



**Preliminary Foundation
Investigation and Design Report
Highway 401 CPR (VIA) Overhead
Structure Replacement
Site No. 16X-0122/B0**

Highway 401 Rehabilitation
Brockville, ON

G.W.P. 4003-19-00

Latitude 44.605552
Longitude -75.693012

Geocres No. 31B-108

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Ministry of Transportation Ontario

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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 CPR (VIA) Overhead Structure Replacement (Site No. 16X-0122/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 and one (1) culvert site, where replacement or rehabilitation of the existing structures are planned. This report presents the results of a preliminary foundation investigation related to the replacement of the CPR (VIA) overhead structure at Site No. 16X-0122/B0. Separate Preliminary Foundation Investigation and Design Reports have been prepared for the other sites included in this assignment.

The purpose of the preliminary foundation investigation was to supplement information on the subsurface conditions at the location of the proposed bridge reconstruction by drilling two (2) boreholes and 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 (FIDR) has been prepared specifically and solely for the proposed replacement of the CPR (VIA) overhead structure at Highway 401. This Preliminary Report is not to be used for the detailed 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

The rail corridor crosses beneath Highway 401 near Station 22+309, in the City of Brockville, Ontario. The site location is shown on Key Plan inset on the Borehole Locations and Soil Strata Plan, Drawing No. 1 in Appendix A.

2.2 SITE DESCRIPTION

At the CPR (VIA) overhead site, Highway 401 is a four-lane divided freeway with two lanes in each direction that is aligned in an approximate southwest-northeast orientation. For the purposes of this report, Highway 401 will be referenced as being orientated east to west.

The overpass structural drawings indicate that the Highway 401 pavement crown is at approximately Elevation 103.8 m, and the rail tracks are at an approximate elevation of 95.3 m. The bridge approach fills are approximately 8.5 m high, have 2 horizontal to 1 vertical (2H:1V) side-slopes, and are about 29 m wide near the bridge.

Beyond the approach fills, the ground surface is relatively flat and generally slopes towards the south. Immediately adjacent to the bridge and approach fill, the ground is grass covered and contains mature trees in some areas. The lands to the northeast and northwest of the site contain industrial/commercial developments, while residential subdivisions are present to the southeast and southwest.

The CPR overhead structure is a single-span, rigid frame slab bridge with a span length of approximately 17.3 m that was constructed in the late 1950's (W.P. 6-56). The centerline of the structure is on a 28°25' skew to the railway. Wing walls are present on both sides of each bridge abutment. A photograph of the bridge looking towards the north is provided below.



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The bridge design drawings indicate that the bridge abutments are supported on 2.4 m wide strip footings founded at approximate Elevation 91.1 m on bedded limestone bedrock. The foundations supporting the wingwalls are founded at approximately 93.2 m, and are irregularly shaped with widths ranging from 1.9 m to 3.7 m.

2.3 EXISTING CONDITIONS

The following items were noted during a site visit carried out by Stantec's staff:

- No visible signs of settlement or deformation of the existing structure was noted.
- The structure exhibits full-length longitudinal cracks in the soffit corresponding to locations of construction joints at the abutments. The cracks have been sprayed with what appears to be temporary sealing material; however, the cracks are visible and are stained, with some locations showing rust stains. Small delaminations were noted on the south fascia.
- The asphalt wearing surface above the structure is in good condition and exhibits transverse cracks above the abutments.
- No signs of embankment settlement or instability were observed.

2.4 SITE DRAINAGE

Regionally, surface drainage is from north to south towards the Saint Lawrence River. Locally, drainage is towards Buells Creek which crosses Highway 401 approximately 180 m east of the overhead structure, then flows west towards the rail alignment, then south along the rail alignment. Where it flows through the south-east quadrant defined by Highway 401 and the rail alignment, Buells Creek is within 50 m of the overhead structure.



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Previous Investigations / Available Information
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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 suggests that the CPR overhead structure is located within a massive to well laminated fine-textured glaciomarine deposit comprised of silt and clay with minor sand and gravel. Zones of stone-poor sandy silt to silty sand-textured till on paleozoic terrain are present to the west and north of the site location.

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.

Review of available water well records for wells located in proximity to the overpass site indicates that bedrock was encountered at depths of approximately 1 m to 4 m below ground surface.

3.0 PREVIOUS INVESTIGATIONS / AVAILABLE INFORMATION

Subsurface information for this site was obtained from MTO Foundation Library Geocres document No. 31B00-014 titled:

- “A report on the Foundation Investigation at St. 250+00, Line “B”, Hwy # 401 for the Proposed Overpass with C.P.R near Brockville”, Project 55-F-7, prepared by Materials Laboratory – Department of Highways Ontario and dated October 13, 1955.

The report included a plan showing the location of borings and sections of sub-strata at the overpass structure location. The report indicates that a total of twenty-one (21) investigation holes, including two (2) bore/coreholes and nineteen (19) auger holes, were advanced at or near the bridge site location.

Of relevance, Drawing Number 55-F-7A from the above-noted report includes two stratigraphic cross-sections, Sections “A-A” and “B-B”, which display the subsurface conditions along the west and east abutment of the bridge. The report also includes a Record of Borehole for Boring No. 2; however, this borehole record is mostly illegible and contains information that appears to contrast with the subsurface strata contained on Section “B-B”. The summary of subsurface information provided below is based on the stratigraphic cross-sections which are inferred to provide relevant information for the site as they were also reproduced on the original structural drawings for the bridge. A copy of Drawing Number 55-F-7A is included in Appendix B for reference.

Sections “A-A” and “B-B” indicate that the subsurface conditions encountered during the 1955 investigation consisted of a surficial layer of topsoil underlain by a deposit of silty sand that extended to a maximum depth of approximately 2 m below original ground surface. The silty sand was underlain by a sandy clay till deposit that extended to a maximum depth of approximately 4 m and which was, in turn, underlain by bedded limestone bedrock. The limestone bedrock was encountered at elevations ranging from approximately 92.2 m to 91.0 m.

The report text states that “silty sand becoming sandy clay till at about 6 ft. overlies bedded limestone at depths ranging from 9 to 13 ft.” Free groundwater surfaces were identified at an elevation of 305 ft (~93 m) in the drill holes.



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4.0 SUBSURFACE CONDITIONS

4.1 FIELD INVESTIGATION

The current investigation consisted of advancing two boreholes, identified as Boreholes CP21-1 and CP21-2. Boreholes CP21-1 was located within the outside paved shoulder of the westbound lanes, west of the rail alignment. Borehole CP21-2 was located within the outside paved shoulder of the eastbound lanes, east of the rail alignment. The borehole locations are shown on the Borehole Locations and Soil Strata Plan, Drawing No. 1, in Appendix A.

Prior to carrying out the investigation, Stantec contacted Ontario One Call and other public utility authorities to clear the borehole locations of public utilities.

The boreholes were advanced between May 10th and 12th, 2021, using a truck-mounted drill rig equipped for soil and bedrock sampling. Hollow-stem continuous flight augers were used to advance through soils, which included rockfill, in Borehole CP21-1. Within Borehole CP21-2, the drilling method was switched at a depth of 1.5 m from hollow-stem augering to advancing drill casing due to difficulties in penetrating the rockfill. Rockfill material is inferred to be present in the highway embankment at both boreholes. NQ size coring equipment was used to advance the boreholes after auger or casing refusal was encountered at depths of approximately 11.8 m and 12.3 m in Boreholes CP21-1 and CP21-2, respectively.

The subsurface stratigraphy encountered in each borehole was recorded in the field by Stantec's geotechnical staff. Standard Penetration Tests (SPTs) were carried out in the overburden and split spoon samples were collected at regular intervals. Minimal to no sample recovery was obtained in the split spoon samplers at various depths within the highway embankment. Material retrieved while advancing the drill casing in Borehole CP21-2 consisted predominantly of gravel, cobble, and boulder-sized particles.

Bedrock was cored in both boreholes using NQ size equipment. The cores were placed in labelled core boxes. All recovered soil samples and rock cores were delivered to our Ottawa laboratory for detailed classification and testing.

A monitoring well, screened from 13.6 m to 15.1 m below ground surface, was installed in Borehole CP21-1. The water level was measured on May 10th and May 11th, 2021; the well was subsequently decommissioned by backfilling the borehole with grout and bentonite.

4.2 LOCATION AND ELEVATION SURVEY

The borehole locations and ground surface elevations 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 summarizes the borehole locations, 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			
CP21-1	4940929	368840.5	104.0	15.1	88.9
CP21-2	4940927.7	368872.6	104.0	18.4	85.6

4.3 Laboratory Testing

All samples were transported to Stantec's Ottawa laboratory where they were examined by a geotechnical engineer. The geotechnical laboratory testing carried out on selected soil and rock samples is summarized in Table 4.2.

Table 4.2: Geotechnical Laboratory Testing Program

Test Description	Number of Tests
Moisture Content	22
Atterberg Limits	3
Grain Size Distribution (sieve & hydrometer)	6
Unconfined Compressive Strength (on rockfill samples)	2
Unconfined Compressive Strength (on bedrock cores)	4

Two soil samples, one from each borehole, were also tested for pH, soluble sulphate content, chloride content, and resistivity 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 subsurface soil, bedrock 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 sections of the subsurface conditions encountered in the boreholes are provided on Drawing Nos. 1 and 2 in Appendix A. Subsurface information contained on the original structural drawings for the bridge was used to supplement the data from the current investigation.

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



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In general, the subsurface stratigraphy encountered in the boreholes consisted of a surficial layer of asphalt underlain by predominantly granular embankment fill materials ranging in composition from sand/silty sand/gravelly silty sand to sand and gravel/sandy gravel to rockfill comprised of gravel, cobbles, and boulders in a matrix of silty sand. The highway embankment fill materials were underlain by a deposit of compact silty sand to sand and gravel in Borehole CP21-2. Glacial till deposits varying from firm to very stiff sandy clayey silt till in Borehole CP21-1, to a loose to very dense sandy silt till in Borehole CP21-2, were encountered beneath the fill and silty sand to sand and gravel deposit. The overburden materials were underlain by dolostone bedrock. The boreholes were terminated within bedrock at the depths of 15.1 m to 18.4 m.

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

5.2 OVERBURDEN

5.2.1 Asphalt

Boreholes CP21-1 and CP21-2 were drilled through the existing asphalt in the western and eastern shoulder lanes of Highway 401, respectively. The asphalt thickness at both borehole locations was approximately 500 mm.

5.2.2 Fill

Predominantly granular fill materials were encountered beneath the asphalt in both boreholes. The fill material is associated with the Highway 401 approach embankments to the CPR overhead structure and generally varies in composition from sand/silty sand/gravelly silty sand to sand and gravel to rockfill comprised of gravel, cobbles and boulders in a matrix of silty sand. The fill materials extended to depths of about 9.1 m and 8.7 m, corresponding to base elevations of about 94.9 m and 95.3 m, in Boreholes CP21-1 and CP21-2, respectively.

Grinding of the augers was noted as they were advanced within the granular fill materials. Additionally, minimal to no sample recovery occurred at various locations. The split spoon sampler was observed to drop slightly at the location of Sample No. 12 in Borehole CP21-1 suggesting the presence of a void within the fill materials at that location.

An approximately 0.5 m thick layer comprised of a mixture of clayey silt and sand fill material was noted in Borehole CP21-1 at a depth of about 1 m below ground surface.

Casing was advanced in Borehole CP21-2 below a depth of about 1.5 m after encountering difficult augering conditions due to the inferred rockfill materials. A photograph of the pieces of rock retrieved during casing advancement from depths of between about 1.5 m and 9.1 m in Borehole CP21-2 is provided below.



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Standard Penetration Test (SPT) N-values ranging from 4 to greater than 100 blows per 0.3 m of penetration were measured within the fill material. Disregarding SPT refusals (i.e., resistance values of greater than 100 blows), which are inferred to have been influenced by the existing gravel, cobbles, and boulders in the fill materials, the remaining SPT N-values recorded within the fill range from 4 to 27 which indicates that the fill materials are typically in a loose to compact condition.

Laboratory testing of samples of the fill materials yielded moisture contents varying between approximately 0% to 13%, expressed as a percentage of the dry weight of the soil.

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

Atterberg Limits tests were also carried out on one of the samples referenced above. The test yielded a Plastic Limit of 13%, a Liquid Limit of 14% and corresponding Plasticity Index of 1%. The results of the tests are illustrated on the borehole records in Appendix C and on Figure No. D2 in Appendix D.

Two (2) samples of the rockfill materials obtained from Borehole CP21-2 were selected for testing to determine their Unconfined Compressive Strengths (UCS). The results of the tests are summarized in Table 5.1 below.

Table 5.1: Results of Unconfined Compressive Strength (UCS) on Samples of Rockfill

Borehole No.	Sample Depth (m below ground)	Sample Elevation (m)	Unit Weight (g/cm ³)	UCS (MPa)
CP21-2	1.8	102.2	2.74	147.5
	6.7	97.3	2.67	116.3

The UCS test results indicate that the pieces of the rockfill tested are very strong (R5), consistent with the strength classification in the Symbols and Terms used on Borehole and Test Pit Records in Appendix C (ISRM, 2007).



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

5.2.3 Silty Sand to Sand and Gravel

A native deposit consisting of silty sand to sand and gravel containing cobbles and trace organic matter was encountered beneath the fill materials in Borehole CP21-2. This deposit was approximately 1.5 m thick and extended to a depth of about 10.2 m below ground surface, corresponding to a base elevation of about 93.8 m.

An SPT 'N' value of 21 blows per 0.3 m of penetration was measured in the granular deposit which indicates that the silty sand to sand and gravel materials are in a compact condition.

Laboratory testing carried out on a sample of the granular deposit yielded a moisture content of approximately 14%.

Gradation analysis was carried out on a sample from Borehole CP21-2. The test results are illustrated on the borehole record in Appendix C and on the gradation curve on Figure No. D3 in Appendix D. Based on the laboratory results, the USCS group symbol for the sample tested is GP.

5.2.4 Glacial Till

5.2.4.1 Sandy Clayey Silt (TILL)

A cohesive glacial till deposit comprised of sandy clayey silt containing trace to some gravel was encountered beneath the fill materials in Borehole CP21-1. Topsoil was noted near the top of the initial sample taken from this stratum. Although not encountered within the clayey silt till, 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.

The cohesive till deposit was approximately 2.7 m thick and extended to a depth of about 11.8 m, corresponding to a base elevation of about 92.2 m.

SPT 'N' values of 10 and 29 blows per 0.3 m of penetration were measured within the cohesive till deposit. Based on the field testing and examination of the samples obtained, the cohesive till deposit is considered to have a firm to very stiff consistency.

Laboratory testing of the samples of the cohesive till yielded moisture contents of approximately 9% and 19%.

Gradation analysis was carried out on a single sample of the cohesive till obtained from Borehole CP21-1. The test results are illustrated on the borehole record in Appendix C and on the gradation curve presented on Figure No. D4 in Appendix D.

An Atterberg Limits test was also carried out on the sample referenced above. The test yielded a Plastic Limit of 18%, a Liquid Limit of 29%, and corresponding Plasticity Index of 11%. The results of the tests are illustrated on the borehole record in Appendix C and on Figure No. D5 in Appendix D.

Based on these results, the cohesive till is classified as sandy clayey silt of low plasticity with a USCS group symbol of CL.



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5.2.4.2 Sandy Silt (TILL)

A predominantly granular glacial till deposit consisting of sandy silt containing various amounts of gravel, cobbles and boulders was encountered underlying the silty sand to sand and gravel layer described in a preceding section in Borehole CP21-2. This deposit was approximately 3.2 m thick and extended to the depth of about 13.4 m, corresponding to a base elevation of about 90.6 m.

Two SPT 'N' values of 6 and greater that 100 blows per 0.3 m of penetration were measured in the granular till deposit. The higher penetration resistance is inferred to have been influenced by gravel or cobbles and/or boulders in the till deposit. Based on the field testing, the granular till deposit is considered to be loose to very dense.

Auger refusal was encountered in the granular till deposit at a depth of about 12.3 m below which NQ coring was carried out to further extend the borehole.

Laboratory testing of the samples of the granular till deposit yielded moisture contents of approximately 11% and 14%.

Gradation analysis was carried out on one sample of the granular till deposit obtained from Borehole CP21-2. The test results are illustrated on the borehole record in Appendix C and on the gradation curve on Figure No. D6 in Appendix D.

Atterberg Limits tests were also carried out on the sample referenced above. The tests yielded a Plastic Limit of 14%, a Liquid Limit of 15% and corresponding Plasticity Index of 1%. The results of the tests are illustrated on the borehole record in Appendix C and on Figure No. D7 in Appendix D.

Based on the laboratory results, the USCS group symbol for this deposit will be Sandy Silt (ML).

5.3 BEDROCK

Bedrock was encountered underlying the overburden described in the preceding sections in both boreholes. The depth to bedrock is summarized in Table 5.2 below.

Table 5.2: Bedrock Surface Depth/Elevation

Borehole	Depth (m)	Elevation (m)
CP21-1	11.8	92.2
CP21-2	13.4	90.6

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

Parameter	Borehole Number							
	CP21-1			CP21-2				
Sample No.	15	16	17 ¹	15 ²	16	17	18	19
Rock Description	Slightly weathered to fresh, light grey to grey Dolostone			Slightly weathered to fresh, light to dark grey Dolostone				
Depth (m below ground)	11.8 - 12.0	12.0 - 13.5	13.5 - 15.1	13.4 - 14.8	14.8 - 15.5	15.5 - 16.3	16.3 - 17.8	17.8 - 18.4
Geodetic Elevation (m)	92.2 - 92.0	92.0 - 90.5	90.5 - 88.9	90.6 - 89.2	89.2 - 88.5	88.5 - 87.7	87.7 - 86.2	86.2 - 85.6
Total Core Recovery, TCR (%)	100	100	100	40	43	96	100	100
Solid Core Recovery, SCR (%)	0	97	98	27	25	96	100	100
Rock Quality Designation, RQD (%)	0	65	60	27	0	65	88	84
Weathering Degree	W2	W2/W1	W2/W1	W1	W2/W1	W2/W1	W2/W1	W2/W1
Fracture Index	10	7	16	0	13	5	3	3

Notes:

¹ A 75 mm thick clay seam was noted in cored sample 17 from Borehole BH21-1

² Cored sample 14 and the upper portion of cored sample 15 from Borehole BH21-2 were within the granular glacial till

Based on the RQD range indicated in the table, the bedrock cores obtained from Borehole CP21-1 can be classified as being of fair quality (except the top 0.2 m which is classified as very poor). The bedrock cores obtained from Borehole CP21-2 can be classified as very poor above a depth of about 15.5 m, and fair to good quality below that depth.

The rock cores from both boreholes were generally characterized as fresh to slightly weathered. An approximately 75 mm thick seam of completely weathered rock was encountered at a depth of about 14.7 m in Borehole CP21-1.

Four (4) samples of the rock cores obtained from the boreholes were selected for testing to determine their Unconfined Compressive Strengths (UCS). The results of the tests are summarized in Table 5.4 below.

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

Borehole No.	Sample No.	Sample Depth (m below ground)	Sample Elevation (m)	Unconfined Compressive Strength (UCS) (MPa)
CP21-1	16	13.1	90.9	166.7
	17	14.3	89.7	262.2
CP21-2	15	14.6	89.4	168.3
	18	16.5	87.5	215



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The UCS test results of the rock cores ranged from 166.7 MPa to 262.2 MPa. The UCS test results indicate that the dolostone bedrock can be classified as being very strong (R5) to extremely strong (R6).

5.4 GROUNDWATER

The water level in a monitoring well installed within the bedrock at Borehole CP21-1 was measured to be at depths of approximately 9.1 m and 9.2 m (corresponding to elevations of about 94.9 m and 94.8 m) on May 10th and 11th, 2021, respectively.

Groundwater levels at the site will be subject to fluctuations due to seasonal changes, snowmelt or precipitation events and the water level within the nearby 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 TESTING

Chemical analyses associated with the potential for corrosion or sulphate attack (i.e., pH, resistivity, and chloride and sulphate content) were carried out by Paracel Laboratories Inc. on one representative sample from each borehole. The analysis results are provided in Table 5.5 and included in Appendix D for reference.

Table 5.5: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Resistivity (Ohm-m)	Chloride (µg/g)	Sulphate (µg/g)
CP21-1	SS14	10.97	7.9	32.8	36	177
CP21-2	SS5	3.35	7.8	16.2	212	51

6.0 MISCELLANEOUS

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

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

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

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

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

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

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



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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;

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PART B - PRELIMINARY FOUNDATION DESIGN REPORT

For
G.W.P 4003-19-00

Highway 401 Rehabilitation, Brockville, Ontario
Highway 401 CPR (VIA) Crossing Overpass Structure Replacement (Site No. 16X-0122/B0)

Brockville, Ontario

8.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

8.1 OVERVIEW

This section of the report provides preliminary foundation design input related to the proposed replacement of the CPR (VIA) overhead structure located at approximately Station 22+308 on Highway 401 in the City of Brockville (Site No. 16X-0122/B0). The new overhead structure is being designed to accommodate the ultimate 8-lane highway configuration.

The interpretation and preliminary recommendations provided in this report are intended solely to provide the designers with information to assess feasible foundation alternatives for the proposed 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 detailed design of this project. A detailed Foundation Investigation and Design Report will need to be prepared after further field investigation is carried out. The foundation recommendations presented in this preliminary report are subject to change, if necessary, based on the findings of the future site investigation.

8.2 PROJECT DESCRIPTION AND BACKGROUND

8.2.1 Project Description

The project involves the preliminary design for a new overpass structure of the railway corridor that is being completed as part of an overall study related to the rehabilitation of Highway 401 in the City of Brockville (GWP 4003-19-00).

Based on preliminary design information, the new bridge will be a single-span structure that will accommodate an ultimate 8-lane highway configuration (four lanes in each direction). This will require a new structure that is wider and longer than the existing bridge.



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8.2.2 Existing Overpass Structure

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 is a single-span rigid frame structure, with an approximately 17.3 m long span, constructed in the late 1950's (W.P. 6-56). The bridge and the tracks underneath the structure are skewed at about 28.4° relative to Highway 401.
- At the bridge site, the Highway 401 pavement surface is at an elevation of approximately 103.8 m while the ground surface within the railway corridor is at an elevation of approximately 95.3 m.
- Retaining walls are present on both sides of each abutment.
- The abutments and associated retaining walls are all supported on strip footings. The abutment foundations consist of strip footings that are approximately 33 m long and 2.4 m wide. The abutment footings are founded at an elevation of approximately 299 feet (~91.1 m) and transition to the retaining wall footings, which are founded at approximately 302 feet (~92.0 m), by means of a series of step footings on both sides of each abutment.
- The retaining wall strip footings have variable lengths and widths. The structural drawings provide the following details for the retaining wall footings:
 - Foundations for the SW and NE wing walls (referred to as Wall "A" on the structural drawings) are approximately 21.8 m long and approximately 3.8 m to 1.9 m wide, with the wider portions located adjacent to the abutments.
 - Foundations for SE and NW wing walls (referred to as Wall "B" on the structural drawings) are approximately 12.4 m long and approximately 3.7 m to 1.9 m wide, with the wider portions located adjacent to the abutments.

8.2.3 Proposed Structure Modifications and Replacement

Based on preliminary design information, the existing overhead structure is planned to be replaced with a new bridge that will accommodate the ultimate, eight lanes of through traffic on Highway 401. The new overpass is planned to consist of a single-span structure with a span length of about 21 m and 6 m long approach slabs on each side of the bridge. The new bridge is planned to be supported on integral abutments.

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, and the existing bridge replaced in stages in order to 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.

Consideration is being given to constructing high-performance RSS walls to serve as false abutments and as retaining/wing walls adjacent to both sides of both abutments (i.e., at all four corners/quadrants of the bridge and the adjoining approach embankments). The proposed RSS retaining wall configuration is shown on Drawing 1 in Appendix A.

The existing, approximately 8.5 m to 9 m high approach embankments will need to be widened from about 30 m to approximately 44 m at highway grade to accommodate the wider bridge; approximately 7 m of embankment widening is anticipated on both the north and south sides of the existing embankments. The portions of the approach embankments above the RSS walls are planned to be constructed with sideslope inclinations of approximately 2H:1V. Beyond the RSS walls, the full embankment slope is also anticipated to be constructed at 2H:1V.



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8.3 DEGREE OF SITE AND PREDICTION MODEL UNDERSTANDING

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

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

8.4 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered during the current investigation at the site consisted of a surficial layer of asphalt underlain by approximately 9 m of fill (pavement structure and approach embankment fill) in both boreholes. At Borehole CP21-1, the fill is directly underlain by glacial till. In Borehole CP21-2, a native silty sand to sand and gravel layer is present underneath the fill and above the underlying glacial till. The till varies in composition from firm to very stiff sandy clayey silt to loose to very dense sandy silt with varying amounts of gravel that contains cobbles and boulders. The overburden is in turn underlain by Dolostone bedrock. The groundwater level was measured to be at approximately 9.2 m depth in the monitoring well screened within the bedrock at Borehole CP21-1; this level is about 0.5 m below the rail corridor ground surface.

The soil profile presented below in Table 8.1 and on Drawing No. E1 in Appendix E can be used for the preliminary design of the bridge replacement. The geotechnical parameters recommended for each soil layer 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 during the investigation.



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Table 8.1: Representative Subsurface Profile – Site 16X-0122/B0

Elevation (m)		Soil Type	Design Parameters			
From	To		Total Unit Weight γ , (kN/m ³)	Drained Friction Angle ϕ' , (°)	Undrained Shear Strength, Su, (kPa)	Soil Modulus E, (MPa)
104.0	95.0	Variable FILL: Loose to compact Silty SAND with GRAVEL (SM), SAND (SP), Gravelly SAND (SP), Gravelly SILTY SAND (SM), SAND & GRAVEL (SP/GP), COBBLES & BOULDERS (ROCKFILL).	23.0	32	N/A	25
95.0	93.8	Compact SILTY SAND (SM), some gravel to SAND and GRAVEL (SP/GP). Contains cobbles.	22.0	30	N/A	20
93.8	91 ⁴	TILL: Firm to Very Stiff SANDY CLAYEY SILT (CL), to loose to very dense SANDY SILT (ML), some gravel to gravelly. Contains cobbles and/or boulders.	22.0	32 ¹	100 ¹	25
91 ⁴	85.6	Strong to extremely strong DOLOSTONE. Very poor to good quality.	27.0	N/A	175 MPa ²	N/A

Notes:

- (1) The friction angles are applicable to drained conditions only while the undrained shear strength is applicable to undrained conditions only.
- (2) The value in the strength column for the bedrock represents the Unconfined Compressive Strength
- (3) Groundwater is assumed to be at an Elevation of 94.9 m for preliminary design purposes. Submerged unit weights (γ') should be used below the groundwater level.
- (4) The elevation of top of bedrock/base of glacial till varies from 90.6 m to 92.2 m. The elevation in the table is an approximation only; refer to borehole information for actual elevations at each test location.

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 native soils at the bridge site consist of thin layers of predominantly granular soils (silty sand to sand and gravel) and/or glacial till deposits. These materials are underlain by bedrock that was encountered at depths of approximately 3 m to 5 m



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below original 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>	<i>S_a(0.2)</i>	<i>PGA_{ref}</i>	Site Class	Site Adjusted <i>PGA</i>
0.166g	0.262g	0.133g	C	0.166g

8.6.3 Liquefaction Potential

The potential for soil liquefaction of the native silty sand to sand and gravel and the sandy silt 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 94.9 m.

Based on the results of these analyses, the factor of safety against liquefaction of these soils is greater than 1.5 under the design earthquake loading conditions, and, as such, these soils are not considered to be liquefiable.

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

8.7 PRELIMINARY FOUNDATION ENGINEERING DESIGN INPUT

The following sections provide preliminary geotechnical engineering design inputs for the design of the foundations for a replacement overpass structure at this site. The inputs provided herein are 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 CPR crossing replacement bridge.



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Table 8.3: Comparison of Foundation Options for Replacement Overpass Structure

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 driven pipe piles for difficult driving conditions 	<ul style="list-style-type: none"> 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
Drilled Pipe Piles (Socketed in Bedrock)	<ul style="list-style-type: none"> Can be advanced through rockfill, till containing cobbles and boulders and into bedrock Reduced settlement Reduces depth of excavations 	<ul style="list-style-type: none"> May not be suitable for integral abutments Lower capacities than H-piles Reduced uplift capacities 	Medium	<ul style="list-style-type: none"> Difficult to verify pile capacity and seating
Drilled Piers / Caissons	<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"> Not suitable for integral bridge abutments Requires use of liners and/or drilling mud to balance water pressures; cannot be visually inspected Bedrock is very strong and auger drilling may not be practical, would likely require the use of a cluster hammer drill or churn drilling techniques Difficult to drill piers/advance liners in till deposits containing boulders and cobbles 	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.
Shallow Foundations Founded on bedrock	<ul style="list-style-type: none"> Pile driving/drilling through difficult deposits to install deep foundations is avoided Can support large structural loads with minimal settlement 	<ul style="list-style-type: none"> Not suitable for integral abutments (Semi-integral abutments possible) Larger foundation areas required compared to integral abutments or drilled piers/greater potential for conflicts with existing foundations Surface of bedrock is up to 13.5 m below highway level necessitating deep excavations/very high temporary protection systems for foundation construction. 	Low to medium	<ul style="list-style-type: none"> Movement/deflection of high temporary protection systems could lead to deformations of adjacent travelled lanes of highway Potential for shallow refusal of temporary protection system components on cobbles and boulders requires pre-drilling
Shallow Foundations Founded within overburden	<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"> Not suitable for integral abutments (Semi-integral abutments possible) Potential for overstressing the silty clay till subgrade leading to large settlements 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

For an integral abutment design, steel H-pile foundations would be a suitable foundation option. The piles would be driven to refusal in the till and would develop most of their load carrying capacity from the end-bearing resistance. Where integral abutments are adopted, the upper portion of the piles would be installed within twin/double 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 existing highway embankment materials if they are not completely removed in the area of the abutments as part of the RSS wall construction.



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Drilled pipe piles installed, and socketed into the bedrock, using a down-the-hole hammer drilling system could also be considered. The use of this system would limit potential difficulties associated with advancing driven piles through the rockfill and the cobble and boulder-laden till deposits. As the interiors of the drilled pipe piles are typically filled with concrete following installation to the design tip elevations, these types of piles may not be suitable for incorporation into integral abutment systems due to their stiffness.

Drilled piers/caisson foundations are not suitable for integral abutments and in this respect are not suitable for support of the replacement overpass structure if integral abutments are planned. Construction of drilled piers/caissons 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.

Support of the abutments on shallow foundations founded within the overburden could result in overstressing of the underlying firm to very stiff silty clay till soils leading to large and unacceptable total and differential settlements, and therefore is not recommended. Consideration could be given to supporting the abutments on strip footings founded on bedrock; however, excavations for this founding option would extend up to 14 m below highway level requiring the installation of substantial temporary protection systems that could pose a risk of ground deformations of the adjacent highway lanes.

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 which is also understood to be the preferred foundation system from a structural engineering perspective. Further discussion and preliminary design input regarding this option is provided below.

8.7.2 Driven Piles

8.7.2.1 Design Considerations

Pile foundations consisting of steel H-piles that are driven to effective refusal within the till or underlying bedrock, and that derive the majority of their capacity from end-bearing, can be used to support the integral abutments of the proposed replacement bridge. Pipe piles are considered to have a higher risk than H-piles of 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 an elevation of approximately 100 m. The surface of the bedrock was encountered at elevations varying from approximately 92 m to 90.6 m which would result in maximum pile lengths of approximately 8 m to 9.5 m.

Effective refusal or pile deflection could occur at shallow depth within the rockfill materials where cobbles and/or boulders are encountered. The rockfill materials in the vicinity of the new abutments are expected to be removed to facilitate construction of the proposed RSS walls. For preliminary design purposes, provision for predrilling through the rockfill materials is recommended to facilitate advancing the piles if the rockfill is not completely removed in the



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area of the abutments as part of the RSS wall construction. The requirement for predrilling should be further reviewed during the detailed design stage.

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

8.7.2.2 Geotechnical Axial Resistance (Driven H-Piles)

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	92.2 m to 90.6	1,800*
HP 310 x 132	92.2 m to 90.6	2,100*

*Note: Due to the potential for piles 'hanging up' in the glacial till above the bedrock surface, the factored geotechnical resistances in the above table have been assessed based on the piles potentially encountering effective refusal to driving within the glacial till. Higher capacities could be achieved for piles driven to refusal on the dolostone bedrock.

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

8.7.2.3 Downdrag and Relaxation of Piles

The native site soils underlying the abutments of the new bridge consist of a surficial layer of loose to compact silty sand to sand and gravel overlying glacial till. Accounting for the removal of near-surface materials as part of the construction of the proposed RSS walls, the abutment piles would only extend through approximately 1 m to 2 m of the native soils before encountering bedrock. Based on this geometry and the site soils conditions, significant downdrag loads are not expected to develop on the abutment piles. 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.



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The site soils generally consist of highway embankment fill materials including rockfill materials, and compact silty sand to sand and gravel that is underlain by firm to very stiff/loose to very dense glacial till with cobbles and/or boulders and then by bedrock. Based on these conditions, removal of, or pre-drilling through, the rockfill is recommended. The piles should also be provided with driving shoes such as Titus “H” Bearing Pile Point (Standard Model) in order to limit possible pile damage should boulders be encountered within the till. 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 for HP 310x110 piles:

- Piles to be fitted with rock points and driven into bedrock in accordance with OPSS.PROV 903. (Note: Pile tip treatment may not be necessary if the piles are installed in holes pre-drilled to the bedrock surface).

The capacity of each pile should be verified in the field by the use of either the Hiley Formula (MTO Standard Structural Drawing SS-103-11) or high-strain dynamic testing (i.e. Pile Driving Analyzer (PDA) testing) to confirm that the specified ultimate capacity is achieved. If consideration is given to driving the piles to bedrock following predrilling through the till such testing would not be required and the piles should be driven to bedrock in accordance with the requirements of 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. The calculated value is based on an estimated pile length of 8 m.

Table 8.5: Recommended Tensile Pile Resistance

Pile Type	Minimum Pile Length(m)	Factored Geotechnical Resistance (Tension) at ULS _r (kN)
HP310x110 or HP310x132	8	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 Drilled Pipe Piles

8.7.3.1 Design Considerations

Drilled pipe piles can also be considered for support of the abutments of the proposed replacement bridge. The piles are installed using a down-the-hole hammer system which is capable of advancing through the rockfill materials and cobbles and boulders within the till with less difficulties in comparison to a driven pile system. The pipe pile foundations would be socketed into the bedrock and derive the majority of their capacity from end-bearing.

For preliminary design purposes, the drilled pipe piles are recommended to be socketed a minimum of 1 m into the bedrock due to the variable quality and degree of fracturing of the upper portion of the rock mass. This would correspond to pile tip elevations of approximately 91 m at the west abutment and 89.5 m at the east abutment. The



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design tip elevations should be reassessed during the detailed design stage once further investigation is completed to better define the bedrock surface and engineering characteristics of the upper portion of the bedrock.

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

8.7.3.2 Geotechnical Axial Resistance (Driven H-Piles)

The typical drilled pipe pile size used by local contractors is understood to have a diameter of approximately 244 mm. For this size of pile, a factored geotechnical resistance at ULS of 1,000 kN may be used for preliminary design. This value includes a resistance factor of 0.4 applied to the ultimate capacity.

The estimated geotechnical reaction at SLS (factored) for 25 mm of vertical settlement for a 244 mm diameter drilled pipe pile socketed 1 m into the bedrock exceeds the factored geotechnical reaction at ULS. Therefore, the ULS (factored) resistances will govern.

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

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

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

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

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

where H is the height of the wall and γ is the unit weight of the backfill soil. Values for K_a , K_p , K_o and γ are provided in Table 8.6 and Table 8.7 for horizontal and sloping (2H:1V) backfill conditions, respectively. The thrusts act at a point



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one third up the height of the wall. For the purposes of preliminary design, a friction angle of 32 degrees has been assumed for the existing embankment fill materials at the site; this value will need to be confirmed and/or reassessed once further subsurface investigation is completed prior to detailed design.

Table 8.6: 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	23
Effective Friction Angle	32°	35°	32°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.47
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.31
Coefficient of Passive Earth Pressure (K_p)	3.25	3.7	3.25

Table 8.7: 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	23
Effective Friction Angle	32°	35°	32°
Coefficient of Earth Pressure at Rest (K_o)	0.68	0.62	0.68
Coefficient of Active Earth Pressure (K_a)	0.47	0.39	0.47

*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



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The seismic earth pressures for structures with horizontal backfill behind the walls may be calculated using the parameters provided in Table 8.8. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate. Table 8.9 and Table 8.10 provide seismic earth pressures for yielding walls with horizontal and 2H:1V backfill slopes behind the walls, respectively.

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

Table 8.8: Seismic Design Parameters to Estimate Lateral Earth Pressures

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

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

Table 8.9: 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 8.10: 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	0.42	0.40



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8.9 RETAINED SOIL SYSTEM WALL DESIGN CONSIDERATIONS

8.9.1 General

Retained Soil System (RSS) walls are being considered to contain the soils behind the abutments (i.e. false abutment configuration) and along the north and south sides of the proposed widened embankments. Based on the proposed wall configuration, as shown on Drawing 1 in Appendix A, the RSS walls at the SW and NE corners of the overpass will extend parallel to the abutments for approximately 10 m beyond the outside edges of the bridge and then turn to parallel the approach embankments. At the SE and NW corners of the bridge, the RSS walls will extend approximately 3 m beyond the abutments, turn approximately 45 degrees away from the rail tracks for about 8 m and then turn again to parallel the approach embankment.

Retained soil systems are listed in the MTO Designated Sources of Materials (DSM) and under Special Provisions 599S22 and 599S23. The RSS should be tendered with the following attributes:

Application: False Abutment
Geometry: Vertical (GV)
Performance: High

8.9.2 Geotechnical Resistance and Reaction

The factored geotechnical resistance at ULS for the RSS walls founded on the site soils at the proposed founding elevation of 93.5 m is 300 kPa. This geotechnical resistance was evaluated based on assumed RSS dimensions, i.e., having a width of $0.7H$ where H is the height of RSS wall (based on CHBDC 2019). A wall height of 6.7 m was assumed; hence, the width of the RSS wall was assumed to be approximately 5 m. An embedment depth of 0.8 m was assumed in the calculations (based on MTO RSS Design Guidelines, 2007).

The factored geotechnical reaction at SLS was estimated to be 250 kPa for 25 mm of total settlement.

The ULS and SLS values provided above include resistance factors of 0.5 (ϕ_{gu}) and 0.8 (ϕ_{gs}), respectively, in accordance with the CHBDC.

8.9.3 Wall Stability

8.9.3.1 Global Stability

The existing CP Rail overpass approach embankments are approximately 8.5 m to 9 m high (above original grades) have crest-to-crest widths of approximately 30 m and have sideslopes of approximately 2H:1V. The new overpass structure is planned to consist of partial height cast-in-place abutment stems with RSS false abutment walls located in front of the abutments. The RSS walls are planned to be approximately 6.7 m high. The new approach embankments are proposed to have sideslope inclinations of approximately 2H:1V above the top of the RSS walls.

Analyses were carried out to evaluate the global stability of the approach embankments incorporating the proposed RSS walls. Stability analyses were carried out for the approach embankment on the east side of the bridge for two configurations: 1) failure surfaces extending beneath the abutment into the railway corridor and 2) failure surfaces on the sideslopes of the approach embankment. The analyses were completed using the commercial program Slope/W



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(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 west side of rail corridor is expected to be similar to or greater than the east 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 8.5 m to 9 m.
- The maximum height and reinforced width of the RSS wall false abutment/retaining walls are about 6.7 m and 5 m, respectively.
- The RSS walls were considered as rigid blocks (i.e., slip surfaces going through the RSS wall were not considered in the stability analysis).
- 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 seismic (pseudo-static) analysis incorporated a k_h value of 0.083g.
- The analyses included allowance for dynamic loading due to traffic by considering a static surcharge load equivalent to 0.8 m of additional fill.

A minimum factor of safety under static conditions of 1.5 (corresponding to a ϕ_{gu} of 0.65) is considered acceptable against deeper-seated failure surfaces extending through the native soils beneath the retaining walls based on the 'Typical' degree of understanding of those soils.

The results of the slope stability analyses for an RSS cross-section near the abutments under static, drained conditions and seismic conditions are provided on Figures E2 and E3, respectively, in Appendix E. The results of the stability analyses indicate that the proposed RSS wall/abutment configuration would provide a factor of safety against instability of approximately 1.5 under static conditions; stability analyses carried out using undrained parameters provided similar or higher factors of safety. A factor of safety of approximately 1.3 was calculated under seismic conditions.

The results of the slope stability analyses for the approach embankment (i.e. failure surfaces oriented perpendicular to the highway) under static, drained conditions and seismic conditions are provided on Figures E4 and E5, respectively, in Appendix E. The results of these stability analyses indicate the factor of safety against instability of a critical failure surface extending beneath the RSS wall is approximately 1.5 under static conditions; stability analyses carried out using undrained parameters provided similar or higher factors of safety. A factor of safety of approximately 1.3 was calculated for this embankment configuration under seismic conditions.

8.9.3.2 Internal Stability of RSS Walls

The internal stability of the RSS blocks/walls is a function of internal design of the wall, which is beyond the scope of this report. It is anticipated that the internal stability of the wall will be assessed by the RSS wall designer/supplier. The recommendations provided in Section 6.19.10 in the CHBDC shall be considered in this respect.



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8.10 EMBANKMENT DESIGN CONSIDERATIONS

8.10.1 Embankment Settlement

The existing CP Rail overpass approach embankments have crest-to-crest widths of approximately 30 m, are approximately 8.5 m to 9 m high (above railway corridor grade) and have sideslopes of approximately 2H:1V. The approach embankments are planned to be widened to approximately 10 m towards both the north and south to accommodate the new, wider bridge structure and ultimate highway configuration. Adjacent to the new bridge, the widened embankments will be supported by the RSS walls described previously. The portions of the new approach embankments located above the top of the RSS walls 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 magnitude of settlement of the soils underlying the embankments due to the proposed widening of the approach embankments. The evaluation of settlements for the embankment were carried out using the commercial program Settle3D (Rocscience 2020) for the embankment on the east side of rail corridor. Based on the subsurface conditions encountered and planned embankment geometries, settlements on the west side of CP rail are expected to be similar to or less than the east side.

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

- The typical soil profile and associated design parameters shown in Table 8.1 were considered in the settlement analyses.
- The new embankment platform involves widening the crest-to-crest width of the existing approach embankments approximately 20 m (about 10 m to the north and to the south) with RSS walls constructed at the toe of embankments.
- The loads from the bridge abutments will be transferred to the bedrock by the bridge foundations and will not contribute significantly to the settlement of the embankment.

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 new approach embankments leading up to the widened bridge is expected to be in the order of 40 mm due to the additional loading imposed by the proposed widening of the approach embankments.

Settlements beneath the existing roadway are calculated to be less than 5 mm. These settlements are anticipated to take place 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.

Self-weight settlement due to compression of the maximum 8.5 m of embankment fill placed during the construction process is expected to be less than 42.5 mm (approximating 0.5 % strain). The bulk of this settlement is expected to be completed almost immediately after the fill has achieved its full height.

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



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8.11 CEMENT TYPE AND CORROSION POTENTIAL

Two soil samples, one from each borehole location, were submitted to Paracel Laboratories for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 5.5.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations of the samples tested were 177 and 51 µg/g. As per Canadian Standards Association (CSA) Standard A23-1.14/A23.2-14, sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH of the samples tested were 7.9 and 7.8 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 one of the reported resistivity values was below that level.

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

9.0 CONSTRUCTION CONSIDERATIONS

9.1 CONSTRUCTION STAGING

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 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.



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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); (struts/rakers as required)	<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

Due to the potential difficulties installing sheet piles through the rockfill (i.e. very dense fill containing frequent cobbles and boulders encountered in both boreholes) 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 to conventional H-piles, consideration could be given to the use of drilled pipe piles installed using a down-the-hole hammer drilling system as the soldier piles; brackets would be required to be welded on to the pipe piles to permit lagging installation.

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.



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Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath the proposed RSS wall footing. 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 abutment foundations would be developed through the existing highway approach embankment fill and native soils. These excavations are expected to encounter fill materials and the native compact silty sand to sand and gravel and the firm to very stiff sandy silty clay/loose to very dense sandy silt till deposits. Where space permits, these excavations may be developed using open-cut methods. The fill materials (above the water table) and the firm to very stiff sandy silty clay till deposit would be classified as Type 3 soils.

OHSA indicates that temporary excavations made within Type 3 soils that are above the water table and/or dewatered prior to excavation should be developed with side slopes no steeper than 1H:1V. Granular soils (fill materials and/or native overburden) below the water table, if encountered, would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements.

9.4 TEMPORARY GROUNDWATER CONTROL

The groundwater level was observed at an elevation of approximately 95 m at the time of the investigation and in the monitoring well installed in Borehole CP21-1. Therefore, excavations required for the removal of existing bridge foundations and construction of the new RSS walls are expected to extend below the ground water level. Given the limited thickness of the native silty sand to sand and gravel soils, temporary unwatering using conventional sump and pump techniques is considered appropriate for shallow excavations at the site developed predominantly within the silty sand to sand and gravel or sandy silt to sandy silty clay till soils.

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.

9.5 CONSIDERATION OF EXISTING FOUNDATIONS

Based on the preliminary design information available for the new bridge, the proposed bridge has an abutment-to-abutment span that is slightly longer than the existing bridge. However, the piles supporting the abutments of the new bridge will generally be located in the area of the abutment walls of the existing bridge. Depending on the final bridge configuration, the new piles could coincide with/encounter the foundations of the existing bridge particularly in the areas of the existing wingwalls where the strip footings are wider than the existing abutment footings.



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Additionally, the existing bridge foundations and abutment walls would be located in the area of construction of the planned RSS walls. Provision should be included for removal of all portions of the existing abutment walls/wingwalls and associated foundations in order to minimize the potential for difficulties installing the abutment piles for the new bridge and RSS walls.

9.6 OBSTRUCTIONS

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

10.0 FURTHER WORK FOR DETAILED DESIGN

Based on the subsurface conditions encountered in the current investigation, driven pile foundations at the abutments are the preferred foundation type to be used in the preliminary design of the replacement of the rail overhead structure 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.
- 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.
- Following completion of the additional investigation and laboratory testing, the soil design parameters and analysis results outlined in this report should be re-evaluated.
- A Final Foundation Investigation and Design Report meeting MTO's standard requirements for foundation engineering assignments should be prepared based on the final structure configuration.



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Specifications
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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



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12.0 CLOSURE

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

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

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

Respectfully submitted,

STANTEC CONSULTING LTD.



Roshan Rashed, P.Eng.
Geotechnical Engineer



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



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



PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0

References
February 2023

13.0 REFERENCES

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- 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.



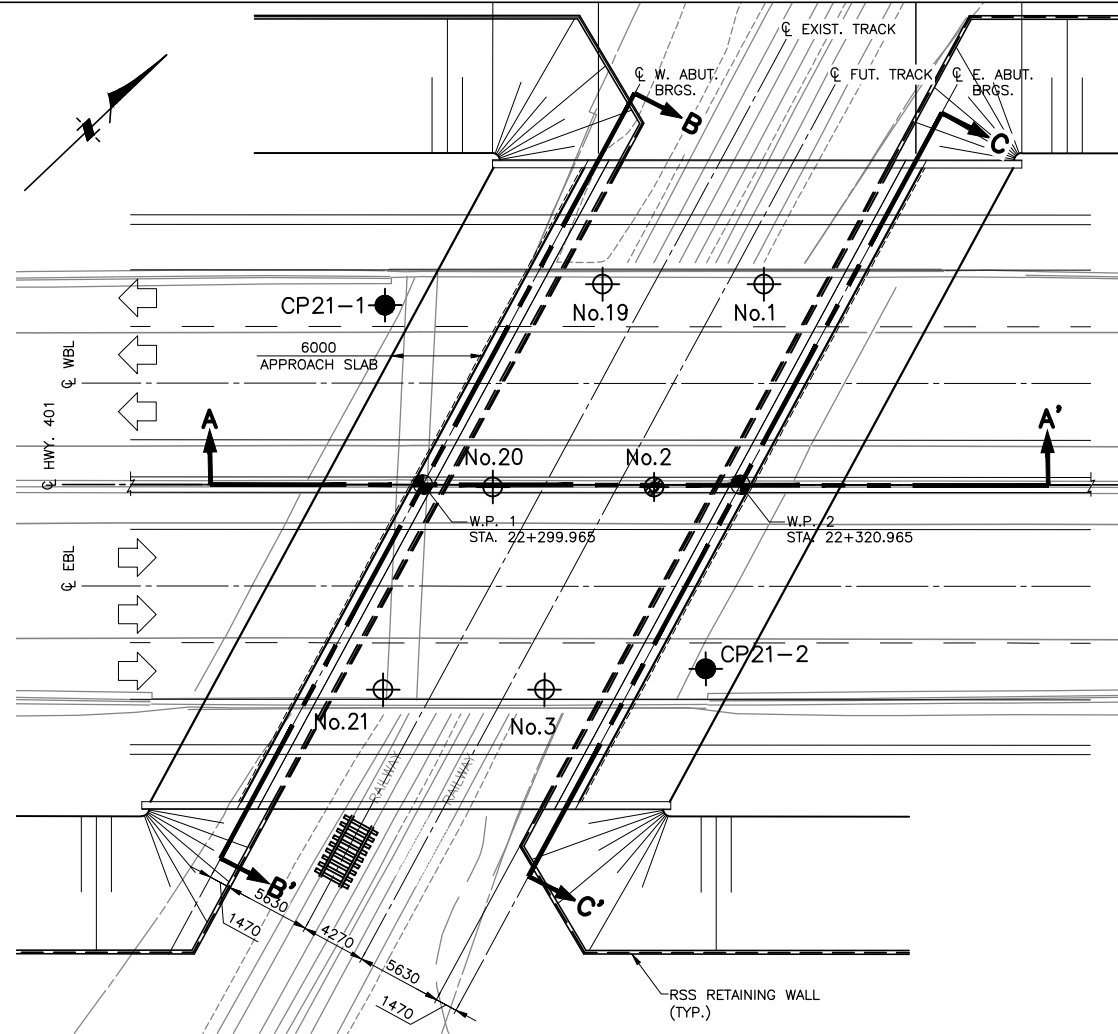
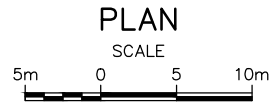
**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD
STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0**

February 2023

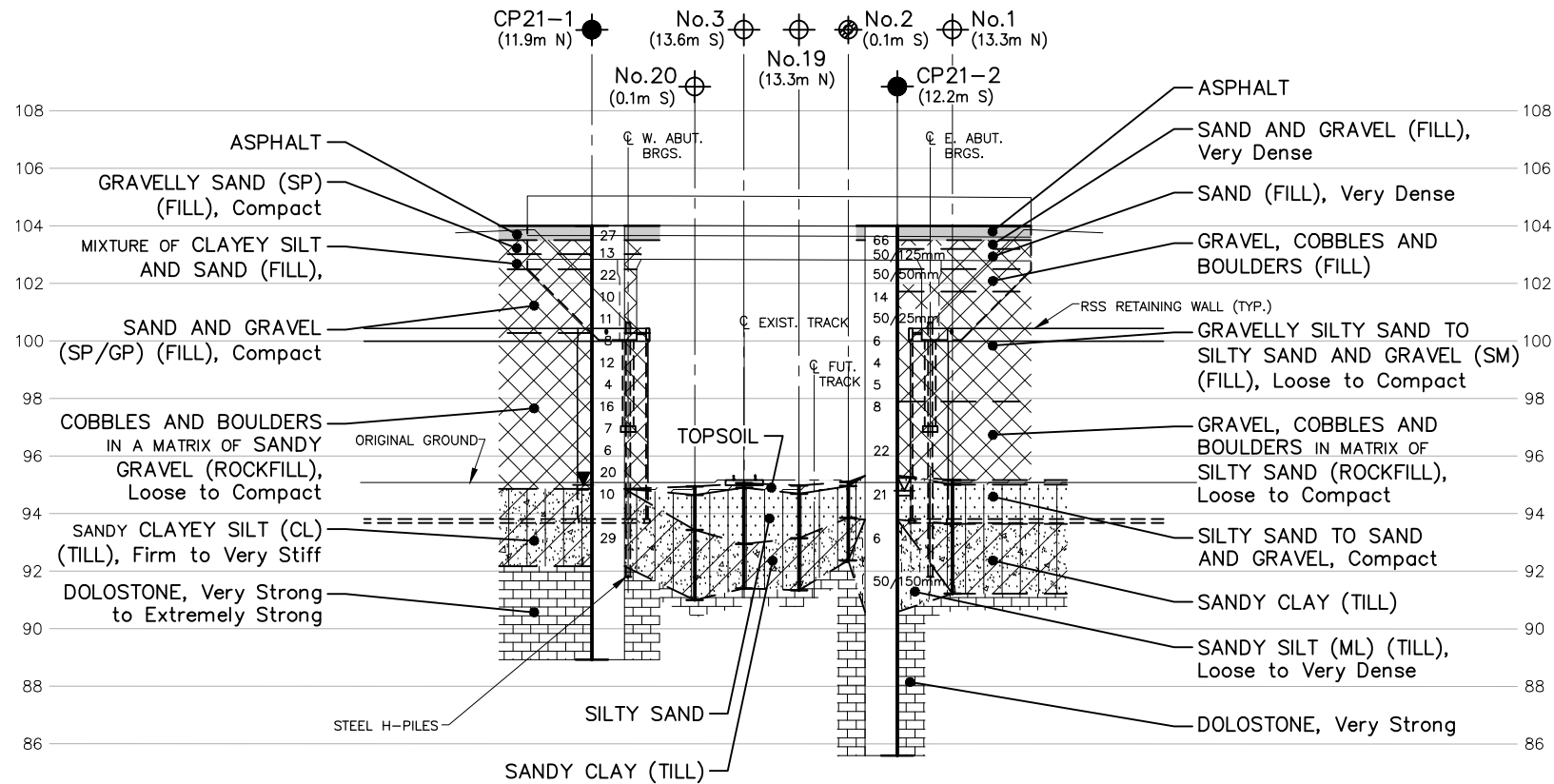
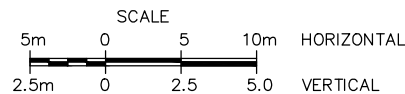
APPENDIX A

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





CROSS SECTION A-A'



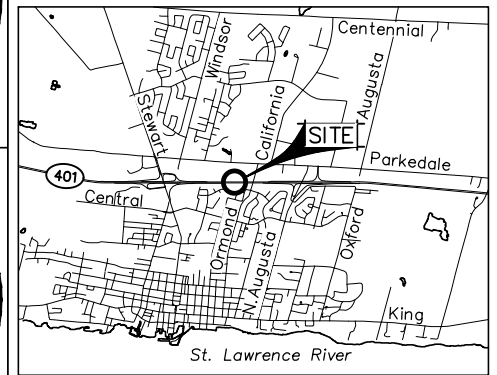
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



PLATE No
CONT
GWP 4003-19-00

HIGHWAY 401
CPR OVERHEAD, BROCKVILLE
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
—



LEGEND

- Borehole (Stantec 2021)
- Penetration & Borehole (MTO 1955)
- Auger Hole (MTO 1955)
- (x.x m) Offset from Cross Section Line in meters
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of Investigation May 2021
- WL Measured on May 11, 2021

No	ELEV	MTM ZONE 9 COORDINATES NORTH	EAST
CP21-1	104.0	4 940 929.0	368 840.5
CP21-2	104.0	4 940 927.7	368 872.6
No.1	94.9	4 940 948.1	368 856.8
No.2	95.1	4 940 933.5	368 861.4
No.3	95.0	4 940 919.1	368 866.2
No.19	95.1	4 940 940.4	368 849.5
No.20	95.2	4 940 925.9	368 854.2
No.21	95.8	4 940 911.4	368 858.8

NOTES

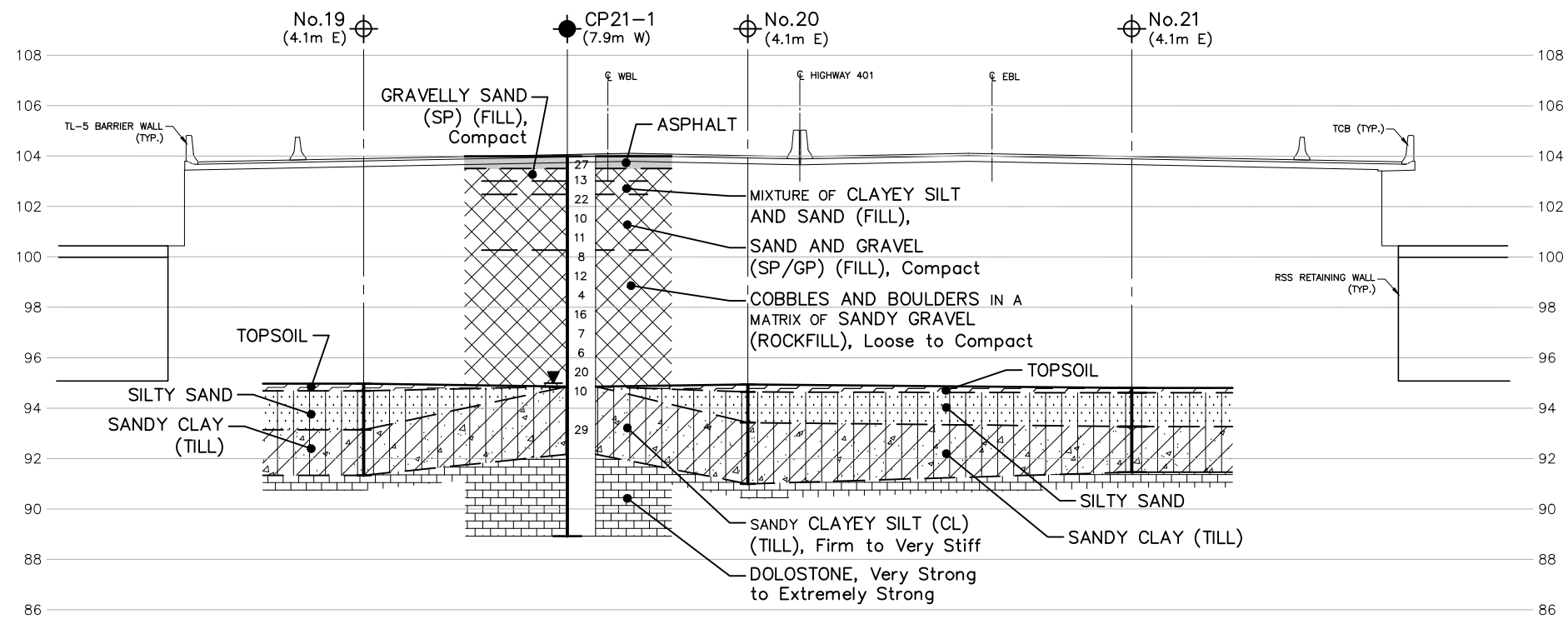
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

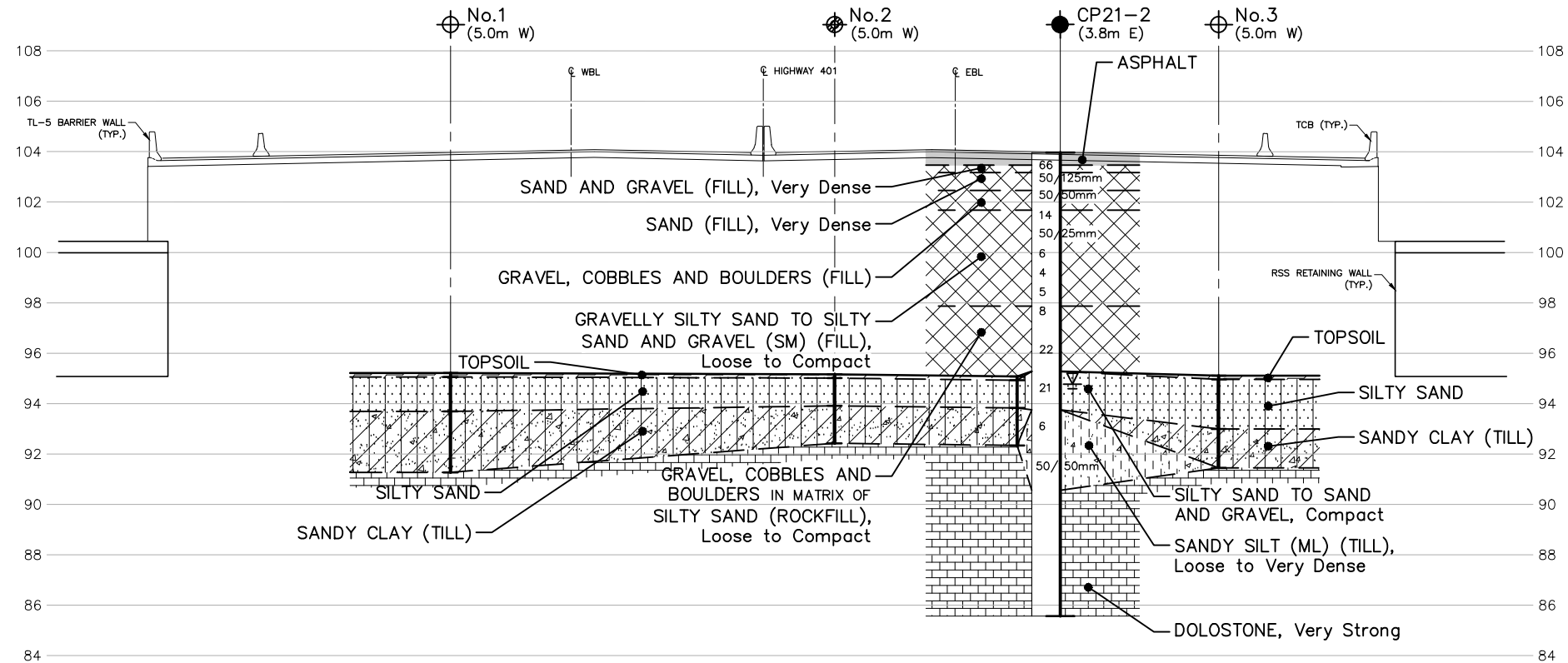
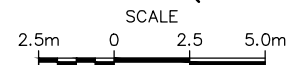
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2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

REVISIONS	DATE	BY	DESCRIPTION

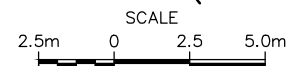
GEORES No 31B-108			
HWY No 401		DIST	
SUBM'D KN	CHECKED	DATE 2023-02-15	SITE 16-122
DRAWN GBB	CHECKED	APPROVED RH	DWG 1



CROSS SECTION B-B' (WEST ABUTMENT)



CROSS SECTION C-C' (EAST ABUTMENT)



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

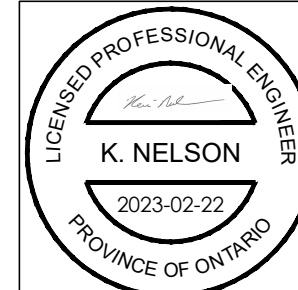
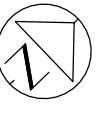
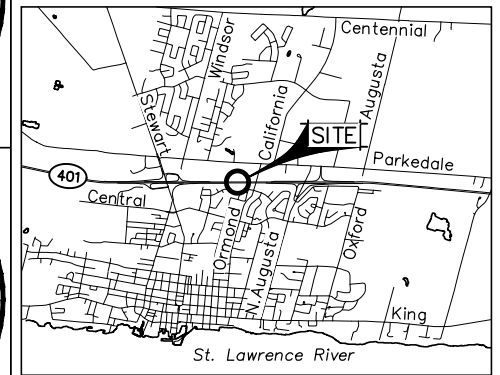


PLATE No
CONT
GWP 4003-19-00

HIGHWAY 401
CPR OVERHEAD, BROCKVILLE
SOIL STRATA



SHEET
—



KEY PLAN
1 km 0 1 2 km

LEGEND

- Borehole (Stantec 2021)
- Penetration & Borehole (MTO 1955)
- Auger Hole (MTO 1955)
- (x.x m) Offset from Cross Section Line in meters
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at time of Investigation May 2021
- WL Measured on May 11, 2021

No	ELEV	MTM ZONE 9 NORTH	COORDINATES EAST
CP21-1	104.0	4 940 929.0	368 840.5
CP21-2	104.0	4 940 927.7	368 872.6
No.1	94.9	4 940 948.1	368 856.8
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No.3	95.0	4 940 919.1	368 866.2
No.19	95.1	4 940 940.4	368 849.5
No.20	95.2	4 940 925.9	368 854.2
No.21	95.8	4 940 911.4	368 858.8

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

GEORES No 31B-108			
HWY No 401		DIST	
SUBM'D KN	CHECKED	DATE 2023-02-15	SITE 16-122
DRAWN GBB	CHECKED	APPROVED RH	DWG 2

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD
STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0**

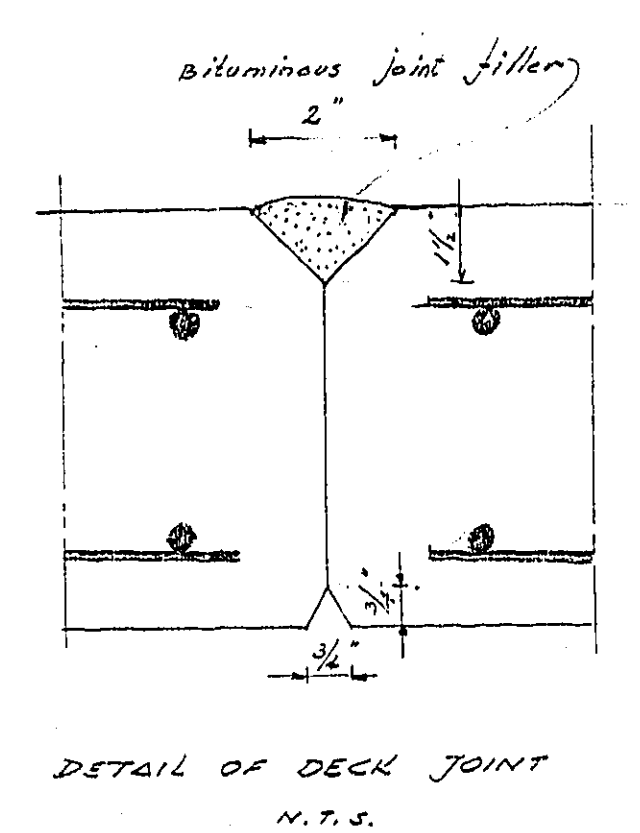
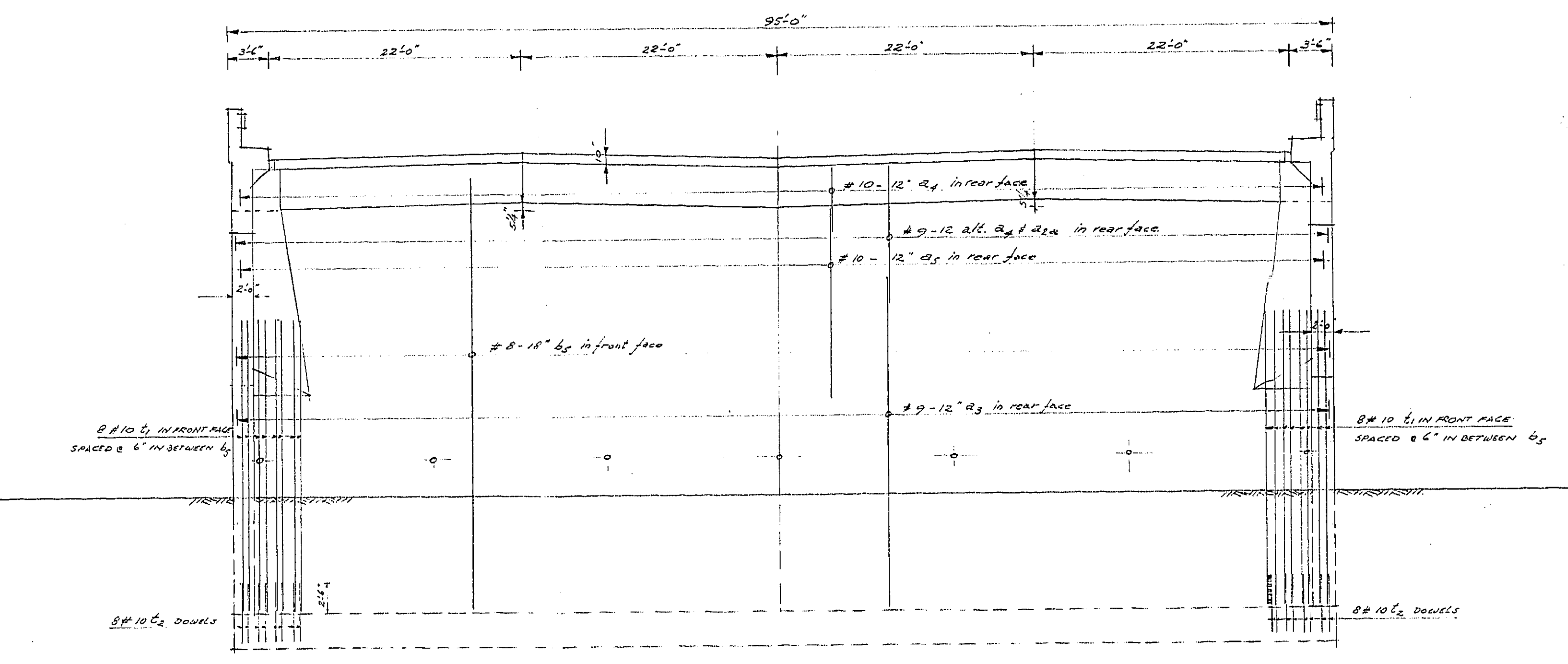
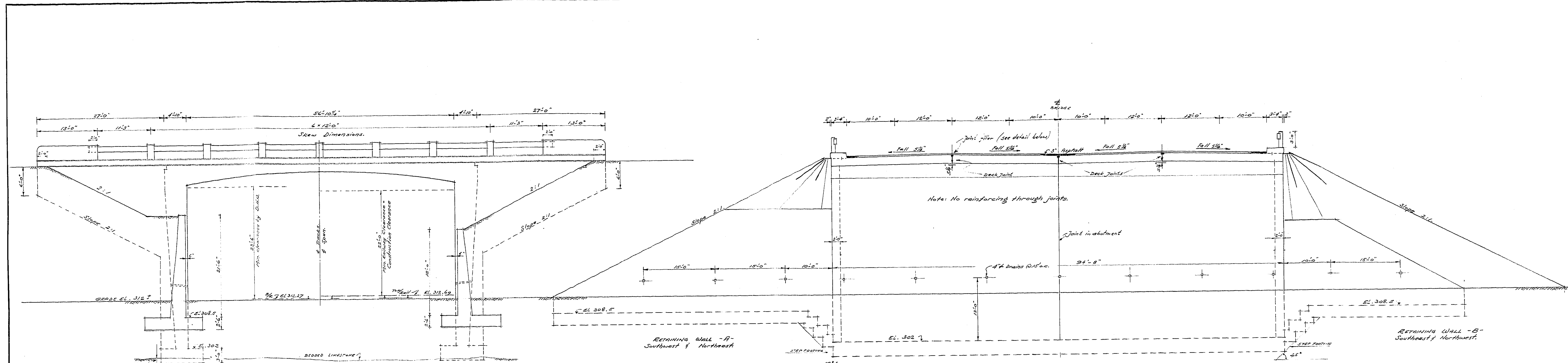
February 2023

APPENDIX B

B.1 EXISTING BRIDGE FOUNDATION PLAN AND AVAILABLE GEOCRETS INFORMATION



ORIGINAL STRUCTURAL DRAWINGS



No.	FOR	DATE
10	REVISION	10.10.57
30	REVISION	10.10.57
13	REVISION	10.10.57

DUNCAN HOPPER & ASSOCIATES
CONSULTING ENGINEERS
1885 WILSON AVENUE
DOWNSVIEW, ONTARIO

W.P. 6-56
S-5639-3

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

ELIZABETHTOWN TWP BRIDGE NO. 12.
C.R.R. OVERHEAD

THE KING'S HIGHWAY No. 401 DIV. No. 8
CO. LEEDS NORTH OF BROCKVILLE.
TWP. ELIZABETHTOWN LOT 10 CON. 1

ELEVATIONS & SECTION.

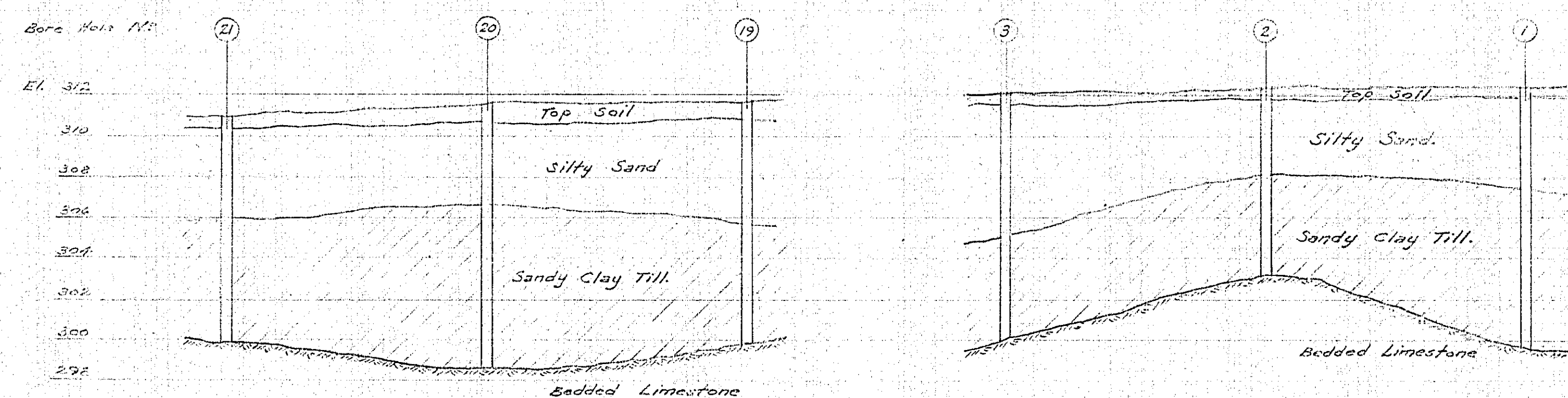
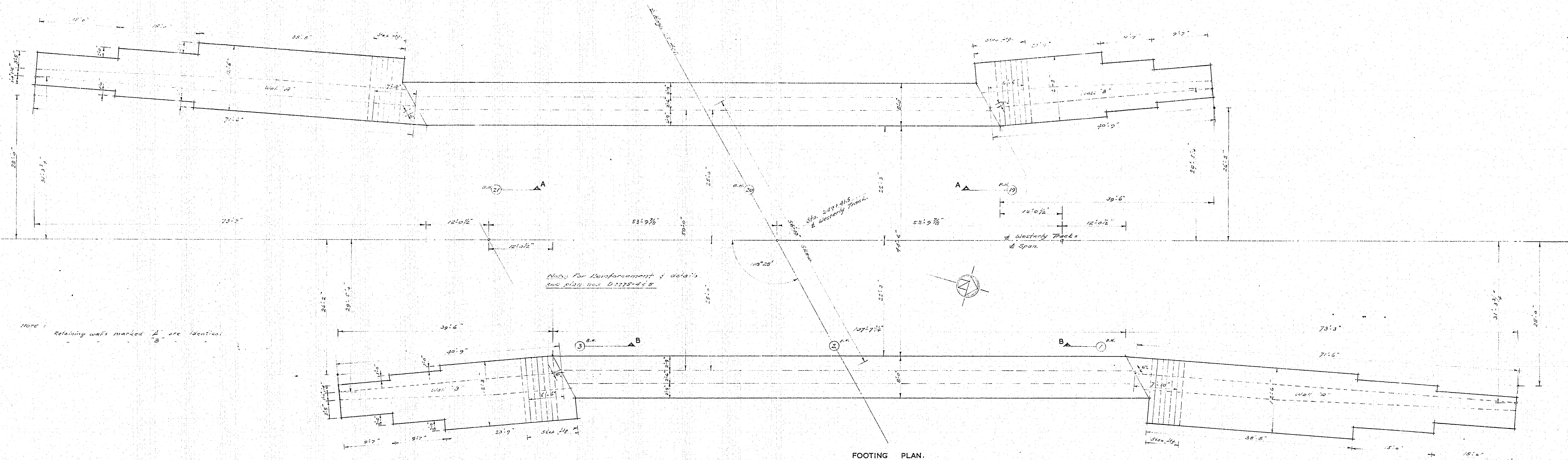
APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER

DESIGN
CHECK
A.D.D.
CONTRACT
NUMBERS
57-208
57-265

DRAWING
CHECK
A.D.D.
LOADING
NUMBERS
410-510
D. 2775-2

DATE 1906.18.1907

TWP 25-122-R-A 56286



SECTION A-A

SECTION B-B

SOIL CONDITION.

BURKMAN HOPPER & ASSOCIATES LTD.
CONSULTING ENGINEERS
591 WILSON AVENUE
BOWENVIEW, ONTARIO

WP 6-56

8-5439-4

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

ELIZABETHTOWN TWP BRIDGE NO. 12.
C.P.R. OVERHEAD.

THE KING'S HIGHWAY No. 401 DIST. No. 8.
CO. LEEDS NORTH OF BROCKVILLE.
TWP. ELIZABETHTOWN LOT 10 CON. 1

FOOTINGS

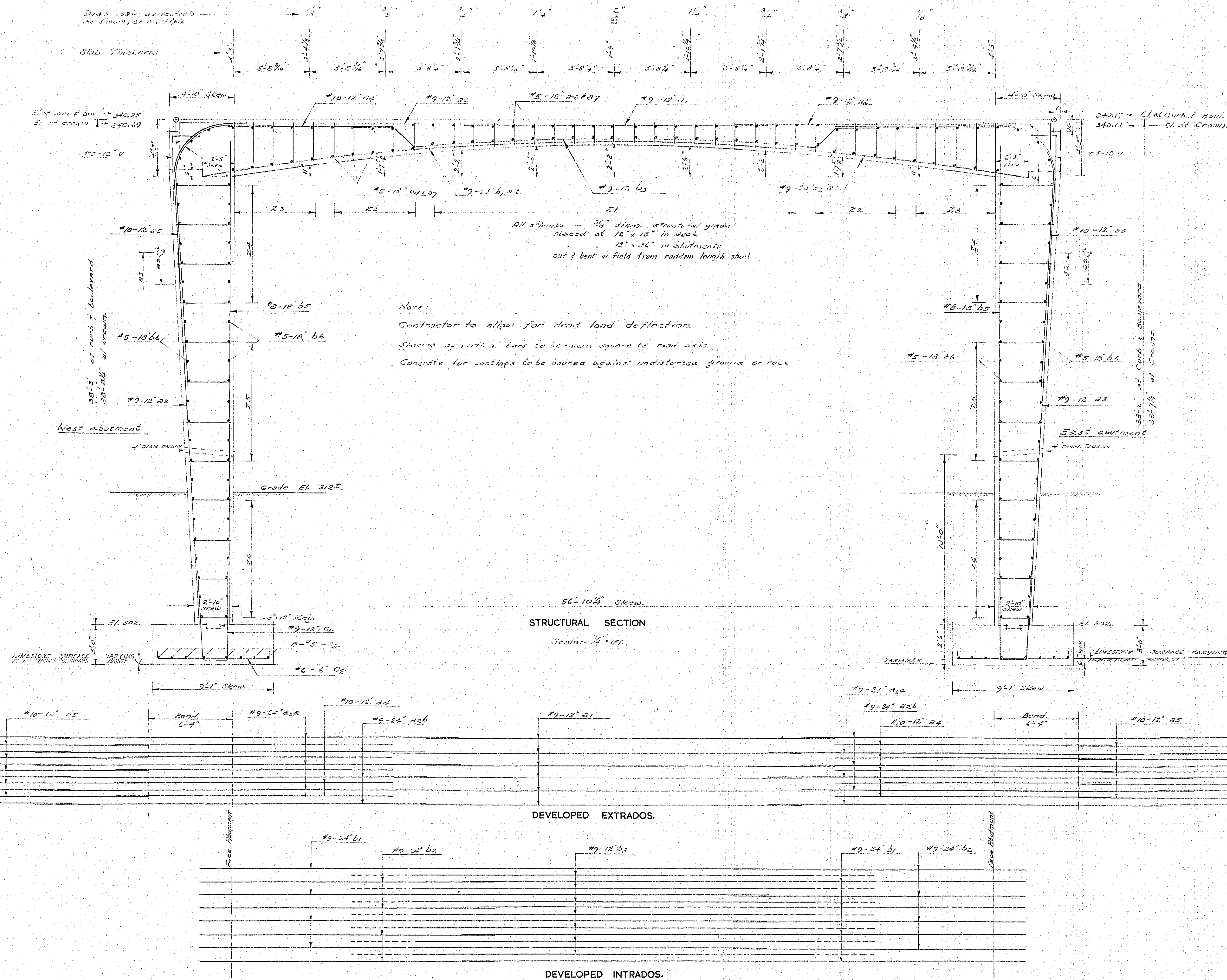
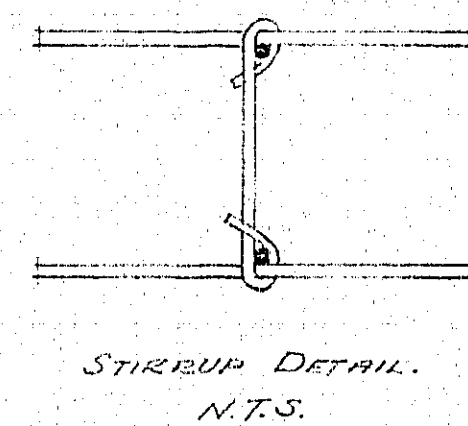
APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER

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TRACING	H.	CHECK	C.H.H.	DRAWING	57-205	57-265
DATE	Feb. 12, 1957			DRAWING	57-205	57-265

TWP # 25-122-3-A

PRINT RECORD		
NO.	FOR	DATE
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20	REVISION	1-1-57
30	REVISION	1-1-57
40	REVISION	1-1-57
50	REVISION	1-1-57
60	REVISION	1-1-57
70	REVISION	1-1-57
80	REVISION	1-1-57
90	REVISION	1-1-57
100	REVISION	1-1-57
110	REVISION	1-1-57
120	REVISION	1-1-57
130	REVISION	1-1-57
140	REVISION	1-1-57
150	REVISION	1-1-57
160	REVISION	1-1-57
170	REVISION	1-1-57
180	REVISION	1-1-57
190	REVISION	1-1-57
200	REVISION	1-1-57

PRINT RECORD		
NO.	FOR	DATE
10	FOR	3.5.57
30	FOR	3.8.57
20	FOR	3.9.57



DUNCAN HOPPER & ASSOCIATES
CONSULTING ENGINEERS
1393 WILSON AVENUE,
DOWNSVIEW, ONTARIO

DEPARTMENT OF HIGHWAYS-ONTARIO-
BRIDGE OFFICE-TORONTO

ELIZABETHTOWN TWP. BRIDGE NO. 12.
C.P.R. OVERHEAD

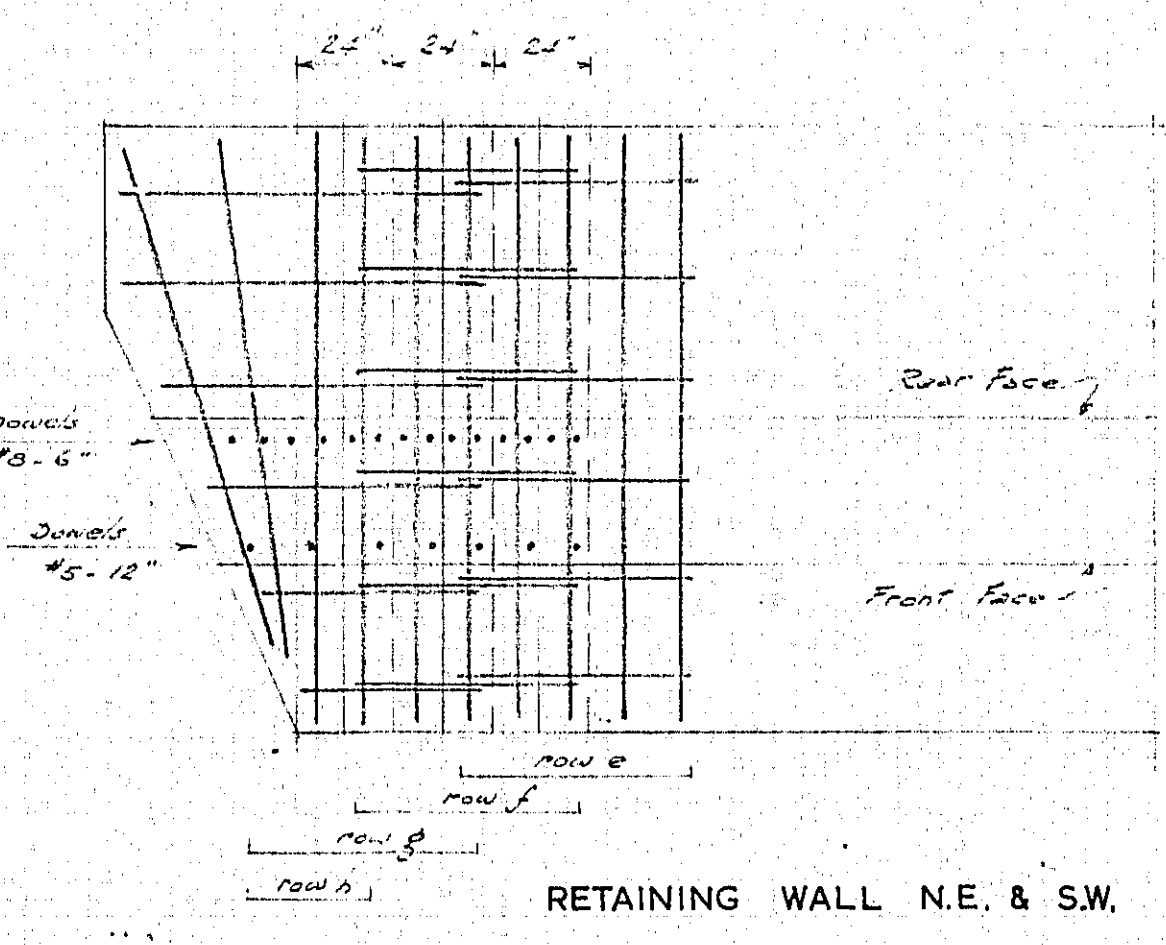
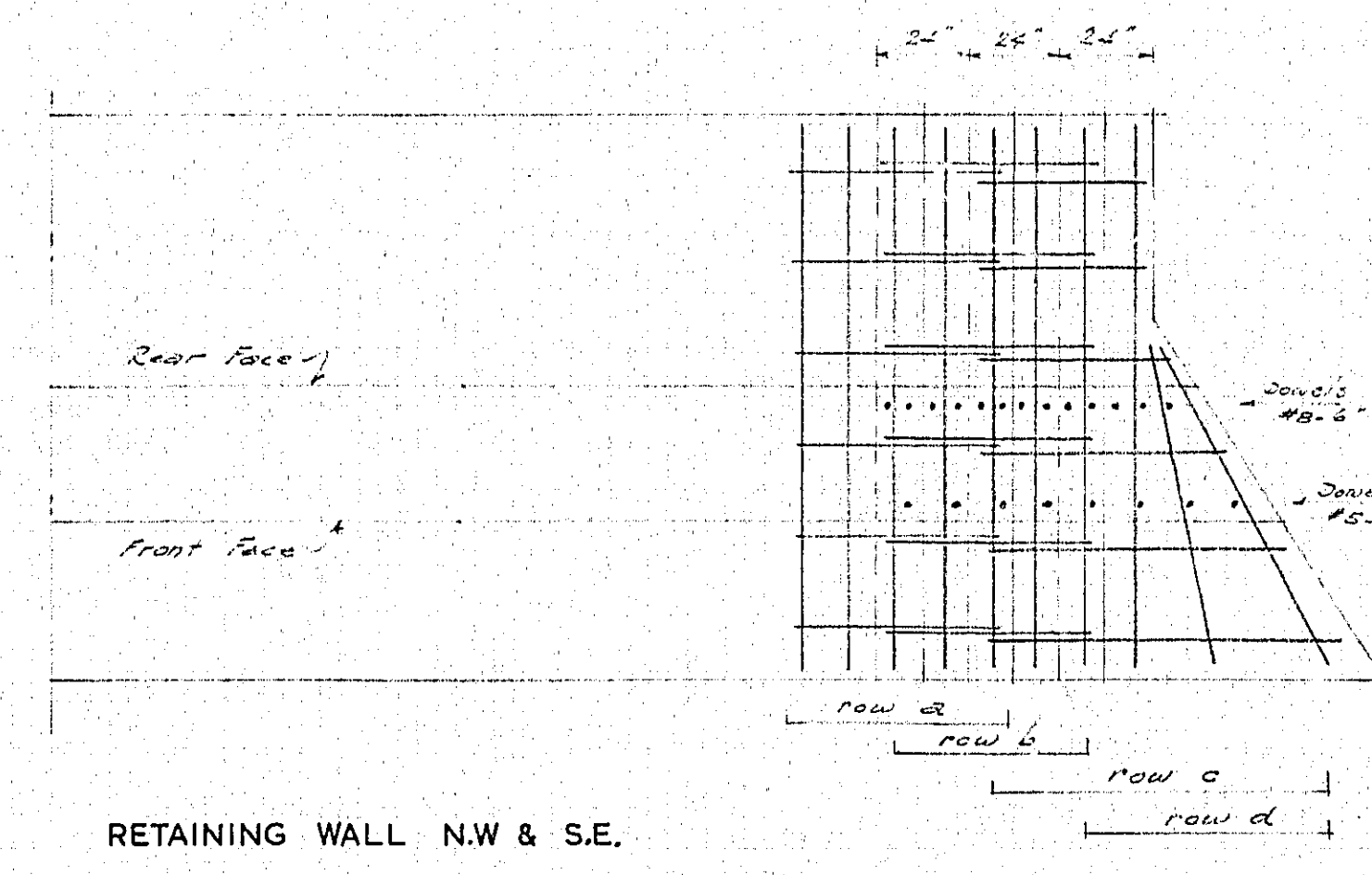
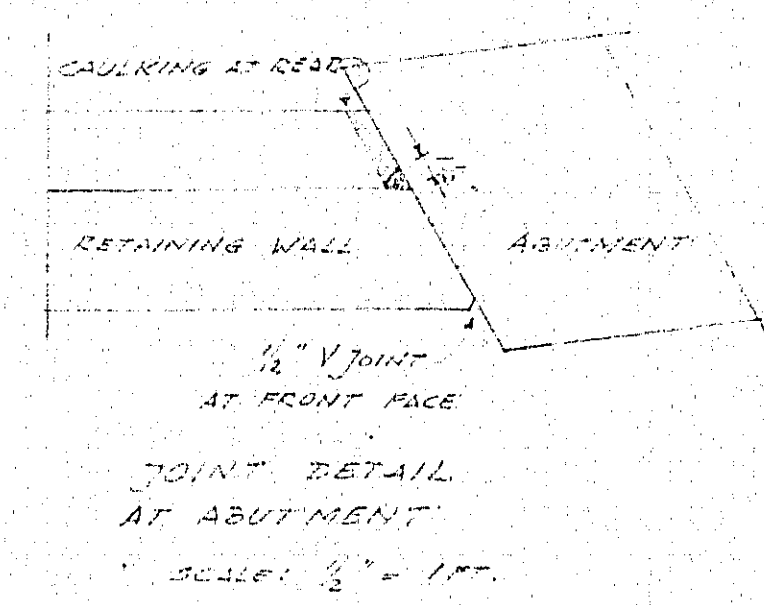
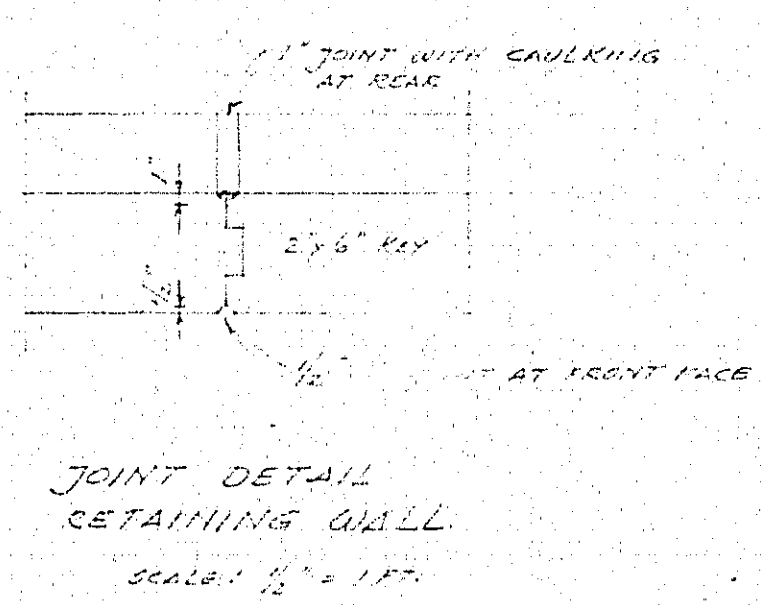
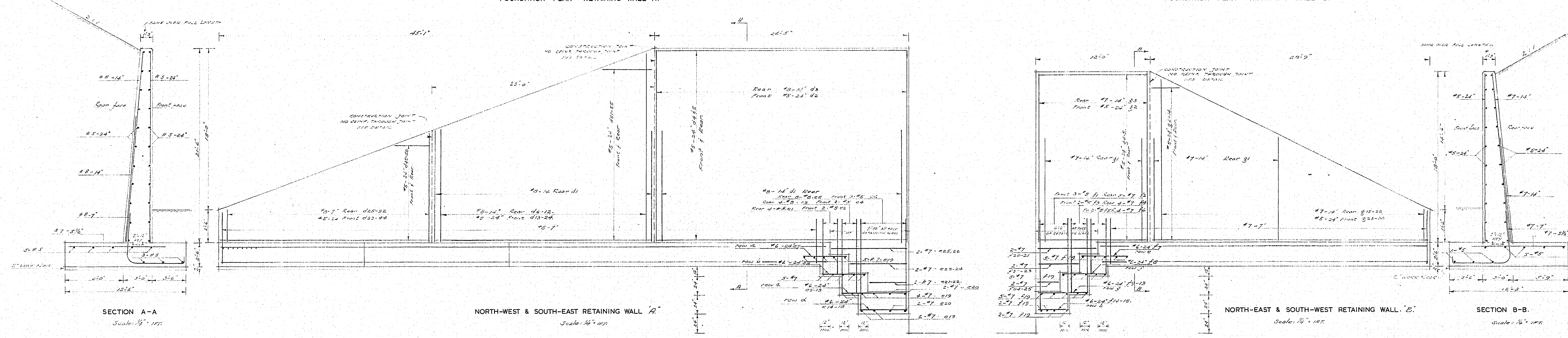
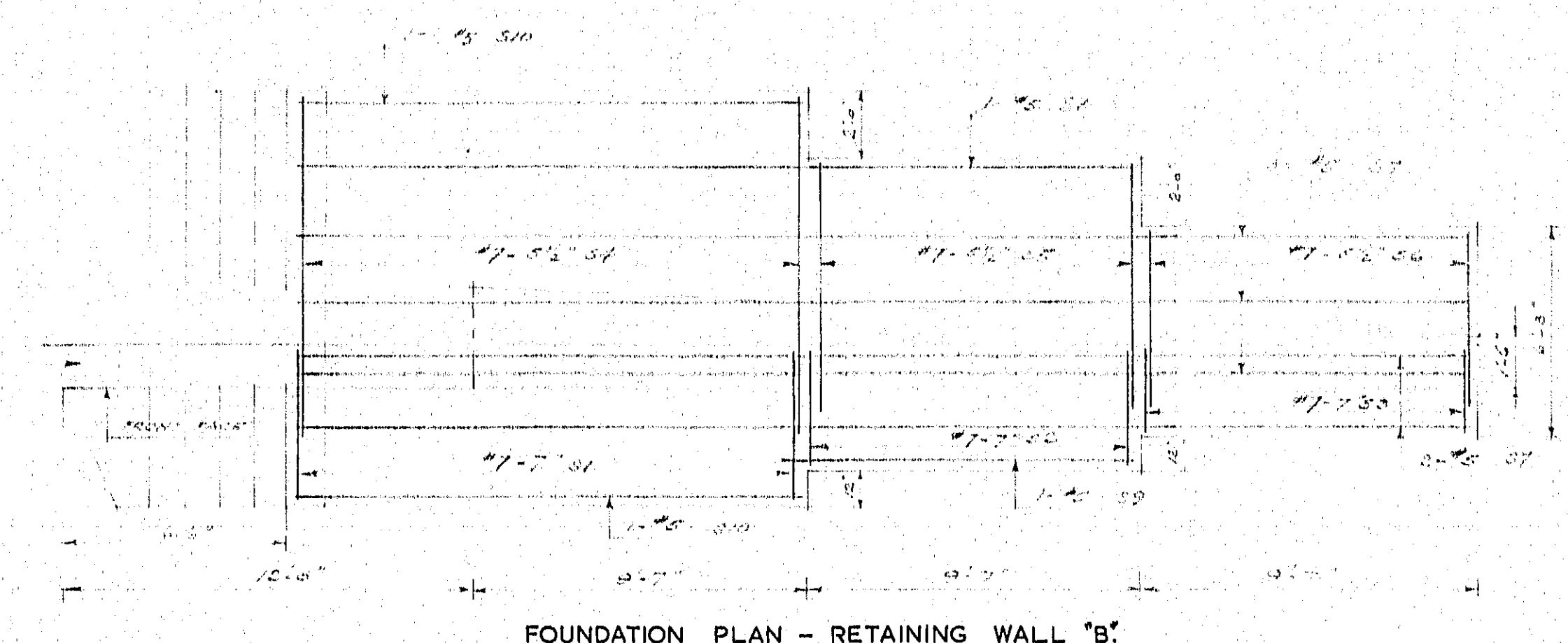
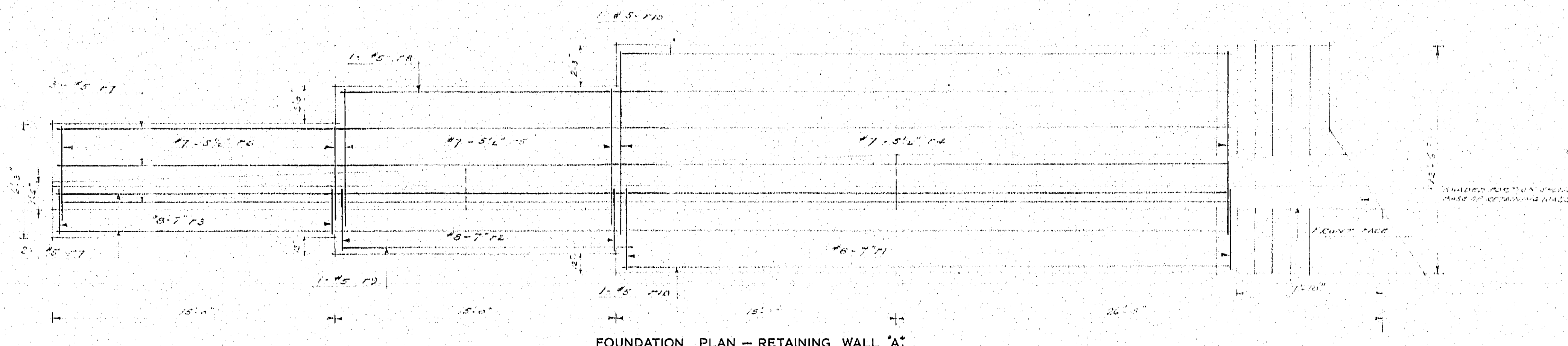
THE KING'S HIGHWAY No. 401 DIST. No. 8
CO. LEEDS NORTH OF BROCKVILLE
TWP. ELIZABETHTOWN LOT 10 CON. 1

FRAME DETAILS.

APPROVED
BRIDGE ENGINEER
DESIGN ENGINEER

DESIGN	CHECK	CONTRACT NUMBER	CANCELLED	57-266
DRAWING	CHECK	H.O.D.	57-266	57-266
TRACING	CHECK	H.O.D.	57-266	57-266
DATE	1957.10.19.57	LOADING	420-5.6	DRAWING NUMBER

TWP. #25-122-4-A



NO.	FOR	DATE
1	APPROVED	3-5-57
2	DESIGNED	3-5-57
3	REVISION	3-5-57

DUNCAN HOPPER & ASSOCIATES
CONSULTING ENGINEERS
1393 WILSON AVENUE,
DOWNSVIEW, ONTARIO

WP 6-56
5-5633-6

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

ELIZABETHTOWN TWP. BRIDGE NO. 12.
C.P.R. OVERHEAD

THE KING'S HIGHWAY No. 401 DIST. No. 8
CO. LEEDS NORTH OF BROCKVILLE
TWP. ELIZABETHTOWN LOT 10 CON. 1

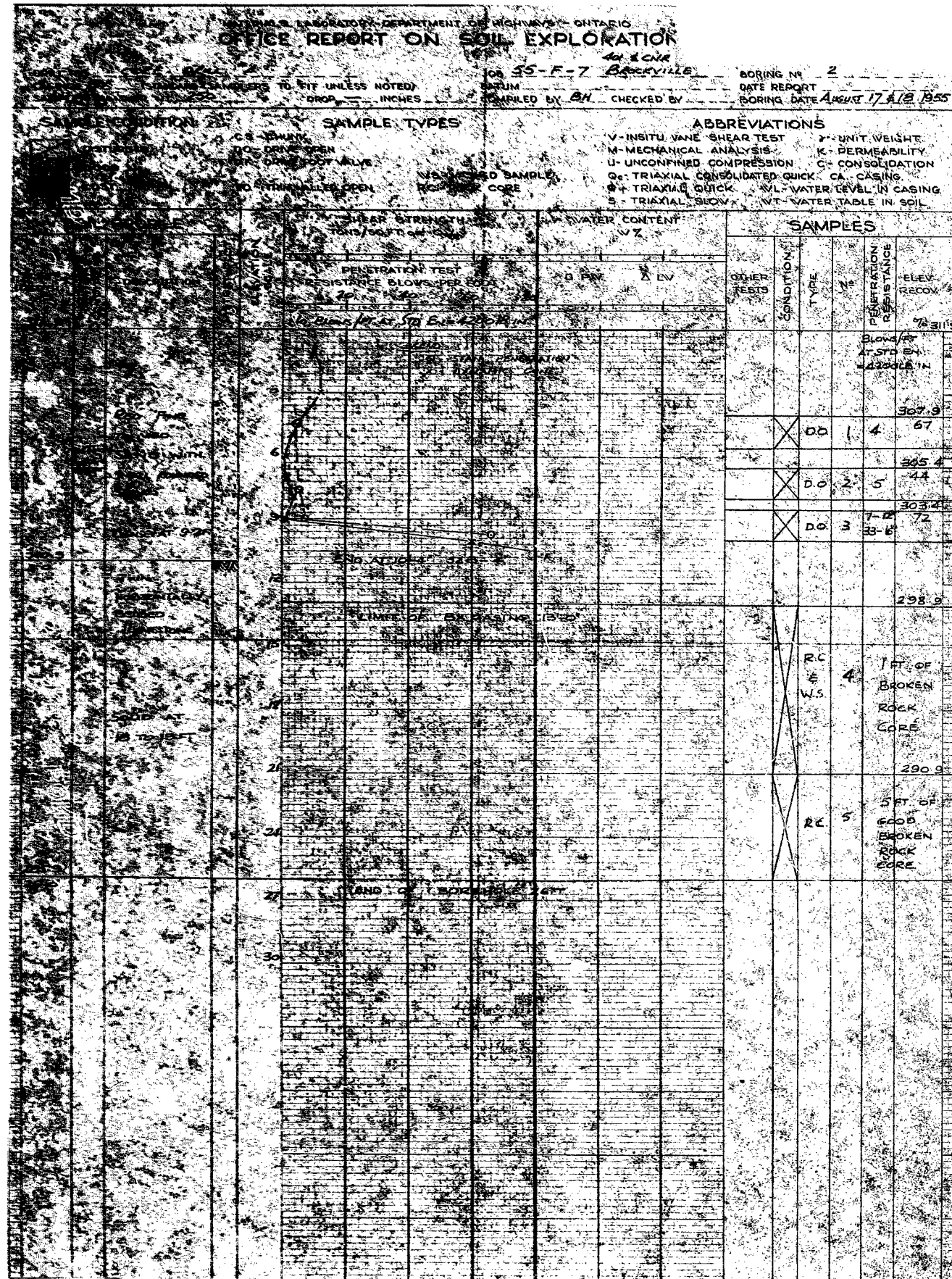
RETAINING WALLS

APPROVED
DESIGN ENGINEER
DESIGN ENGINEER

DESIGN	C.H.H.	CHECK	A.W.D.	CONTRACT	57-205	GENERAL	57-265
DRAWING	C.H.H.	CHECK	D.B.D.	LOADING	420-516	DRAWING	2-3775-5
TRACING	H.C.	CHECK	J.B.H.	DATE	FEB. 18 - 1957		

TWP. 25-122-5-A

**INFORMATION FROM GEOCRES DOCUMENT
NO. 31B00-014**



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD
STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0**

February 2023

APPENDIX C

C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS

C.2 BOREHOLE RECORDS

C.3 ROCK CORE PHOTOGRAPHS



SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<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:

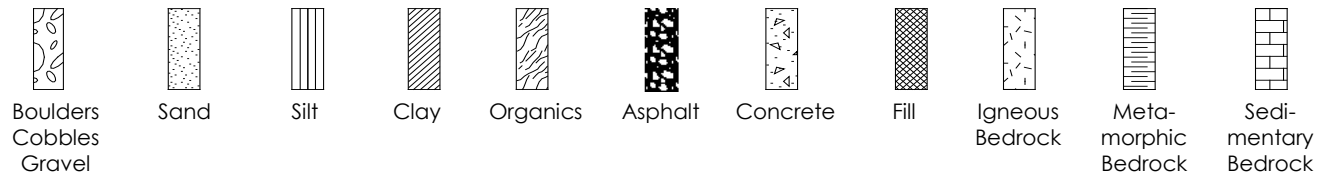
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

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

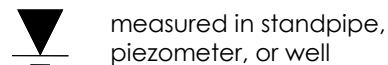
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



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



measured in standpipe, piezometer, or well



inferred

RECOVERY

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

N-VALUE

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

DYNAMIC CONE PENETRATION TEST (DCPT)

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

OTHER TESTS

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

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

RECORD OF BOREHOLE No CP21-1

1 OF 2

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 CPR Overhead - Brockville N: 4940929.0 E: 368840.5 ORIGINATED BY KT
 DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring COMPILED BY RR
 DATUM Geodetic DATE 2021.05.10 - 2021.05.10 LATITUDE 44.605519 LONGITUDE -75.693221 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
104.0 0.0	500 mm ASPHALT						20 40 60 80 100							GR SA SI CL
103.5 0.5	GRAVELLY SAND (SP), trace silt (FILL) Compact Grey Moist		1	SS	27				○					
103.0 1.0	Mixture of SAND, trace gravel (SP) and CLAYEY SILT (CL) (FILL) Brown Moist		2	SS	13					○				
102.5 1.5	SAND and GRAVEL (SP/GP), trace to some silt, (FILL). Contains cobbles and boulders Compact Brown Dry to moist Grinding of augers noted throughout layer.		3	SS	22					○			48	48 3 1
			4	SS	10					○				
			5	SS	11					○				
100.3 3.7	COBBLES and BOULDERS in a matrix of SANDY GRAVEL (FILL) Loose to compact Brown Dry to moist Minimal to no sample recovery for samples 6 to 12.		6	SS	8									No Recovery
			7	SS	12					○			69	27 3 1
			8	SS	4									No Recovery
			9	SS	16					○				
			10	SS	7									No Recovery
			11	SS	6					○				
			12	SS	20					○				
94.9 9.1	Sandy CLAYEY SILT (CL), trace to some gravel (TILL) Firm to stiff Grey Moist Topsoil in top of sample 13		13	SS	10								4	40 43 13
94.0														

Continued Next Page

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




ONTARIO MTO 165001160_Hwy 401_BROCKVILLE.GPJ ONTARIO MTO.GDT 5/16/22

RECORD OF BOREHOLE No CP21-1

2 OF 2

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 CPR Overhead - Brockville N: 4940929.0 E: 368840.5 ORIGINATED BY KT
 DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger + NQ Rock Coring COMPILED BY RR
 DATUM Geodetic DATE 2021.05.10 - 2021.05.10 LATITUDE 44.605519 LONGITUDE -75.693221 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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×					
							20	40	60	80	100		20	40	60		
10.0	Sandy CLAYEY SILT (CL), trace to some gravel (TILL) Very stiff Grey Moist																
			14	SS	29												
92.2	Auger Refusal at 11.8 m		15	NQ	-												
11.8	DOLOSTONE Light grey to grey Fair quality (very poor quality in top 0.2 m) Fresh to slightly weathered Very strong to extremely strong																
			16	NQ	-												

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

RECORD OF BOREHOLE No CP21-2

1 OF 2

METRIC

W.P. GWP 4003-19-00 LOCATION Highway 401 CPR Overhead - Brockville N: 4940927.7 E: 368872.6 ORIGINATED BY KT
 DIST East HWY HWY 401 BOREHOLE TYPE Hollow Stem Auger, Casing Advanced below 1.5 m + NQ Rock Coring COMPILED BY RR
 DATUM Geodetic DATE 2021.05.12 - 2021.05.12 LATITUDE 44.605504 LONGITUDE -75.692818 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 ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE	WATER CONTENT (%)							
104.0 0.0	500 mm ASPHALT						20	40	60	80	100		20	40	60		GR SA SI CL
103.5 0.5	SAND and GRAVEL (SP/GP), trace silt (FILL) Very dense Brown		1	SS	66												
103.2 0.8	SAND (SP), trace gravel (FILL) Very dense Dry		2	SS	50/ 25mm												
102.5 1.5	GRAVEL, COBBLES, and BOULDERS (ROCKFILL) Grey Dry		3	SS	50/ 50mm												UCS=147.5 MPa
101.7 2.3	Gravelly SILTY SAND to Silty SAND and GRAVEL (SM), trace clay (FILL). Contains cobbles and boulders. Loose to compact Moist		4	SS	14												No Recovery
			5	SS	50/ 25mm												
			6	SS	6												35 36 22 7
			7	SS	4												
			8	SS	5												No Recovery
97.9 6.1	GRAVEL, COBBLES and BOULDERS in a matrix of Silty SAND (ROCKFILL) Loose to compact Grey Moist		9	SS	8												UCS=116.3 MPa
			10	SS	22												
95.3 8.7	SILTY SAND, some gravel to SAND and GRAVEL (SM to GP), trace organic matter Contains cobbles Compact Brown Wet		11	SS	21												Bit jammed; removed and reinstalled casing
																	52 36 10 0

Continued Next Page

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

ONTARIO MTO 165001160 HWY 401 BROCKVILLE GPJ ONTARIO MTO.GDT 5/16/22

METRIC

[illegible]

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



Project No.: 165001160

Project Name: Hwy 401 Brockville

Rock Core
Photographs



Rock Core Photo No.: 1

Borehole: CP21-1

Depth: 11.8 m to 15.1 m



Rock Core Photo No.: 2

Borehole: CP21-2

Depth: 12.3 m to 18.4 m

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD
STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0**

February 2023

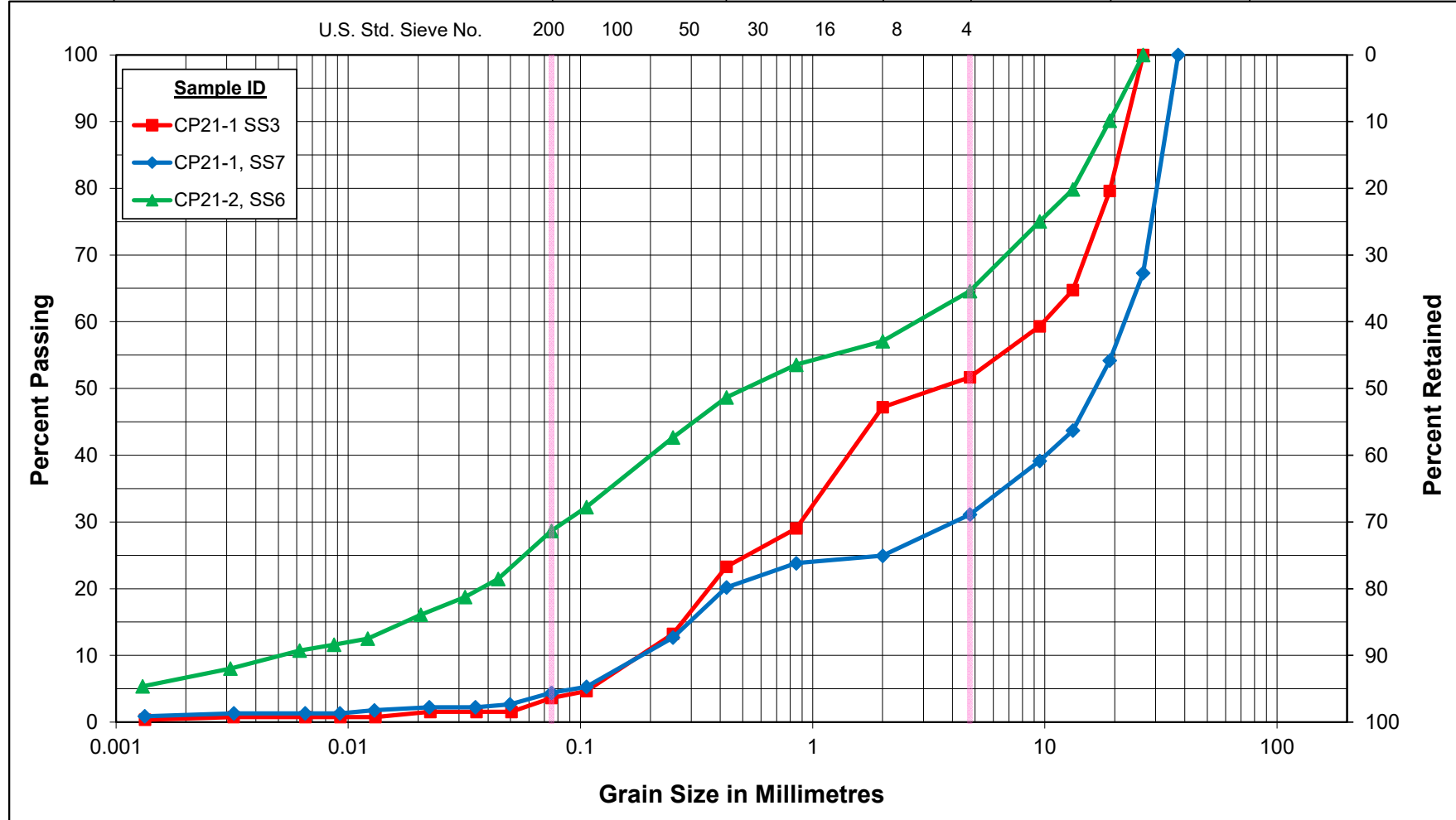
APPENDIX D

D.1 LABORATORY TEST RESULTS



Unified Soil Classification System

			SAND			Gravel	
CLAY & SILT			Fine	Medium	Coarse	Fine	Coarse



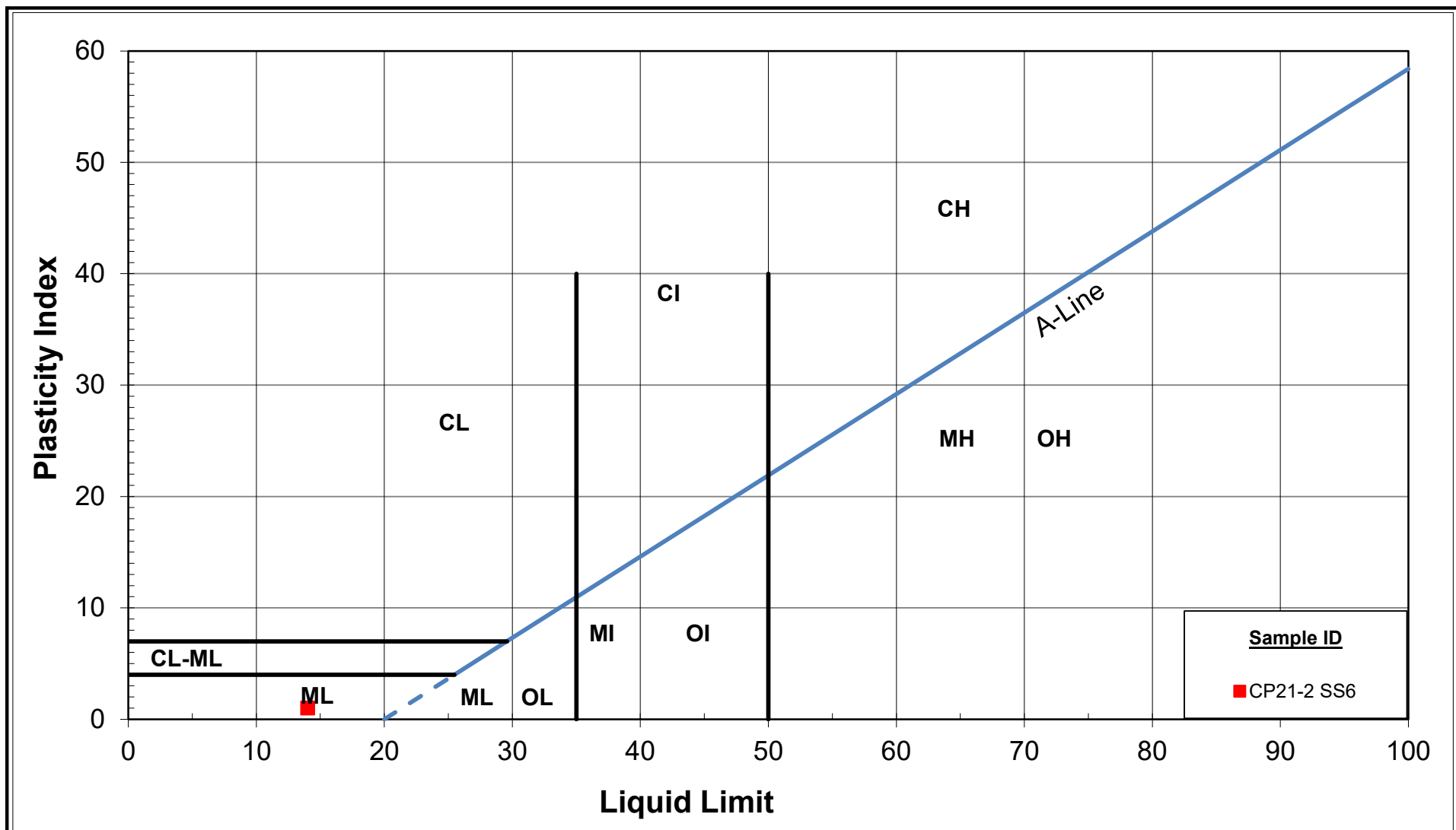
GRAIN SIZE DISTRIBUTION

FILL: Silty SAND and GRAVEL (SM) to SANDY GRAVEL (GP)

Hwy 401 Brockville - CPR Overhead

Figure No. D1

Project No. 165001160



FILL: Silty SAND and GRAVEL (SM)
Hwy 401 Brockville - CPR Overhead
PLASTICITY CHART

Figure No. D2

Project No. 165001160

	SAND			Gravel	
CLAY & SILT	Fine	Medium	Coarse	Fine	Coarse



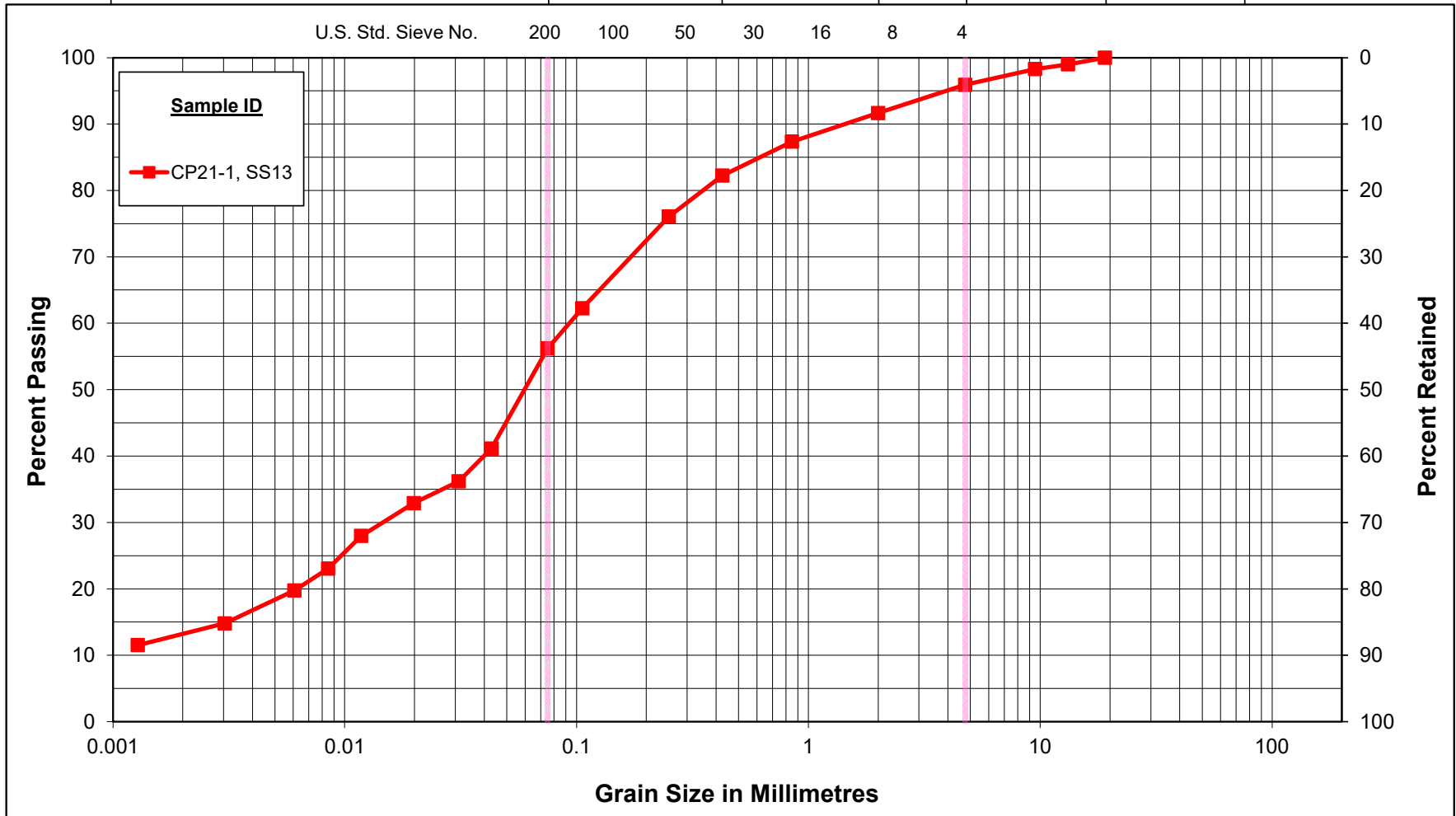
SAND and GRAVEL (GP)

Figure No. D3

Project No. 165001160

Unified Soil Classification System

CLAY & SILT		SAND			Gravel	
		Fine	Medium	Coarse	Fine	Coarse



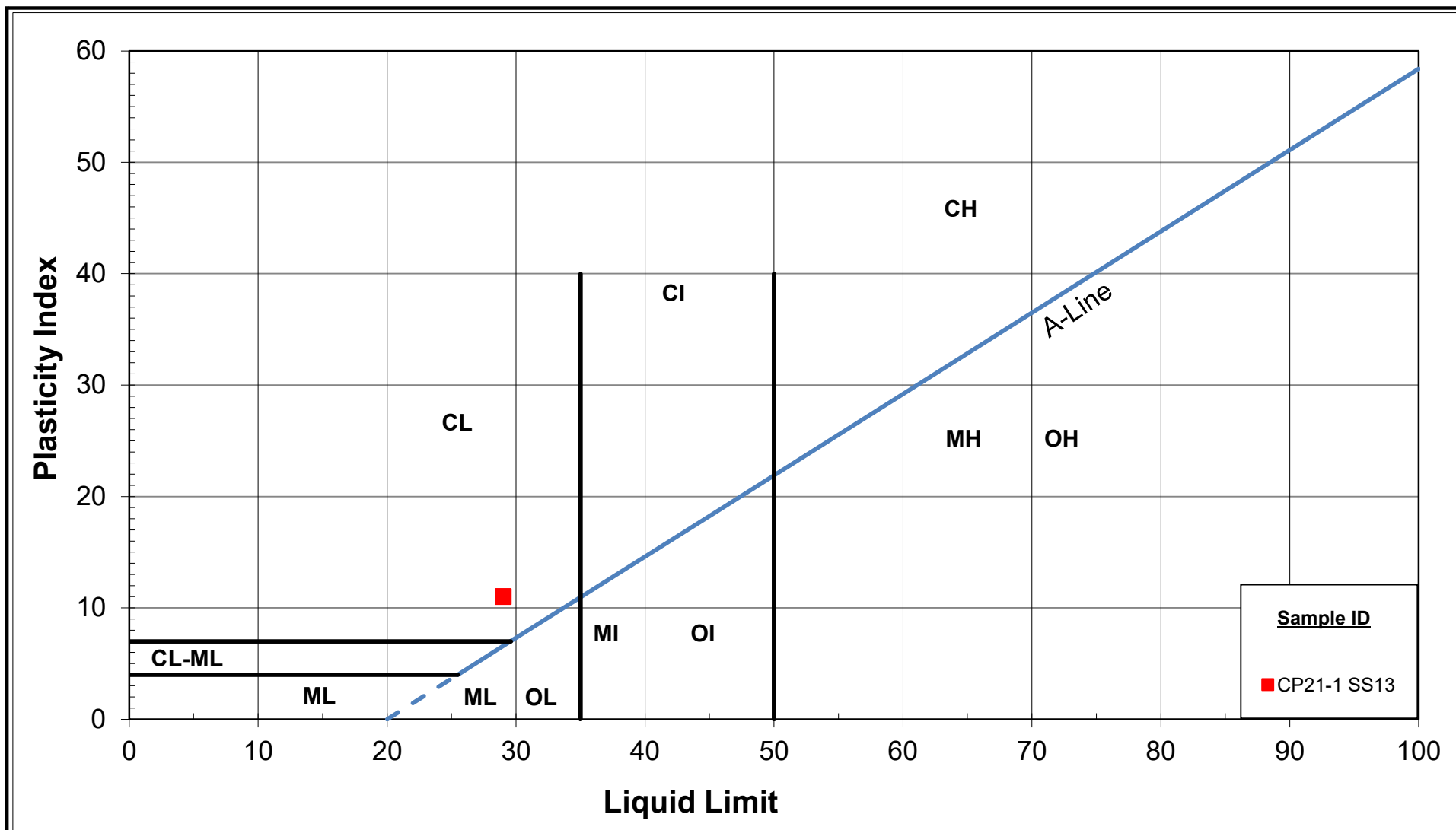
GRAIN SIZE DISTRIBUTION

TILL: SANDY CLAYEY SILT (CL)

Hwy 401 Brockville - CPR Overhead

Figure No. D4

Project No. 165001160



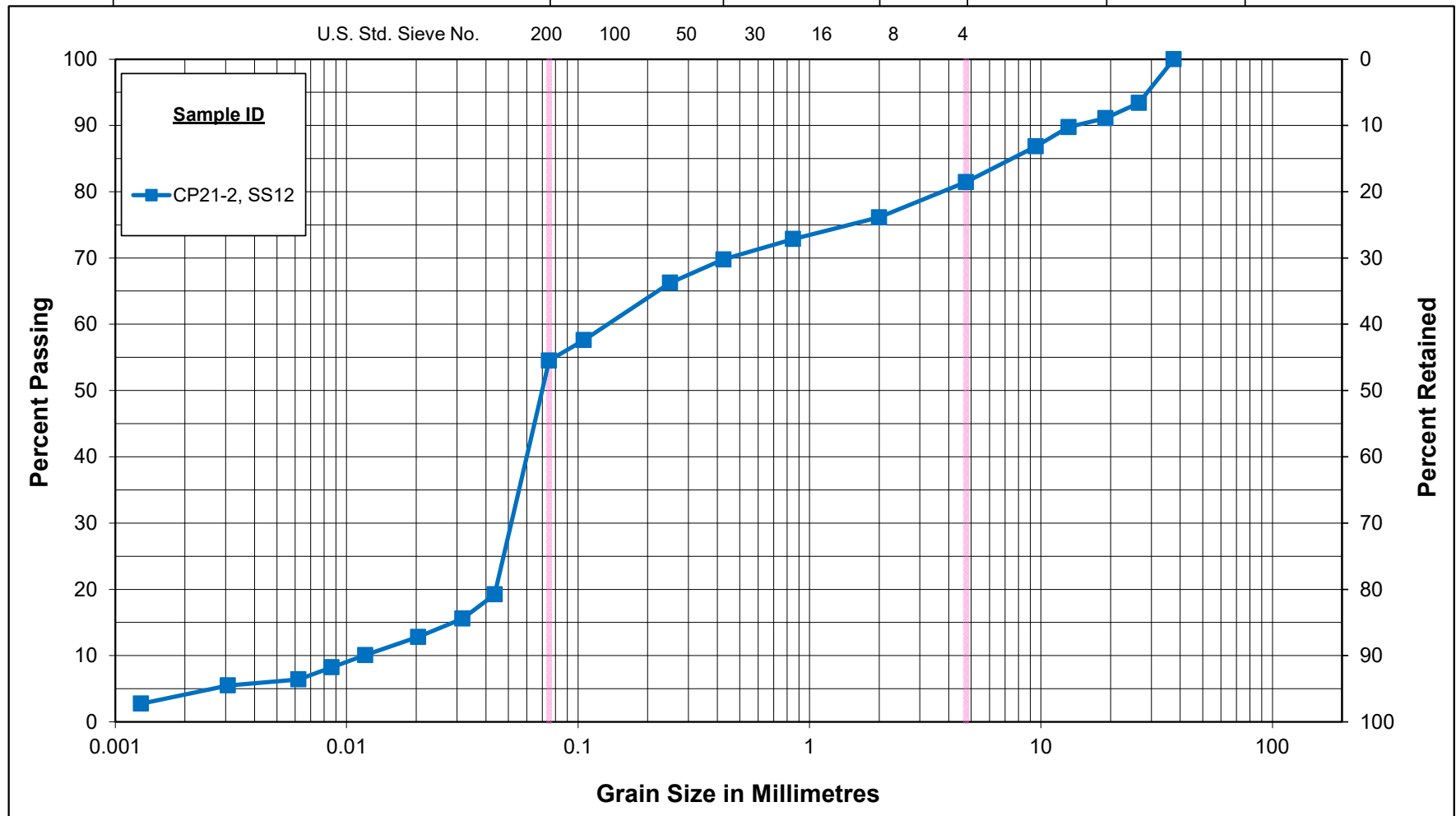
TILL: Sandy CLAYEY SILT (CL)
Hwy 401 Brockville - CPR Overhead
PLASTICITY CHART

Figure No. D5

Project No. 165001160

Unified Soil Classification System

CLAY & SILT		SAND			Gravel	
		Fine	Medium	Coarse	Fine	Coarse



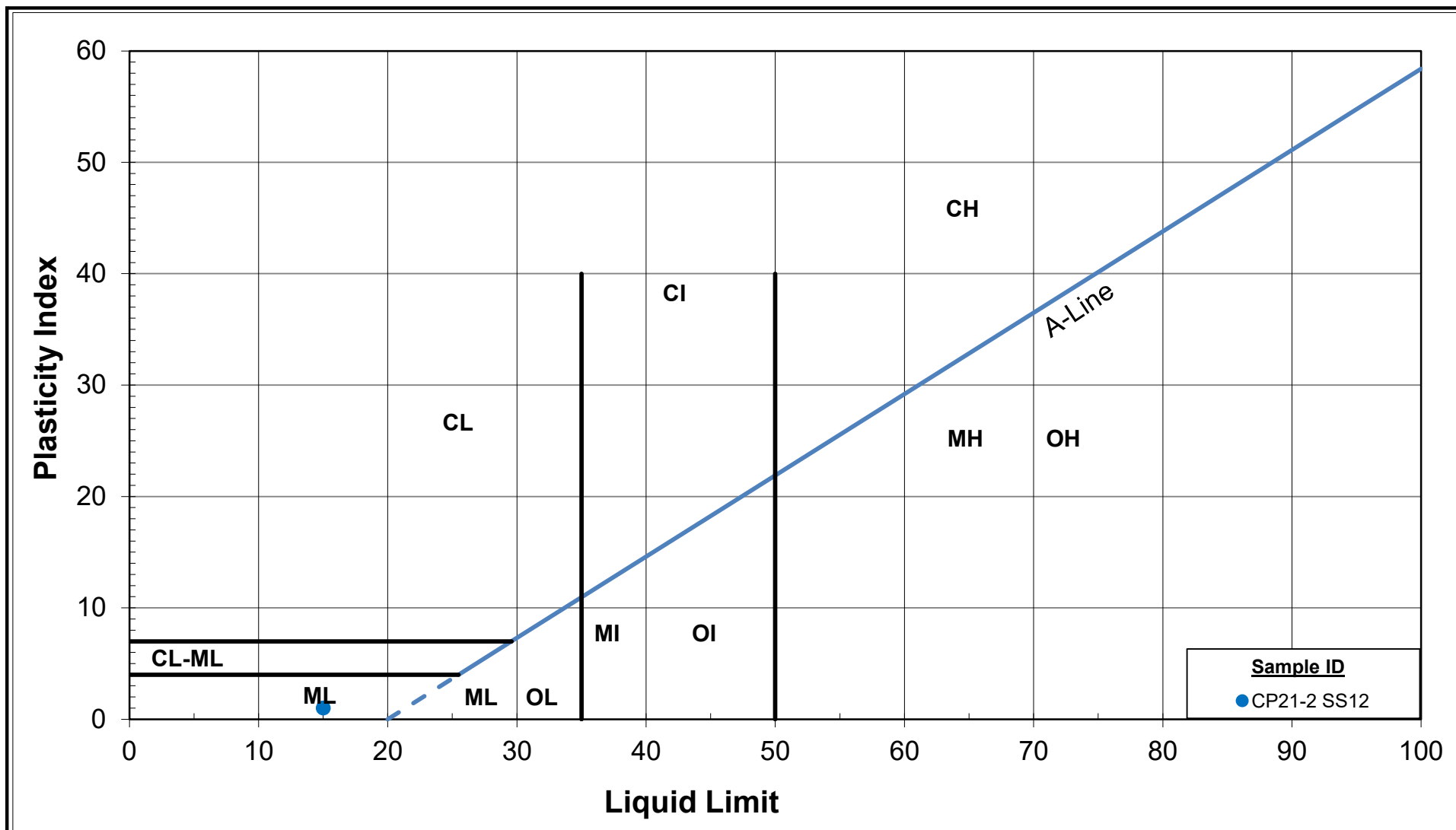
GRAIN SIZE DISTRIBUTION

TILL: SANDY SILT, some gravel (ML)

Hwy 401 Brockville - CPR Overhead

Figure No. D6

Project No. 165001160



TILL: Sandy SILT, some gravel (ML)
Highway 401 Brockville - CPR Overhead
PLASTICITY CHART

Figure No. D7

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
----------	--------------	------	------	------	------

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]	-	-

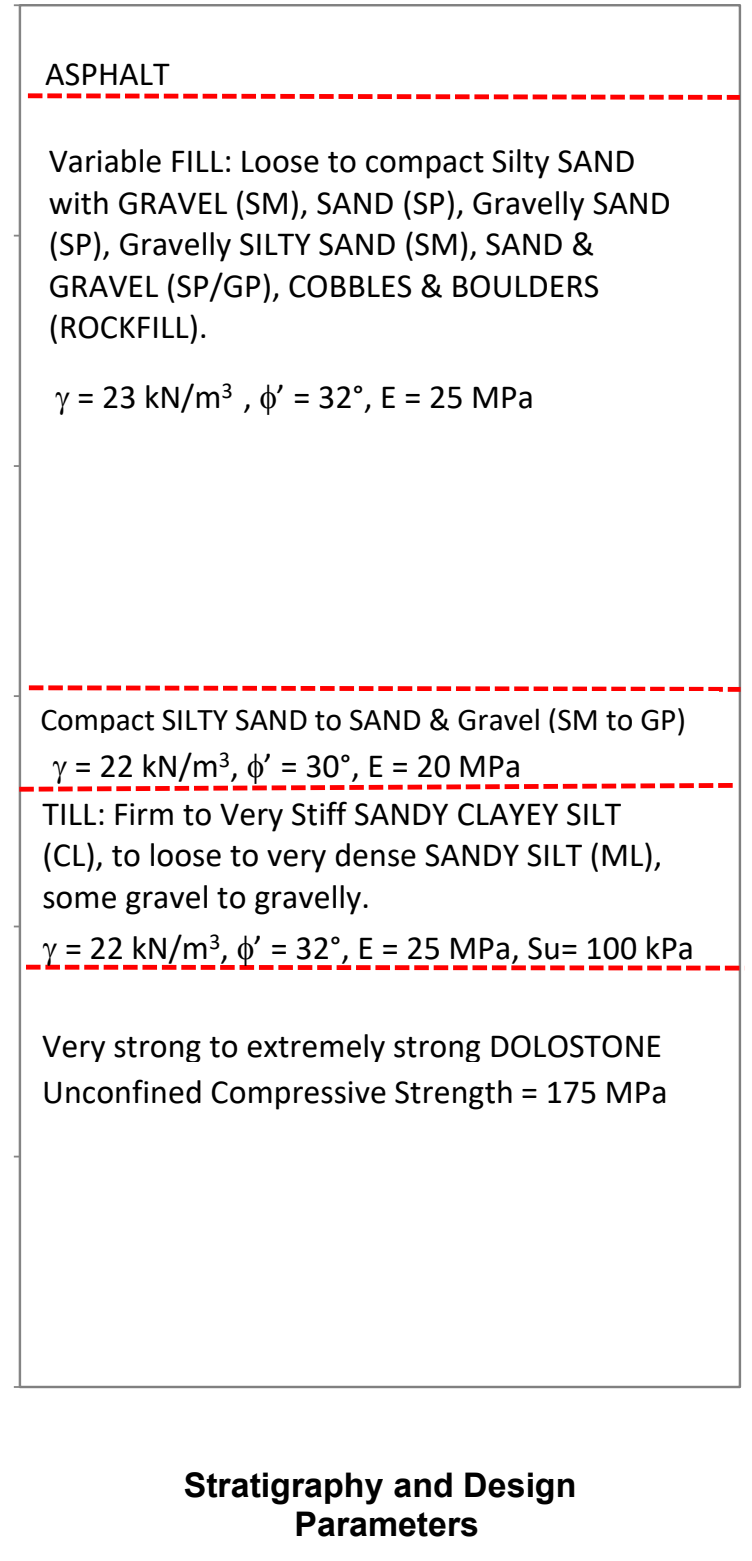
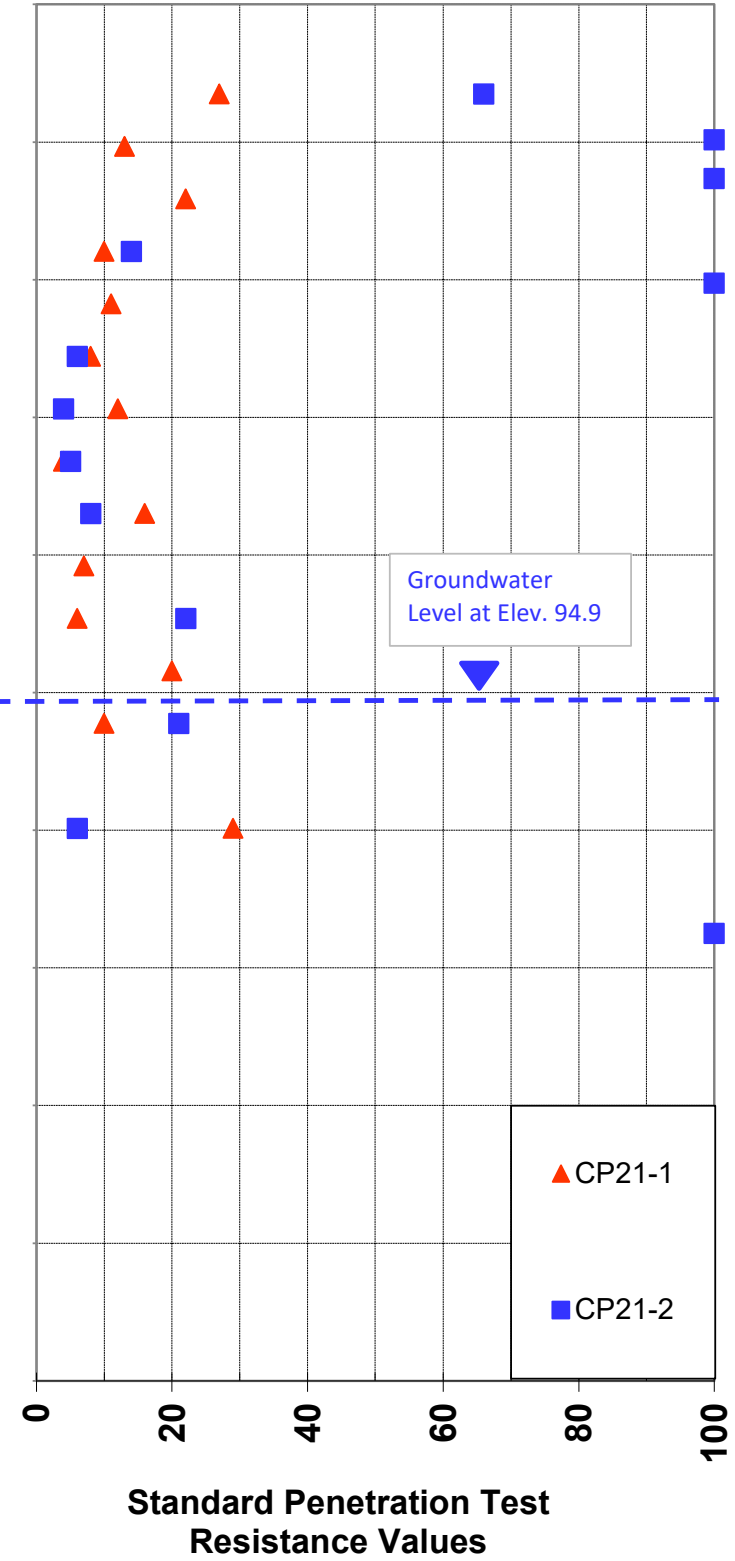
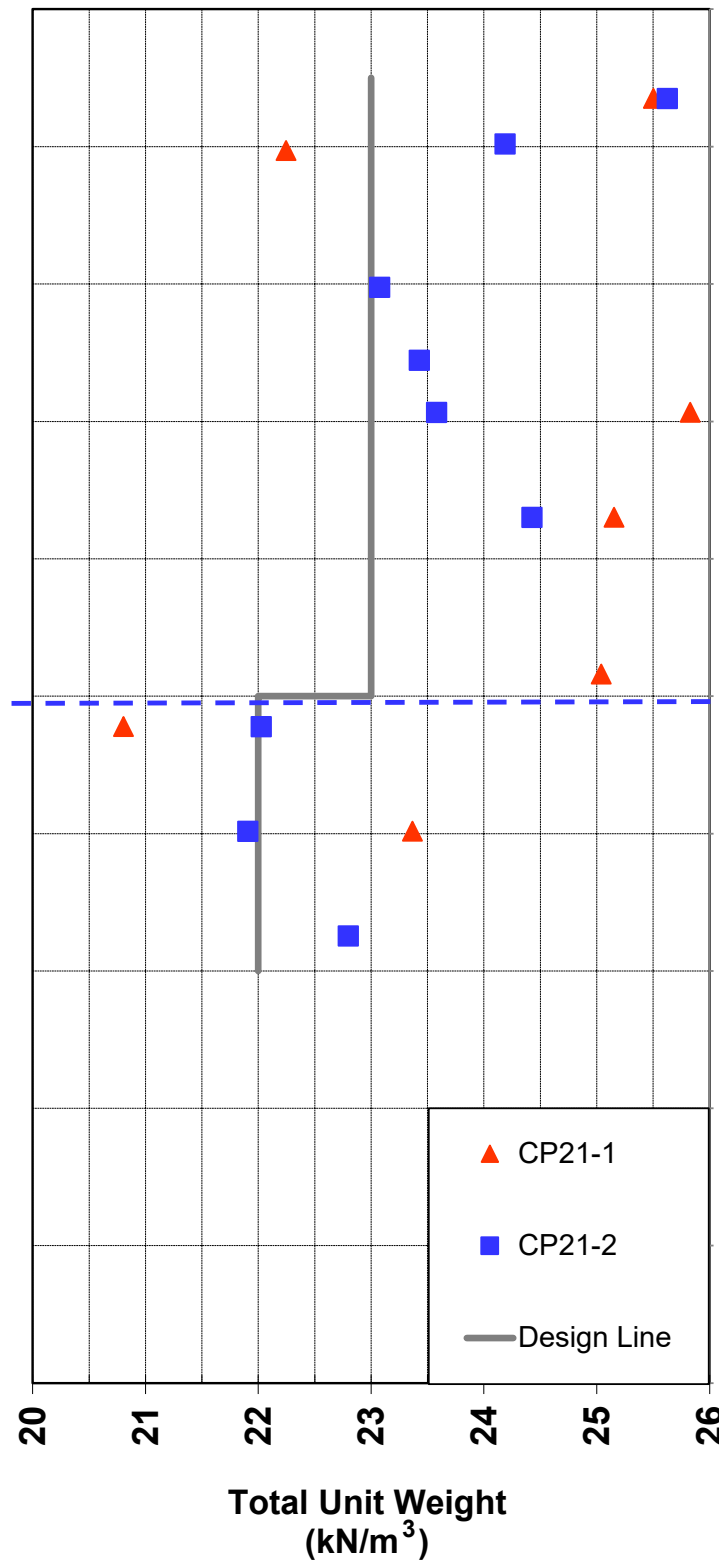
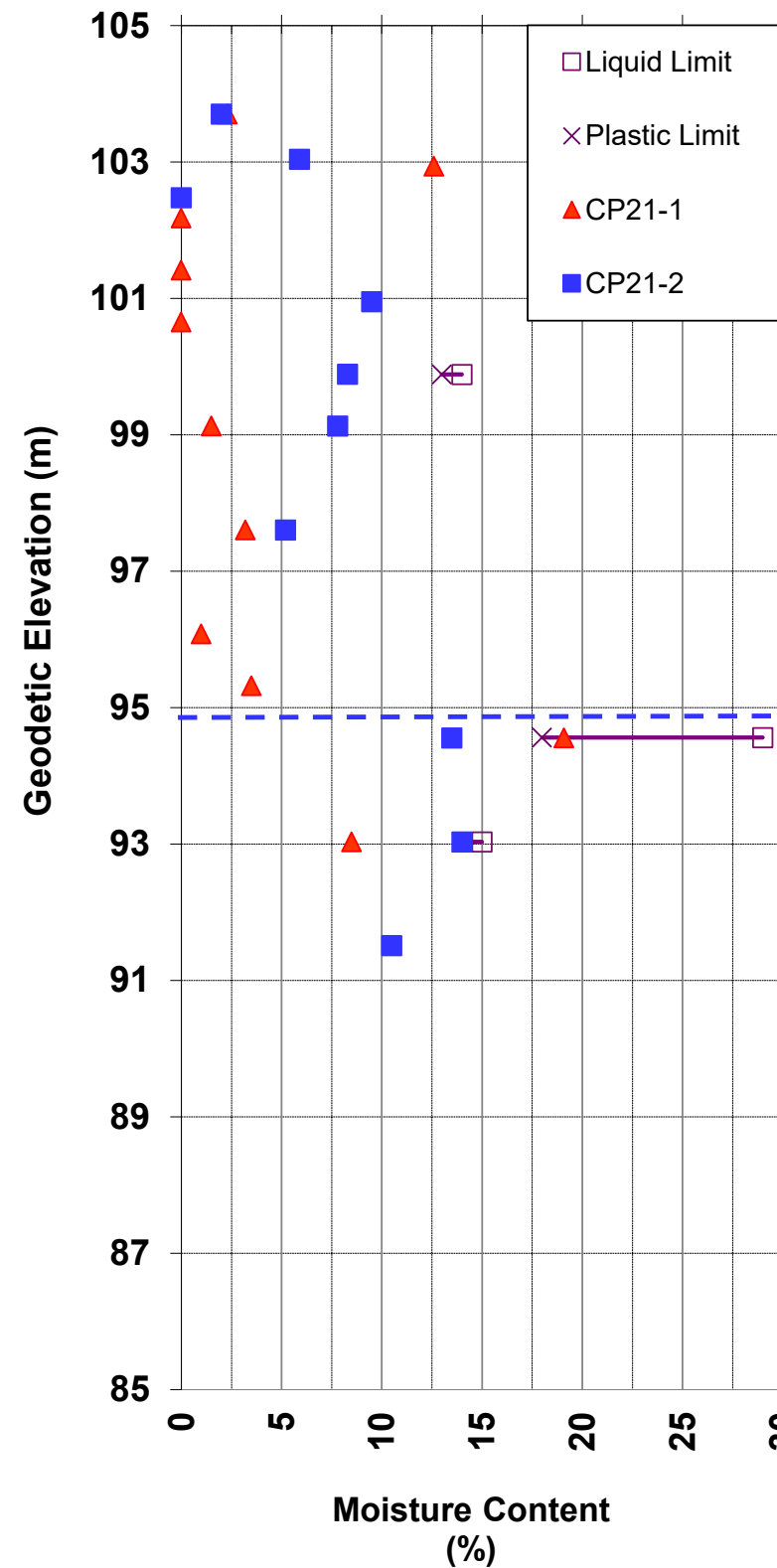
**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD
STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0**

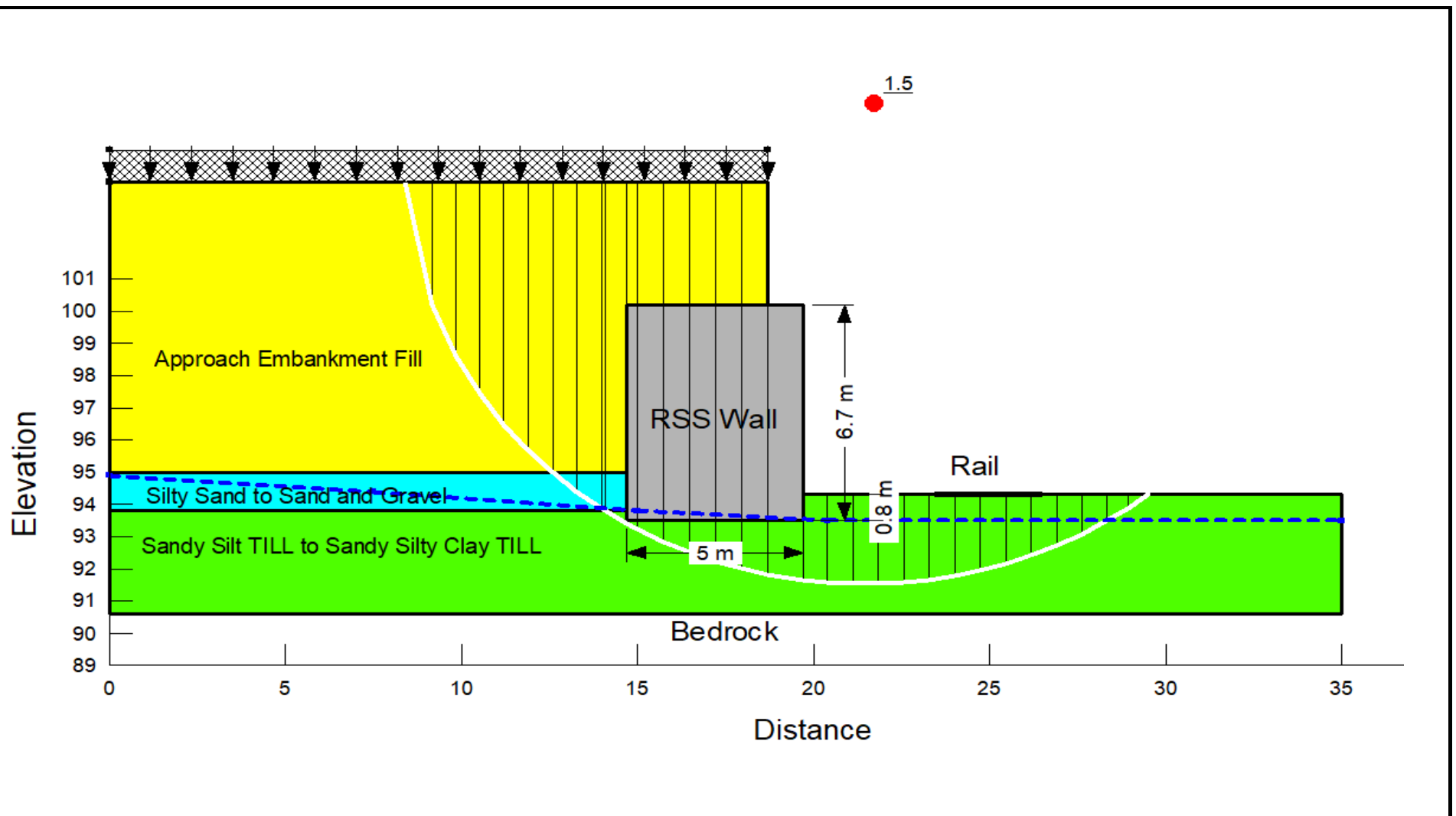
February 2023

APPENDIX E

E.1 DRAWINGS E1 TO E5 - GEOTECHNICAL SOIL MODEL AND SLOPE STABILITY ANALYSES







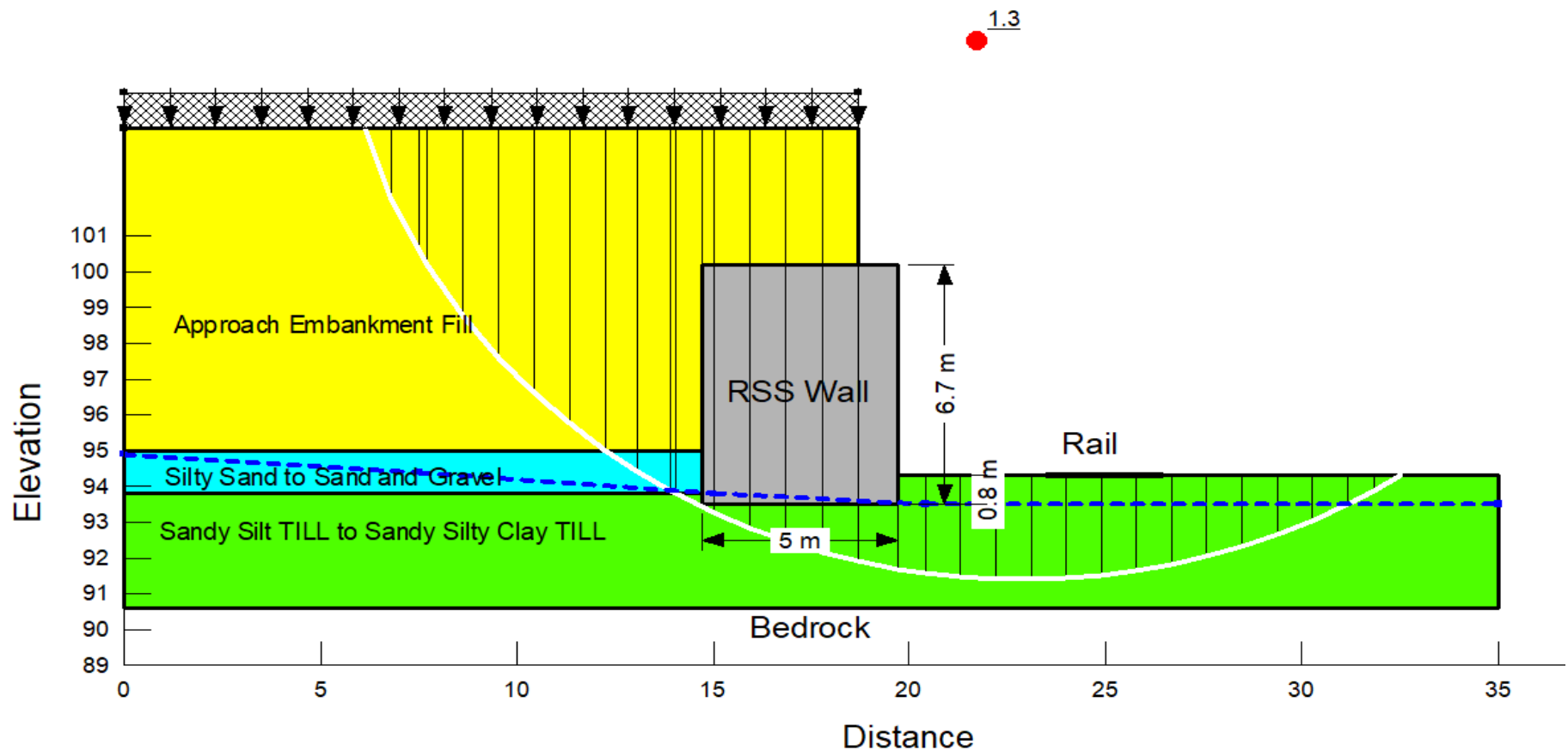
Slope Stability Analysis (Static) False Abutment Configuration with RSS Wall

Highway 401 Reconstruction, CPR Overhead

Figure E2

Project No. 165001160

GWP No. 4003-19-00



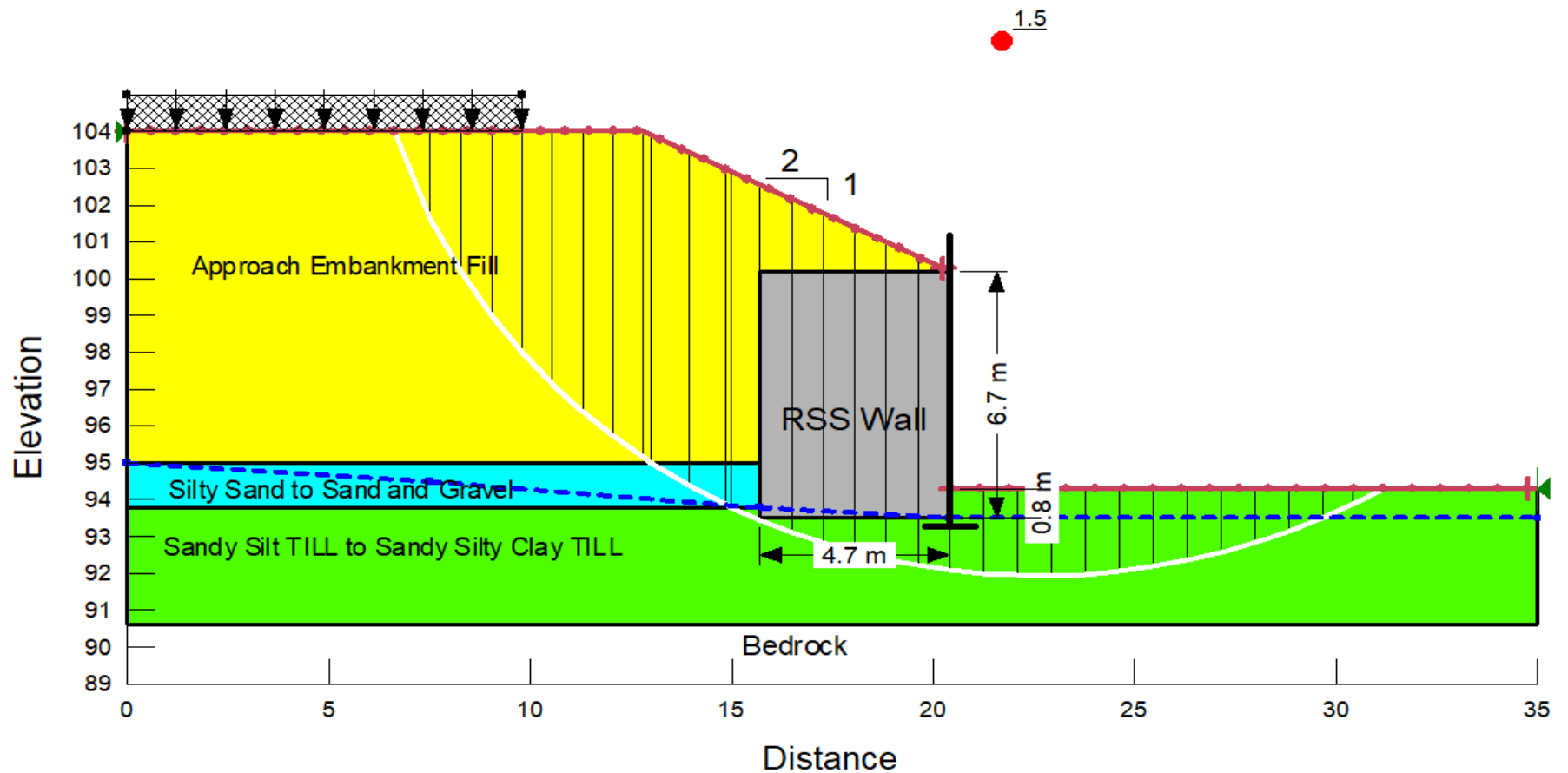
Slope Stability Analysis, (Seismic, $k_h = 0.083$)
False Abutment Configuration with RSS Wall

Highway 401 Reconstruction, CPR Overhead

Figure E3

Project No. 165001160

GWP No. 4003-19-00

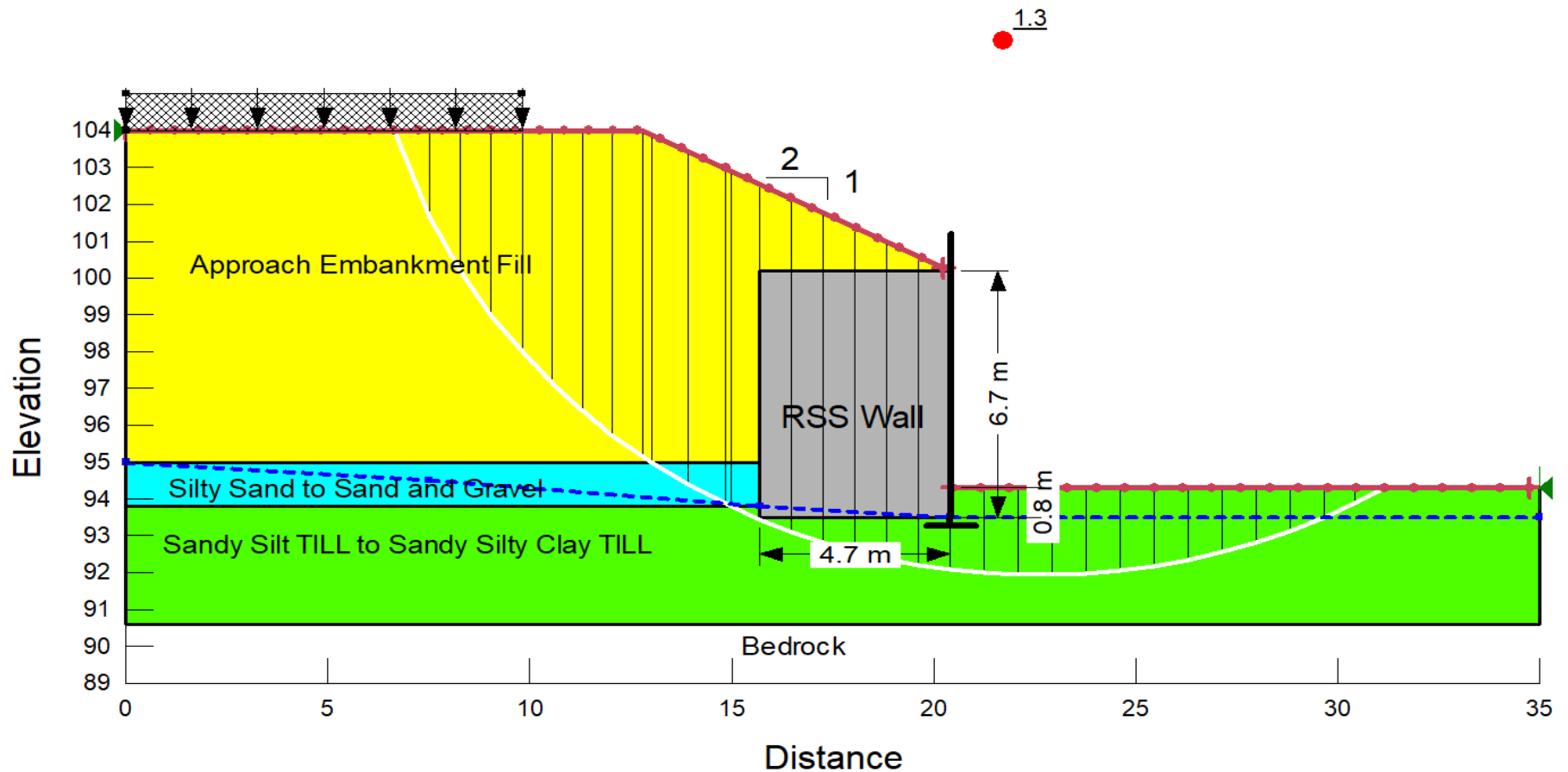


Slope Stability Analysis (Static)
Approach embankment with RSS Wall
Highway 401 Reconstruction, CPR Overhead

Figure E4

Project No. 165001160

GWP No. 4003-19-00



Slope Stability Analysis, (Seismic, $k_h = 0.083$)
Approach Embankment with RSS Wall

Highway 401 Reconstruction, CPR Overhead

Figure E5

Project No. 165001160

GWP No. 4003-19-00

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT - CPR (VIA) OVERHEAD
STRUCTURE REPLACEMENT – SITE NO. 16X-0122/B0**

February 2023

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: 44.606N 75.693W

User File Reference: Highway 401 - CP Rail Crossing - Brockville

2022-01-14 14:01 UT

Requested by: Stantec

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.237	0.145	0.094	0.031
Sa (0.1)	0.294	0.187	0.124	0.044
Sa (0.2)	0.262	0.168	0.113	0.042
Sa (0.3)	0.209	0.135	0.091	0.034
Sa (0.5)	0.158	0.101	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	0.104	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