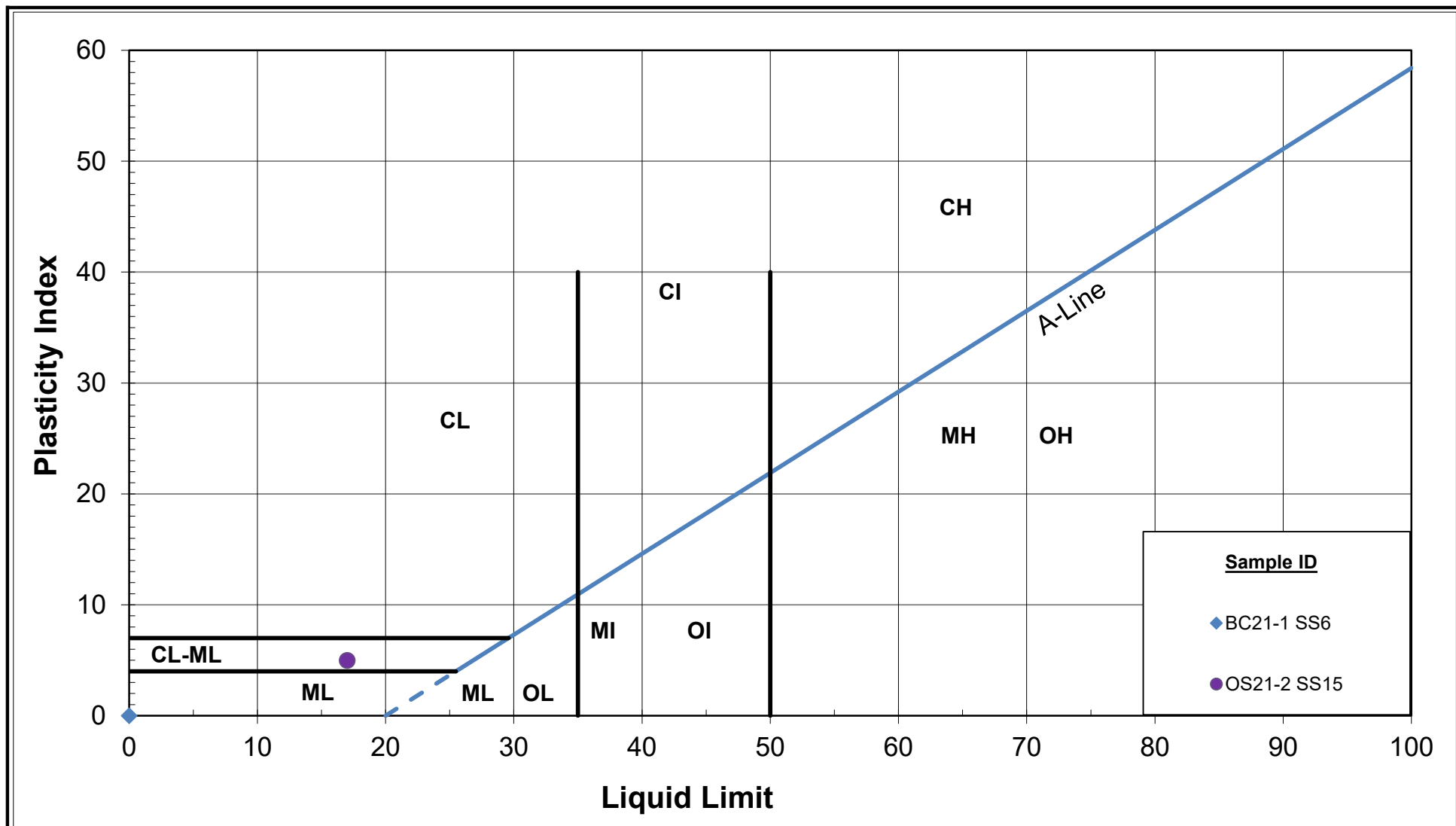
Unified Soil Classification System



GRAIN SIZE DISTRIBUTION
 SILTY SAND to SILT and SAND (TILL) (SM to CL-ML)
 Hwy 401 Brockville - Ormond Street Overpass

Figure No. D7

Project No. 165001160



SILTY SAND (SM) tp SILT and SAND (CL-ML) (TILL)
 Hwy 401 Brockville - Ormond Street Overpass
PLASTICITY CHART

Figure No. D8

Project No. 165001160

Certificate of Analysis

Report Date: 16-Jun-2021

Client: Stantec Consulting Ltd. (Ottawa)

Order Date: 11-Jun-2021

Client PO: Hwy 401 Brockville EA

Project Description: 165001160.309

Client ID:	CP21-1, SS14.10.668-11.278m	BC21-1,SS3.1.524-2 .134m	BC21-2,SS3.1.524-2. 134m	NA21-1,SS3.1.524- 2.134m
Sample Date:	10-May-21 09:00	05-May-21 09:00	05-May-21 09:00	03-May-21 09:00
Sample ID:	2124634-01	2124634-02	2124634-03	2124634-04
MDL/Units	Soil	Soil	Soil	Soil

Physical Characteristics

% Solids	0.1 % by Wt.	90.5	78.7	77.4	82.1
----------	--------------	------	------	------	------

General Inorganics

pH	0.05 pH Units	7.90 [1]	7.48 [1]	7.81 [1]	7.45 [1]
Resistivity	0.10 Ohm.m	32.8	16.3	15.1	39.8

Anions

Chloride	5 ug/g dry	36 [1]	244 [1]	264 [1]	27 [1]
Sulphate	5 ug/g dry	177 [1]	72 [1]	49 [1]	26 [1]

Client ID:	NA21-2,SS15.12.192- 12.802m	OS21-1,SS2.0.254- 0.609m	OS21-2, SS3B.1.829-2.134m	SB21-2,SS3.1.524- 2.134m
Sample Date:	06-May-21 09:00	07-May-21 09:00	11-May-21 09:00	04-May-21 09:00
Sample ID:	2124634-05	2124634-06	2124634-07	2124634-08
MDL/Units	Soil	Soil	Soil	Soil

Physical Characteristics

% Solids	0.1 % by Wt.	87.0	80.0	99.5	99.5
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General Inorganics

pH	0.05 pH Units	7.94 [1]	7.62 [1]	7.91 [1]	7.58 [1]
Resistivity	0.10 Ohm.m	12.6	44.3	30.2	80.0

Anions

Chloride	5 ug/g dry	388 [1]	22 [1]	118 [1]	13 [1]
Sulphate	5 ug/g dry	86 [1]	8 [1]	16 [1]	6 [1]

Client ID:	SB21-2,SS5.3.048-3.3 53m	CP21-2,SS5.3.048- 3.658m	-	-
Sample Date:	04-May-21 09:00	12-May-21 09:00	-	-
Sample ID:	2124634-09	2124634-10	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	100	99.2	-	-
----------	--------------	-----	------	---	---

General Inorganics

pH	0.05 pH Units	7.89 [1]	7.81 [1]	-	-
Resistivity	0.10 Ohm.m	102	16.2	-	-

Anions

Chloride	5 ug/g dry	11 [1]	212 [1]	-	-
Sulphate	5 ug/g dry	10 [1]	51 [1]	-	-

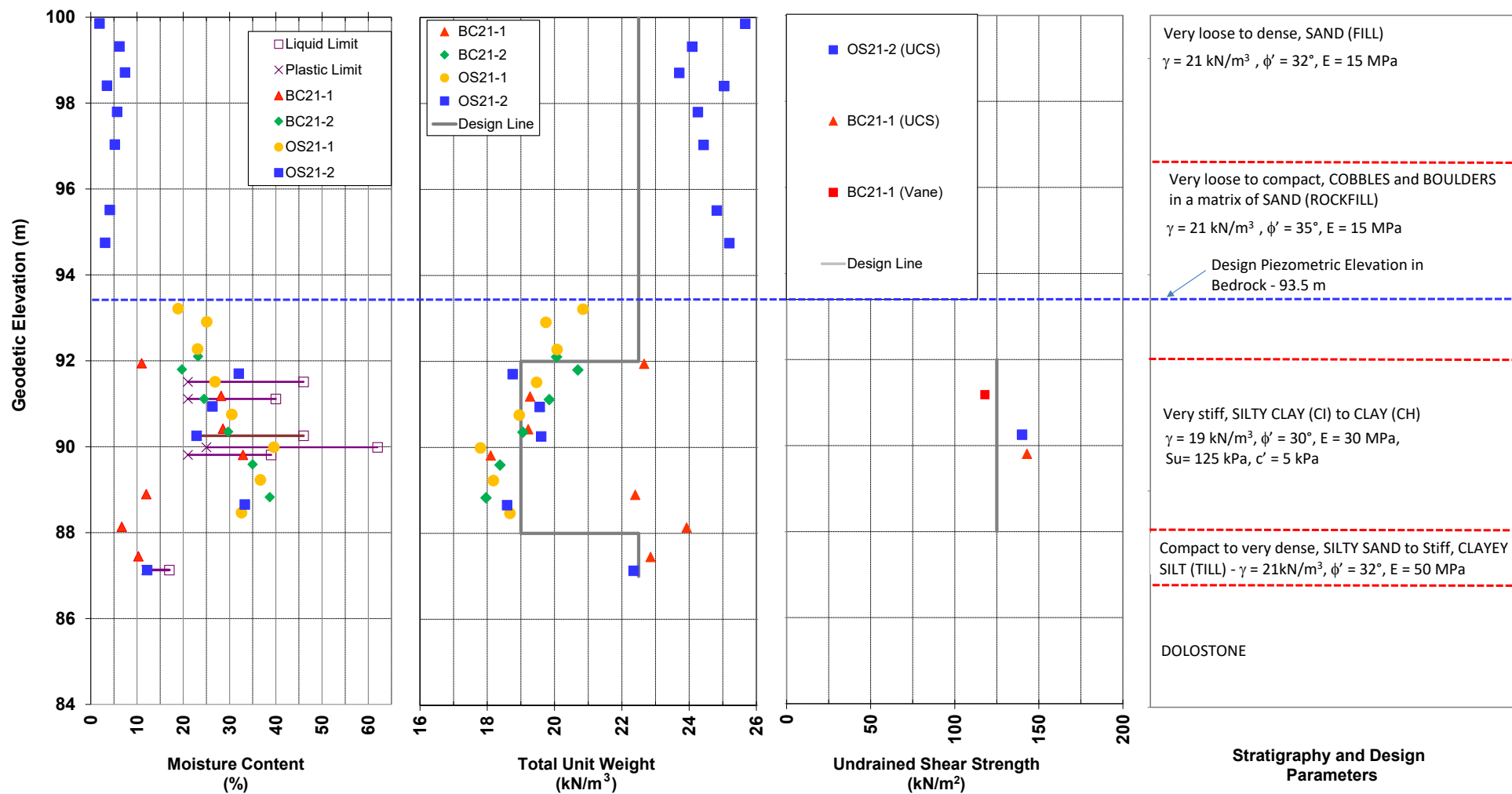
February 2023

APPENDIX E

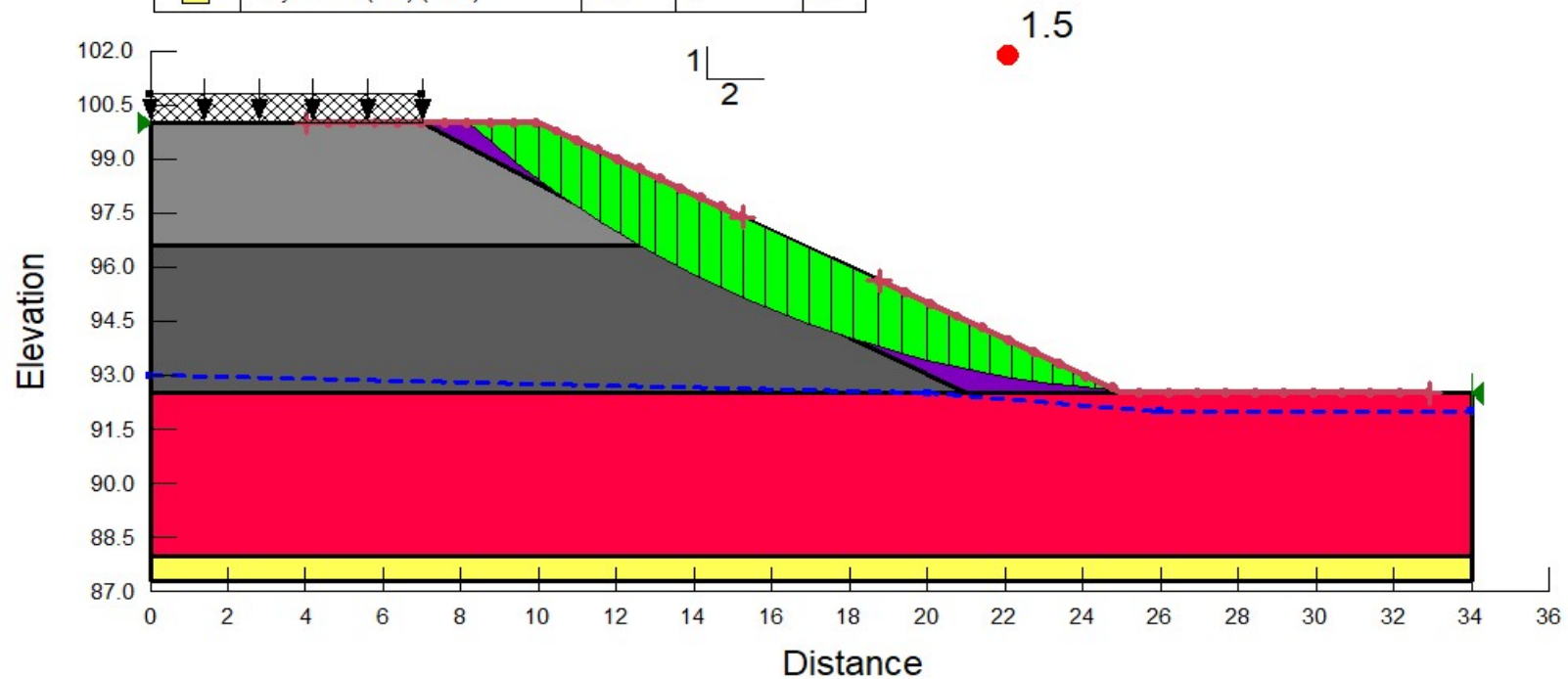
E.1 DRAWING E1 - GEOTECHNICAL SOIL MODEL

E.2 DRAWINGS E2 TO E4 – SLOPE STABILITY ANALYSIS RESULTS





Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
█	Existing ROCKFILL	21	0	35
█	Existing Sand FILL	21	0	32
█	Select Subgrade Material (SSM)	20	0	32
█	SILTY CLAY (CI) to CLAY (CH)	19	5	30
█	Silty SAND (SM) (TILL)	22.5	0	32








Slope Stability Analysis (Static)

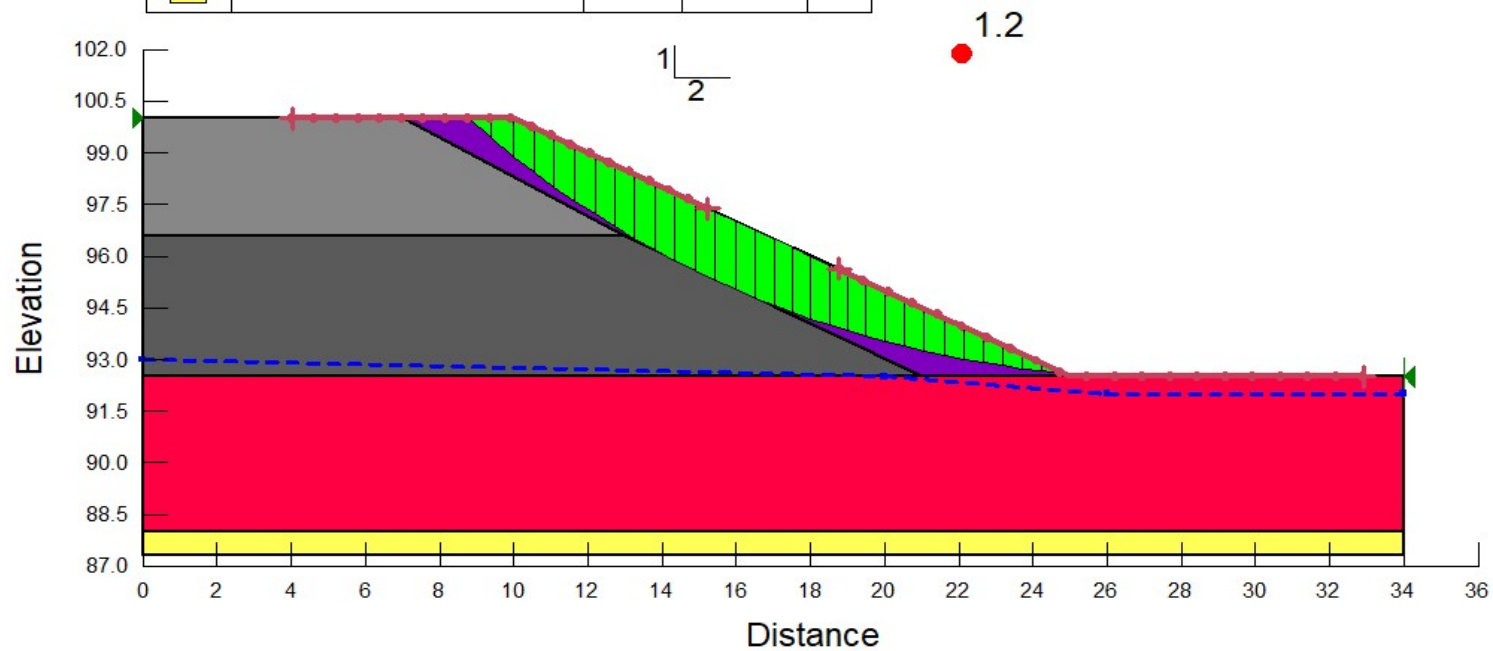
Highway 401 Brockville, Ormond Street Overpass

Figure E2

Project No. 165001160

GWP No. 4003-19-00

Color	Name	Unit Weight (kN/m ³)	Cohesion' (kPa)	Phi' (°)
	Existing ROCKFILL	21	0	35
	Existing Sand FILL	21	0	32
	Select Subgrade Material (SSM)	20	0	32
	SILTY CLAY (CI) to CLAY (CH)	19	125	0
	Silty SAND (SM) (TILL)	22.5	0	32








Slope Stability Analysis (Seismic)

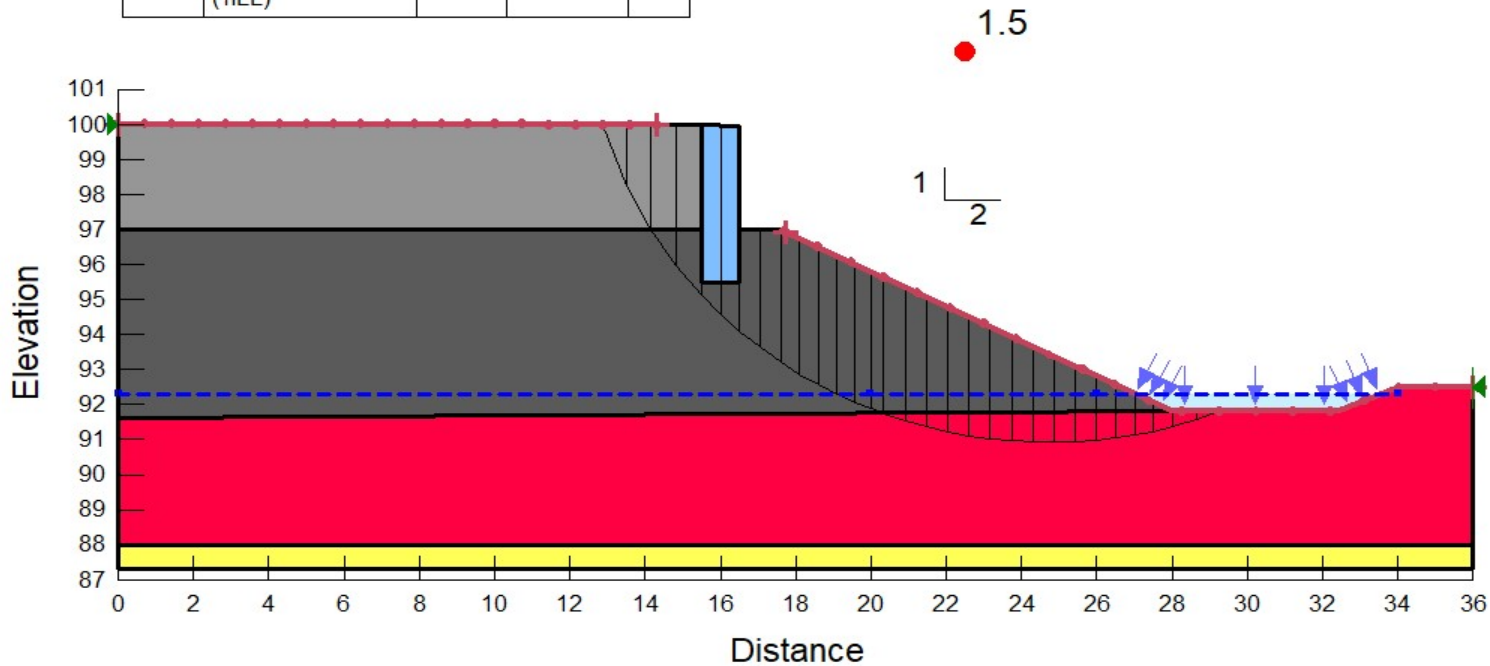
Highway 401 Brockville, Ormond Street Overpass

Figure E3

Project No. 165001160

GWP No. 4003-19-00

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
	Concrete	26	1,000	0
	Existing ROCKFILL	21	0	35
	Existing Sand FILL	21	0	32
	SILTY CLAY (CI) to CLAY (CH)	19	5	30
	Silty SAND (SM) (TILL)	22.5	0	32



Slope Stability Analysis (Static with Open Channel)

Highway 401 Brockville, Ormond Street Overpass

Figure E4

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: 44.607N 75.691W

User File Reference: Highway 401, Ormond Street Overpass

2021-10-08 19:13 UT

Requested by: Stantec Consulting Ltd.

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.238	0.146	0.094	0.031
Sa (0.1)	0.295	0.188	0.125	0.044
Sa (0.2)	0.263	0.169	0.113	0.042
Sa (0.3)	0.210	0.135	0.091	0.035
Sa (0.5)	0.158	0.102	0.068	0.026
Sa (1.0)	0.086	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)	0.166	0.105	0.069	0.024
PGV (m/s)	0.133	0.081	0.052	0.017

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

References

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

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

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

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



Natural Resources
Canada

Ressources naturelles
Canada

Canada