Foundation Investigation and Design Report - Roseville Road Underpass Replacement, Hwy 401, Township of North Dumfries, ON

MTO Site No. 33X-0177/B0
Latitude 43.3514
Longitude - 80.4167
G.W.P. 3204-16-00

Geocres No. 40P8-272

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Prepared for:

Ministry of Transportation Ontario

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FOUNDATION INVESTIGATION REPORT

For

G.W.P 3204-16-00

Roseville Road Underpass Replacement, Highway 401
Site No. 33X-0177/B0

Township of North Dumfries, ON

1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation of Ontario (MTO) to undertake a foundation investigation to support the detailed design of the replacement of the existing Roseville Road Underpass at Highway 401 (Site No. 33X-0177/B0). The bridge is located in the Township of North Dumfries, Ontario approximately 8 km west of the City of Cambridge.

The project involves the replacement of the existing two-lane, four-span underpass on the same alignment as the existing bridge. The new underpass, which is being designed to accommodate the future widening of Highway 401 to a ten-lane configuration, will consist of a three-span structure that is longer and slightly wider than the existing bridge. An existing structural culvert (Site No. 33X-0421/C0) that currently passes beneath the Roseville Road embankment immediately south of the existing south abutment of the underpass will be removed to create an open creek channel beneath the southernmost span of the new bridge.

The purpose of the foundation investigation was to assess the subsurface conditions at the site of the bridge replacement by drilling 12 boreholes and carrying out in-situ testing to supplement existing borehole information and completing a laboratory testing program on selected soil samples obtained from the boreholes. Geophysical testing was also completed on both sides of Highway 401 to measure the shear wave velocities of the soil strata.

This Foundation Investigation and Design Report (FIDR) has been prepared specifically and solely for the proposed bridge replacement project described above.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 SITE LOCATION

The existing bridge is located on Highway 401 at Station 18+035, approximately 8 km west of the City of Cambridge, in the Township of North Dumfries, Ontario. The location is shown on the Key Plan inset to the Borehole Locations & Soil Strata Plans, Drawing Nos. 1 to 2 in Appendix A.

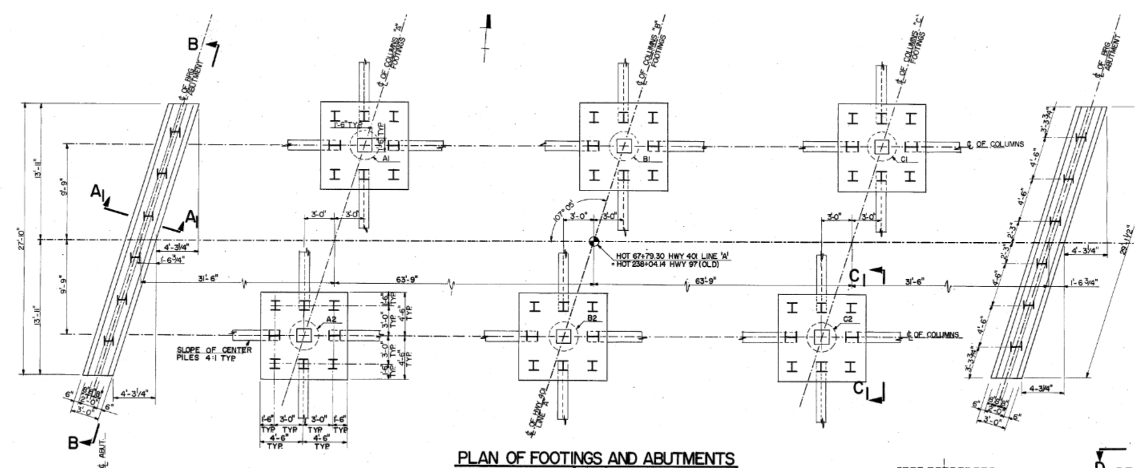
2.2 SITE DESCRIPTION

At the location of the bridge, Highway 401 is oriented approximately north-south while Roseville Road is oriented approximately east-west. However, Highway 401 is classified as an east-west highway. Therefore, for the purposes of this report, the bridge will be referenced as being oriented north-south.

At the location of the bridge, Highway 401 is a six-lane divided highway and Roseville Road (also known as County Road 46) is a two-lane undivided roadway. The existing Roseville Road underpass is a four-span structure constructed circa 1960 and is approximately 58 m long and 10.3 m wide. The bridge is on a 17.1° skew to the orientation of the highway.

The existing Roseville Road surface is at an elevation of just below 311 m and the asphalt surface on Highway 401 is at an elevation of approximately 304 m to 304.5 m. The original ground surface adjacent the bridge and approach embankments was at an elevation of about 303 m to 304.5 m, similar to the current elevation of Highway 401. The existing approach embankment has maximum heights of about 6 m to 7 m and sideslopes in the range of 2 to 2.5 horizontal to 1 vertical (2 to 2.5H:1V).

The available structural drawings for the existing underpass indicate the bridge abutment and pier foundations are supported on 12 BP 53 (HP310x79) steel H-Piles. The abutments are each supported on a single row of 6 piles. Each pier is supported on 2 pile caps that each contain 8 piles (e.g. resulting in a total of 16 piles per pier); half of the pier piles are inclined/battered. An extract from the structural drawings is provided below for reference.



Section A_1- $A_{1\ Typ.}$ contained on the 'Layout & Reinforcement of Footings, Abutments & Columns, Boreholes and Piles' drawing from the original 1958 structural drawings for the bridge identifies piles for the north abutment would have an approximate tip elevation of 970 ft (~295.6 m); no information is provided for the tip elevations of other piles.

The structural culvert at Site No. 33X-0421/C0conveys water from the Cedar Creek channel beneath the south approach embankment to the underpass. Flow in the creek is from east to west. The creek merges with another arm of the creek about 15 m west of the west end of the culvert.

A gas pipeline is located near the west toe of the south approach embankment; this pipeline crosses the creek near the west end of the culvert.

2.3 PHYSIOGRAPHIC DESCRIPTION

The site is located within a physiographic region known as the Guelph Drumlin Field (Chapman and Putnam, 1984). The prevailing soils in the vicinity of the site are predominantly granular in nature consisting of sands and gravels with some silts and stoney tills which contain cobbles and boulders.

Nearby water well records encountered limestone and/or shale bedrock at depths typically in excess of 40 m to 50 m below ground surface.

In the vicinity of the project site, the terrain is generally undulating to gently sloping with a regional slope towards the Grand River to the northeast of the site.

3.0 INVESTIGATION PROCEDURES

3.1 REVIEW OF EXISTING INVESTIGATION

Subsurface information from a foundation investigation that was carried out to support the design of the existing bridge was available from the following document:

- Report titled "Subsoil Investigation of Site of Proposed Highway 401 and Old Highway 97 Grade Separation, N. Dumfries Township, Near Galt" prepared by the Dominion Soil Investigation Ltd. and dated November 6th, 1958 (GEOCRES Reference No. 40P08-017).

The investigation consisted of advancing boreholes with adjacent dynamic cone penetration tests (DCPTs) at four locations (designated as BH1, BH2, BH5 and BH6) and an additional two DCPTs (designated as BH3 and BH4) between the dates of September 1st to 6th, 1958. The boreholes were advanced using a diamond drill adapted for soil sampling. Standard Penetration Tests (SPTs) were conducted at regular intervals in the boreholes.

The borehole locations and a strata plot incorporating information from the previous investigation (and current investigation as discussed below) are shown on Drawing Nos. 1 and 2 (Borehole Location and Soil Strata Drawing) in Appendix A.

Copies of the borehole location plan, the borehole and DCPT records and associated laboratory testing results from the previous investigation are included in Appendix B.

Review of the information from the previous investigation indicates that the soils within the vicinity of the planned bridge replacement consist of surficial organic soils (e.g. topsoil and peat) underlain by predominantly cohesionless/granular soil deposits typically varying in composition from silty sand to sand and gravel.

The report identifies that a surficial layer of topsoil and peat extended to a depth of approximately 1.4 m below ground surface at the location of BH1 and that a marshy area was present on the south (west) side of the highway. A layer of sandy peat was also identified at a depth of about 1.4 m in BH2. The organic soil deposits were encountered down to elevations estimated to vary from about 302.4 m to 303 m.

Standard Penetration Test (SPT) ‘N’ values recorded in the upper portion of the granular soils that underlie the near-surface topsoil and organic materials typically varied from about 8 to 30 blows per 0.3 m of penetration indicating these materials are typically loose to compact, with occasional higher SPT ‘N’ values recorded within soils containing significant proportions of gravel. SPT ‘N’ values recorded in the granular soils below an elevation of approximately 297 m varied from 60 to 100 blows per 0.3 m of penetration indicating these soils are in a very dense state.

The groundwater table was reported at elevations of about 302.5 to 303 m.

3.2 FIELD INVESTIGATION

The current foundation investigation for the proposed bridge replacement consisted of advancing twelve (12) boreholes, designated as Boreholes BH19-01 to BH19-12 at the site between the dates of April 30th and August 28th, 2019.

Boreholes BH19-01 to 19-06, BH19-08 and BH19-10, were drilled at/near the proposed pier and abutment locations. The remaining boreholes were drilled in the areas of the approach embankments. The borehole locations were selected in consultation with MTO personnel prior to completing the fieldwork. The locations of these boreholes are shown on the Borehole Locations Plan, Drawing No. 1, contained in Appendix A.

Prior to carrying out the investigation, Stantec contacted public utility authorities to provide utility locates/ clear the borehole location for drilling.

Drilling was carried out with a track-mounted auger drill rig equipped for soil sampling. The boreholes were advanced using continuous flight hollow-stem augers. Drilling mud was added during drilling at the majority of the borehole locations to counteract ‘flowing’ sand conditions encountered within the augers.

The subsurface stratigraphy encountered in each borehole was recorded in the field by Stantec’s geotechnical engineering staff. Split spoon samples were collected at regularly spaced intervals (typically every 760 mm up to 6.5 m below existing ground surface, every 1.5 m between depths of 6.5 m to 20 m and 3 m thereafter.). All samples recovered were returned to Stantec’s Ottawa laboratory for detailed classification and testing.

Observations of the groundwater conditions were made in the open boreholes at the time of drilling. To permit further measurement of the groundwater level, a temporary well was installed to a depth of 3 m below ground surface in BH19-05. The stabilized water level was measured at this location the following day after which the well was removed/decommissioned.

The boreholes were grouted on completion of drilling and sampling. Boreholes advanced on the highway/roadway were sealed at surface with granular fill and cold patch asphalt.

A geophysical testing program, consisting of conducting Multichannel Analysis of Surface Waves (MASW) surveys was undertaken on April 11, 2019 to measure the shear wave velocity of the site soils on both sides of the highway. Further details on the geophysical testing program are provided in Section 4.5.

3.3 LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations were determined in the field by Stantec Geomatics personnel. The survey data is accurate to within 0.1 m for both co-ordinates and elevation.

Table 3.1 below provides a summary of borehole information including co-ordinates, elevations, borehole depths, termination elevations and the number of samples collected at each borehole.

Table 3.1: Borehole Information Summary

	Borehole Numbers					
	19-01	**19-02**	**19-03**	**19-04**	**19-05**	**19-06**
MTM Zone 10 Coordinates Northing Easting	4801721 230522	4801697 230523	4801699 230491	4801683 230484	4801707 230461	4801681 230451
Ground Surface Elevation, m	304.2	310.7	304.2	304.5	304.4	310.6
Total Depth Drilled, m	23.3	31.1	20.1	20.1	21.5	28.0
End of Borehole Elevation, m	280.9	279.6	284.1	284.4	282.9	282.6
Number of Soil Samples	19	22	18	18	19	22
	19-07	**19-08**	**19-09**	**19-10**	**19-11**	**19-12**
MTM Zone 10 Coordinates Northing Easting	4801676 230428	4801701 230544	4801705 230564	4801677 230543	4801689 230588	4801727 230540
Ground Surface Elevation, m	310.1	310.3	309.7	304.0	303.8	303.5
Total Depth Drilled, m	15.8	31.1	15.8	27.9	9.8	9.8
End of Borehole Elevation, m	294.3	279.2	293.9	276.1	294.0	293.7
Number of Soil Samples	15	21	14	20	10	10

3.4 LABORATORY TESTING

All samples obtained from the investigation were visually reviewed by a Geotechnical Engineer. The geotechnical laboratory testing program for the borehole samples is summarized in Table 3.2.

Table 3.2: Geotechnical Laboratory Testing Program

Test Description	Number of Tests
Moisture Content	202
Atterberg Limits	10 (includes 5 non-plastic results)
Grain Size Distribution (sieve & hydrometer)	53
Organic Content	3
Direct Shear	2
pH, resistivity, soluble sulphate and chloride content tests*	2

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.

4.0 SUBSURFACE CONDITIONS

4.1 OVERVIEW

The detailed soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are shown on the Record of Borehole sheets contained in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix B. Copies of the borehole records for the boreholes drilled at the site as part of the 1958 investigation for the existing bridge are also included in Appendix B. The results of geotechnical laboratory testing completed by Stantec as part of the current investigation are presented on Figures C1 to C13 in Appendix C.

A borehole location plan and a stratigraphic profile of the soils encountered at the borehole locations along the bridge is provided on Drawing No. 2 in Appendix A. The stratigraphic boundaries on the borehole records and strata plots are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units. The subsoil conditions will vary between and beyond the borehole locations.

The subsurface stratigraphy encountered in the boreholes typically consisted of asphalt and roadway embankment fill materials overlying an extensive deposit of predominantly granular/cohesionless soils. Deposits of topsoil/peaty topsoil were encountered, either at ground surface or at shallow depth, in several boreholes which were typically outside of the limits (i.e. beyond the toe) of the existing Roseville Road embankments. The granular/cohesionless soil deposits extended to depths in excess of 30 m below ground surface and generally varied in composition from sand/silty sand to sand and gravel/sandy gravel but also contained sporadic/occasional layers of finer-grained silt and sandy silt. The upper portions of the granular/cohesionless soil deposits were generally compact with occasional loose or dense zones; the predominantly compact materials were encountered down to elevations of about 296 m to 297 m in the vicinity of the existing bridge and to about 294 m to 295 m to the south (east) of the bridge. Below those elevations, the soils become dense to very dense. A layer of hard clayey silt was encountered at a depth of about 28 m (~Elev. 283 m) in BH19-06.

Detailed descriptions of the soils are provided in the following subsections.

4.2 OVERBURDEN

4.2.1 Topsoil/Organic Soils

Surficial topsoil deposits were encountered at ground surface at the locations of Boreholes 19-01, and 19-10 to 19-12. The topsoil was generally comprised of sandy silt/silty sand containing wood pieces and organic matter. The topsoil layer was approximately 0.1 m to 0.6 m thick with the base of the deposit at elevations of approximately 303.1 m to 304 m at these borehole locations.

Laboratory testing of a sample of the surficial topsoil from BH19-11 measured an organic content of 5% and a natural moisture content of approximately 29 percent, expressed as a percentage of the dry weight of the soil.

Layers of buried topsoil/peaty topsoil were encountered at depths of about 0.9 m in Borehole BH19-03 and 1.1 m in BH19-10. These layers were about 0.2 m (in BH19-03) and 0.3 m (in BH19-10) thick with the base of the deposits encountered at approximately 302.6 m to 303.1 m. These elevations are similar to the elevations of the base of the organic soils encountered during the 1958 investigation at the site.

Laboratory testing of the buried topsoil/peaty topsoil layers measured organic contents ranging from about 2 to 21 percent and natural moisture contents of approximately 29 to 42 percent.

4.2.2 Asphalt

Boreholes 19-03 and 19-04 were located in the paved median of Highway 401 and Boreholes 19-02 and 19-06 to 19-09 were located in the travelled surface of Roseville Road. The thickness of the asphalt varied between 127 mm and 165 mm at these borehole locations.

4.2.3 Fill

Fill was encountered at ground surface in Borehole BH19-05 and below the topsoil or asphalt pavement in Boreholes BH19-01 to BH19-04 and BH19-06 to BH19-10.

The fill materials encountered in the boreholes advanced through the Roseville Road approach embankments extended to depths varying from 5.9 m to 7.2 m below ground surface corresponding to elevations of approximately 302.5 m to 304.8 m

The fill materials encountered in boreholes drilled within the Highway 401 median and near the toe of the Roseville Road approach embankments extended to depths of 0.9 m to 1.5 m below ground surface corresponding to elevations of 302.7 m to 303.3 m.

The fill materials were typically comprised of granular/cohesionless fill varying in composition from sand/silty sand to sand and gravel. However, zones of cohesive fill were encountered near the base of the approach embankment fill materials at several borehole locations.

Further details on the fill materials, divided into cohesionless and cohesive fill for ease of reference, are provided in the following sections.

4.2.3.1 Cohesionless Fill

The asphalt pavement in Boreholes BH19-02 to BH19-04, and BH19-06 to BH19-09 was underlain by granular fill associated with the Highway 401 and Roseville Road pavement structures. The granular fill typically varied in composition from sand containing some silt and gravel to sand and gravel.

The fill materials encountered in the Roseville Road approach embankments consisted predominantly of cohesionless fill varying in composition from sand/silty sand to sand and gravel and contained localized inclusions of organic matter and/or silt. Cobbles and/or boulders were also inferred to be present within these fill materials in several boreholes based on auger grinding and drilling progress.

Standard Penetration Test (SPT) ‘N’ values obtained in the cohesionless fill in the boreholes drilled through Highway 401 and/or Roseville Road embankments ranged from 12 to 78 blows per 305 mm penetration. Based on the SPT ‘N’ values recorded, these roadway/highway embankment fill materials are in a compact to very dense state. SPT ‘N’ values varying from 7 to 23 blows per 305 mm penetration were recorded in the fill materials present beyond the toe of the roadway embankments; these fill materials are in a loose to compact state.

Laboratory testing of the granular/cohesionless fill materials yielded moisture contents that typically ranged from approximately 3% to 14% expressed as a percentage of the dry weight of the soil.

Gradation analyses were carried out on ten (10) representative samples of the cohesionless fill materials. The results of the tests are illustrated on the gradation curves on Figure No. C1 in Appendix C. The results of a consolidated drained direct shear test is also contained in Appendix C.

4.2.3.2 Cohesive Fill

The fill materials in BH19-05 and the fill present in the lower portions of the Roseville Road approach embankments in Boreholes BH19-07, BH19-08 and BH19-09 were comprised of, or contained, cohesive materials.

The cohesive fill in BH19-07 to BH19-09 was generally comprised of gravelly clayey silt with sand and contained trace organic matter. Cobbles and boulders are inferred to be present within this fill based on auger grinding and drilling progress. A possible hydrocarbon odour was noted within the fill materials in BH19-08 below about 5.3 m depth. The fill in BH19-05 was comprised predominantly of silt containing some sand and topsoil with zones of clayey silt.

The cohesive fill materials in Boreholes BH19-07 to BH19-09 were encountered at depths of about 5.3 m to 6 m, were approximately 1.2 m to 1.9 m thick, and extended to depths of approximately 7.2 m to 7.5 m below ground surface corresponding to elevations of approximately 302.5 m to 303.1 m. The fill in BH19-05 was encountered at ground surface, was approximately 1.5 m thick, with a base elevation of about 302.9 m.

Standard Penetration Test (SPT) 'N' values obtained in the cohesive embankment fill materials typically ranged from 11 to 14 blows per 305 mm penetration. A higher SPT 'N' value of 63 was measured in BH19-08; this value is inferred to have been influenced by the presence of gravel or cobbles.

Laboratory testing of the cohesive fill materials yielded moisture contents ranging from approximately 8% to 22%. Gradation analyses were carried out on three (3) samples of the cohesive fill materials obtained from Boreholes BH19-07 to BH19-09. The results of the tests are illustrated on the gradation curves on Figure No. C2 in Appendix C.

Atterberg Limits tests were carried out on the three samples noted above. These tests yielded Liquid Limits of between 25% and 34%, Plastic Limits of between 19% and 22%, and Plasticity Indices of between 5% and 12%. The results of the Atterberg Limits test are illustrated on Figure C3 in Appendix C.

4.2.4 Sand/Silty Sand to Sandy Gravel (SP/SM to GP)

An extensive sequence of layered deposits of cohesionless soils was encountered beneath the asphalt, topsoil/organic soil deposits and fill materials at all borehole locations. These soils were variable in composition, typically ranging from sand to sandy gravel, and contained varying amounts of silt as well as cobbles and boulders. In several areas, these soils had a more broadly-graded, 'till-like' composition.

Where fully penetrated, the thickness of the sequence of cohesionless deposits encountered during the current investigation varied from about 6 m at BH19-12 to in excess of 26.5 m at BH19-10. Boreholes BH19-01 to BH19-05 and BH19-07 to BH19-11 were terminated in the silty sand/sand to gravelly sand deposits at depths varying from approximately 9.8 m to 31.1 m below the existing ground surface, corresponding to elevations of between about 276.1 m and 294.3 m.

Layers/zones of silt/sandy silt and silty fine sand were encountered sporadically within the overall sequence of sand to sandy gravel soils. Specific examples of soil layers comprised predominantly of silt include an approximately 0.8 m thick layer of silt at 7.9 m depth in BH19-01, an approximately 1.6 m layer of sandy silt at 27.7 m in BH19-02, and a greater than 3.5 m thick layer of silt (transitioning into clayey silt) at 6.3 m depth in BH19-12; the latter deposit is described in more detail in the following section of this report. In addition to the above, thinner seams of silt were encountered at various borehole locations.

The native, cohesionless soil deposits can broadly be divided into two zones based on the degree of compactness of these soils. The upper zone, that extends down to elevations of about 295 m to 297 m in the vicinity of the existing bridge and to about 294 m to 295 m to the south (east) of the existing bridge, is generally in a compact state though loose zones were encountered. SPT 'N' values measured within this zone varied from 9 to 60 blows but were more typically in the range of 10 to 25 blows. In the lower zone, the granular/cohesionless soils become dense to very dense. SPT 'N' values measured in the lower, dense to very dense zone varied from 28 blows per 305 mm penetration to 100 blows per 165 mm but were typically in excess of 40 blows per 305 mm.

Laboratory testing of samples of the native silty sand to sandy gravel deposits yielded moisture contents ranging from approximately 5% to 27% with values generally less than 15%.

Gradation analyses were carried out on thirty-three (38) samples of the cohesionless soil deposits obtained from Boreholes BH19-01 to BH19-12. The results of these tests are illustrated on the gradation curves on Figure Nos. C4 to C9 in Appendix C. Figures C4 to C7 display the results of testing on samples of the coarser portions of these deposits, Figure C8 displays testing carried out samples of the silty fine sand while gradation analyses completed on two samples of the silt/sandy silt interlayers are illustrated on Figure No. C9. The results of a consolidated drained direct shear test of a sample of the cohesionless deposits is also included in Appendix C.

4.2.5 Upper Silt to Clayey Silt (ML to CL-ML)

A deposit of silt that transitions into clayey silt was encountered at a depth of 6.3 m below ground surface (~Elevation 297.3 m) in BH19-12. The upper portion of this deposit consists of dense brown silt with trace sand which grades/transitions into very stiff to hard, clayey silt with trace sand and gravel, below a depth of about 7.5 m (~Elevation 296 m).

A SPT ‘N’ value of 37 was recorded in the silt while SPT ‘N’ values of 25 to 31 were measured within the underlying clayey silt. The results of a gradation analysis completed on a sample of the clayey silt are illustrated on Figure No. C10 in Appendix C. An Atterberg Limit test conducted on this sample yielded a Liquid Limit of 22 percent, a Plastic Limit of 13 percent and a corresponding Plasticity Index of 9 percent. The results of this test are displayed on Figure No. C11 in Appendix C.

BH19-12 was terminated within the upper clayey silt deposit at a depth of 9.8 m below ground surface (~Elevation 293.7 m).

4.2.6 Lower Clayey Silt (CL)

A deposit of clayey silt containing trace sand was also encountered below the sequence of sand/silty sand to sandy gravel soil deposits at a depth of 27.6 m, corresponding to an elevation of approximately 283 m, in BH19-06. BH19-06 was terminated in the clayey silt deposit at a depth of 28 m below ground surface (~Elevation 282.6 m).

A SPT penetration resistance of 62 blows for 76 mm of penetration was recorded in the clayey silt indicating this material has a hard consistency.

The results of a gradation analysis completed on a sample of the clayey silt are illustrated on Figure No. C12 in Appendix C. An Atterberg Limits test conducted on this sample yielded a Liquid Limit of 34 percent, a Plastic Limit of 14 percent and a corresponding Plasticity Index of 20 percent. The results of this test are illustrated on Figure No. C13 in Appendix C.

4.3 BEDROCK

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

4.4 GROUNDWATER

Observations of groundwater levels were made in all the open boreholes during drilling (drilling dates of April 30th to August 19th, 2019). Information on the measured groundwater level depths and elevations are provided below in Table 4.1 and on the Borehole Records in Appendix B.

Table 4.1: Groundwater Levels (Stantec Boreholes)

Borehole No.	Ground Surface Elevation (m)	Approximate Groundwater Levels			
		Inferred During Drilling (April 30th to May 15th, 2019)		Inferred During Drilling (August 19th to 29th, 2019)	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
BH19-01	304.2	0.8	303.3	N/A	N/A
BH19-02	310.7	7.2	303.5	N/A	N/A
BH19-03	304.2	1.2	303.0	N/A	N/A
BH19-04	304.5	1.5	303.0	N/A	N/A
BH19-05	304.4	*1.1	*303.3	N/A	N/A
BH19-06	310.6	7.6	303.0	N/A	N/A
BH19-07	310.1	N/A	N/A	6.9	303.2
BH19-08	310.3	N/A	N/A	7.1	303.2
BH19-09	309.7	N/A	N/A	6.4	303.3
BH19-10	304.0	N/A	N/A	0.8	303.2
BH19-11	303.8	N/A	N/A	0.6	303.2
BH19-12	303.5	N/A	N/A	0.5	303.0

*Note: A temporary well was established at the location of BH19-05 on May 1st, 2019 to permit the water level within the borehole to stabilize overnight. The water level at this location was measured to be at a depth of 1.1 m below ground surface, corresponding to an elevation of 303.3 m, on May 2nd, 2019. The well was removed and the borehole grouted following measurement of the water level.

The groundwater levels shown in the table above are generally consistent with but slightly higher than the water levels measured as part of the original 1958 foundation investigation where groundwater level elevations of between about 302.5 to 303 m were reported.

The water level in Cedar Creek was surveyed to be at an elevation of approximately 303.1 m in January 2019. The water level in the channel was observed to typically be at or just below the top of the inside of the box culvert at various times during the foundation field investigation program (refer to photograph below).

Groundwater levels at the site will be subject to fluctuation due to seasonal changes, precipitation and snow melt events as well as variations in the water level in the adjacent ditches and Cedar Creek channel. The water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation or snow melt.

4.5 GEOPHYSICAL TESTING

Multichannel Analysis of Surface Waves (MASW) surveys were completed on either side of the highway to measure the shear wave velocity of the site soils. The MASW seismic surveys were completed by ClearView Geophysics Inc.

A report providing the methodology and results of the geophysical testing program is included in Appendix D. The results and conclusions of the report identified the following:

- At MASW survey/Spread 1 which was located on the south (east) side of the highway, the average shear-wave velocity from ground surface to a depth of 30 meters ($\bar{V}_S30$) was calculated to be 380 m/s (east end of Spread) to 425 m/s (west end of spread).
- At MASW survey/Spread 2 which was located on the north (west) side of the highway, the average shear-wave velocity from ground surface to a depth of 30 meters ($\bar{V}_S30$) was calculated to be 389 m/s (west end of Spread) to 445 m/s (east end of spread).
- All of the measured $\bar{V}_S30$ values fall within the range of shear wave velocities for a Site Class C designation (360 to 760 metres per second) which is termed Very Dense Soil and Soft Rock in the CHBDC.

4.6 CHEMICAL TESTING

Two (2) samples retrieved from BH19-08 were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, water soluble sulphates and chloride concentrations, and resistivity. The analysis results are summarized in Table 4.2.

Table 4.2: Results of Chemical Analysis

Borehole and Sample No.	Material Type	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH19-08 SS3	Gravelly SAND (FILL)	1.5 to 2.1	7.86	565	19	10.3
BH19-08 SS12	SAND	10.7 to 11.3	7.87	37	42	56.3

5.0 MISCELLANEOUS

The field work was carried out under the supervision of Karen Thrams, P.Eng. and David Lee, P.Eng., under the direction of Kevin Nelson, P.Eng.

The utility locates for the boreholes advanced as part of the foundation investigation were carried out by Stantec personnel.

The boreholes were advanced using rubber-track mounted drilling equipment that was supplied and operated by London Soil Testing of London, Ontario.

Location and elevation survey of all the boreholes was carried out by Stantec Geomatics staff. The borehole survey data is considered to meet MTO’s vertical and horizontal accuracy requirements of 0.1 m and 0.5 m, respectively.

Traffic control service was provided by On Track Safety Ltd. of Thornhill, Ontario and Direct Traffic Management of Hamilton, Ontario.

Geotechnical laboratory testing was carried out at Stantec’s Ottawa laboratory. Direct shear testing was conducted by Golder Associates Ltd. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was carried out by Paracel Laboratories of Ottawa.

This report was prepared by Bridgit Bocage, P.Eng. and Kevin Nelson, P.Eng., and reviewed by John Brisbois, P.Eng., Designated Principal MTO Foundation Contact.

6.0 CLOSURE

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

Respectfully Submitted;

STANTEC CONSULTING LTD.

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

John J. Brisbois, MScE., P. Eng.
MTO Designated Principal Foundation Contact

FOUNDATION INVESTIGATION AND DESIGN REPORT

For

G.W.P 3204-16-00

Roseville Road Underpass Replacement, Highway 401
Site No. 33X-0177/B0

North Dumfries, ON

7.0 DISCUSSIONS AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design input related to the proposed replacement of the Roseville Road bridge over Highway 401. The recommendations provided herein are based on interpretation of the factual data obtained from subsurface investigations completed at this site.

The interpretation and recommendations provided in this report are intended solely to provide the designers with information to assess foundation options and carry out the design of the bridge foundations and embankments. As such, where comments are made regarding construction, the comments are provided only to highlight those aspects which could affect the design of the project. Those parties requiring information on aspects of construction should make their own interpretation of the factual information provided herein as it may affect equipment selection, proposed construction methods, scheduling and the like.

7.1 PROJECT DESCRIPTION AND BACKGROUND

7.1.1 Project Description

The project involves the replacement of the existing four-span bridge carrying Roseville Road over Highway 401. The new structure is being designed to accommodate the ultimate 10-lane configuration of Highway 401 (one HOV lane plus 4 lanes in each direction). Roseville Road is planned to be closed during the construction of the new underpass structure.

7.1.2 Proposed Bridge Structure and Approach Embankments

A new three-span bridge structure is proposed to be constructed along approximately the same alignment as the existing bridge. The new bridge will be longer than the existing bridge to allow for removal of the Culvert at Site No. 33X-0421/C0 to restore Cedar Creek to an open channel. A post-tensioned box girder structure with integral abutments and span lengths of 48 m, 33 m and 30 m (total length of 111 m) is proposed. The new bridge will also be widened to approximately 15.7 m to allow for construction of sidewalks.

Key information associated with the existing bridge and proposed replacement bridge is as follows:

Approximate Existing Roseville Road Grade at Bridge:	311 m
Approximate Apex of Proposed Roseville Road Grade at Bridge:	312.9 m
Approximate Elevation of Underside of Proposed Abutment Walls	308.6 m (S) to 309.4 m (N)
Existing Highway 401 Grade at Bridge:	304.1 m to 304.5 m
Approximate Underside Elevation of Proposed Pier Pile Caps:	~302.0 m (S) to 302.3 m (N)
Approximate Elevation of Underside of Existing Pier Pile Caps:	~302.3 m

The approach embankments on Roseville Road are planned to be widened and raised by about 1.5 m above the existing roadway grade in the vicinity of the replacement structure. The new, widened embankments will have maximum embankment heights of about 8 m and side slopes of 2H:1V.

The gas pipeline currently located near the toe of the south approach embankment (west side of Roseville Road) is planned to be relocated outside of the area of the new, widened embankment.

7.1.3 Degree of Site and Prediction Model Understanding

The Canadian Highway Bridge Design Code (CHBDC) [December 2014] 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 considers the geotechnical properties of the soils underlying 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 except that a "High" degree of understanding has been adopted for assessment of embankment stability where slip surfaces develop through imported/manufactured granular fill materials. The consequence classification has been selected as "Typical Consequence" as per Section 6.5 of the Commentary on CSA S6-14, Canadian Highway Bridge Design Code (CHBDC), (S6, 1-14).

7.2 GEOTECHNICAL DESIGN PARAMETERS

The primary soil strata encountered in the area of the replacement bridge generally consist of:

- Asphalt and pavement/roadway embankment fill materials beneath Highway 401 and in the approaches to the existing bridge structure;
- Layered sequences of predominantly cohesionless soils, varying in composition from silt/sandy silt to sandy gravel, that typically extend to depths in excess of 20 m to 30 m below ground surface. The upper portion of these soils is typically in a compact state with occasional loose or dense zones. The lower portion of these soils below elevations of about 294 m to 297 m is in a dense to very dense state; and
- A deposit of hard clayey silt was encountered beneath the cohesionless soils at a depth of about 28 m (approximately Elevation 283 m) in one of the boreholes.

Due to the variability of the subsurface conditions across the site, the soil profiles identified in Tables 7.1 and 7.2 below and on Figures E1 to E2 in Appendix E have been developed for use in the design of the bridge foundations and embankments on the north side of the bridge (including the north abutment and north pier) and the south side of the bridge (including the south abutment and south pier), respectively.

The geotechnical parameters identified in the soil profiles were developed based on the synthesis of measured SPT ‘N’ values and laboratory index test results (including moisture contents) of soil samples obtained from the current and previous boreholes advanced at the site.

Table 7.1: Representative Soil Profile (North Side of Bridge)

Elevation		Soil Type	Design Parameters				
From	To		Total Unit Weight (kN/m³) γ	Drained Friction Angle (degrees) ϕ' [3]	Undrained Shear Strength (kPa) S_u [3]	Modulus (MPa) E	Constant of Subgrade Reaction n_h (kN/m³)
Ground Surface	303	FILL: Predominantly dense to very dense sand to sand and gravel	23	35	-	50	7500
303	296	Compact to Dense SAND/Silty SAND to Gravelly SAND (SP/SM)	21.5	34	-	40	6500
296	286	Very Dense Sand to Sand and Gravel (SP to GP)	23	40	-	100	11000
286	283	Very Dense Sand to Silty Sand (SP/GP)	22	35	-	60	11000
< 283		Hard CLAYEY SILT	21	31	250	100	--

Table 7.2: Representative Soil Profile (South Side of Bridge)

Elevation		Soil Type	Design Parameters				
From	To		Total Unit Weight (kN/m^3) γ	Drained Friction Angle (degrees) ϕ' [3]	Undrained Shear Strength (kPa) S_u [3]	Modulus (MPa) E	Constant of Subgrade Reaction n_h (kN/m^3)
Ground Surface	305	FILL: Predominantly dense to very dense sand to sand and gravel	23	35	-	50	--
305	303	FILL: Gravelly CLAYEY SILT with sand	22	32	100	25	--
303	295	Compact to dense, SAND and GRAVEL (SP/GP) to SAND/Silty SAND (SP/SM)	22	34	-	40	6500
295	285.5	Dense to very dense, SAND and GRAVEL (SP/GP) to SAND/Gravelly SAND (SP)	23.5	40	-	100	11000
285.5	281	Very dense, SAND to SILT and SAND (SP to SM)	21.5	35	-	60	11000
< 281		Very dense, SAND/Gravelly SAND (SP) to Sandy GRAVEL (GP)	23.5	38		100	11000

Note:

(1) Groundwater is assumed to be at an Elevation of 303.3 m for design purposes. Submerged unit weights (γ') should be used below the groundwater level.

(2) Cobbles and boulders were encountered in the fill and the cohesionless sand to gravelly sand deposits. These materials should be expected to present in all fill materials and native, cohesionless strata at this site.

(3) The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only. The results of direct shear tests completed on select samples indicated higher friction angles than identified in the above tables; those values were considered in selection of the friction angles but were considered higher than the typical shear strength of the bulk of the soils at the site.

(4) The values of nh (constant of subgrade reaction) provided in the tables above considered the subsurface conditions encountered in the boreholes and the framework provided in CFEM, 3rd Edition.

7.3 FROST PENETRATION AND PROTECTION

In accordance with OPSD 3090.101, a design frost penetration depth for foundations, f, of 1.4 m would apply at this site. 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.

Based on information provided by the design team, it is understood that the MTO is considering relaxing the frost protection requirement (i.e. reducing the depth of soil cover) for the perched abutments of the replacement bridge. This is understood to be an approach used by transportation authorities/departments in other provinces and states.

The relaxation of frost protection requirements should only be considered where site conditions warrant and measures are implemented to reduce the potential for frost-related impacts on the structure. The following criteria are recommended to be reviewed when assessing this concept:

- The level of the groundwater table and any future fluctuations in the groundwater table must be below the depth of frost penetration;
- The backfill surrounding (i.e. in front, below and behind) the abutments should consist of free-draining granular materials; and,
- Surface water and/or infiltration from above should be directed away from the abutments.

With respect to the Roseville Road site, the base of the abutment stems/walls are planned to be perched within the existing approach embankment fill materials and would be located approximately 4 m above the recorded groundwater level at the site. The fill materials in the approach embankment typically consist of sand/gravelly sand to sand and gravel fill that is considered to have a low susceptibility to frost action. The design drawings provided indicate that surface water from the roadway will be shed/directed away from the abutment locations (i.e. towards the embankment crests and along Roseville Road away from the bridge).

Based on these conditions, a relaxation in the frost protection requirement may be considered for this site provided that:

- The contract documents specify that free draining backfill is placed around and behind the abutment stem and adjacent to the CSPs that contain the abutment piles. The backfill materials should consist of Ontario Provincial Standard Specification (OPSS) Granular A or B Type II materials extending a minimum of 1.4 m below and behind the abutment walls. As the performance of the backfill is vital to preventing freeze-thaw movement in this case, it is recommended that the fines content of the Granular A or B Type II material be restricted to less than 5% passing the 200 sieve; and
- A drainage system consisting of geotextile encapsulated, 150 mm diameter perforated pipe(s) connected to a frost-free outlet(s) be installed at the base of the fill.

It is further recommended that consideration be given to supplementing the above by installing insulation (e.g. extruded polystyrene insulation) below the abutment stem and walls, and by installing concrete abutment slope protection to further reduce the potential for frost impacts at the abutments. This is considered the preferred approach from a foundations engineering perspective to further minimize the risk of foundation related movements due to frost action. If the abutment/wall foundations are provided with insulation to fully protect them against frost action, the abutment backfill can be placed in accordance with Ontario Provincial Standard Drawing (OPSD) 3101.150 rather than the backfill configuration outlined above.

The following provides design parameters and input regarding the use of insulation for abutment foundation protection purposes.

- Mean Freezing Index (Roseville): 625 Degree-Days below 0° C, 1,125 Degree-Days below 0° F
- Design Freezing Index 907 °C-days
- Minimum recommended depth of cover: 0.6 m
- Minimum thickness of insulation: 50 mm

The minimum recommended cover and minimum thickness of insultation provided above are for preliminary assessment purposes. Final design details associated with the insulation, including the length that the insulation should extend beyond the edges of the foundations and the required compressive strength of the insulation, are dependent on the type of insulation used, the design depth of cover and the final insulation geometry/configuration and should comply with all design guidelines from the insulation supplier.

In preparation for the insulation, a levelling mat consisting of 25 millimetres of concrete/mortar sand or lean concrete should be placed on the approved bearing surface. Care must be taken to ensure that the insulation is not damaged during construction. Joints should be carefully lap jointed and glued to prevent separation.

7.4 SEISMIC CONDITIONS

7.4.1 Site Class

Based on the results of the current investigation, the bridge site is underlain by an extensive sequence of predominantly cohesionless soils varying in composition from silt sand/silt and sand to sandy gravel that extend to depths in excess of 30 m below ground surface. These deposits are typically compact to dense to depths of about 10 m below ground surface and become dense to very dense at greater depths.

MASW seismic surveys were carried out on both sides of the highway. These surveys measured average shear wave velocities within the upper 30 m ($Vs_{(30)}$ values) ranging from 380 m/s to 445 m/s.

Based on the subsurface conditions and the measured shear wave velocities, it is recommended that Site Class C as defined in Section 4.4.3 of the CHBDC (2014) be used in the seismic design.

7.4.2 Peak Ground Acceleration (PGA)

Seismic hazard values for this site were obtained from Natural Resources Canada (2015 National Building Code). The 2015 NBC Seismic Hazard calculation sheet for this site is provided in Appendix F. Table 7.3 summarizes the parameters based on a 2475-year return period to be used in force-based design.

Table 7.3: Peak Ground Acceleration Data

PGA	$S_a(0.2)$	PGA_{ref}	Site Class	Site Adjusted PGA
0.08g	0.131g	0.064	C	0.08g

7.4.3 Liquefaction Potential

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

- The blow count data from boreholes.
- A Site Adjusted PGA of 0.08g.
- An earthquake magnitude Mw of 6.55.

The analysis indicates a factor of safety against liquefaction of 2.0 or greater, and therefore liquefaction is not a concern at this site.

7.5 REPLACEMENT BRIDGE FOUNDATION OPTIONS

Both shallow and deep foundation options were evaluated for the proposed replacement bridge.

Table 7.4 presents the advantages, disadvantages, relative costs and risks/consequences for various foundation options for the pier and abutment foundations of the proposed replacement bridge.

Table 7.4: Comparison of Foundation Options for Roseville Road Underpass Structure

Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
H Piles in Overburden	• Reduced total and differential settlement compared to shallow foundations • Reduced pile length compared to piles driven to bedrock • Suitable for integral abutments • Reduced depth of excavation at abutments	• Structural capacity of piles may not be fully utilized • Difficulty driving piles through cobbles and boulders • Potential conflicts with existing bridge piles at north pier location • Traffic impacts due to large crane for vibratory extraction of existing piles, if pile removal is required • Dewatering systems required and groundwater cut-off (e.g. sheet piles) may be needed at pier locations.	Medium	• Pile damage during installation • Shallow refusal of piles on cobbles and boulders requires pre-drilling • Risk existing piles at north pier cannot be removed by vibro-extraction causing redesign of foundations during construction, if pile removal is required
Shallow Foundations	• Excavation in/through difficult deposits avoided • Lower foundation costs than deep foundations; however, groundwater control and temporary protection measures required to protect subgrade would increase costs compared to typical shallow foundations	• Upper portions of native soils are compact (locally loose) resulting in potential for large, unacceptable total and differential settlement. • Differential settlement performance at pier locations where existing piles are present • Deep excavations needed to remove existing approach embankment fill materials at abutment locations. • Not suitable for integral abutments (Semi-integral Abutments possible) • Dewatering systems required and groundwater cut-off (e.g. sheet piles) may be needed at pier locations.	Low to medium	• Potential for large total and differential settlement due to large footing areas and varying (locally loose) subgrade conditions • Groundwater inflows into excavations can lead to disturbance of subgrade soils

Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
Drilled Caissons	• Can transmit very large axial and lateral loads • Shorter construction time than shallow or pile foundations	• Conflicts with existing central pier pile foundations and pile caps will obstruct installation • Caissons would extend below the water table and into the loose to compact cohesionless soils • Requires use of drilling mud to balance water pressures; **bases cannot be visually inspected** • Difficult to drill and advance liners though deposits containing boulders and cobbles • Not suitable for integral bridge abutment	High	• Risk existing piles cannot be removed causing re-design of foundations during construction • Drilled pier installation through saturated granular soils can result in soil disturbance and collapse of sidewalls. Liners and drilling mud required due to groundwater issues. • Use of “wet” installation methods **precludes ability to review/confirm materials at the base of the caissons and assess the potential for reduced capacity**
Piles end-bearing on bedrock	• High geotechnical resistances • Negligible settlement	• Excessive pile lengths • Potential conflicts with existing bridge piles at north pier location	High	• Bedrock depth is not known but could be in excess of 50 m • Potential for damage to piles during installation

The use of drilled piers/caisson foundations is not considered practical for the piers at this site for the following reasons:

- Potential conflicts with the existing bridge foundations would require pile cap and pile removal. Difficulties may be encountered removing the existing piles, particularly the inclined piles, present at the north pier location;
- The disturbance of soils during removal of the existing piles would reduce caisson capacity;
- Difficulties associated with installation of the caissons through cohesionless soil deposits containing below the water table will require mitigation measures (i.e. use of liners and drilling mud/“wet’ installation methods). Liner installation would be hindered by the presence of cobbles and boulders;
- Interlayers of silt/sandy silt/clayey silt were encountered sporadically and at varying elevations within the overall sand/sand and gravel deposits at the site. These materials would provide significantly less base resistance than materials comprised predominantly of sand and/or gravel. The use of “wet” installation methods would preclude the ability to review/confirm the materials present at the base of the caissons and assess the potential for reduced capacity increasing the risk of unsuitable foundation performance.

The use of shallow foundations (i.e. spread footings) is not recommended and/or considered feasible at the pier locations due to the potential overlap and interaction with the existing piles and due to the potential for large total and differential settlements. The use of shallow foundations at the abutment locations is not considered practical as the existing approach embankment fills are not considered suitable for the support of the bridge abutments; therefore, such a foundation system would require removal of the existing fill materials and replacement with suitable approved material.

Based on the above, the preferred foundation option from a geotechnical/foundations engineering perspective is to support the central pier and abutments for the replacement bridge structure on driven steel piles that derive their load carrying capacity from a combination of shaft adhesion and tip/end-bearing resistance within the dense granular soils present below Elevation 294. Flexible piles will likely be required at the abutments to enable an integral abutment bridge configuration. As such, where integral abutments are adopted, the upper portion of the piles would need to be installed within sand-filled, corrugated steel pipe (CSP) liners to provide suitable flexibility of the steel H-piles.

Further details on the preferred foundation option are provided in the following sections.

7.6 FOUNDATION RECOMMENDATIONS

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, 2014).

7.6.1 Driven Pile Foundations – Bridge Replacement

7.6.1.1 Design Considerations

Pile foundations consisting of steel H-piles that are driven through the upper, loose to compact portions of the native site soils and into the underlying dense to very dense portions of the sand to sand and gravel deposits present below an elevation of about 294 m (south portion of bridge) to 296 m to 297 m (north portion of bridge) can be used to support the integral abutments and pier foundations of the proposed bridge. These piles would derive their capacity from a combination of shaft adhesion and tip/end-bearing resistance within the dense granular soils. Driven pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected from their design orientation due to the presence of cobbles and/or boulders within the granular soils and could lead to increased soil displacement/heave during installation. Therefore, H-piles are recommended for use at this site.

The foundation for the north pier of the new bridge coincides with the location of the central pier of the existing bridge which is supported on H-Pile foundations, including battered/inclined piles. In this regard, there is potential for the existing piles to conflict with the installation of new piles. In addition, piles from the existing south abutment could conflict with deep foundations for the new south pier. The new pile locations should be reviewed/selected at the detailed design stage to avoid conflicts with existing piles.

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

Given the variability in both the composition and density of the cohesionless soil deposits, the depth of pile penetration into these soils is expected to vary considerably. Pile tip elevations of approximately 284 m have been considered herein for design purposes. However, we note that effective pile driving refusal could also be encountered within the very dense portions of the site soils above this elevation particularly where cobbles and/or boulders or zones of soil with SPT ‘N’ values greater than 100 are encountered. Piles driven/advanced below 284 m have an increased potential for encountering finer-grained soils (e.g. clayey silt till present in Borehole BH19-06) that could result in reduced pile tip resistances and are not recommended.

A NSSP for the CSPs for the integral abutments is provided in Appendix G.

7.6.1.2 Consideration of Existing Piles

The north pier of the new bridge is being constructed at the location of the central pier of the existing bridge. Accordingly, the design of piles for this pier will need to take into consideration the presence of the existing pile foundations (e.g. new piles will need to be located to avoid conflicts with existing piles).

The deviation of the existing piles from the intended/design locations depends on several factors including quality of construction, potential that obstructions were encountered during installation, and pile length/stiffness (e.g. long/slender or flexible piles have a greater potential for deviation than shorter or stiffer piles). There is currently no information available on the installation of the existing piles (e.g. pile driving records, as-built pile locations, or similar). The lengths of the existing piles were not identified on the available structural drawings but Section A1-A1 Typ. from the structural drawings indicates that the piles were installed to an approximate pile tip elevation of 970 feet (~296 m) which is about 7 m below the base of the pile caps of the existing piers. The subsurface conditions through which the piles were installed consisted predominantly of compact sand or sand and gravel; occasional cobbles were inferred to be present within these strata during the current investigation. The short length of the piles and the compact (i.e. generally not hard or dense to very dense) state of the soils would reduce the potential for pile deviation.

Section 21.3 (Location and Alignment) of the Canadian Foundation Engineering Manual states “it is usually impractical to limit location deviations to less than 70 mm for deep foundation units” and “current practice is to limit the total deviation from design alignment to a certain percentage of the final length of the deep foundation unit; 2% is a value in common use”.

Based on the information noted above, deviation of the existing pier piles could be in the order of 200 mm provided that quality construction methods were implemented during pile installation. In this regard, it is recommended that new piles be installed at least one pile width away from the location of the existing piles. There is a potential that piles could have encountered obstructions (e.g. cobbles) which could have resulted in increased deviations. In this regard, the new pier piles and associated pile caps must be designed to accommodate minor adjustment to the location of the new piles in the event that the existing piles are encountered during driving.

7.6.1.3 . Geotechnical Axial Resistance

The axial resistances at Ultimate Limit State (ULS) for driven steel 310x110 and 360x108 H-piles were assessed using the American Petroleum Institute (API) design method with the program APILE developed by Ensoft (Ensoft, 2007). The geotechnical models outlined in Tables 7.1 and 7.2 and on Figures E1 and E2 were used as input to these analyses.

The factored geotechnical resistances at Ultimate Limit States (ULS) outlined in Table 7.5 may be used in design. These values include a resistance factor of 0.4 applied to the ultimate capacity.

Table 7.5: Recommended Factored Geotechnical Resistances (ULS) - Pile Foundations

Pile Type	Foundation Unit	Anticipated Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kN)
HP 310 x 110 HP 360 x 108	North Abutment	284	1,400 1,650
HP 310 x 110 HP 360 x 108	North Pier	284	1,100 1,300
HP 310 x 110 HP 360 x 108	South Pier	284	1,100 1,300
HP 310 x 110 HP 360 x 108	South Abutment	284	1,400 1,650

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

Axial Resistance in Tension

For design against uplift, the tensile resistance provided in Table 7.6 is recommended. This value is based on a minimum pile length of 12 m.

Table 7.6: Recommended Uplift Resistance – Pile Foundations

Pile Type	Assumed Minimum Pile Length (m)	Factored Geotechnical Resistance (Tension) at ULSf (kN)
HP 310 x 110	12	200
HP 360 x 108	12	240

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

Downdrag

The proposed bridge will be constructed in the same location as the existing bridge. The proposed grade raise in the vicinity of the new underpass is typically less than about 1.5 m above existing site grades and the revised/lengthened bridge configuration will result in removal of portions of the existing embankment fill present on the highway side of the new abutment locations. In addition, the native site soils are typically comprised of compact to very dense granular/cohesionless soils. Based on these conditions, significant downdrag loads are not anticipated.

Relaxation of Piles

Relaxation and reduction of pile capacity will not be of concern for H-piles that are founded within the very dense sand to sand and gravel deposits. However, the foundation investigation has identified that there are deposits/layers of very dense silt/sandy silt at varying locations/elevations. The driving of piles into these soils can result in the generation of reduced (negative) pore water pressure and increased effective stresses due to soil dilation. These effects can lead to an apparent temporary increase in driving resistance and strength leading to a 'false set'. After driving, the reduced pore pressures stabilize with time causing a reduction in the capacity and the energy required to advance the pile. Due to these conditions, retapping of all piles should be completed a minimum of 24 hours after the end of initial driving to confirm that the design pile capacities have been achieved.

Pile Driving and Capacity Testing Considerations

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

The subsurface conditions consist of embankment fill overlying compact to very dense cohesionless/granular soils typically varying in composition from silty fine sand to sandy gravel. Cobbles and boulders were inferred in the fill and throughout the native overburden soils based on observations made during drilling. It is essential that the compatibility of the pile driving equipment, the soil conditions, and the pile type being driven is properly accounted for to achieve the required pile penetration and a satisfactory pile foundation. The piles should be provided with driving shoes such as Titus "H" Bearing Pile Point (Standard Model) or equivalent to facilitate penetration into/through the dense to very dense soils, cobbles and boulders and reduce the potential for damage to the piles during driving.

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 70,000 J.

The following "Pile Driving Note" should be included:

- Piles to be driven in accordance with MTO Standard Drawing SS 103-11 using ultimate geotechnical resistances of 2,800 kN per pile at bridge abutments and 2,200 kN at bridge piers. **Piles must be driven below Elevation 294 m and not below Elevation 284 m without the approval of the engineer**. {NOTE: This note is based on the use of 310x110 H-piles and the geotechnical resistances should be adjusted to '3,300 kN per pile at bridge abutments and 2,600 kN at bridge piers' at ULS if 360x108 H-piles are used}.

As outlined in MTO's Structural Manual (MTO, 2014), MTO's typical pile driving control tool is the Hiley Formula. The manual indicates that the Ultimate Pile Resistance (R) calculated in the field using the Hiley Formula must be greater than twice the Design Load at ULS determined by the structural engineer (which is less than or equal to two times the factored geotechnical resistance at ULS). The capacity of each pile should be verified in the field using the Hiley Formula (MTO Standard Structural Drawing SS-103-11) to confirm that the specified ultimate capacity is achieved. Overdriving of the piles needs to be avoided. In this regard, the Hiley formula testing should be conducted at regular intervals during pile installation to assess when pile capacities have been achieved.

In addition, high-strain dynamic testing (i.e. Pile Driving Analyzer (PDA) testing) is to be carried out on a minimum of 2 piles per pier/abutment to verify the ultimate pile capacities with CAPWAP analyses completed on each pile. The PDA testing is to be carried out at the end of initial drive (EOID). A Non-

Standard Special Provision (NSSP) should be included with the contract documents that notifies the contractor of the requirements for the high-strain dynamic / PDA testing. A sample NSSP has been included in Appendix G.

The “Hiley Formula Pile Resistance” and PDA test results for all piles shall be submitted to the project foundation engineer for comparison. In the event of a discrepancy between the calculated results, the capacities assessed by PDA testing will govern.

Driven piles generally gain capacity after driving has been completed and excess pore pressures have dissipated. If the specified ultimate resistance is not achieved during EOID testing, retesting using PDA methods should be completed on the same piles (after a minimum of 3 days to allow for soil setup to occur) to assess the associated gain in pile capacity.

7.6.1.4 Geotechnical Lateral Resistance

P-Y Curves

The response of a pile to lateral loads is a non-linear relationship. Non-linear elastic-plastic springs (i.e. p-y curves representing the load intensity per unit length of pile (p) versus the lateral deflection of the pile) can be used in evaluating the structural response of the pile in response to lateral loads.

The program LPile 2016 developed by Ensoft, Inc. (Ensoft, 2016) was used to develop p-y curves for 310x110 and 360x108 H-piles at this site. The geotechnical input parameters that were used in the analyses for the piles for the north side of the bridge (i.e. north abutment and north pier) and the south side of the bridge (i.e. south pier and south abutment) are displayed in Tables 7.1 and 7.2, respectively with strength parameters associated with the sand backfill placed within the CSP liners used for the top 3 m of the piles at the abutment locations.

The p-y curve values versus depth for the two piles sizes identified above at each foundation unit are presented in Tables E-1 to E-8 in Appendix E. These tables provide a series of curves obtained from the LPILE program generated for selected depths below the pile head. The p-y curves can be used in the structural evaluation of the H-piles noting that the p-y curves provided are unfactored and that appropriate resistance factors (i.e. as outlined in Table 6.2 of the CHBDC, 2014) should be applied when assessing the geotechnical lateral resistances of the piles at ULS and SLS.

Coefficient of Horizontal Sub-grade Reaction

The lateral resistance of pile foundations may also be evaluated for deflection and resistance using the coefficient of horizontal sub-grade reaction (k_s) and lateral resistance (p_{ult}) as follows:

For cohesionless soils:

$$k_s = n_h \cdot Z / D \qquad [kN/m^3]$$
$$p_{ult} = 3 \cdot Kp \cdot [\gamma D_w + (\gamma - \gamma_w)(z - D_w)] \qquad [kPa]$$

For cohesive soils:

$k_s = 67 \cdot S_u / D$ [kN/m^3]

$p_{ult} = 3 \cdot [\gamma z + 2\ S_u]$ [kPa]

where:

z: depth of embedment of pile	[m]
D: pile diameter	[m]
n_h: constant of horizontal subgrade reaction	[kPa/m]
γ: unit weight of soil	[kN/m^3]
γ_w: unit weight of water	(9.8 kN/m^3)
D_w: depth of groundwater below excavation base	[m]
K_p = passive earth pressure coefficient	[unitless]
S_u: undrained shear strength of cohesive soil	[kPa]

The soil stratigraphy and design parameters provided in Tables 7.1 and 7.2 and a design water table elevation of 303.3 m can be used in this analysis. A n_h value of 3000 kN/m^3 can be used for the sand backfill within the CSPs for integral abutments.

Appropriate resistance factors (i.e. as outlined in Table 6.2 of the CHBDC, 2014) should be applied when assessing the geotechnical lateral resistances and reactions of the piles at ULS and SLS.

Group Action

The horizontal resistance of piles should take into account group action of piles (pile interaction) in accordance with Section 6.11.3.4 and the associated commentary of the CHBDC.

The effect of interaction between piles can be considered by applying a reduction factor to the soil resistance (i.e. the p-multiplier) of a single pile to obtain p-y curves for the pile group. The reduction factors to be applied are dependent on the pile spacing/group geometry. The reduction factors (i.e. p-multipliers) outlined in Figures C6.11.3(r), C6.11.3(s) and C6.11.3(t) of Section C6.11.3.4 of the CHBDC should be used.

7.6.2 Foundations for Falsework Support System

A 'Falsework' support system is understood to be required to provide temporary support to the post-tensioned deck of the bridge during construction. The native, mineral soils at the site typically consist of cohesionless/granular deposits that are generally compact to depths of 5 m or more. These soils or the granular, highway embankment/pavement structure fill materials are considered suitable for the support of temporary foundations for the falsework support system. For preliminary assessment purposes with consideration for foundations having a dimension in the order of 1.5 m placed at the ground surface, these soils are considered capable of supporting loads of 100 kPa at SLS for 25 mm of settlement. The final factored geotechnical resistance at ULS and the corresponding geotechnical reaction at SLS will be dependent on the foundation geometry/size and founding depth and must be determined by contractor as part of their design.

As discussed further in Section 8.2.2., layers of buried soils containing organic materials were identified at the site. These soils will need to be removed or accounted for in the design of the falsework systems to provide satisfactory performance of the falsework support system.

7.7 LATERAL EARTH PRESSURES

7.7.1 Abutment Backfill

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge abutments. The materials used as backfill behind the proposed bridge abutments 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. If consideration is given to reducing the design soil cover for frost protection purposes at the abutment locations, the backfill type(s) and geometry discussed in Section 7.3 should be implemented.

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures. Backfill materials should meet the requirements of OPSS.PROV 1010 and be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.

7.7.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and retained soil systems (if any). These structures should be backfilled using imported free-draining granular fill materials meeting the gradation requirements of OPSS Granular A or Granular B Type I materials.

Computation of earth pressures should be in accordance with Section 6.17.3 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as outlined in Section 6.12.3 and as shown in Figure 6.6 of the CHBDC. Where applicable (i.e. where unbalanced water pressures may develop), the structures should also be designed to account for hydrostatic pressures.

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

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

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

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

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 Tables 7.7 and 7.8 for horizontal and sloping (2H:1V) backfill conditions, respectively. The thrust acts at a point one third up the height of the wall.

Table 7.7: Recommended Static Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Granular B Type I	OPSS Gran. A and Gran. B Type II
Bulk Unit Weight, γ (kN/m3)	21	22
Effective Friction Angle	32º	35º
Coefficient of Earth Pressure at Rest (Ko)	0.47	0.43
Coefficient of Active Earth Pressure (Ka)	0.31	0.27
Coefficient of Passive Earth Pressure (Kp)	3.25	3.7

Table 7.8: Recommended Static Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Granular B Type I	OPSS Gran. A and Gran B. Type II
Bulk Unit Weight, γ (kN/m3)	21	22
Effective Friction Angle	32º	35º
Coefficient of Earth Pressure at Rest (Ko)	0.68	0.62
Coefficient of Active Earth Pressure (Ka)	0.47	0.39

7.7.3 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 under seismic loading conditions.

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

$$P_{AE} = ½\, K_{AE}\, \gamma\, H^2 (1 - k_V)$$

$$P_{PE} = ½\, K_{PE}\, \gamma\, H^2 (1 - k_V)$$

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

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values as per CHBDC 2014.

Table 7.9: Seismic Design Parameters to Estimate Lateral Earth Pressures

Site Adjusted PGA	Horizontal Acceleration Coefficient, k_{ho}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding $(wall\ movements\ of\ 25\ mm\ to\ 50\ mm)$
0.08g	0.08	0.04
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 2014, section C4.6.5.		

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

The seismic earth pressures may be calculated using the parameters detailed in Tables 7.10 and 7.11 for horizontal and 2H:1V backfill configurations, respectively.

Table 7.10: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran. B Type I	OPSS Gran. A and Gran. B Type II
Bulk Unit Weight, γ (kN/m^3)	21	22
Effective Friction Angle	32º	35º
Passive Earth Pressure, (K_{PE})	3.18	3.61
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.3277	0.328
Yielding Wall		
Active Earth Pressure (K_{AE}) for Yielding Wall	0.33	0.29
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.352	0.353
Non-Yielding Wall		
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.35	0.32
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.369	0.371

Table 7.11: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran. B Type I	OPSS Gran. A and Gran. B Type II
Bulk Unit Weight, γ (kN/m^3)	21	22
Effective Friction Angle	32º	35º
Passive Earth Pressure, (K_{PE})	8.52	10.71
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.331	0.331
Yielding Wall		
Active Earth Pressure (K_{AE}) for Yielding Wall	0.54	0.45
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.369	0.365
Non-Yielding Wall		
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.66	0.51
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.411	0.396

7.8 EMBANKMENT DESIGN CONSIDERATIONS

No signs of substantial differential settlement (e.g. dips in the Roseville Road surface leading up to the bridge, buried or exceptionally thick asphalt layers indicative of previous padding activities etc.) were noted between the bridge structure and approach embankments during the foundation investigation.

The existing approach embankments are proposed to be raised by up to approximately 1.5 m with associated minor widening on both sides of the embankment. Typically, the new, widened embankments will have maximum embankment heights of about 8 m and side slopes of 2H:1V. As per OPSD 202.010, a mid-slope bench should be implemented on the widened embankment side slopes if embankment heights exceed 8 m.

All raised and widened embankments are recommended to be constructed using granular fill materials meeting the requirements of OPSS Granular B Type 1 or Type 2 materials or Select Subgrade materials. Any embankment widening should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

7.8.1 Slope Stability Evaluation

A slope stability evaluation was carried out for each of two critical cross-section of the new raised and widened approach embankment (i.e. sections near the south abutment corresponding to the greatest embankment height and weakest subgrade soils) using a commercial program Slope/W (Geo-Slope, 2010). Analyses were completed for one cross-section oriented perpendicular to Roseville Road and one cross section extending north from the new south abutment towards the Cedar Creek channel/Highway 401 based on embankment geometries/cross-sections provided by the design team. The analyses incorporated the design parameters outlined in Section 7.2 and included allowance for dynamic loading due to traffic by considering a static surcharge load equivalent to 0.8 m of additional fill as per Section 6.9.5 of the CHBDC.

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

The results of the slope stability analysis for static, drained conditions for the cross-sections in the area of the south abutment foreslope and perpendicular to Roseville Road and are provided in Figures E3 and E4 in Appendix E, respectively. The results of the stability analyses indicate that the proposed embankment configurations, which incorporate slope angles of 2H:1V, would provide a factor of safety against instability of 1.5 for a critical failure surface extending up to the crest of the embankment. A factor of safety of greater than 1.4 was calculated under seismic conditions. Stability analyses carried out using undrained parameters provided similar or higher factors of safety.

7.8.2 Embankment Settlements

Settlement of the soils underlying the embankments due to the raising and widening of the embankments was evaluated. The following assumptions were made in this respect:

- The typical soil profile and associated design parameters for the south side of the bridge shown in Table 7.2 were considered in the settlement analyses. Settlements on the north side of the bridge are be expected to be similar or less based on the subsurface conditions.
- The maximum height of the embankment grade raise is limited to about 1.5 m.
- The new embankment platform, which is understood to have a crest-to-crest width of about 16 m, has been assumed to be centered over the existing embankment platform.
- The load from the bridge abutments will be transferred to deep, very dense strata by the piles and hence will not contribute significantly to the settlement of the embankment.
- The estimated preconsolidation pressures of the clayey silt deposits are expected to be higher than the anticipated post-construction stresses in these deposits. Therefore, substantial consolidation settlements of the cohesive native soils are not expected to occur and only immediate (elastic) settlement was considered in the analyses.
- Soil moduli used in the analyses were based on typical values provided in the CHBDC.

The evaluation of settlements for the embankment on the south side of the highway was carried out using the commercial program Settle3D (Rocscience 2009) and incorporated the design parameters outlined in Section 7.2. Settlements on the north side of highway are expected to be similar to or less than the south side based on the subsurface conditions. The analysis included evaluation of settlements under the current embankment configuration and the higher, wider embankment planned for the new underpass; the settlement model conservatively incorporated a 100 m length of embankment with the maximum heights of the current and planned embankments. The results of the analysis for these two conditions are presented in Figures E-5 and E-6 in Appendix E. The results of the analyses indicate that, for the conditions presented herein, the maximum incremental vertical settlement of the native soils beneath the approach embankments leading up to the new bridge is expected to be less than 25 mm due to the additional loading imposed by the proposed grade raise in these areas. Settlements of less than 15 mm are expected at the abutment locations. These settlements are anticipated to take place relatively rapidly and to be predominantly complete during construction of the embankments.

Self-weight settlement due to compression of the maximum 1.5 m of embankment fill placed during the construction process is expected to be less than 10 mm (approximating 0.5 % strain). Similarly, the settlement of the existing underlying embankment fill materials resulting from the raising of the embankment is expected to be approximately 5 mm or less. The bulk of this settlement is expected to be completed almost immediately after the fill has achieved its full height.

7.9 EROSION AND SCOUR PROTECTION

The near-surface soils present in the vicinity of the abutment locations include highly variable fill materials, ranging in composition from clayey silt with sand to sandy gravel, and native cohesionless soils that include zones of silt/sandy silt/silty fine sand. The fine sand and silty materials are expected to be highly susceptible to erosion and scour. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g., rip-rap within water channels, granular sheeting or concrete slope protection below abutments) be provided for/within the portion of the Cedar Creek channel present between the south abutment and south pier (i.e. the section of the channel being 'daylighted' as a result of the culvert removal) and for any embankment fill slopes adjacent to the abutments (e.g. in the abutment foreslope areas) to protect the foundations/pile caps from being exposed. The scour and erosion protection should be designed in accordance with Section 1.9.4 of the CHBDC. The rip rap or granular sheeting should be consistent with the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous).

Vegetation on the new highway embankment slopes should be established as soon as possible after completion of the embankment construction to minimize the potential for surficial erosion. The supply and placement of topsoil and seed materials to establish vegetation on the embankments should meet the requirements of OPSS 802 (Construction Specification for Topsoil) and OPSS.PROV 804 (Construction Specification for Seed and Cover).

The above treatments should be applied at the discretion of the Hydraulic Design Engineer.

7.10 CEMENT TYPE AND CORROSION POTENTIAL

Two samples of the site soils were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to provide data for assessing 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 Section 4.6.

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 maximum soluble sulphate concentration for the two samples tested was 42 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The test results provided in Table 4.3 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. The soil pH values were 7.8 and 7.9 which are within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. However, a resistivity value of 10.3 ohm-meters was measured in the sample of the fill material tested indicating a highly to severely corrosive environment.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 CONSTRUCTION STAGING AND DETOUR

A local detour is anticipated to be required for the construction of the new underpass structure as Roseville Road is planned to be closed to traffic during the construction of the new bridge.

The construction of the foundations for the new central pier of the bridge is anticipated to involve staging and lane-reductions on Highway 401 using appropriate traffic control. The use of a temporary roadway protection system may be required near the centerline of existing Highway 401.

8.2 TEMPORARY WORKS

8.2.1 Temporary Roadway Protection

Temporary roadway protection is anticipated to form part of the staged construction approach that will be required to protect traffic on Hwy 401 during excavations for the construction of both pier foundations.

In addition, the foundation for the proposed south bridge pier will be located in close proximity to, and extend below the water level in, the adjacent Cedar Creek channel/ditch. As such, a temporary excavation support system may be required to separate/isolate the excavation from the adjacent channel to facilitate construction of this pier foundation.

Based on the subsurface conditions at the site, the following table compares the available roadway protection options considered for the bridge pier foundation excavations adjacent to Highway 401.

Table 8.1: Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet piles (SSP)	• Simple installation process • Provides cut-off to groundwater seepage from sides of excavation thereby reducing overall dewatering volumes	• Difficult to drive/install where cobbles and/or boulders are present • May require large sections where cantilever design is adopted	Medium	• Possible damage to sheet piles during driving
Soldier piles with timber lagging; (struts/rakers as required)	• Simple installation process	• Additional labour required • Groundwater control (dewatering) must be implemented prior to installation where wall extends below water level. • Removal of soldier piles can be difficult	Low	• Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented • Potential for minor loss of ground at rear of lagging

Both support systems are considered feasible for use at this site provided that the water table is lowered to below the base of the excavations prior to installation. However, the use of steel sheet pile walls / enclosures is considered to be more practical where excavations will extend below the groundwater level as the use of this type of system will reduce dewatering requirements.

Table 8.2 below provides unit weight and lateral earth pressure coefficients that can be considered in the preliminary assessment of the TPS systems at the new bridge piers. The contractor will ultimately be responsible to develop and implement a roadway protection system/temporary excavation support system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters for use in the detailed design of the TPS systems.

Table 8.2: Earth Pressure Parameters (Temporary Protection Systems at Bridge Piers)

Parameter	Existing Fill (Ground Surface to Elev. 303)	Compact SAND to Gravelly SAND (Below Elev. 303)
Bulk Unit Weight, γ (kN/m^3)	23	22
Effective Friction Angle, ϕ' (degrees)	30º	32º
Coefficient of Earth Pressure at Rest (Ko)	0.5	0.53
Coefficient of Active Earth Pressure (Ka)	0.33	0.31
Coefficient of Passive Earth Pressure (Kp)	3	3.2

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

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

8.2.2 Falsework Supports

A ‘Falsework’ support system is understood to be required to provide temporary support to the post-tensioned deck of the bridge during construction. The contractor will be fully responsible for the design and installation/implementation of the falsework system.

A buried topsoil layer was encountered beneath the pavement structure fill in one of the boreholes (BH19-03) drilled in the highway median near the existing center pier. The presence of this material, which extended to a depth of 1.1 m, indicates that the surficial topsoil/organics may not have been stripped entirely before fill placement. Topsoil/peaty topsoil deposits were also encountered beyond the toe of the embankment in boreholes on the south side of Roseville Road, east of the highway. The application of loads from the falsework system could result in compression/settlement of soils containing organic matter;

these soils should either be removed or be accounted for in the design and performance of the falsework systems.

A Non-Standard Special Provisions (NSSP) should be included in the Contract to alert the contractor to this issue.

8.3 EXCAVATION AND BACKFILLING

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

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 should be benched into the existing embankments in accordance with OPSD 208.010.

All side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects (OHSA). The excavations required for the new abutments will extend into the existing approach embankment fill to depths in the order of 3 m to 4 m below the existing Roseville road grade. The excavations required for construction of the pier foundations are expected to encounter topsoil, fill materials and native soils varying in composition from silty sand to gravelly sand/sand and gravel and extend below the measured groundwater levels at the site.

Where space permits, these excavations may be developed using open-cut methods. The existing fill materials and compact sandy native soil deposits above the water table would be classified as Type 3 soils. OHSA indicates that temporary excavations in these materials above the water table should be developed with side slopes no steeper than 1H:1V. Granular soils (fill materials and/or native overburden) below the water table would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements.

8.4 TEMPORARY SURFACE WATER AND GROUNDWATER CONTROL

The water level elevations measured/observed during the foundation investigation program were typically in the range of 303 m to 303.5 m which is 1 m or less below portions of the travelled surface of Highway 401. In addition, it is noted that water levels at the site will fluctuate with the water level in the Cedar Creek channel which is anticipated to fluctuate/rise quickly as a result of precipitation events.

Excavations for construction of the bridge pier foundations and for the removal of the existing box culvert and associated regrading of the Cedar Creek channel will extend approximately 1 m to 1.5 m below the groundwater table and the water level in the creek channel.

The near surface soils present at the site are typically comprised of highly permeable granular soils. Significant groundwater inflows will occur into excavations extending into the saturated sand to sand and gravel materials present within the excavation zones for the new pier foundations and culvert removal.

The design of dewatering, unwatering, and temporary flow passage systems is the responsibility of the contractor. Depending on the water taking/dewatering volumes and source(s) of water, the dewatering activities may require a Permit to Take Water (PTTW) from the Ministry of Environment, Conservation and Parks (MOECP) or registration of the water taking activity in the Environmental Activity and Sector Registry (EASR). The permit/registration requirements are outlined in Table 1.0 of CDED B517.

Groundwater control consisting of external dewatering systems (e.g. well points or dewatering wells) could be used to control inflows and lower the water level in these deposits. Cut-off systems (e.g. steel sheet pile enclosures extending below the base of the excavations in conjunction with internal dewatering) could be considered to reduce the volume of dewatering required.

Control of the surface water, including creek flows, will also be necessary to allow excavation, foundation construction in the south pier area, and regrading of the creek channel to be carried out in dry conditions. Surface water should be directed away from the area of the planned excavations and creek water diverted by pumping from behind temporary cofferdams.

All groundwater control and temporary flow passage systems required for the culvert rehabilitation works should be designed and implemented in accordance with NSSP No. FOUN 0003 (Amendment to OPSS 902) Dewatering Structure Excavations. The following inputs should be included in NSSP No. FOUN 0003:

- Section 902.04.01.01 (Dewatering) – The temporary flow passage should be designed for a design storm return period of 2 years;
- Section 902.04.02.02 (Preconstruction Survey) – Given that the extent of dewatering would only require lowering of the groundwater level slightly (i.e. ~1 to 2 m below measured levels), off-site impacts are anticipated to be minor. A condition survey of structures and wells within 100 m of the site is recommended to be specified in this section.

A version of NSSP No. FOUN 0003 incorporating the inputs identified above is provided in Appendix G.

8.5 OBSTRUCTIONS

Cobbles and/or boulders are present in the fill and native cohesionless soil deposits that underlie this site. These materials could obstruct excavations and the installation of deep foundations and temporary roadway protections systems.

The foundation for the north pier of the new bridge coincides with the location of the central pier of the existing bridge which is supported on H-Pile foundations, including battered/inclined piles. The existing bridge foundations (e.g. piles and pile caps) could also obstruct the installation of pile foundations for the replacement bridge and temporary roadway protections systems. It is understood that portions of the existing pier pile caps that overlap with the proposed pile caps will be removed and the locations of the new pile foundations will be selected to avoid conflicts with the existing piles. Following removal of the existing pier pile cap(s), the locations of the existing piles should be surveyed to confirm that they will not conflict/interfere with the installation of the new piles.

Non-Standard Special Provisions (NSSPs) should be included in the contract to address these items. Draft versions of sample NSSPs are provided in Appendix H.

8.6 PILE INSTALLATION

Extraction of the existing piles is not recommended as this may result in loosening of the surrounding soils and could lead to settlement of the ground surface and/or instability of adjacent infrastructure. The proposed pile foundations should be installed to avoid conflicts with existing piles to avoid damage to the new piles. Once the existing piles are exposed and the locations are verified, the Contractor should be required to identify any potential conflicts with existing piles and/or any locations where new piles are not a minimum of one pile width from any existing piles prior to pile installation.

The Contractor should also be required to stop pile driving operations and inform the Contract Administrator immediately in the event that an existing pile is encountered during driving of new piles. The Contractor should have suitable equipment available to extract and relocate the new pile if required.

The piles are to be installed between the specified pile tip elevation range of 284 m to 294 m and tested to confirm that the design pile capacities are achieved.

The Contractor should be required to select/supply pile lengths that avoid pile splices within the CSP zone of the integral abutments and that allow for sufficient pile stick-up above ground surface to permit PDA testing of piles.

9.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 9.1: Specifications Referenced in Report

Document	Title
OPSD 202.010	Slope Flattening Using Surplus Excavated Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSS.PROV 206	Construction Specification for Grading
OPSS 802	Construction Specification for Topsoil
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protections Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 1004	Material Specification for Aggregates
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
SP 206S03	Earth Excavation, Grading

10.0 REFERENCES

Canadian Foundation Engineering Manual (CFEM). 1992 and 2006. Third and Fourth Editions. Canadian Geotechnical Society

CHBDC. 2014. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

Dominion Soil Investigation Ltd. November 6, 1958. Subsoil Investigation of Site of Proposed Highway 401 and Old Highway 97 Grade Separation, N. Dumfries Township, Near Galt (GEOCRES Reference No. 40P08-017).

Ensoft, 2016. User's Manual for Computer Program LPILE Plus Version 6.0. Ensoft, Inc., Austin, Texas.

GEO-SLOPE International Ltd. 2010. Stability Modeling with SLOPE/W 2012©. Calgary, AB.

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

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

Rocscience, Settle3D Settlement and Consolidation Analysis, v 3.009: Theory Manual, Rocscience, Inc.

11.0 CLOSURE

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

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

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

Respectfully submitted,

STANTEC CONSULTING LTD.

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

John J. Brisbois, MScE., P. Eng.
MTO Designated Principal Foundation Contact

APPENDIX A

A.1 DRAWING NOS. 1 TO 2 – BOREHOLE LOCATION PLANS AND SOIL STRATA PLOT

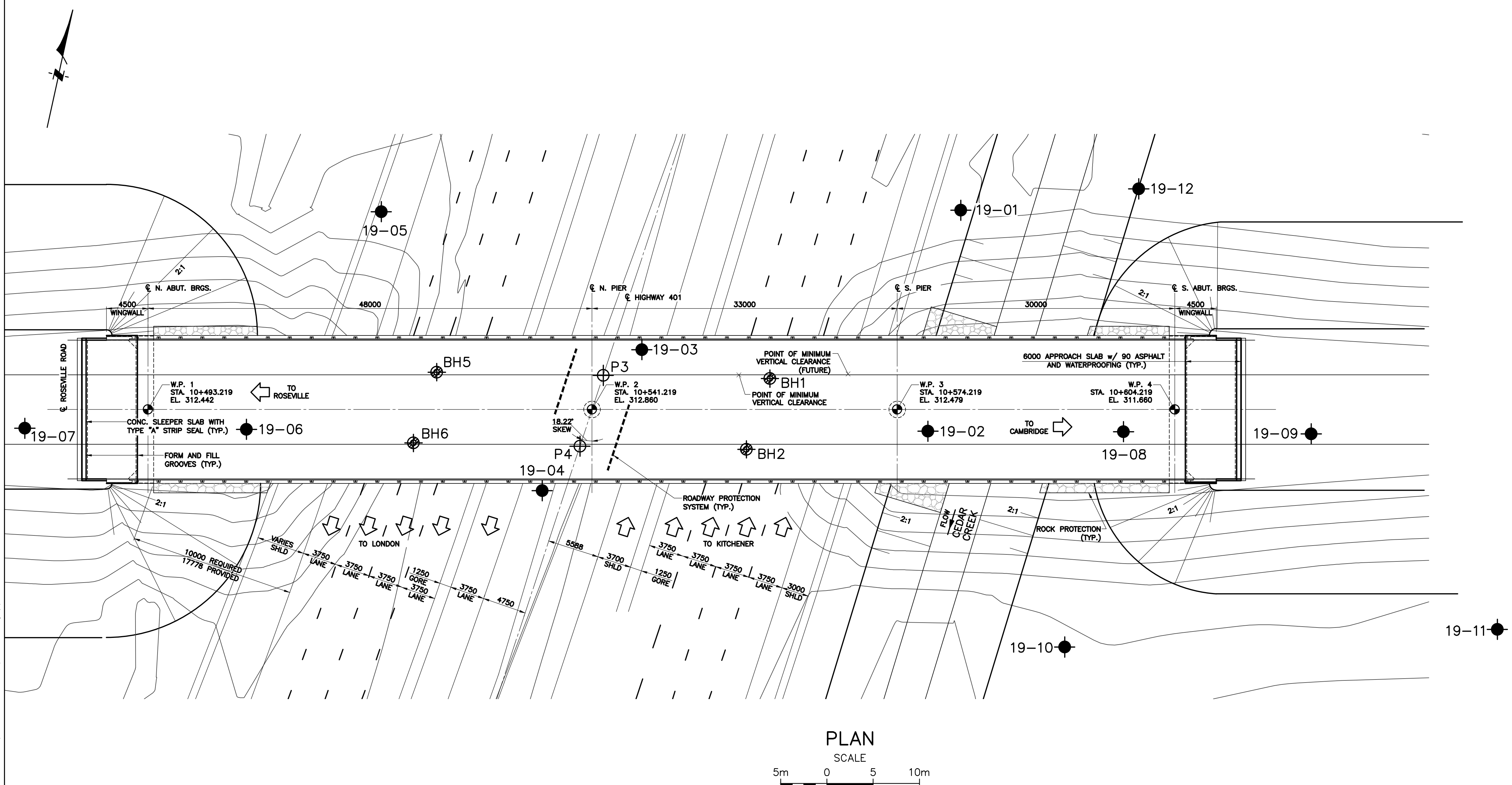
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

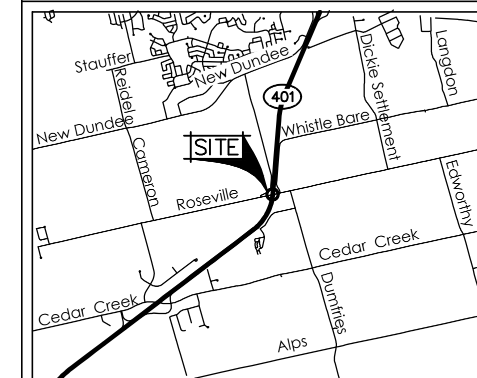
PLATE No
CONT
GWP 3204-16-00

HIGHWAY 401 & ROSEVILLE ROAD, CAMBRIDGE, ON.
BOREHOLE LOCATIONS

SHEET
–

Stantec





LEGEND

- Borehole
- Borehole by Others (Approx. Location)
- Dynamic Cone Penetration Test by Others (Approx. Location)

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	EAST
19–01	304.2	4 801 720.9	230 521.8
19–02	310.7	4 801 696.8	230 523.4
19–03	304.2	4 801 698.8	230 491.3
19–04	304.5	4 801 682.5	230 484.2
19–05	304.4	4 801 707.3	230 460.6
19–06	310.6	4 801 681.2	230 451.4
19–07	310.1	4 801 676.2	230 428.0
19–08	310.3	4 801 701.3	230 544.1
19–09	309.7	4 801 705.4	230 563.9
19–10	304.0	4 801 677.1	230 542.6
19–11	303.8	4 801 689.1	230 588.1
19–12	303.5	4 801 727.3	230 540.0
BH1	–	4 801 698.7	230 505.5
BH2	304.1	4 801 690.7	230 504.6
P3	303.4	4 801 695.2	230 487.8
P4	304.0	4 801 687.2	230 487.0
BH5	303.4	4 801 691.7	230 470.2
BH6	304.3	4 801 683.6	230 469.3

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

REVISIONS		
DATE	BY	DESCRIPTION

GEOCRES No

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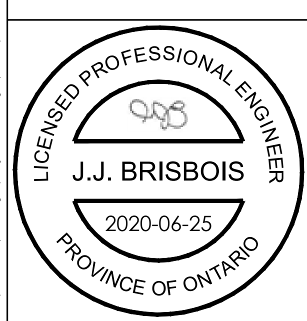
SUBM'D KN	CHECKED	DATE 2020–01–14	SITE 33X–0177/B0
DRAWN GBB	CHECKED	APPROVED	DWG 1



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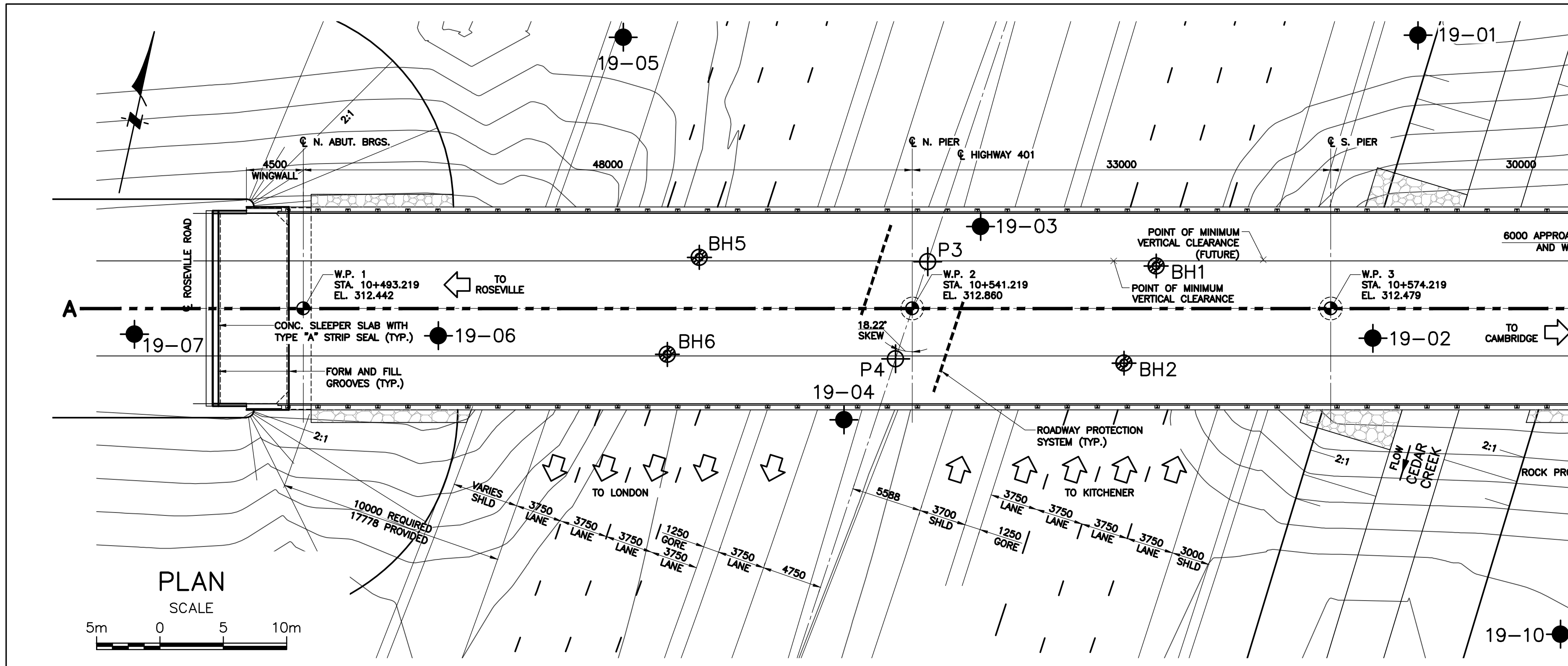
MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05



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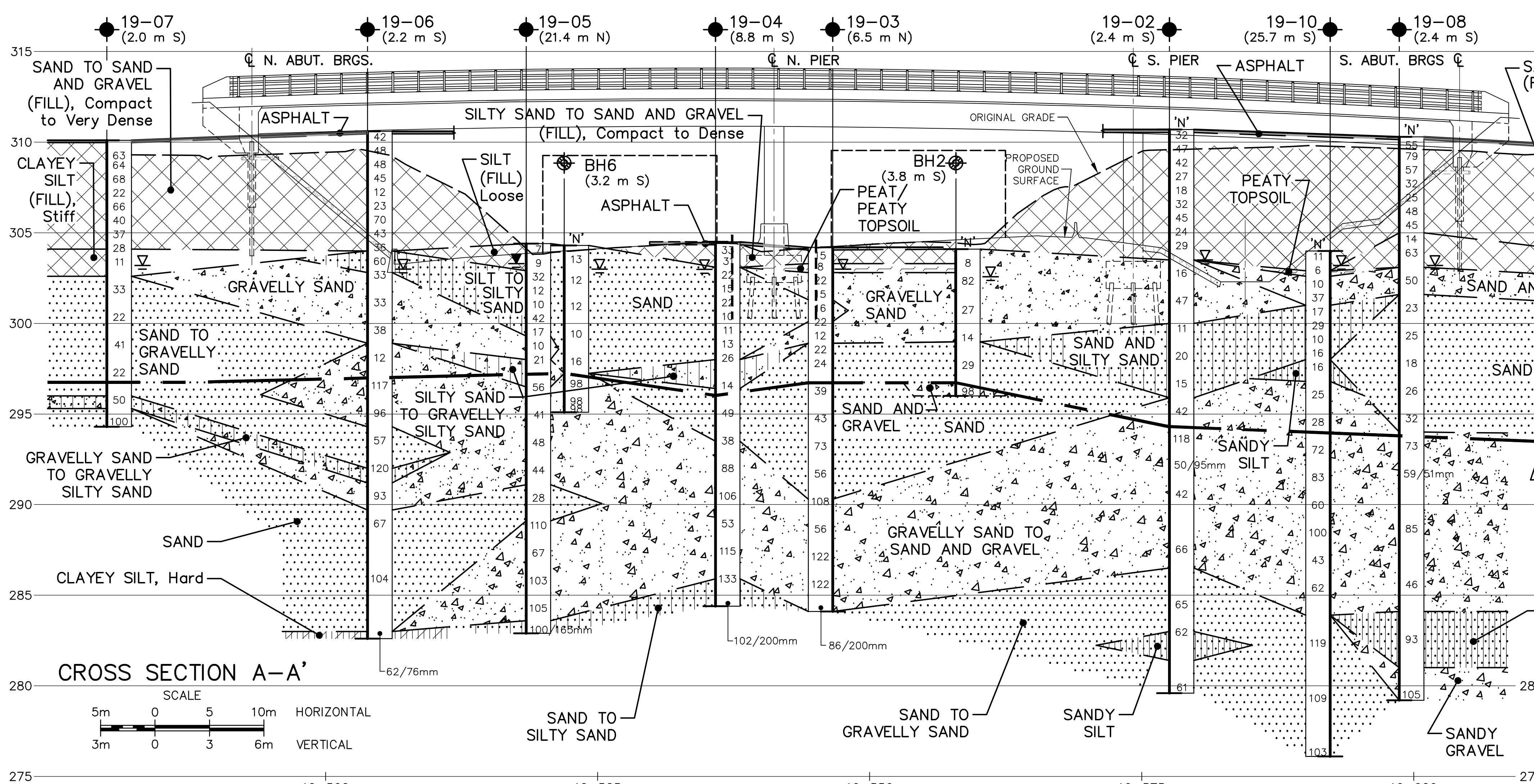


=NOTES=

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.

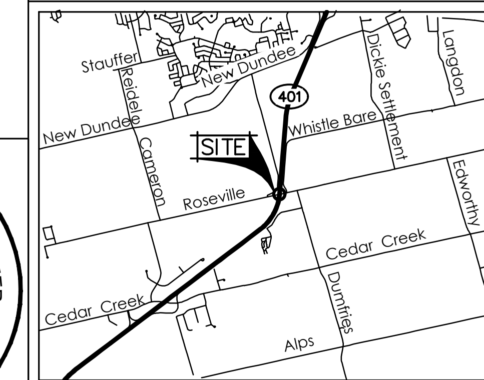
PLATE No

CONT
GWP 3204-16-00

HIGHWAY 401 & ROSEVILLE ROAD, CAMBRIDGE, ON.
BOREHOLE LOCATIONS & SOIL STRATA

SHEET
–

Stantec



KEY PLAN
1.5 km 0 1.5 3.0 km

LEGEND

- Borehole
- Borehole by Others (Approx. Location)
- Dynamic Cone Penetration Test by Others (Approx. Location)
- (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 2019
- WL Measured

No	ELEVATION	MTM ZONE 10 COORDINATES NORTH	EAST
19-01	304.2	4 801 720.9	230 521.8
19-02	310.7	4 801 696.8	230 523.4
19-03	304.2	4 801 698.8	230 491.3
19-04	304.5	4 801 682.5	230 484.2
19-05	304.4	4 801 707.3	230 460.6
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19-10	304.0	4 801 677.1	230 542.6
19-11	303.8	4 801 689.1	230 588.1
19-12	303.5	4 801 727.3	230 540.0
BH1	–	4 801 698.7	230 505.5
BH2	304.1	4 801 690.7	230 504.6
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BH5	303.4	4 801 691.7	230 470.2
BH6	304.3	4 801 683.6	230 469.3

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

REVISIONS DATE	BY	DESCRIPTION

GEOCRES No

HWY No 401			DIST WEST
SUBM'D KN	CHECKED	DATE 2020-01-18	SITE 33X-0177/B0
DRAWN GBB	CHECKED	APPROVED	DWG 2

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MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05

APPENDIX B

B.1 SYMBOLS AND TERMS USED ON STANTEC BOREHOLE RECORDS

B.2 BOREHOLE RECORDS

B.3 BOREHOLE RECORDS FROM PREVIOUS INVESTIGATION (GEOCRES REPORT NO. 40P08-017)

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

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

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

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

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

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

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

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

ROCK DESCRIPTION

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

Terminology describing rock quality:

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

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

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

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

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

Terminology describing rock with respect to discontinuity and bedding spacing:

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

Terminology describing rock strength:

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

Terminology describing rock weathering:

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

STRATA PLOT

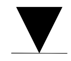
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.

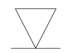
Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Igneous Bedrock	Meta-morphic Bedrock	Sedi-mentary Bedrock

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

Stantec Borehole Records

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No BH19-01

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.04.30	LATITUDE 43.3517087 LONGITUDE -80.4162639	CHECKED BY KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
304.2	TOPSOIL & GRASS
304.2 / 0.0	SILTY SAND (TOPSOIL), trace organic matter Dark brown Wet
304.0 / 0.2	SILTY SAND to SANDY GRAVEL (SM-GM) (Possible FILL), occasional cobbles Loose to compact Brown Moist, wet below 0.9 m
302.7 / 1.5	Gravelly SAND to SAND and GRAVEL (SP/GP), trace silt to silty, occasional cobbles Dense Brown Wet Compact below 3.8 m
299.7 / 4.5	SAND (SP) some silt to SILTY SAND (SM), trace gravel Contains sand and gravel seams less than 0.1 m in thickness 0.1 m thick SILT layer at 5 m depth Compact Brown Wet
297.0 / 7.2	SILTY SAND (SM), some gravel (TILL-LIKE Composition) Dense Grey-brown Wet
296.3 / 7.9	SILT (ML), trace sand Contains sand seams Dense Grey Wet
295.5 / 8.7	GRAVELLY SAND to SAND and GRAVEL (SP/GP), trace silt to silty Contains cobbles and boulders Very dense Grey-brown Wet
289.2	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	9	
2	SS	23	
3	SS	40	
4	SS	40	
5	SS	31	
6	SS	17	33 61 4 2
7	SS	16	0 4 96 0
8	SS	17	
9	SS	18	
10	SS	34	16 33 43 8 Non-Plastic
11	SS	110	
12	SS	101	33 56 9 2
13	SS	78	
14	SS	60	

GROUND WATER CONDITIONS | ELEVATION SCALE: 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No BH19-01

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY KT

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.04.30 LATITUDE 43.3517087 LONGITUDE -80.4162639 CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
15.0	GRAVELLY SAND to SAND and GRAVEL (SP/GP), trace silt to silty Contains cobbles and boulders Dense Wet	15	SS	35		
		16	SS	49		
286.0 18.2	SILTY FINE SAND to SILT and SAND (SM) Very dense Grey-brown Wet Laminated	17	SS	146		0 62 36 2
		18	SS	150		
		19	SS	130		
280.9 23.3	End of Borehole					

Elevation scale: 289, 288, 287, 286, 285, 284, 283, 282, 281

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No BH19-02

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY DL

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.05.07 LATITUDE 43.3514919 LONGITUDE -80.4162402 CHECKED BY KN

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.7	TOPSOIL & GRASS					
310.5 / 0.0	165 mm ASPHALT					
0.2	SAND (SP), some silt and gravel (FILL) Contains zones of SILTY SAND (SM) and occasional silt pockets Compact to dense Brown Moist	1	SS	32		
		2	SS	47		
		3	SS	42		10 73 15 2
		4	SS	27		
		5	SS	18		
307.0 / 3.7	Gravelly SAND (SP), some silt (FILL) Contains occasional cobbles Compact to dense Brown Moist	6	SS	32		
		7	SS	45		33 54 10 3
		8	SS	24		
304.8 / 5.9	SILTY SAND (SM) (FILL) Compact Dark brown to black Moist					
304.6 / 6.1	Gravelly SAND (SP), some silt (FILL) Compact Brown Wet	9	SS	29		
304.2 / 6.6	SILTY SAND (SM) (FILL) Compact Dark brown to grey Compact Moist to wet				∇	
303.5 / 7.2	SAND (SP), some gravel to gravelly, trace silt Contains occasional cobbles ~225mm thick gravel layer at 9.4 m depth Compact to dense Brown Wet	10	SS	16		
		11	SS	47		
300.0 / 10.7	SILTY SAND (SM) to SAND (SP), some silt, occasional gravel seams Compact Brown Wet	12	SS	11		
		13	SS	20		0 77 21 2
		14	SS	15		
295.9 / 14.8						

Chart columns: STRAT PLOT; ELEVATION SCALE (310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m^3.

Continued Next Page

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

ONTARIO MTO 165001107 HWY 401 AND ROSEVILLE ROAD BRIDGE.GPJ ONTARIO MTO.GDT 18/2/20

RECORD OF BOREHOLE No BH19-02

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY DL
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.05.07	LATITUDE 43.3514919 LONGITUDE -80.4162402	CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT w (approx. %)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL (SP/GP) to gravelly SAND (SP), trace silt Frequent cobbles Dense to very dense Grey Wet *(continued)*	15	SS	42		
		16	SS	118		
292.9 17.8	SAND and GRAVEL (SP/GP), some silt (TILL-LIKE Composition) Very dense Grey Wet	17	SS	50/ 95mm		38 46 13 3
291.3 19.4	Gravelly SAND (SP) to sandy GRAVEL (GP), trace silt Contains cobbles and/or boulders Dense to very dense Grey Wet	18	SS	42		
	Auger grinding/bouncing on cobble or boulder near 22 m depth	19	SS	66		62 31 5 2
286.5 24.2	SAND (SP), some gravel to gravelly, trace silt Very dense Grey Wet	20	SS	65		
283.0 27.7	Sandy SILT (ML) Very dense Grey Wet	21	SS	62		0 37 54 9 Non-Plastic
281.4 29.3	Gravelly SAND (SP), trace silt Very dense Grey Wet					

Elevation scale: 295, 294, 293, 292, 291, 290, 289, 288, 287, 286, 285, 284, 283, 282, 281

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No BH19-02

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY DL

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.05.07 LATITUDE 43.3514919 LONGITUDE -80.4162402 CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20, 40, 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Gravelly SAND (SP), trace silt Very dense Grey Wet *(continued)*		22	SS	61		280		○		
279.6 31.1	End of Borehole										

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

RECORD OF BOREHOLE No BH19-03

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.04.15	LATITUDE 43.3515066 LONGITUDE -80.4166366	CHECKED BY KN

Column headings: SOIL PROFILE (ELEV/DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
304.2	ASPHALTIC CONCRETE				
304.0 / 0.0	150 mm ASPHALT				
0.2	SAND and GRAVEL (SP/GP), trace silt and asphalt to SILTY SAND (SM) trace gravel (FILL) Compact Brown	1	SS	15	
303.3 / 0.9	Peaty TOPSOIL Loose Black	2	SS	8	Org. Content SS2A = 1.8 %
303.1 / 1.1	SILTY SAND (SP), trace clay and gravel Loose Brown Moist				
302.8 / 1.4	Gravelly SAND (SP), some silt, some cobbles Compact Grey Wet	3	SS	22	32 54 11 3
		4	SS	15	
301.3 / 2.9	SAND (SP), some silt, occasional cobbles Compact Grey Wet	5	SS	16	
300.1 / 4.1	Gravelly SAND (SP), some silt, occasional cobbles Compact Grey to grey-brown Wet	6	SS	22	
		7	SS	12	32 54 13 1
298.9 / 5.3	SAND (SP), some gravel, trace to some silt Contains zones/inclusions of sand to sandy silt Compact to dense Grey Wet	8	SS	22	
		9	SS	24	
		10	SS	39	
295.5 / 8.7	SAND (SP), trace to some sand, trace silt Dense Grey Wet	11	SS	43	
294.0 / 10.2	SAND (SP), trace to some silt and gravel Very dense Grey Wet	12	SS	73	
		13	SS	56	
290.3 / 13.9	SAND and GRAVEL (SP/GP) to sandy GRAVEL (GP), trace to some silt Occasional cobbles Very dense Grey	14	SS	108	55 35 9 1
289.2					

Elevation scale: 304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290

Continued Next Page

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

RECORD OF BOREHOLE No BH19-03

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY KT

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.04.15 LATITUDE 43.3515066 LONGITUDE -80.4166366 CHECKED BY KN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT	GR SA SI CL
15.0	Wet SAND and GRAVEL (SP/GP) to sandy GRAVEL (GP), trace to some silt Occasional cobbles Very dense Grey Wet				289		
		15	SS	56		o	
					288		
		16	SS	122	287	o	55 39 5 1
					286		
		17	SS	122		o	
					285		
284.1		18	SS	86/200mm		o	
20.1	End of Borehole						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

ONTARIO MTO 165001107 HWY 401 AND ROSEVILLE ROAD BRIDGE.GPJ ONTARIO MTO.GDT 18/2/20

RECORD OF BOREHOLE No BH19-04

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.05.14	LATITUDE 43.3513593 LONGITUDE -80.4167215	CHECKED BY KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
304.5	
304.3 / 0.0	165 mm ASPHALT
0.2	Gravelly SAND (SP), some silt (FILL) Dense Brown Moist
303.6 / 0.9	Gravelly SAND (SP), some silt, contains cobbles and trace asphalt (FILL) Dense Grey-brown Moist
303.1 / 1.4	SAND (SP), trace to some silt Contains occasional cobbles and seams containing some gravel Compact Grey-brown Wet
300.4 / 4.1	SAND (SP), trace to some gravel, trace silt and occasional cobbles Compact Grey-brown Wet
298.0 / 6.5	Silty gravelly SAND (SM) (TILL-LIKE Composition) Contains cobbles and boulders Compact Grey-brown Wet
296.4 / 8.1	SAND (SP), some silt Contains cobbles, boulders and layers of sand and gravel Dense Grey-brown Wet Grinding of augers below 10 m
293.5 / 11.0	SAND and GRAVEL (SP/GP), trace to some silt Contains occasional cobbles and/or boulders Very dense Grey-brown Wet
289.5	

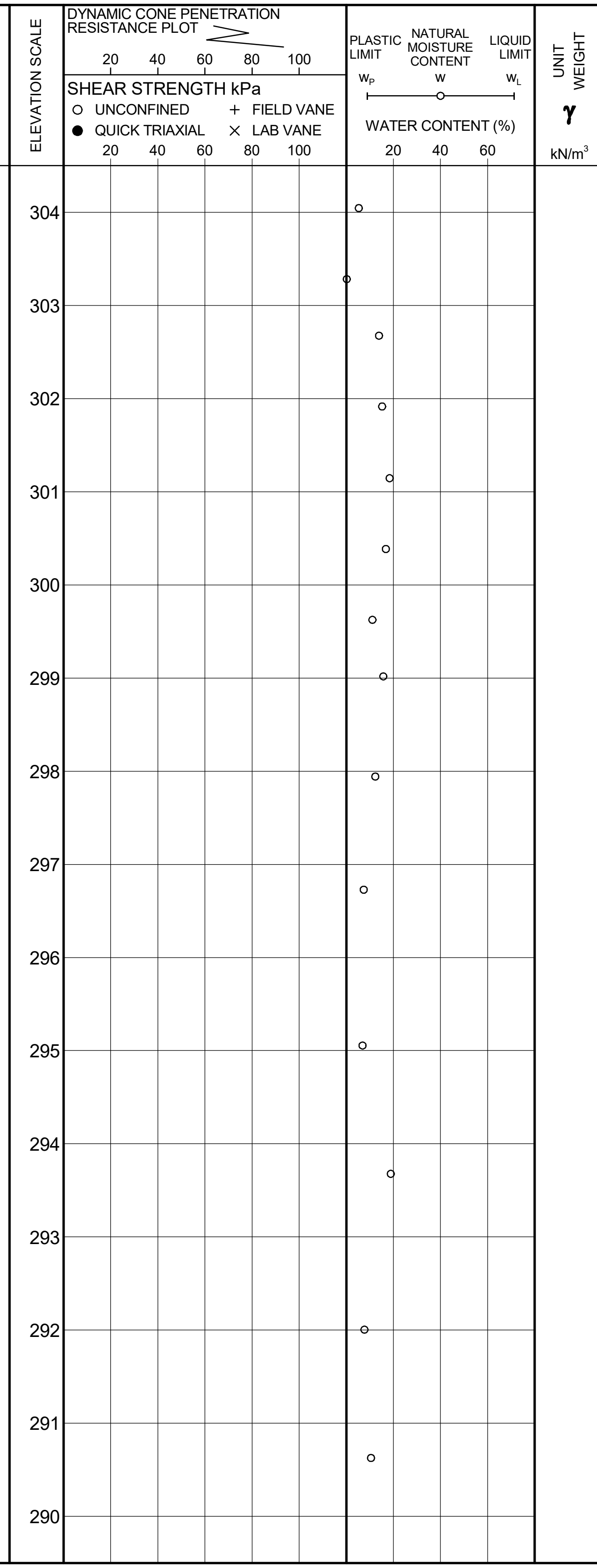
SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	33
2	SS	31
3	SS	22
4	SS	15
5	SS	22
6	SS	10
7	SS	11
8	SS	13
9	SS	26
10	SS	14
11	SS	49
12	SS	38
13	SS	88
14	SS	106

GROUND WATER CONDITIONS: water level symbol at approx. elev. 303

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Elevation (approx.)	GR	SA	SI	CL
302.8	28	51	17	4
300.9	0	93	5	2
297.8	27	39	30	4
294.9	0	91	8	1



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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No BH19-04

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.05.14	LATITUDE 43.3513593 LONGITUDE -80.4167215	CHECKED BY KN

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
15.0	SAND and GRAVEL (SP/GP), trace to some silt Contains occasional cobbles and/or boulders Very dense Grey-brown Wet	15	SS	53		52 38 9 1
		16	SS	115		
285.9 18.6	SAND to SILTY SAND (SP/SM) trace gravel Seams of clayey silt (<25 mm thick) at base of SS17 Very dense Grey-brown Wet	17	SS	133		
284.4		18	SS	102/ 200mm		
20.1	End of Borehole					

Elevation scale: 289, 288, 287, 286, 285

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No BH19-05

METRIC

W.P.	3204-16-00	LOCATION	Roseville Road at Highway 401	ORIGINATED BY	KT
DIST	WEST HWY 401	BOREHOLE TYPE	Hollow Stem Auger - Split Spoon	COMPILED BY	SR
DATUM	Geodetic	DATE	2019.05.01 LATITUDE 43.3515803 LONGITUDE -80.4170165	CHECKED BY	KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
304.4 / 0.0	SILT (ML), trace to some sand and topsoil (FILL) Contains zones of clayey silt Loose Dark brown to yellow-brown Moist
302.9 / 1.5	SILTY SAND (SM) Compact to dense Grey-brown Wet
301.0 / 3.4	SILT and SAND (SM), trace to some gravel Compact to dense Grey-brown Wet
299.3 / 5.1	Gravelly SAND (SP), some silt Compact to dense Grey-brown Wet
298.0 / 6.4	SILTY SAND (SM), some gravel (TILL-LIKE Composition) Contains occasional to frequent cobbles and/or boulders Compact to very dense Grey-brown Wet
296.5 / 7.9	SAND and GRAVEL (SP/GP) trace to some silt Contains layers of sand, trace silt and gravel Dense Grey-brown Wet
291.1 / 13.3	SAND (SP), some silt, trace to some gravel Compact Grey-brown Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	7				
2	SS	9				
3	SS	32				
4	SS	12				
5	SS	10	0	81	17	2
6	SS	42				
7	SS	17	33	55	11	1
8	SS	10				
9	SS	21	12	56	28	4
10	SS	56				
11	SS	41				
12	SS	48	42	51	6	1
13	SS	44				
14	SS	28				

STRAT PLOT	
GROUND WATER CONDITIONS	
ELEVATION SCALE	304, 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290
DYNAMIC CONE PENETRATION RESISTANCE PLOT	20 40 60 80 100
SHEAR STRENGTH kPa	○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L	WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ	kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No BH19-05

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY KT

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.05.01 LATITUDE 43.3515803 LONGITUDE -80.4170165 CHECKED BY KN

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
289.1 15.3	Gravelly SAND to SAND and GRAVEL (SP/GP), trace silt Cobbles and interlayers of sand, trace silt Very dense Grey-brown Wet	15	SS	110	289	
					288	
		16	SS	67	287	
		17	SS	103	286	
					285	
		18	SS	105	284	
283.6 20.8	SAND, some silt to SILTY SAND (SM) Very dense Grey-brown Wet					0 83 14 3 Non-Plastic
282.9 21.5	End of Borehole Temporary well installed to a depth of 3 m on May 1, 2019. Stabilized water level in augers measured at 1.1 m depth (~Elev. 303.3 m) on May 2, 2019.	19	SS	100/ 165mm	283	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No BH19-06

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY DL
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.05.06	LATITUDE 43.3513444 LONGITUDE -80.4171263	CHECKED BY KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
310.6	
310.5 / 0.1	140 mm ASPHALT
	SAND and GRAVELLY SAND (SP), some silt (FILL) Contains silt pockets/inclusions Compact to very dense Brown Moist Frequent cobbles and/or boulders at 4m depth
304.0 / 6.6	SAND (SP), some silt and gravel to SILTY SAND (SM), occasional to frequent cobbles Dense to very dense Dark brown Wet
303.2 / 7.4	SILT to sandy SILT (ML) Compact Brown Wet
302.8 / 7.8	Gravelly SAND (SP), trace to some silt Contains layers of silty fine SAND Compact Brown Wet
298.9 / 11.7	SAND and GRAVEL (SP/GP), trace to some silt Compact to very dense Grey Wet

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	42	
2	SS	48	0 81 16 3
3	SS	48	
4	SS	45	
5	SS	12	
6	SS	23	
7	SS	70	33 49 15 3
8	SS	43	
9	SS	36	
10	SS	60	
11	SS	33	
12	SS	33	0 70 28 2
13	SS	38	
14	SS	12	
15	SS	117	

GROUND WATER CONDITIONS: ∇ at approx. elev. 303.0

ELEVATION SCALE: 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No BH19-06

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY DL

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.05.06 LATITUDE 43.3513444 LONGITUDE -80.4171263 CHECKED BY KN

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL (SP/GP), trace to some silt Compact to very dense Grey Wet *(continued)*	16	SS	96	295	37 50 11 2
294.3 16.3	SAND (SP), trace to some gravel, trace silt Very dense Grey Wet	17	SS	57	294, 293	
292.0 18.6	Gravelly SILTY SAND (SM) Occasional cobbles (TILL-LIKE Composition) Very dense Grey Wet	18	SS	120	292	
291.2 19.4	SAND and GRAVEL (SP/GP), trace silt, frequent cobbles Very dense Grey Wet	19	SS	93	291, 290	50 43 6 1
289.7 20.9	SAND (SP) trace to some gravel, trace silt Contains occasional cobbles and/or boulders Very dense Grey Wet	20	SS	67	289, 288, 287	
		21	SS	104	286, 285	
	Grinding of augers below 25.9m				284	
283.0 27.6	CLAYEY SILT (CL), trace sand Hard Grey	22	SS	62/ 76mm	283	0 5 48 47
282.6 28.0	End of Borehole					

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No BH19-07

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.08.28	LATITUDE 43.3512968 LONGITUDE -80.4174146	CHECKED BY KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
310.1	ASPHALTIC CONCRETE
310.0 / 0.0	127 mm ASPHALT
309.9 / 0.1	SAND and GRAVEL (SP/GP), trace silt (FILL) Very dense Brown Moist
309.8 / 0.3	SAND (SP), some gravel, trace silt (FILL) Very dense Brown Moist
309.3 / 0.8	SAND (SP), some gravel, trace to some silt (FILL) Very dense Brown Moist
307.6 / 2.5	GRAVELLY SAND to SAND and GRAVEL (SP/GP), some silt (FILL) Compact to very dense Brown Moist 150 mm thick layer of dark brown silty sand, some gravel, trace organic matter at 3.5 m depth
304.1 / 6.0	CLAYEY SILT (CL) with sand, trace gravel and organic material (FILL) Contains occasional boulders Stiff Grey-brown Moist to wet
302.6 / 7.5	SAND (SP), some gravel to gravelly, trace to some silt Compact to dense Brown Wet
296.0 / 14.1	Gravelly SAND (SP), some silt (TILL-LIKE Composition) Brown Very dense Wet
295.3 / 14.8	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	63		
2	SS	64		16 65 15 4
3	SS	68		
4	SS	22		
5	SS	66		
6	SS	40		
7	SS	37		30 51 15 4
8	SS	28		
9	SS	11	∇	2 50 38 10
10	SS	33		
11	SS	22		
12	SS	41		
13	SS	22		
14	SS	50		

ELEVATION SCALE: 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

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

RECORD OF BOREHOLE No BH19-07

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY KT

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.08.28 LATITUDE 43.3512968 LONGITUDE -80.4174146 CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	SAND (SP), trace silt and garvel *(continued)*						295							
294.3			15	SS	100				o		1	89	8	2
15.8	End of Borehole													

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

RECORD OF BOREHOLE No BH19-08

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.08.26	LATITUDE 43.3515343 LONGITUDE -80.415986	CHECKED BY KN

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
310.3						
310.1 / 0.0	152 mm ASPHALT					
309.9 / 0.2	SAND and GRAVEL (FILL) Very dense					
0.4	SAND (SP/SM), some silt to silty, some gravel to gravelly (FILL) Contains occassional cobbles Very dense Brown Moist	1	SS	55		
		2	SS	79		
		3	SS	57		
308.1 / 2.2	SAND (SP) some silt, trace to some gravel (FILL) Contains occasional cobbles Sa5 contains clayey silt inclusions and trace organic matter Compact to dense Brown Moist to wet	4	SS	32		7 73 17 3
		5	SS	25		
		6	SS	48		
		7	SS	45		
305.0 / 5.3	Gravelly CLAYEY SILT (CL-ML), with sand (FILL) Contains cobbles Stiff to hard Dark brown Moist Possible hydrocarbon odour	8	SS	14		25 53 18 4
		9	SS	63		
303.1 / 7.2	SAND and GRAVEL (SP/GP), some silt Very dense Light brown Wet				∇	
		10	SS	50		46 41 11 2
301.6 / 8.7	SAND (SP/SM), trace silt to silty, trace to some gravel Contains gravelly zones and occasional cobbles Compact Brown to grey-brown Wet	11	SS	23		
		12	SS	25		
		13	SS	18		
		14	SS	26		17 48 33 2 Non-Plastic

ELEVATION SCALE: 310, 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

RECORD OF BOREHOLE No BH19-08

METRIC

W.P. 3204-16-00 | LOCATION Roseville Road at Highway 401 | ORIGINATED BY KT

DIST WEST HWY 401 | BOREHOLE TYPE Hollow Stem Auger - Split Spoon | COMPILED BY SR

DATUM Geodetic | DATE 2019.08.26 | LATITUDE 43.3515343 LONGITUDE -80.415986 | CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND (SP/SM), trace silt to silty, trace to some gravel Contains gravelly zones and occasional cobbles Compact Brown to grey-brown Wet *(continued)* Gravelly below 15 m	15	SS	32	295	o	
294.0 16.3	SAND (SP), some gravel to sandy GRAVEL (GP), trace silt Dense to very dense Brown Wet	16	SS	73	294 / 293	o	
		17	SS	59/ 51mm	292	o	
		18	SS	85	291 / 290 / 289	o	
		19	SS	46	288 / 287 / 286	o	52 40 7 1
284.1 26.2	SILT and SAND (SP to ML) Very dense Grey-brown Wet	20	SS	93	285 / 284 / 283	o	0 62 37 1 Non-Plastic
281.0 29.3	Sandy GRAVEL (GP), some silt Very dense Grey-brown Wet				282 / 281		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No BH19-08

METRIC

W.P.	3204-16-00	LOCATION	Roseville Road at Highway 401	ORIGINATED BY	KT
DIST	WEST HWY 401	BOREHOLE TYPE	Hollow Stem Auger - Split Spoon	COMPILED BY	SR
DATUM	Geodetic	DATE	2019.08.26 LATITUDE 43.3515343 LONGITUDE -80.415986	CHECKED BY	KN

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Sandy GRAVEL (GP), some silt Very dense Grey-brown Wet *(continued)*						280				
279.2			21	SS	105				o		
31.1	End of Borehole										

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

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No BH19-09

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY KT
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.08.28	LATITUDE 43.3515733 LONGITUDE -80.4157415	CHECKED BY KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
309.7	ASPHALTIC CONCRETE
309.5 / 0.0	175 mm ASPHALT
0.2	SAND, some silt and gravel (FILL) Contains cobbles Very dense Brown
308.9 / 0.8	SAND (SP), some silt, trace to some gravel (FILL) Transitions to dark brown SILTY SAND, trace organic matter
306.0 / 3.7	SAND (SP), some gravel to SAND and GRAVEL trace silt to silty (FILL) Compact to very dense Grey-brown Moist to wet Sa7 conatins silt inclusions and trace organic matter
303.7 / 6.0	Gravelly CLAYEY SILT (CL-ML), with sand, trace organic matter (FILL) Stiff Grey Wet
302.5 / 7.2	SAND and GRAVEL (SP/GP), trace to some silt, occasional cobbles and boulders Very dense Grey-brown Wet
301.0 / 8.7	SAND (SP), trace silt and gravel Contains cobbles and/or boulders and layers of sandy silt Compact to dense Grey-brown Wet
294.7	

SAMPLES

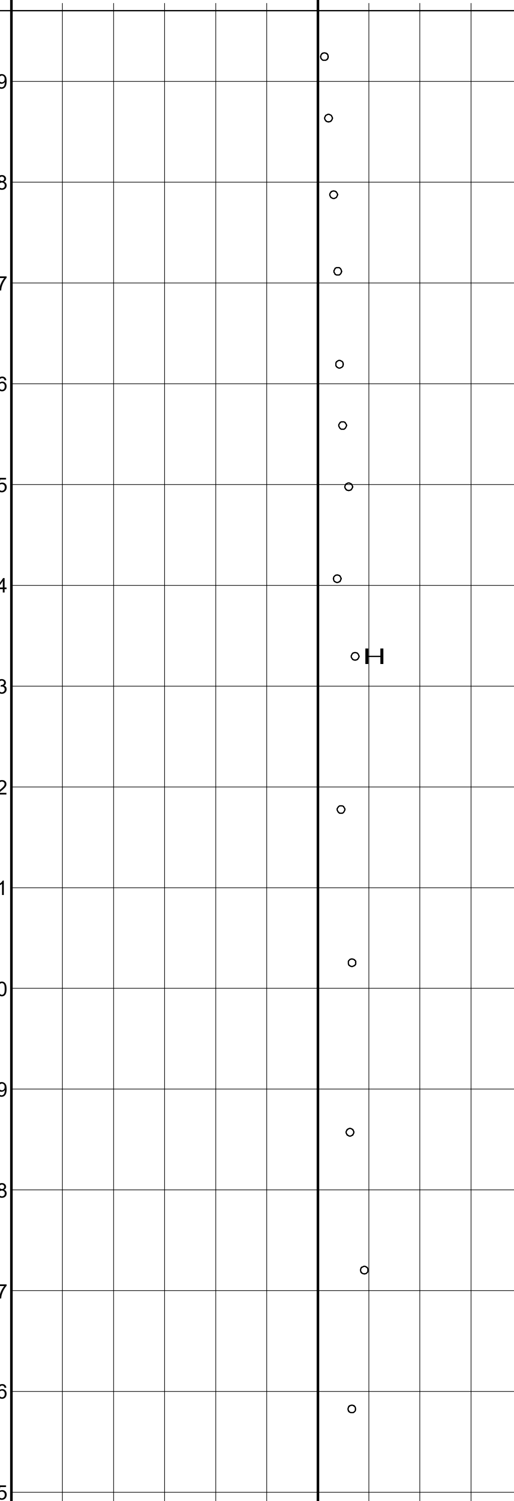
NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	58		
2	SS	48		
3	SS	40		4 78 15 3
4	SS	30		
5	SS	25		
6	SS	36		
7	SS	44		
8	SS	60		43 39 14 4
9	SS	14	▽	26 42 25 7
10	SS	59		
11	SS	23		
12	SS	21		
13	SS	15		1 91 7 1
14	SS	31		

ELEVATION SCALE: 309, 308, 307, 306, 305, 304, 303, 302, 301, 300, 299, 298, 297, 296, 295

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³



Continued Next Page

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

RECORD OF BOREHOLE No BH19-09 2 OF 2 METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY KT

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.08.28 LATITUDE 43.3515733 LONGITUDE -80.4157415 CHECKED BY KN

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
15.0	SAND (SP), trace silt and gravel Contains cobbles and/or boulders and layers of sandy silt Compact to dense Grey-brown Wet		15	SS	26		294		○		
293.9 15.8	End of Borehole										

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

RECORD OF BOREHOLE No BH19-10

METRIC

W.P. 3204-16-00	LOCATION Roseville Road at Highway 401		ORIGINATED BY DL
DIST WEST HWY 401	BOREHOLE TYPE Hollow Stem Auger - Split Spoon		COMPILED BY SR
DATUM Geodetic	DATE 2019.08.19	LATITUDE 43.3513166 LONGITUDE -80.4160008	CHECKED BY KN

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
304.0	
303.9 / 0.1	SANDY SILT (TOPSOIL) Contains wood pieces/organic matter
302.9 / 1.1	Gravelly SAND (SP) trace to some silt (Probable FILL) Loose to compact Brown Moist to wet
302.6 / 1.4	Peaty TOPSOIL Loose Black Fibrous
301.0 / 3.0	SAND and GRAVEL, trace to some silt Compact to dense Brown Wet
298.0 / 6.0	SAND (SP), some silt to SILTY SAND (SM) Compact Brown Wet Contains some gravel and silt seams below 4.5 m
296.8 / 7.2	SANDY SILT (ML), some gravel to gravelly (TILL-LIKE composition) Compact Brown Wet
289.0	GRAVELLY SAND (SP) to sandy GRAVEL (GP), trace silt to silty Compact to dense Brown Wet

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	11	
2	SS	6	Organic Content = 20.5%
3	SS	10	
4	SS	37	46 43 10 1
5	SS	17	
6	SS	29	0 71 28 1
7	SS	10	
8	SS	16	
9	SS	16	21 28 49 2
10	SS	25	
11	SS	28	
12	SS	72	50 42 7 1
13	SS	83	
14	SS	60	

Elevation scale: 303, 302, 301, 300, 299, 298, 297, 296, 295, 294, 293, 292, 291, 290

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60
UNIT WEIGHT γ kN/m³

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

RECORD OF BOREHOLE No BH19-10

METRIC

W.P. 3204-16-00 | LOCATION Roseville Road at Highway 401 | ORIGINATED BY DL

DIST WEST | HWY 401 | BOREHOLE TYPE Hollow Stem Auger - Split Spoon | COMPILED BY SR

DATUM Geodetic | DATE 2019.08.19 | LATITUDE 43.3513166 | LONGITUDE -80.4160008 | CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
15.0	GRAVELLY SAND (SP) to sandy GRAVEL (GP), trace silt to silty Compact to dense Brown Wet	15	SS	100						
					288					
		16	SS	43	287	o				
					286					
		17	SS	62	285					
283.9 / 20.1	SAND (SP), trace to some gravel trace silt Very dense Grey-brown Wet				284					
					283					
		18	SS	119	282	o				
					281					
					280					
		19	SS	109	279					
					278					
					277					
		20	SS	103		o	2	87	9	2
276.1 / 27.9	End of Borehole									

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No BH19-11

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY DL

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.08.29 LATITUDE 43.3514287 LONGITUDE -80.4154409 CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
303.8 / 0.0	Sandy SILT (ML) (TOPSOIL) Contains wood and fibous organic matter Very loose	1	SS	3	▽	Org. Content SS1 = 5.0 %
303.2 / 0.6	Interlayered SAND and gravelly SAND (SP), some silt Compact to dense Brown Wet	2	SS	18		
		3	SS	40		
301.5 / 2.3	SAND (SP) some silt, trace gravel Loose to compact Brown Wet	4	SS	12		0 87 12 1
		5	SS	11		
		6	SS	11		
		7	SS	22		4 84 11 1
		8	SS	9		
		9	SS	25		
295.7 / 8.1	Gravelly SAND to SAND and GRAVEL (SP/GP) Contains sand seams and frequent cobbles and/or boulders Compact Brown Wet	10	SS	28		
294.1 / 9.8	End of Borehole					

ELEVATION SCALE: 303, 302, 301, 300, 299, 298, 297, 296, 295

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No BH19-12

METRIC

W.P. 3204-16-00 LOCATION Roseville Road at Highway 401 ORIGINATED BY DL

DIST WEST HWY 401 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR

DATUM Geodetic DATE 2019.08.19 LATITUDE 43.3517679 LONGITUDE -80.4160391 CHECKED BY KN

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
303.5						
0.0	SILTY SAND (TOPSOIL) Dark brown to black	1	SS	12	∇	
303.1 / 0.4	Interlayered SAND and SAND and GRAVEL (SP/GP), trace silt, Dense, Brown, Wet	2	SS	34		51 39 8 2
		3	SS	50		
		4	SS	34		
300.5 / 3.0	SAND trace to some silt and gravel, Contains zones of SILT and SAND, Compact, Wet	5	SS	14		
		6	SS	13		
		7	SS	14		8 41 43 8
297.3 / 6.3	SILT (ML), trace sand, Dense, Brown, Wet	8	SS	37		
295.9 / 7.6	CLAYEY SILT (CL), trace sand, Very stiff to hard, Brown	9	SS	25		
		10	SS	31		4 9 64 23
293.8 / 9.8	End of Borehole					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; ELEVATION SCALE (303, 302, 301, 300, 299, 298, 297, 296, 295, 294); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

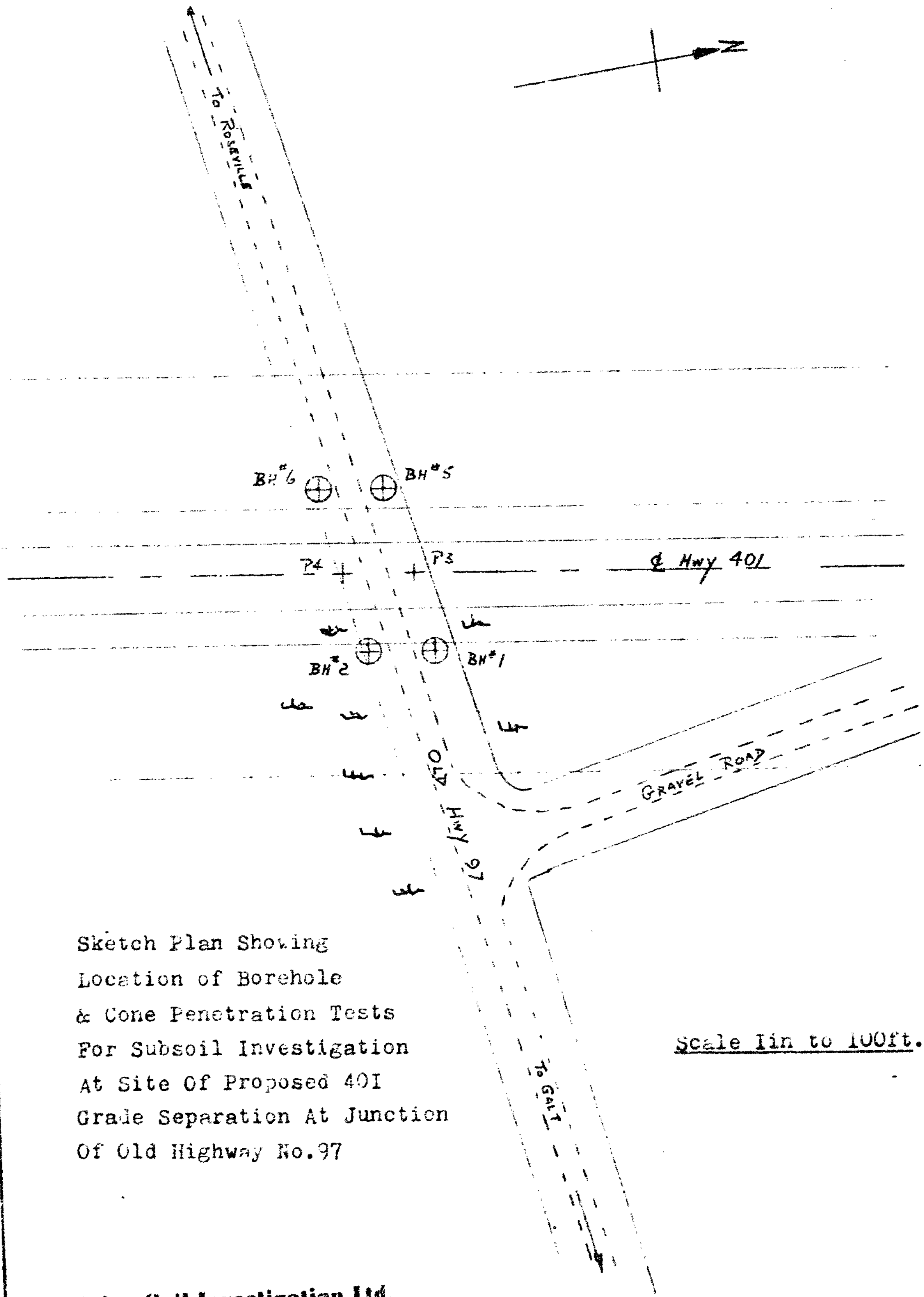
+^{3}, ×^{3}: Numbers refer to Sensitivity   O$^{3\%}$ STRAIN AT FAILURE

Borehole Records from Previous Investigation (Geocres Report No. 40P08-017)

Enclosure No. 2

Order No. I05/F8I

Prep. By P.M.



Sketch Plan Showing
Location of Borehole
& Cone Penetration Tests
For Subsoil Investigation
At Site Of Proposed 40I
Grade Separation At Junction
Of Old Highway No.97

Scale Iin to 100ft.

Dominion Soil Investigation Ltd.

Order No. I05/F8I

Enclosure No. 3

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: I

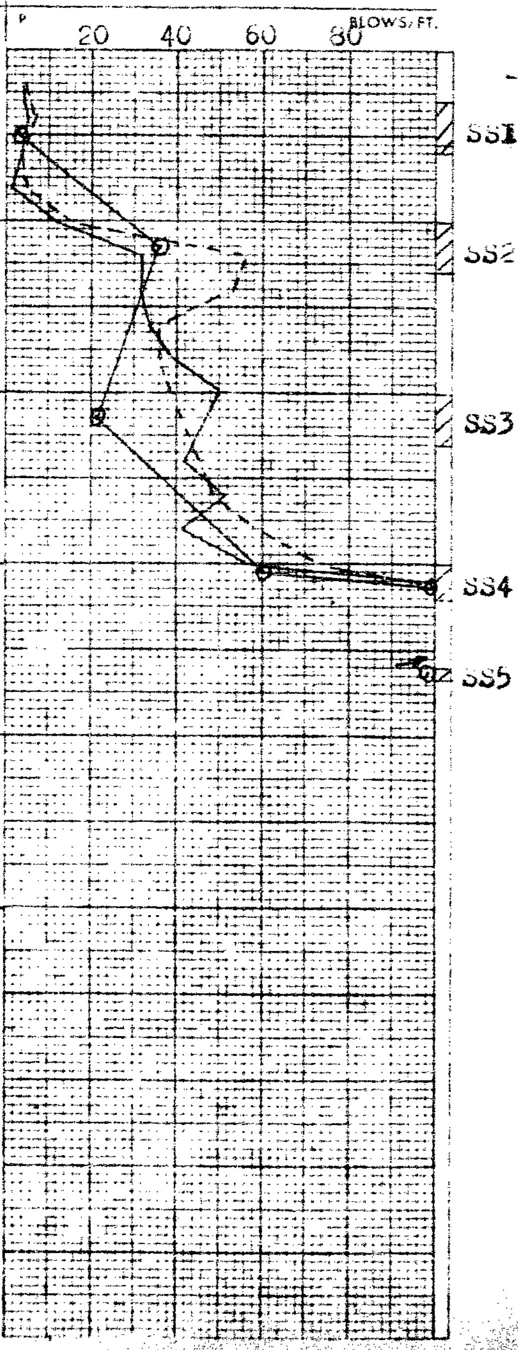
Project: Grade Separation No I90
Location: N Dunfries Old Hwy No 97
Hole Location: See Enclosure No 2
Hole Elevation and Datum:
Field Supervisor: T.S. Prep.: P.M.
Driller: A.B. Checked:

Date: I0/9/58

LEGEND
Shear Strength (C)
Unconfined compression
Vane test and sensitivity (S)
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

Sampling Method
2" Dia. split tube
2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE
	Loose dark grey topsoil			
	W/L 3/3/58 Peat			
	Medium dense to dense fine to coarse sand and gravel with traces of silt		5	
	Medium dense grey brown sand some coarse gravel		10	
	Grey white dense coarse sand fine to coarse gravel		15	
	End of cone test			
	Very dense partially cemented fine to coarse sand & gravel			
	End of hole		20	
			25	



Order No. IO5/F8I

Enclosure No. 4

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 2

Date: 11/9/58

Project: Grade Separation No I90
Location: N Dunfries Old Hwy No 97
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997.4 Geo.
Field Supervisor: T.S. Prep.: P.M.
Driller: A.B. Checked:

LEGEND

Shear Strength (C)
- Unconfined compression ⊕
- Vane test and sensitivity (S) $+^{S}$

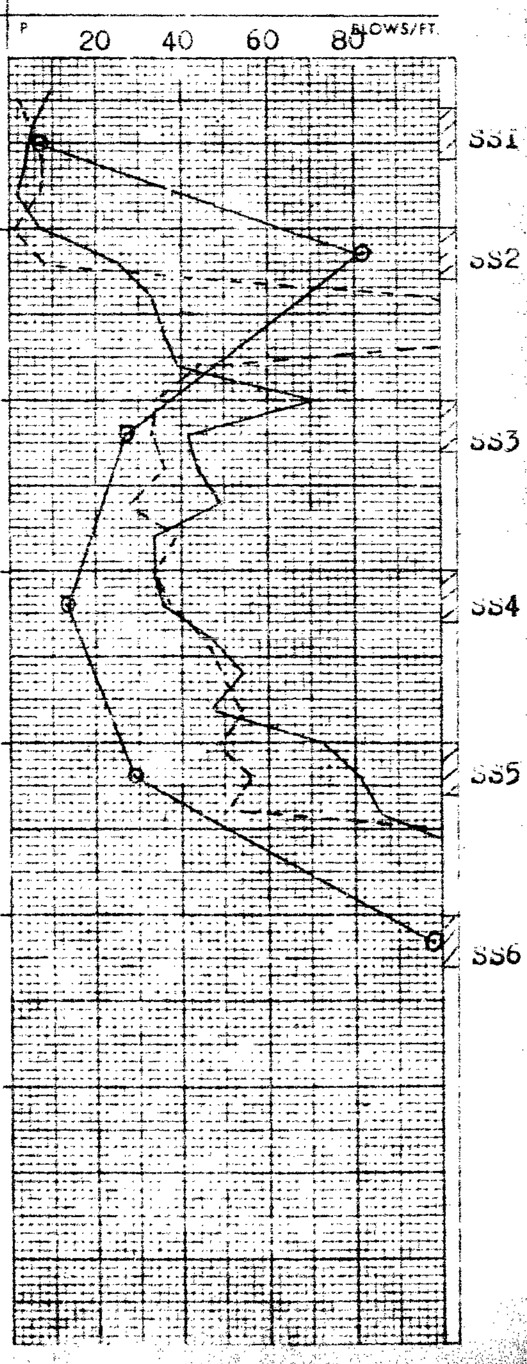
Penetration Resistance (P)
- 2" Split tube
- 2" Dia. Cone
- Casing

Sampling Method
- 2" Dia. split tube
- 2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
		997.4	
	Loose coarse sand and gravel		
	W/L 6/9/58 Peat		5
	Medium dense to dense fine grey sand and coarse gravel		
			10
	Medium dense very fine grey and brown sand		15
	Medium dense going to dense fine to coarse grey and brown sand and gravel		20
	End of cone test		
	Very dense partially cemented sand and gravel		25
	End of hole		
			30

STRENGTH AND PENETRATION RESISTANCE

C P.S.F.



Order No. I05/F8I

Enclosure No. 5

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 3

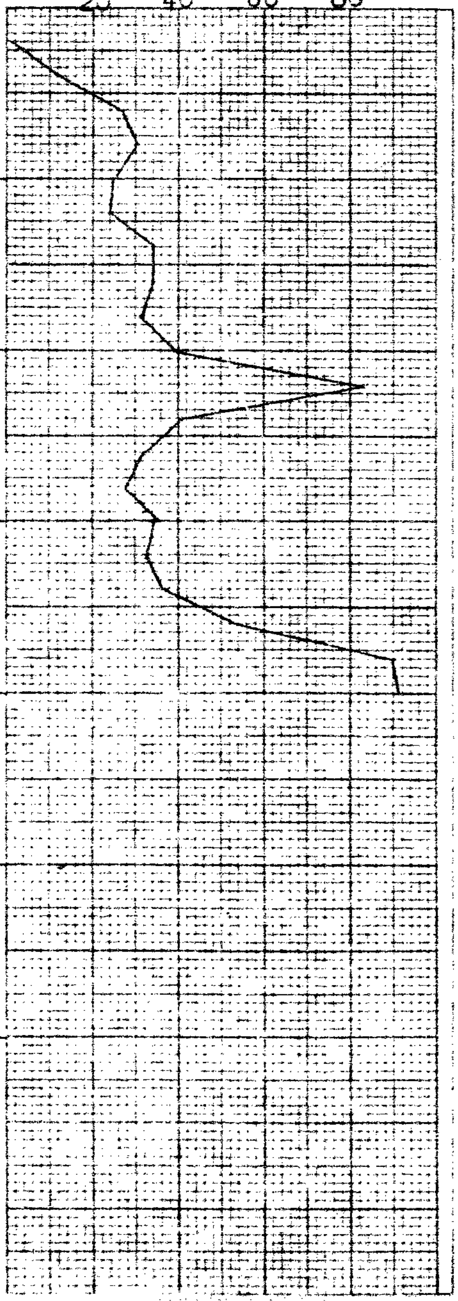
Date: 10/9/53

Project: Proposed Grade Separation
Location: N Dunfries Old Hwy No 97
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 995.0 Geo.
Field Supervisor: T.S. Prep.: P.M.
Driller: A.B. Checked: -

LEGEND
Shear Strength (C)
Unconfined compression ⊕
Vane test and sensitivity (S) $+^{S}$
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

Sampling Method
2" Dia. split tube
2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE C P.S.F. / P BLOWS/FT.
		995.0		20 40 60 80
	End of cone test		5, 10, 15, 20, 25	



Order No. I05/F8I

Enclosure No. 6

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 4

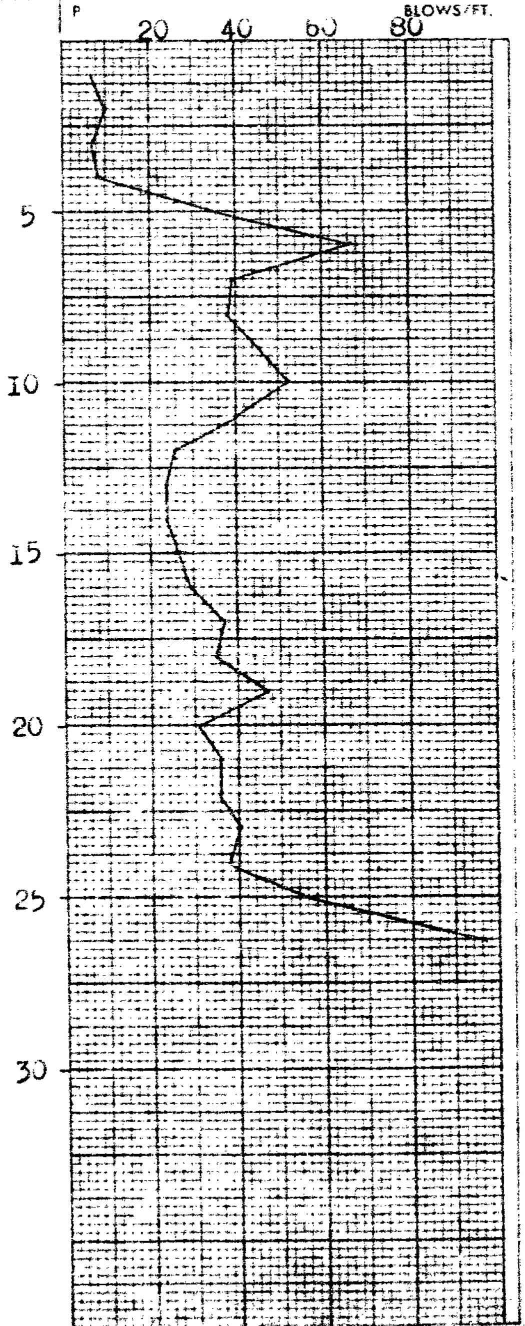
Date: I0/9/58

Project: Grade Separation No I90
Location: N Dunfries Old Hwy No 97
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 997.2 Geo.
Field Supervisor: T.S. Prep.: P.M.
Driller: A.B. Checked:

LEGEND
Shear Strength (C)
Unconfined compression ⊕
Vane test and sensitivity (S) $+^{S}$
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

Sampling Method
2" Dia. split tube
2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE C P.S.F. / P BLOWS/FT.
		997.2		
	— End of cone test			



Order No. I05/F8I

Enclosure No. 7

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 5

Date: 10/9/58

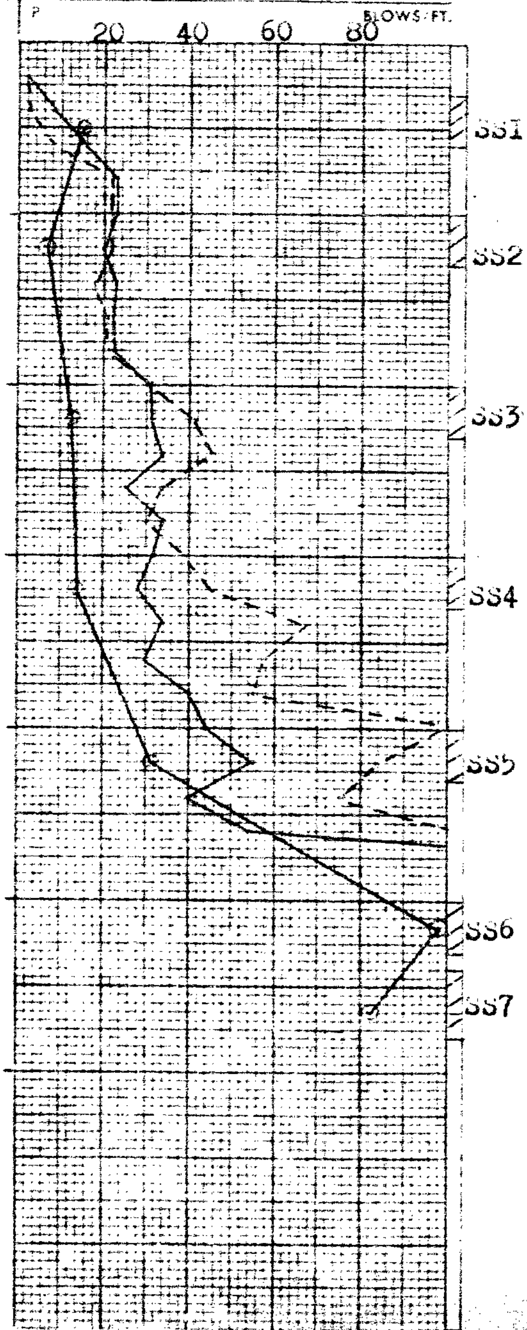
Project: Grade Separation No I90
Location: N Dunfries Old Hwy No 97
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 995.3 Geo.
Field Supervisor: T.S. Prep.: P.M.
Driller: A.B. Checked:

LEGEND
Shear Strength (C)
Unconfined compression ⊕
Vane test and sensitivity (S) +S
Penetration Resistance (P)
2" Split tube
2" Dia. Cone
Casing

Sampling Method
2" Dia. split tube
2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
	Loose brown fine sand and coarse gravel	995.3	
	W/L 5/9/58		
	Loose very fine grey sand		5
	Medium dense grey fine to coarse sand and gravel		10
	Medium dense coarse sand and fine to coarse gravel		15
			20
	End of cone test		
	Very dense partially cemented coarse sand and gravel		25
	End of hole		30

STRENGTH AND PENETRATION RESISTANCE
C P.S.F.
P BLOWS/FT.



Order No. I05/F8I

Enclosure No. 8

Dominion Soil Investigation Ltd.

Engineering Data Sheet for Borehole: 6

Date: II/9/58

Project: Grade Separation No I90
Location: N Dumfries Old Hwy No 97
Hole Location: See Enclosure No 2
Hole Elevation and Datum: 998.0 Geo.
Field Supervisor: T.S. Prep.: P.M.
Driller: A.B. Checked:

LEGEND

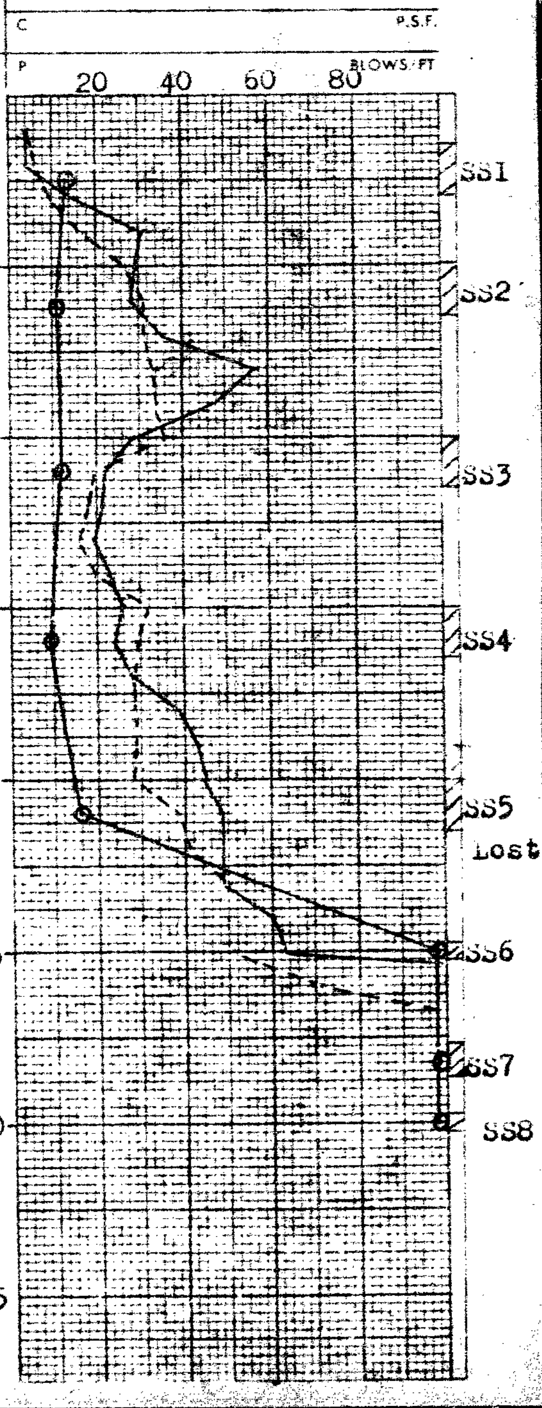
Shear Strength (C)
- Unconfined compression
- Vane test and sensitivity (S)

Penetration Resistance (P)
- 2" Split tube
- 2" Dia. Cone
- Casing

Sampling Method
- 2" Dia. split tube
- 2" Shelby tube

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET
		998.0	
	Loose dark brown sandy topsoil		
	W/L 4/9/58		
	Loose brown silty sand and coarse gravel		5
	Loose grey brown coarse sand		10
	Loose grey brown coarse sand		15
	Medium dense grey fine to medium sand		
	Medium dense grey fine to medium sand		20
	End of cone test		25
	Very dense grey fine to coarse sand and gravel		
	End of hole		30
			35

STRENGTH AND PENETRATION RESISTANCE

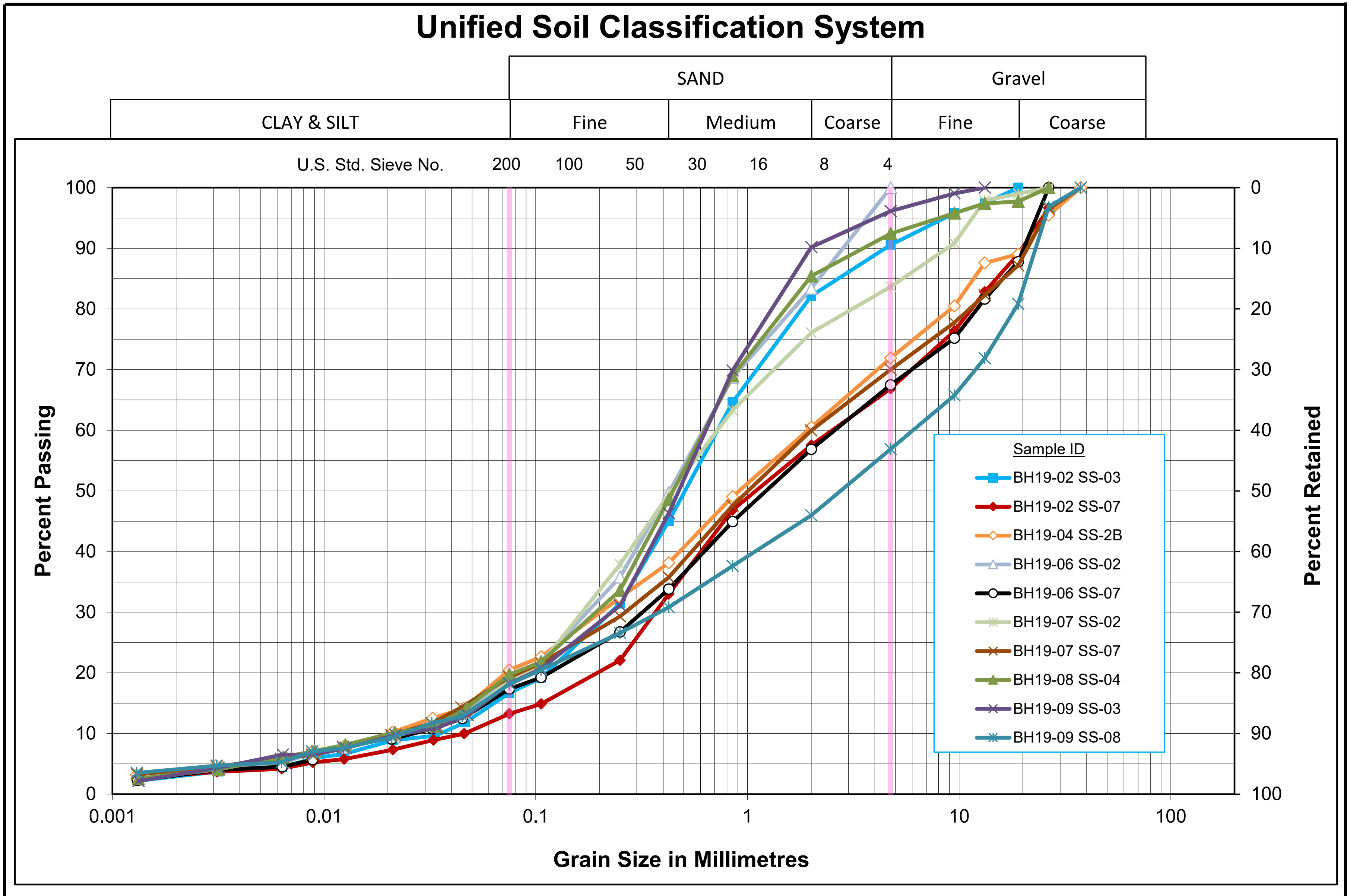


APPENDIX C

C.1 LABORATORY TEST RESULTS

C.2 FIGURES C1 TO C13: GRAIN SIZE DISTRIBUTION PLOTS AND PLASTICITY CHARTS

C.3 DIRECT SHEAR AND CORROSIVITY TESTING RESULTS



Stantec

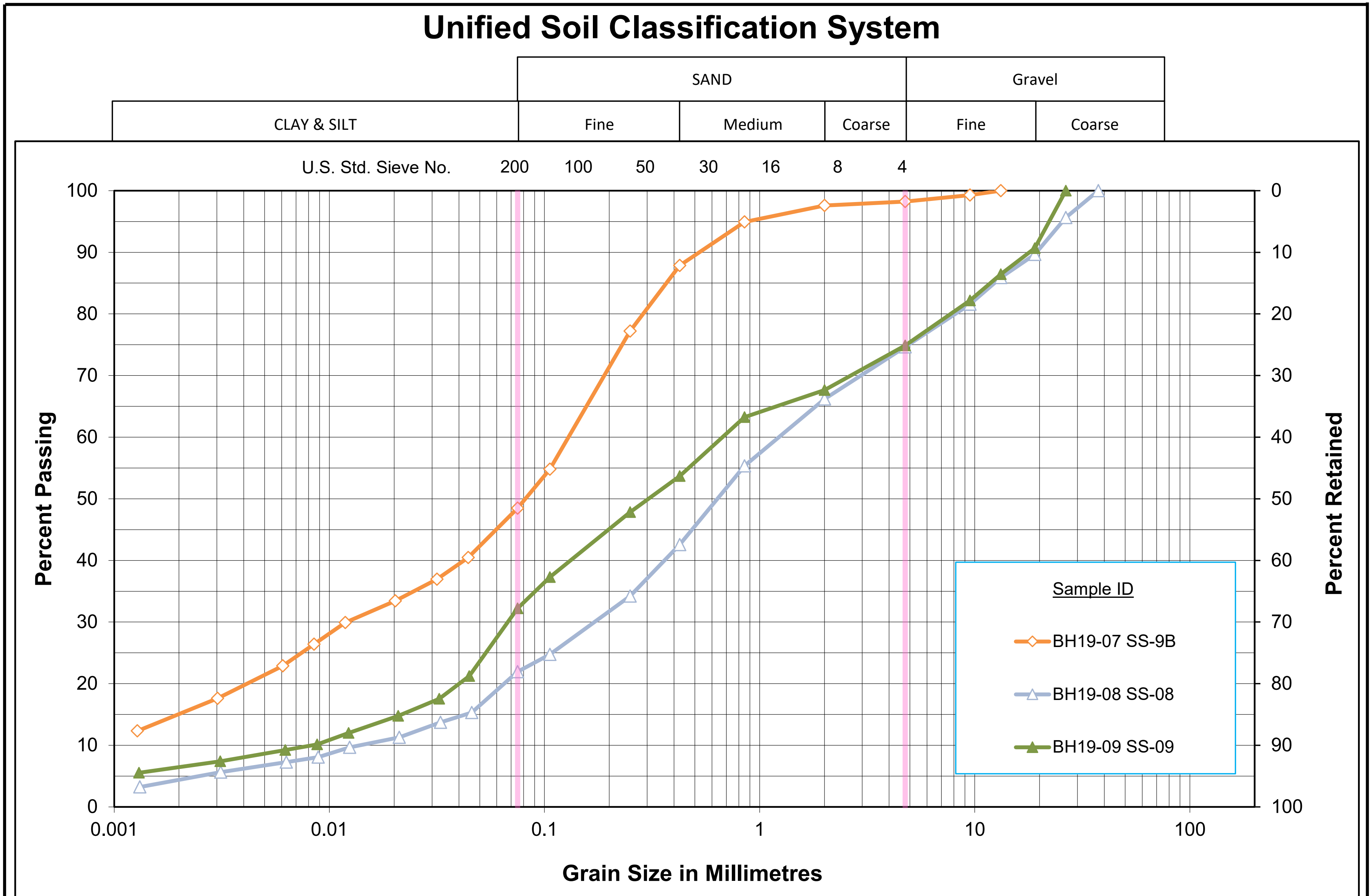
GRAIN SIZE DISTRIBUTION

FILL: Silty SAND to SAND and GRAVEL

Hwy 401 Roseville Road Bridge

Figure No. C1

Project No. 165001107



Stantec

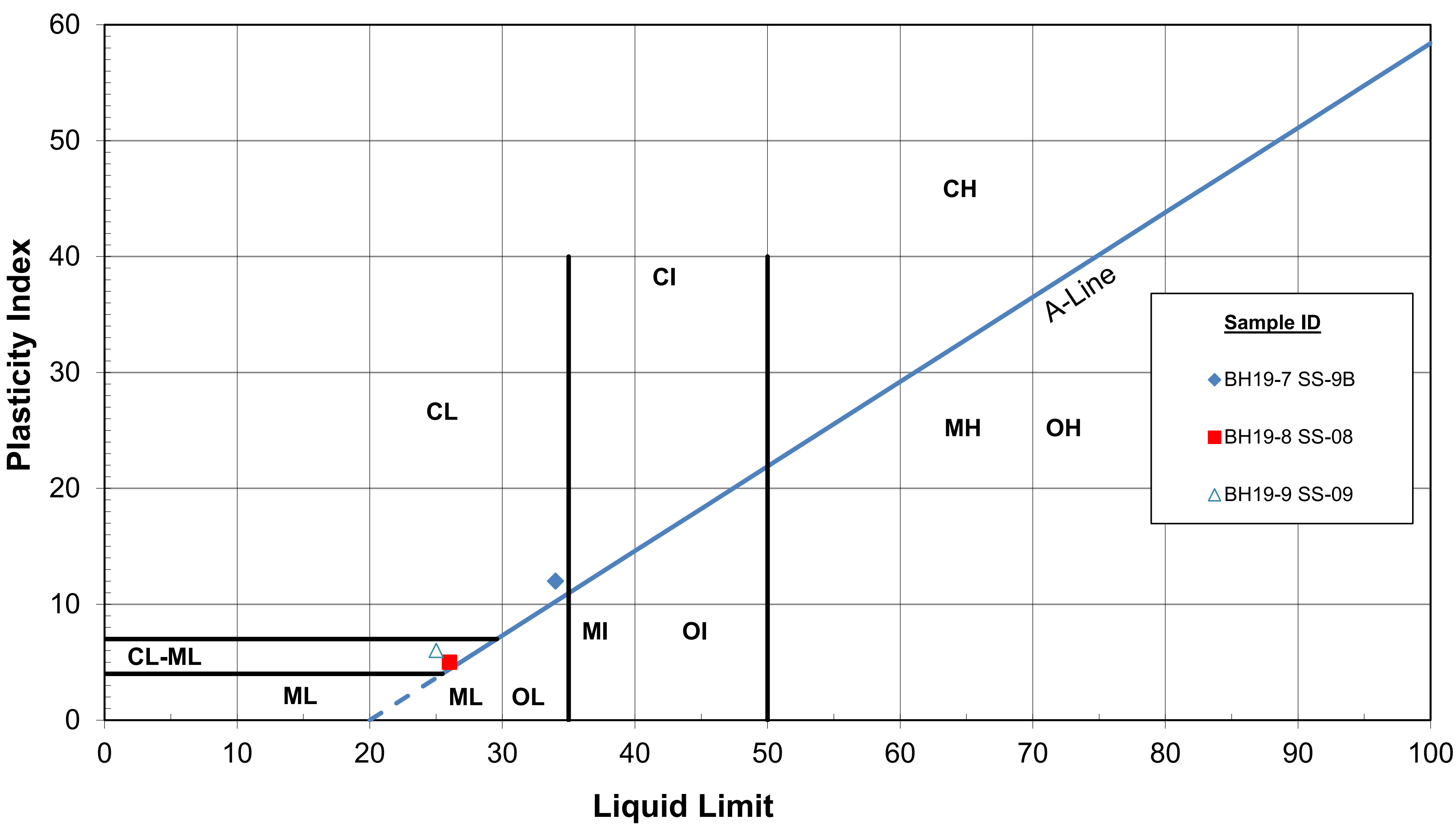
GRAIN SIZE DISTRIBUTION

FILL: CLAYEY SILT with sand

Hwy 401 Roseville Road Bridge

Figure No. C2

Project No. 165001107



Stantec

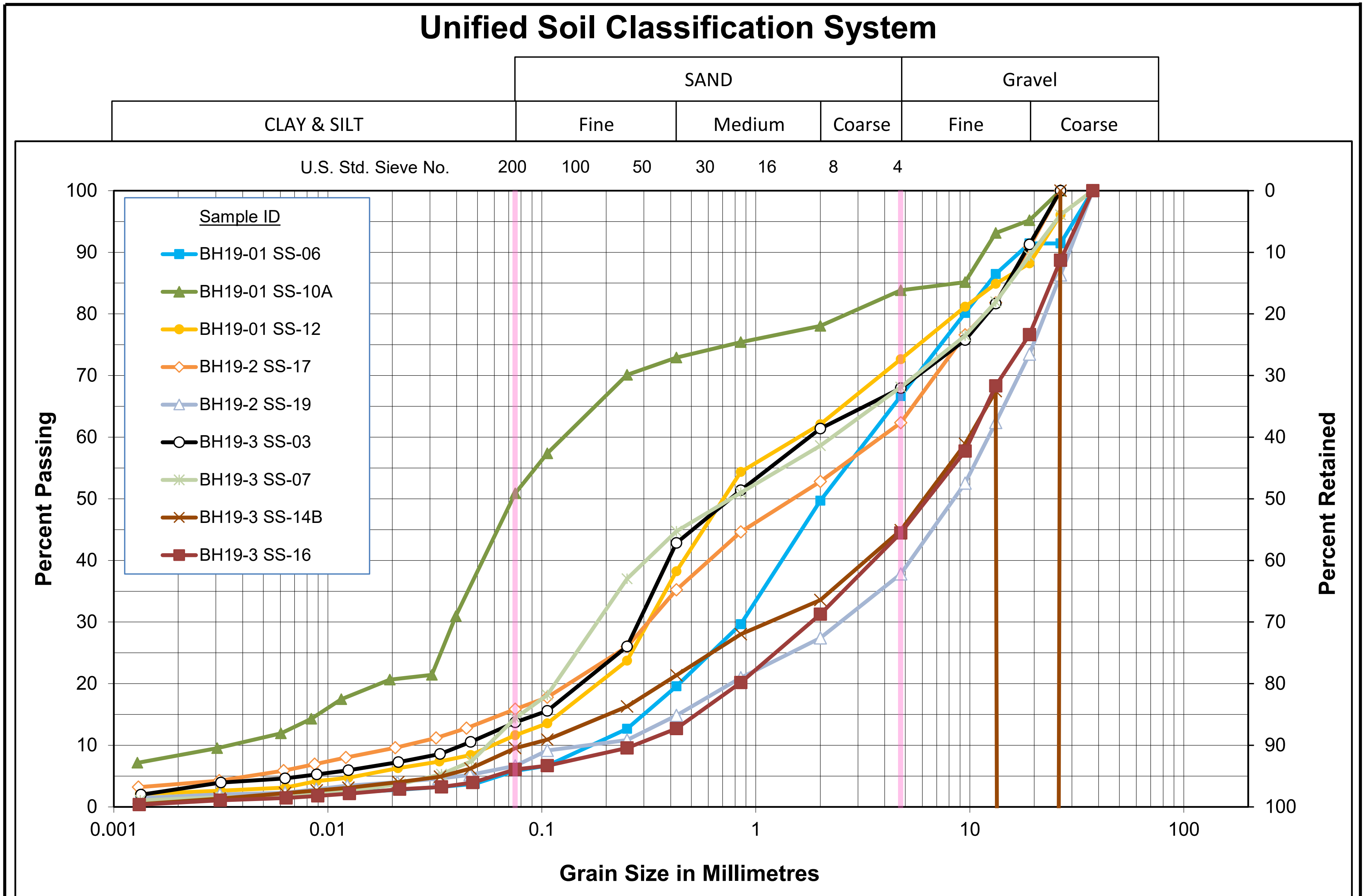
FILL: CLAYEY SILT with sand

Hwy 401 Roseville Road Bridge

PLASTICITY CHART

Figure No. C3

Project No. 165001107



Stantec

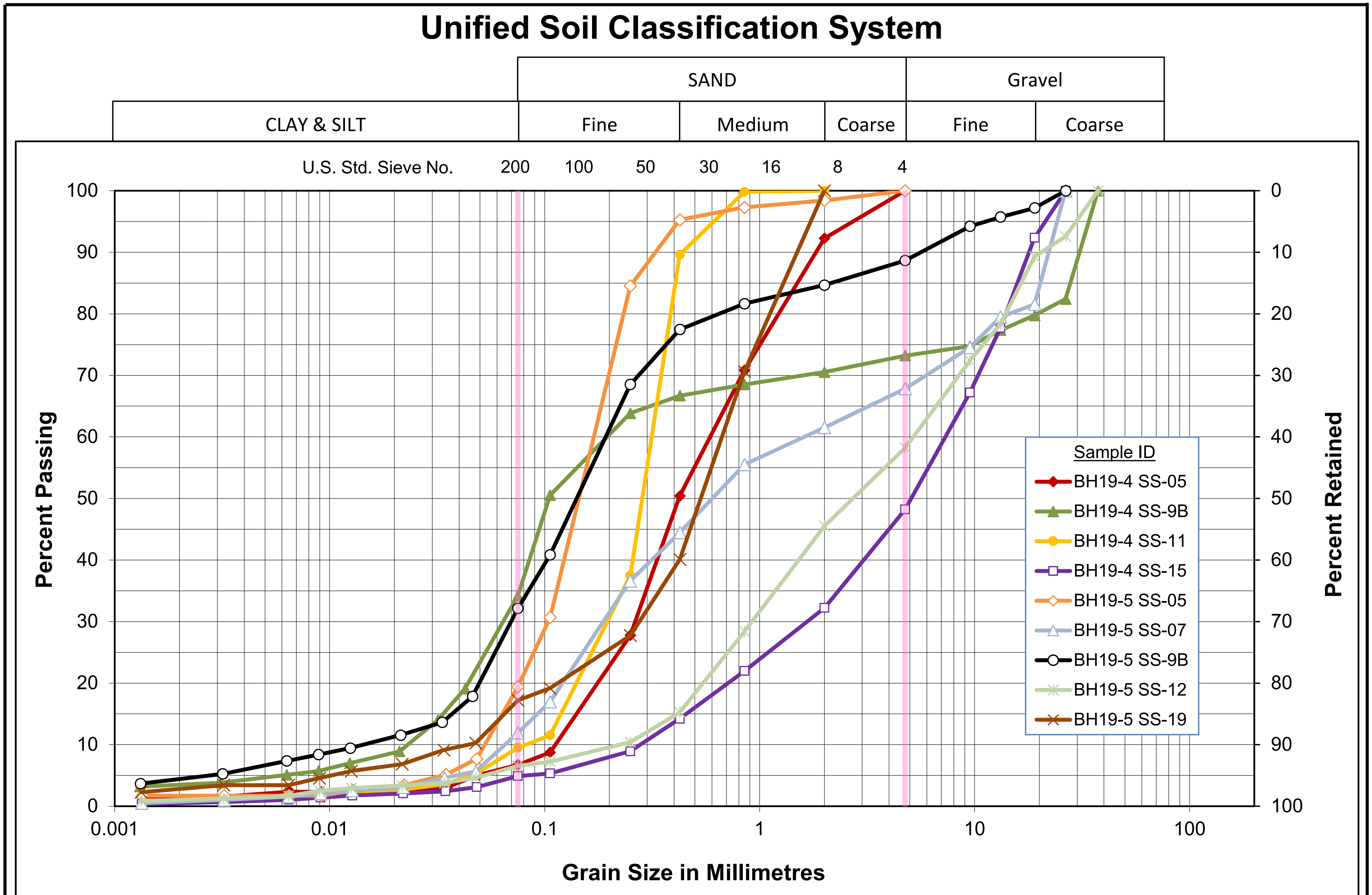
GRAIN SIZE DISTRIBUTION

SILT and SAND (SM) to SAND & GRAVEL (GP)

Hwy 401 Roseville Road Bridge

Figure No. C4

Project No. 165001107



Stantec

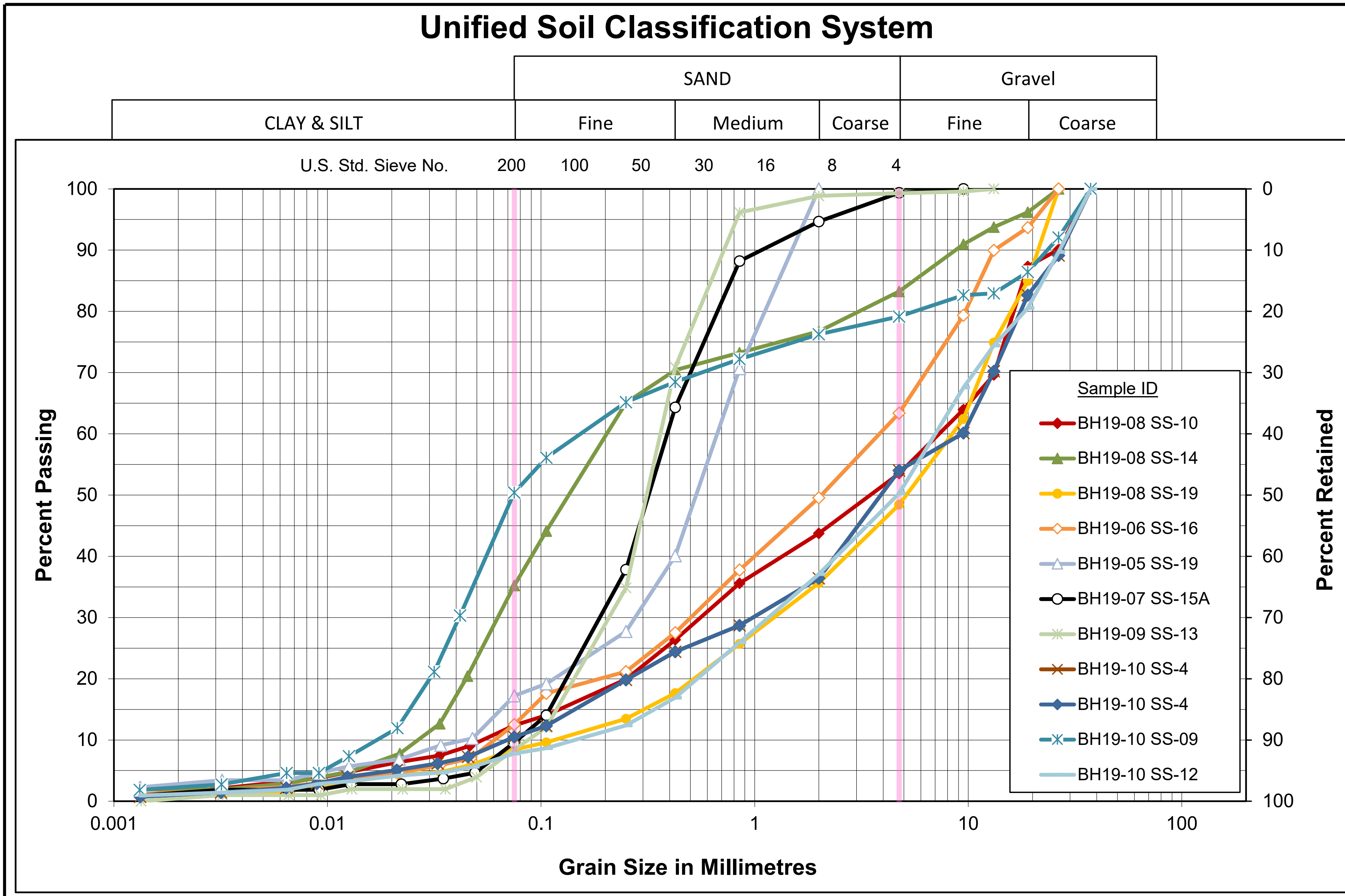
GRAIN SIZE DISTRIBUTION

Silty Sand (SM) to Sand and Gravel (SP/GP)

Hwy 401 Roseville Road Bridge

Figure No. C5

Project No. 165001107



Stantec

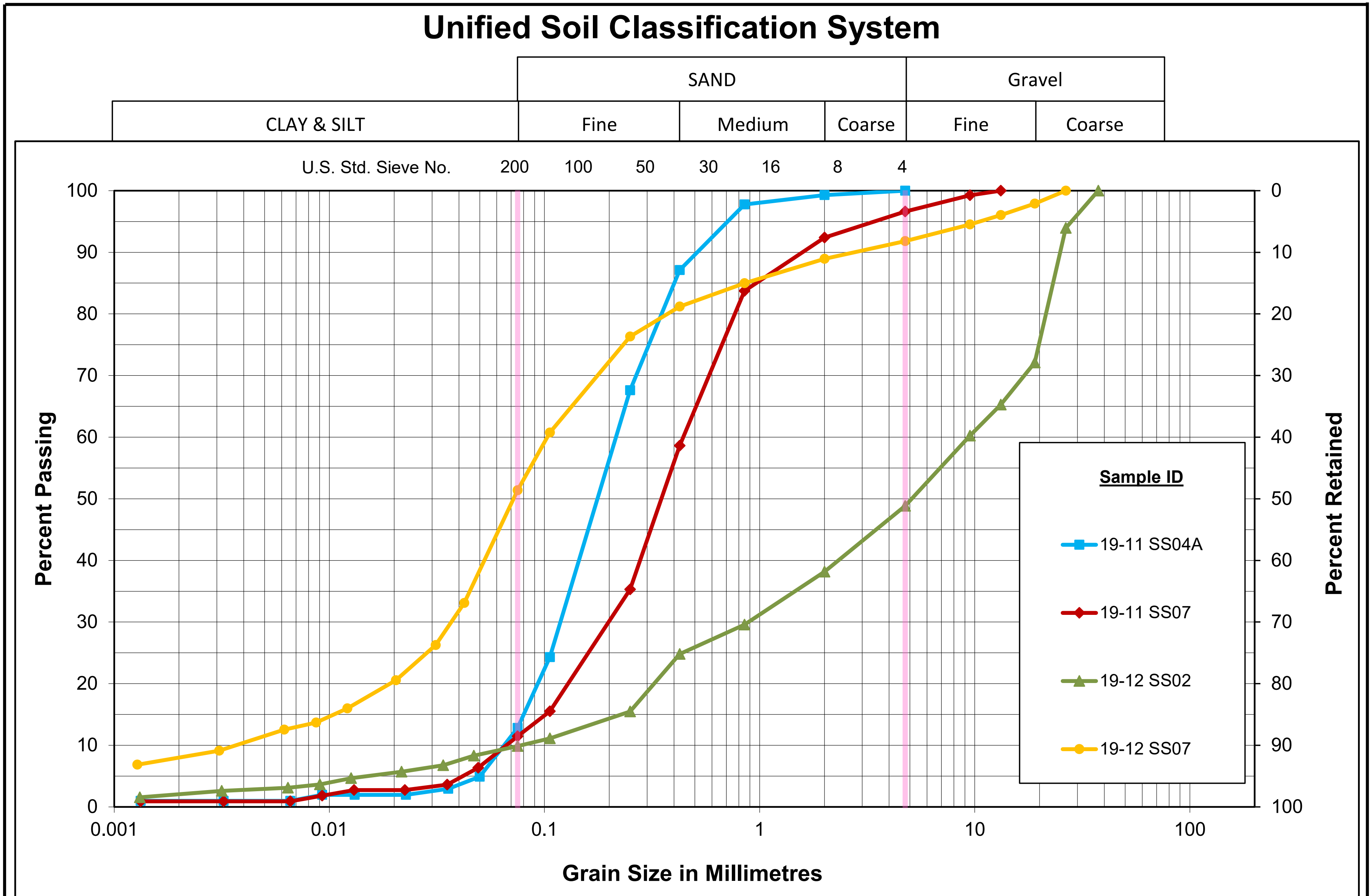
GRAIN SIZE DISTRIBUTION

SILT and SAND (SM) to SAND & GRAVEL (GP)

Hwy 401 Roseville Road Bridge

Figure No. C6

Project No. 165001107



Stantec

GRAIN SIZE DISTRIBUTION

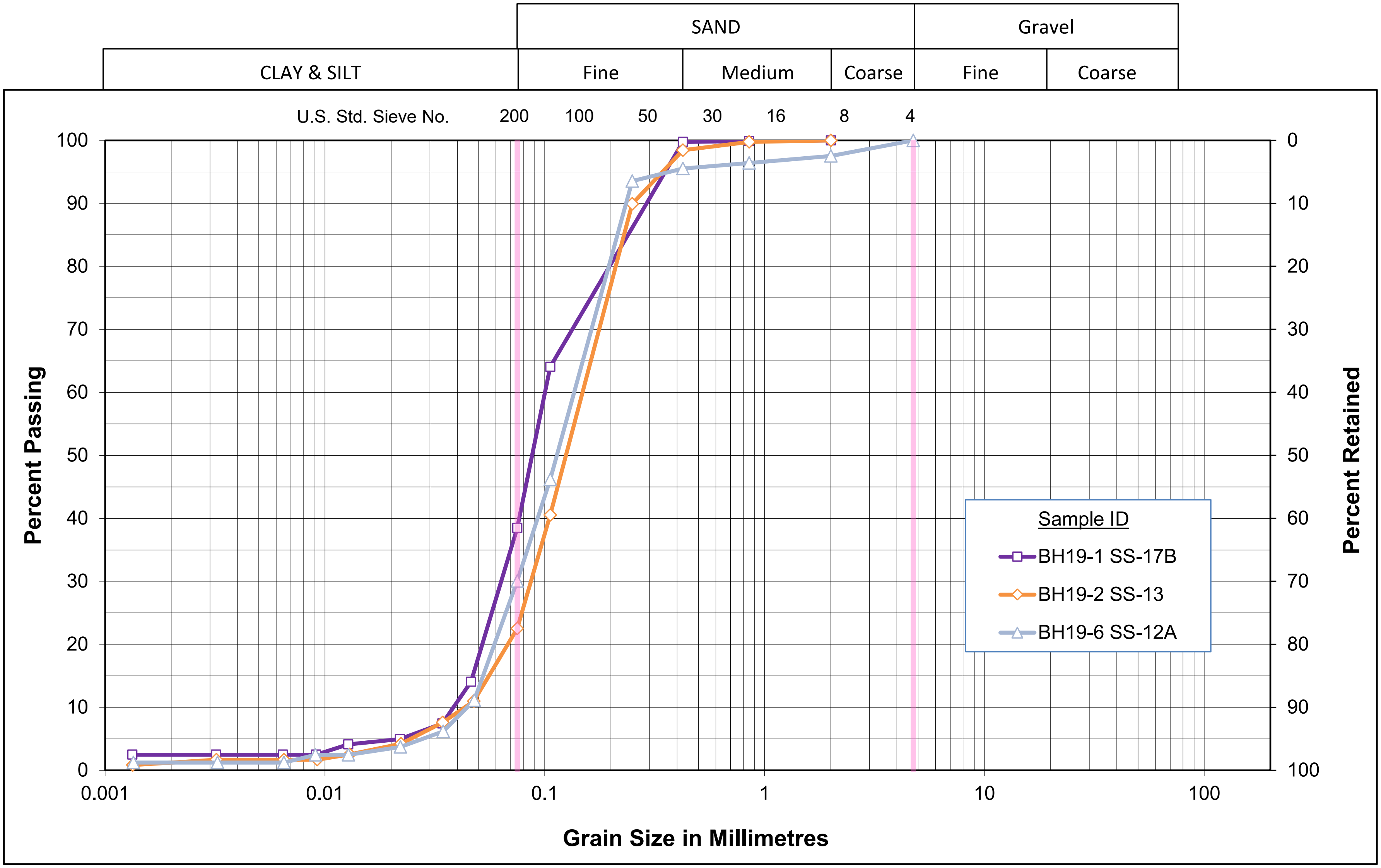
SILT and SAND (SM) to SAND & GRAVEL (GP)

Hwy 401 Roseville Road Bridge

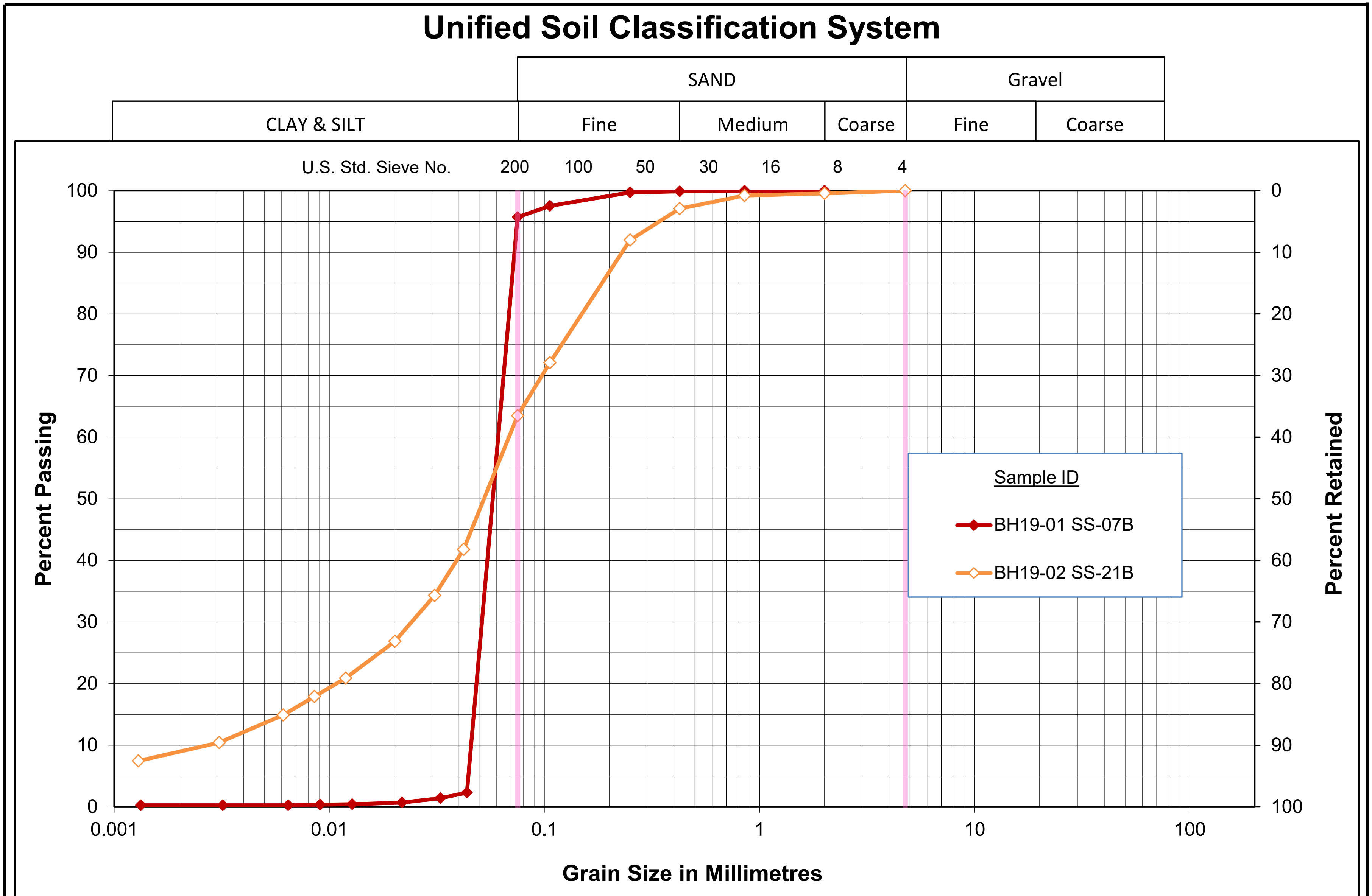
Figure No. C7

Project No. 165001107

Unified Soil Classification System



Stantec	**GRAIN SIZE DISTRIBUTION** Silty Fine SAND (SM) Hwy 401 Roseville Road Bridge	Figure No. C8 Project No. 165001107



Stantec

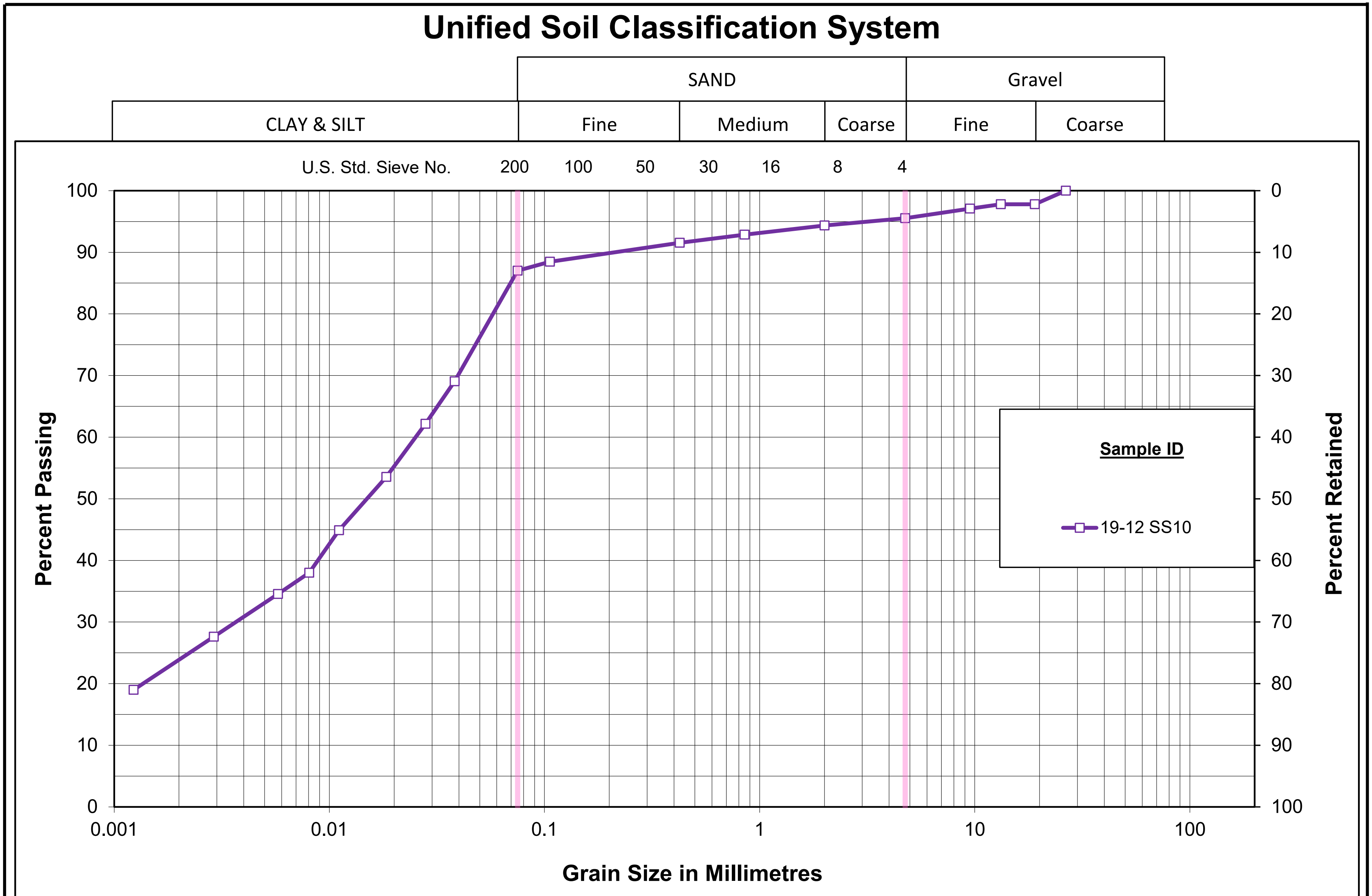
GRAIN SIZE DISTRIBUTION

SILT / Sandy SILT (ML)

Hwy 401 Roseville Road Bridge

Figure No. C9

Project No. 165001107



GRAIN SIZE DISTRIBUTION

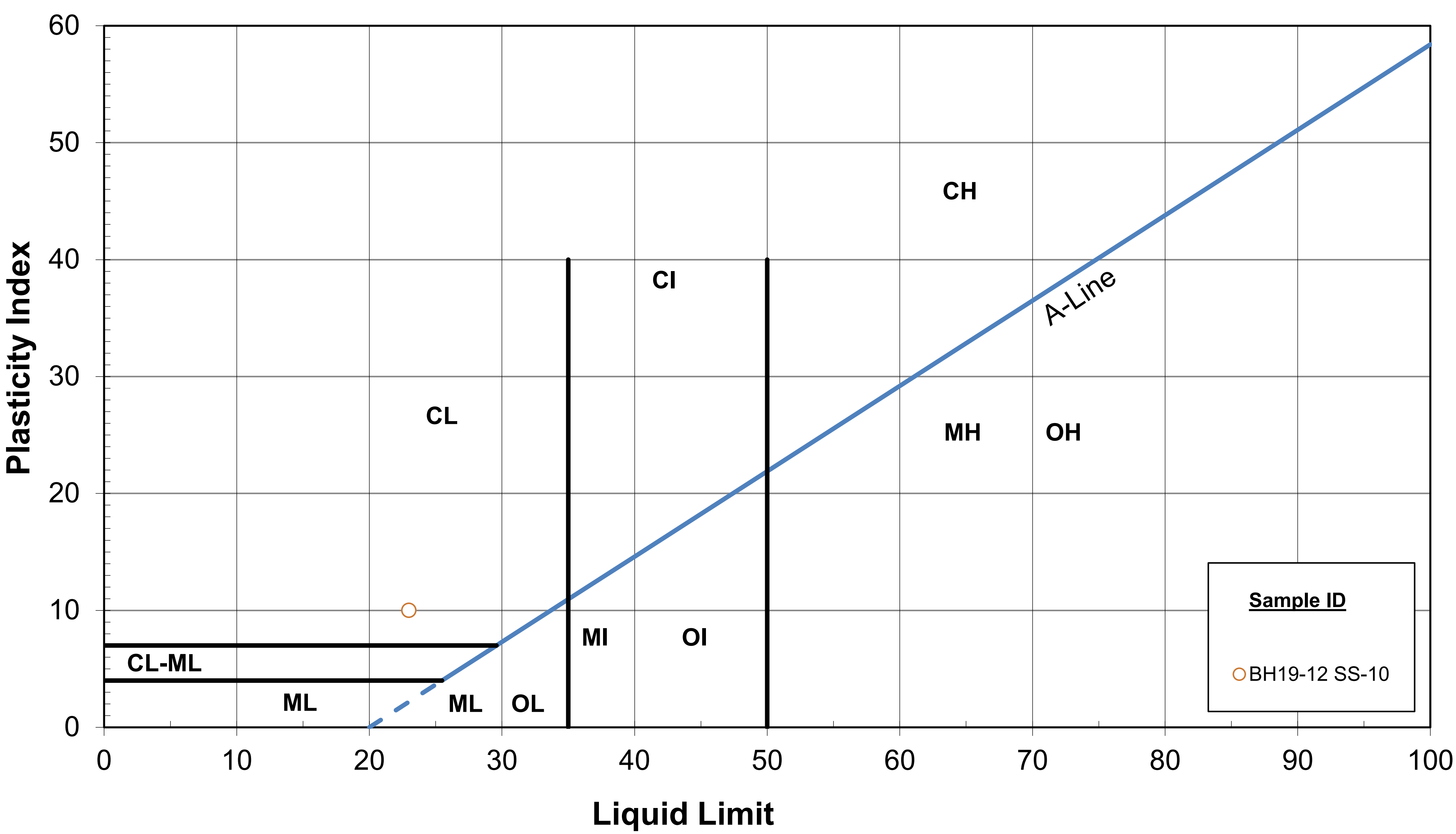
Upper CLAYEY SILT (CL)

Hwy 401 Roseville Road Bridge

Stantec

Figure No. C10

Project No. 165001107



Stantec

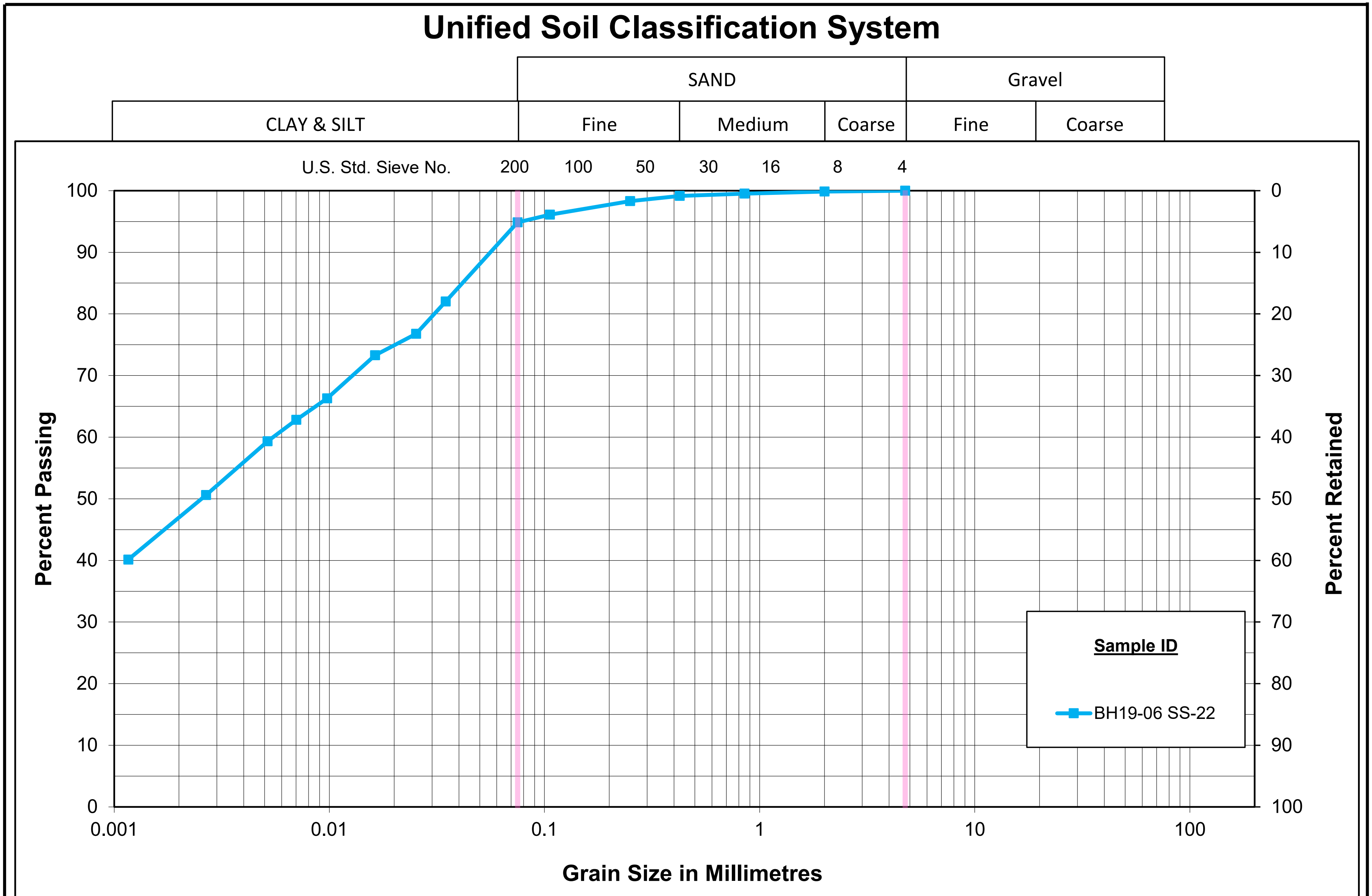
Upper CLAYEY SILT (CL)

Hwy 401 Culverts

PLASTICITY CHART

Figure No. C11

Project No. 165001107



Stantec

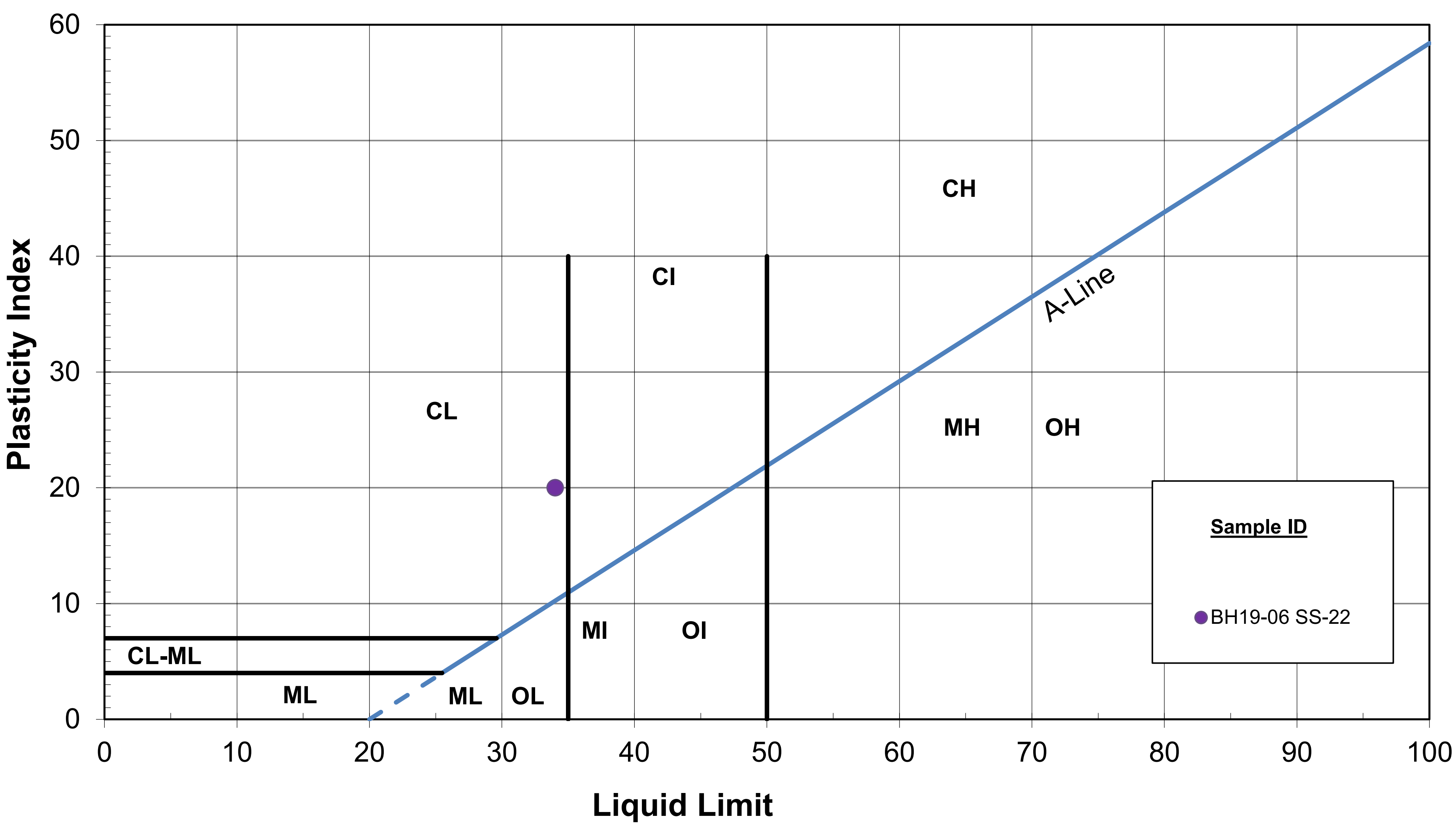
GRAIN SIZE DISTRIBUTION

Lower CLAYEY SILT (CL)

Hwy 401 Roseville Road Bridge

Figure No. C12

Project No. 165001107



Stantec

Lower CLAYEY SILT (CL)

Hwy 401 Roseville Road Bridge

PLASTICITY CHART

Figure No. C13

Project No. 165001107

CONSOLIDATED DRAINED DIRECT SHEAR TEST ASTM D3080 SHEET 1 OF 3	FIGURE		
TEST STAGE	A	B	C
BOREHOLE NUMBER		-	
SAMPLE		19-09	
SAMPLE DEPTH, (m)		1.5	
SAMPLE HEIGHT, (mm)	27.20	26.90	27.60
SAMPLE LENGTH, (mm)	60.00	60.00	60.00
WATER CONTENT, BEFORE TEST, (%)	8.0	8.0	8.0
NORMAL (CONSOLIDATION) STRESS, (kPa)	40	80	120
WATER CONTENT, AFTER TEST, (%)	11.8	10.6	12.2
DISPLACEMENT RATE, mm/min	0.024	0.024	0.024
TIME TO FAILURE, hours	1	1	2
PEAK SHEAR STRESS[1], (kPa)	43.6	107.7	173.6
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	2.0	1.6	2.6
DRY DENSITY, initial, Mg/m^3	2.02	2.05	2.03
WET DENSITY, initial, Mg/m^3	2.18	2.21	2.19

TEST NOTES:

1 In the absence of a peak, the shear stress reported is at 10 percent relative horizontal displacement (ASTM D3080).

2 Normal stresses asigned by the client

3 Specimens compacted to a target density $2.2 g/cm^3$ at 8% moisture content; achieved 99%, 100% and 100% compaction respectively.

4 Direct Shear Tests carried out submerged, per clients instruction.

5 Sample screened through #4 sieve (about 12% retained) piror testing.

Date: 10/24/2019

Project No. 1786672(1000)

Golder Associates

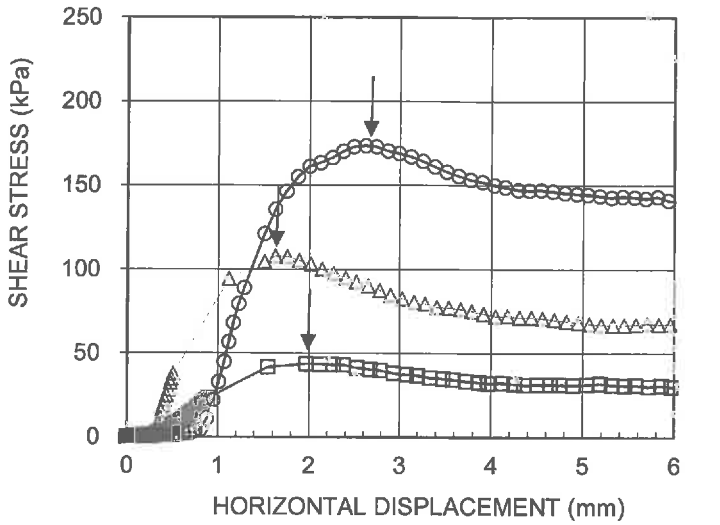
Prepared By: LH

Checked By: [signature]

CONSOLIDATED DRAINED DIRECT SHEAR TEST ASTM D3080 SHEET 2 OF 3	FIGURE

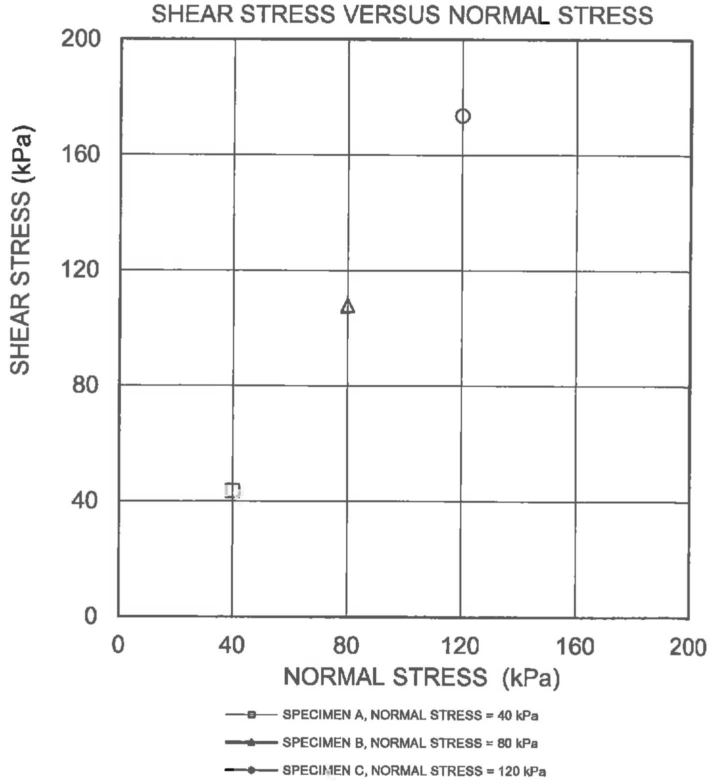
SA 19-09

SHEAR STRESS VERSUS HORIZONTAL DISPLACEMENT



SA 19-09

SHEAR STRESS VERSUS NORMAL STRESS



Date: 10/24/2019

Project No. 1786672(1000)

Golder Associates

Prepared By LH

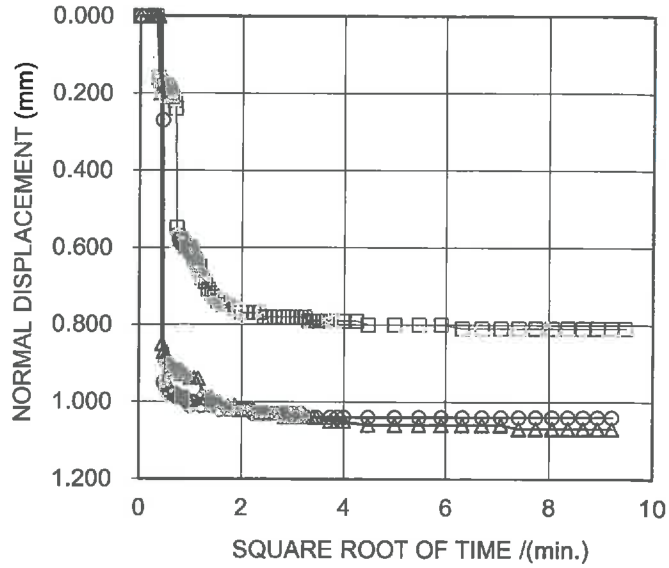
Checked By: [signature]

CONSOLIDATED DRAINED DIRECT SHEAR TEST
ASTM D3080
SHEET 3 OF 3

FIGURE

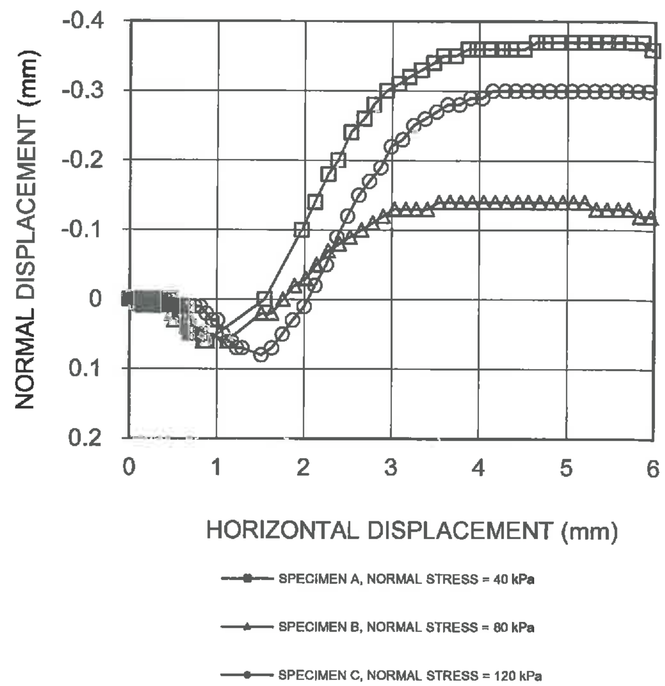
SA 19-09

NORMAL DISPLACEMENT VERSUS SQUARE ROOT OF TIME



SA 19-09

NORMAL DISPLACEMENT VERSUS HORIZONTAL DISPLACEMENT



Date: 10/24/2019

Project No. 1786672(1000)

Golder Associates

Prepared By LH

Checked By: [signature]

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 1 OF 3	FIGURE

TEST STAGE	A	B
BOREHOLE NUMBER	19-12	19-12
SAMPLE NUMBER	-	-
SAMPLE DEPTH, (m)	0.75-1.35	0.75-1.35
SAMPLE HEIGHT, (mm)	57.20	58.40
SAMPLE LENGTH, (mm)	152.30	152.30
WATER CONTENT, BEFORE TEST, (%)	8.24	8.24
NORMAL (CONSOLIDATION) STRESS, (kPa)	60.00	100.00
WATER CONTENT, AFTER TEST, (%)	8.93	9.35
DISPLACEMENT RATE, mm/min	0.43	0.43
TIME TO FAILURE, min	12	14
PEAK SHEAR STRESS[1], (kPa)	75.39	98.98
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	5.04	5.95
DRY DENSITY, initial, Mg/m^3	2.10	2.06
WET DENSITY, initial, Mg/m^3	2.27	2.23

TEST NOTES:

1 In the absence of a peak, the shear stress reported is at 10 percent relative horizontal displacement (ASTM D3080).

2 Normal stresses assigned by the client

3 Specimens compacted to a target density of 2.2g/cm^3 at 8% moisture content; achieved 100% compaction respectively.

4 Direct Shear Tests carried out submerged

5 Sample screened through a 3/8" (9.5mm) sieve prior to testing. Approximately 54% was retained.

Date: 10/28/2019

Project No. 1786672(1000)

Golder Associates

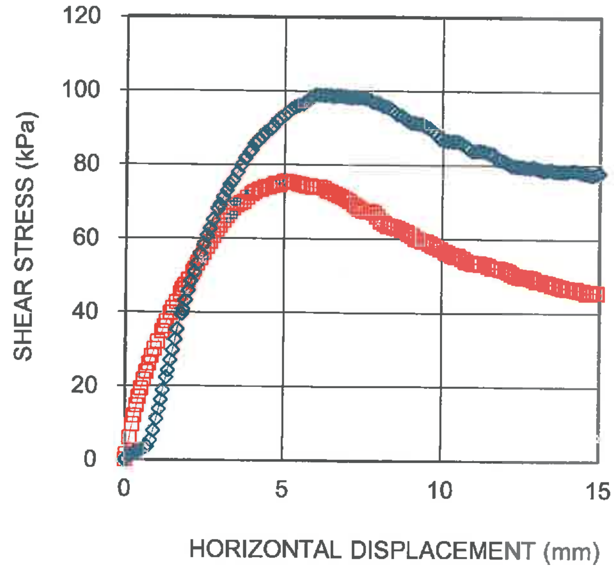
Prepared By: AH

Checked By: [signature]

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 2 OF 3	FIGURE

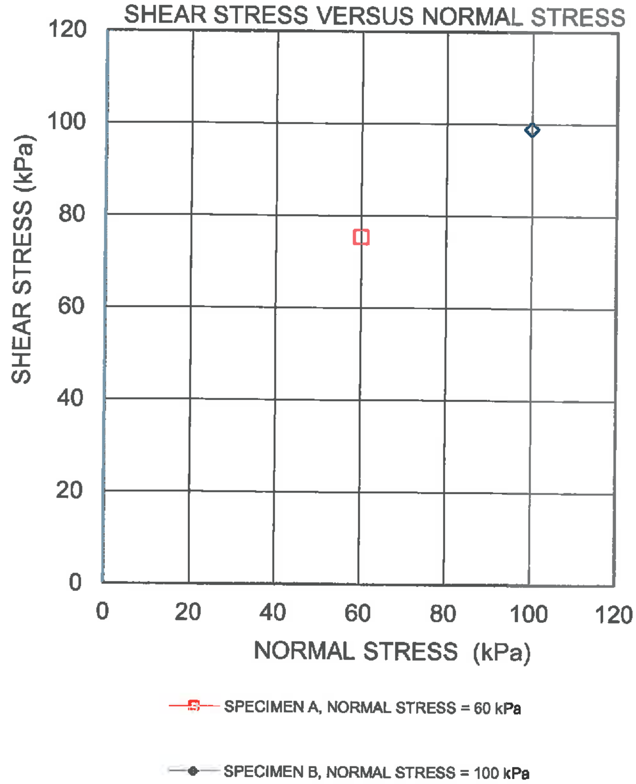
SA 19-12

SHEAR STRESS VERSUS HORIZONTAL DISPLACEMENT



SA 19-12

SHEAR STRESS VERSUS NORMAL STRESS



Date: 10/28/2019

Project No. 1786672(1000)

Golder Associates

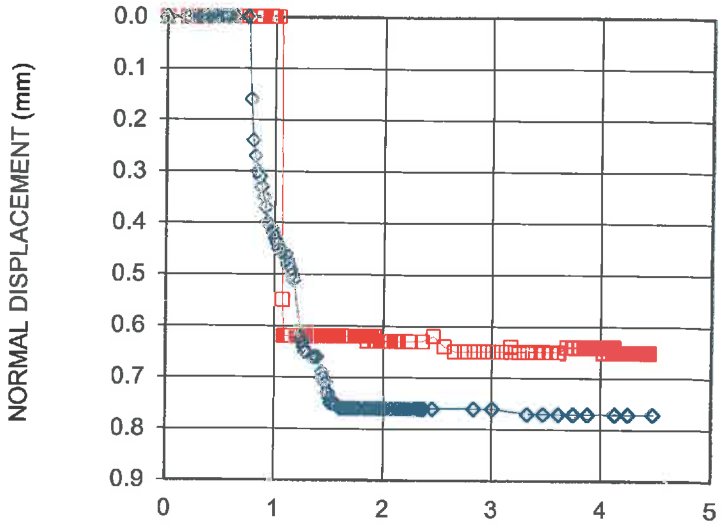
Prepared By AH

Checked By: [signature]

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 3 OF 3	FIGURE

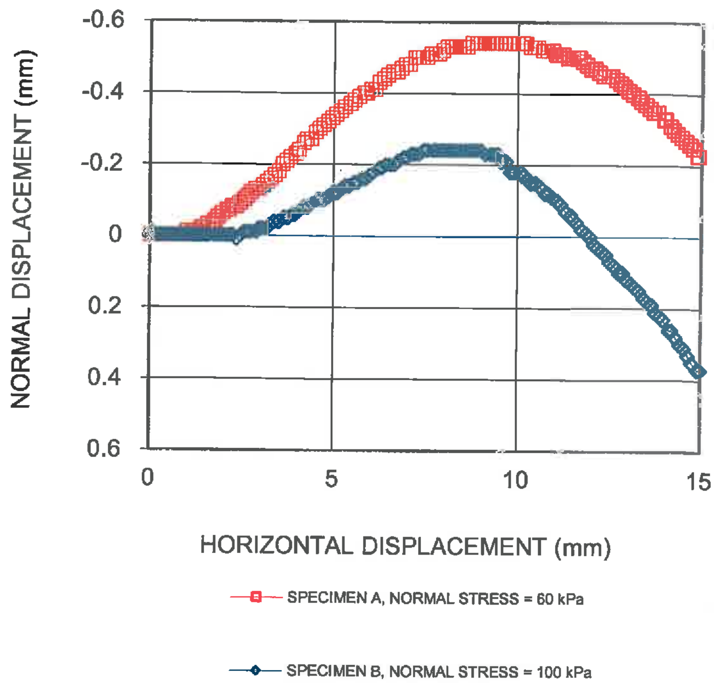
SA 19-12

NORMAL DISPLACEMENT VERSUS SQUARE ROOT OF TIME



SA 19-12

NORMAL DISPLACEMENT VERSUS HORIZONTAL DISPLACEMENT



Date: 10/28/2019

Project No. 1786672 (1000)

Golder Associates

Prepared By AH

Checked By: [signature]

Certificate of Analysis
Client: Stantec Consulting Ltd. (Ottawa)
Client PO: 165001107.340

Report Date: 23-Sep-2019
Order Date: 17-Sep-2019
Project Description: 165001107.340

Client ID:		BH19-08, SS3, 5'-7'	BH19-08, SS12, 35'-37'	-	-
Sample Date:		16-Aug-19 09:00	16-Aug-19 09:00	-	-
Sample ID:		1938243-01	1938243-02	-	-
	MDL/Units	Soil	Soil	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	95.1	90.8	-	-
General Inorganics					
pH	0.05 pH Units	7.86 [1]	7.87 [1]	-	-
Resistivity	0.10 Ohm.m	10.3	56.3	-	-
Anions					
Chloride	5 ug/g dry	565 [1]	37 [1]	-	-
Sulphate	5 ug/g dry	19 [1]	42 [1]	-	-

APPENDIX D

D.1 GEOPHYSICAL TESTING RESULTS

APRIL 30, 2019

Report on Seismic Surveys at the HWY 401 / Roseville Road Bridge Replacement Site

North Dumfries, Ontario

Stantec Project No. 165001107

ClearView Geophysics Inc.

Report on
Seismic Surveys
at the
HWY 401 / Roseville Road Bridge Replacement Site
North Dumfries, Ontario
Stantec Project No. 165001107

On behalf of:

STANTEC CONSULTING LTD.
400-1331 Clyde Avenue
Ottawa, Ontario
K2C 3G4

telephone: (613) 722-4420
facsimile:
E-mail: kevin.nelson@stantec.com

Contact: Mr. Kevin Nelson

By:

ClearView Geophysics Inc.
12 Twisted Oak Street
Brampton, Ontario
L6R 1T1

telephone: 905.458.1883
facsimile: 905.792.1884
cellular: 416.617.1884
E-mail: general@geophysics.ca

Contact: Mr. Joe Mihelcic, P.Eng., M.B.A., Geophysicist

ClearView Ref: W0817
Issued: April 30, 2019

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2. ***Personnel*** ---------- ***1***
3. ***Seismic Method*** ---------- ***1***
4. ***Discussion of Results*** ---------- ***4***
5. ***Conclusions*** ---------- ***12***

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The file is available for download until November 25, 2019 (e.g., using Filezilla https://filezilla-project.org/):

Host=	geophysics.ca
Protocol=	SFTP – SSH File Transfer Protocol
Logon Type=	Normal
Use=	u83505156-stantec
Password=	TrGy26&<%?z

1. Introduction

ClearView Geophysics Inc. carried out MASW (Multichannel Analysis of Surface Waves) seismic surveys for **STANTEC CONSULTING LTD.** at the HWY 401 / Roseville Road Bridge Replacement Site, North Dumfries, Ontario (Stantec Project No. 165001107). The purpose of the work is to determine a seismic site classification as part of a geotechnical investigation.

The fieldwork was completed on April 11, 2019 at the Seismic Spread Locations depicted in Figure 1.

Figure 1: Seismic Spread Locations.

2. Personnel

Joe Mihelcic, P.Eng.; Geophysicist:

Mr. Mihelcic carried out the fieldwork. He processed, plotted and interpreted the data presented in this report and is responsible for data quality.

3. Seismic Method

The **Geometrics Geode Seismograph** is utilized for these surveys. The module is connected to a 5-metre take-out multi-conductor cable that is connected to a line of 24 geophones. The geophones are at 4.5 Hz.

The Geode is powered by a 12V battery. A laptop, running specialized software, is connected to the Geode to record the seismic event immediately following the trigger. The software allows for data viewing, recording and processing in the field. For the present surveys it was set to record over a period of 1 second after the shot.

The Refraction and MASW methods consist of inside, end and offset shots. Ideally, they are typically at multiples of the geophone receiver spacings. All data are preserved in *Geometrics Geode* digital format.

The **Interpex** Limited software *IXRefraX* ver.1.14 was used for the data processing and presentation of refraction results. *IXRefraX* is an integrated software package for processing and interpreting seismic refraction data using the Generalized Reciprocal Method (GRM) of Dereke Palmer.

Geode data were imported into *IXRefraX*. First break picking was carried out manually. Initial estimates of number of layers, layer velocities and layer thicknesses were determined from an examination of the travel time curves plotted by the software.

The MASW data processing was carried out using ***ParkSEIS*** software. MASW is based on the fact that surface waves are easy to generate and detect, propagate parallel to the surface, have the strongest energy, amplitudes decrease with depth from the surface and waveforms spread out away from the source (i.e., exhibit dispersion). Surface waves are different from body waves including compression waves (P) and shear waves (S).

Most of the energy from surface waves are confined near the surface to a depth of approximately one wavelength. Longer wavelengths penetrate deeper and therefore their velocities are affected by the deeper materials. If the surface wave velocity of each wavelength (frequency) is measured, it is possible to obtain the corresponding material properties, mainly shear wave velocities, of the layers.

For a uniform half-space, there is no dispersion. For a layered medium with increased velocity with depth, the surface waves exhibit dispersion. The lower frequency components propagate faster than the higher frequency components so the waveform gradually spreads out away from the source.

Just like first arrivals picking is the most important part of refraction analyses, the most important task in surface wave data processing is to accurately measure the phase velocity of surface waves. That is, generating the dispersion curve.

Note that the longer the wavelength and therefore the deeper the penetration, the larger the uncertainty. Converse to refraction analyses, an increase in geophone spacing will improve the measurement of phase velocity for MASW. Also, the wave fields near the source are very complicated, with refracted, reflected, air and direct waves all mixed together and the low frequency surface waves have not yet completely developed. That is why the off-end shots are necessary to allow accurate measurement of the slower surface waves later in time once the faster waves have already passed all of the geophones.

The final step in surface wave data processing is to derive the velocity structure of the subsurface from dispersion curves through inversion. The dispersion curve is generally a simple curve presenting the trend of velocity variation with depth. However and not trivially, inversion exhibits non-uniqueness and therefore several sets of shear wave velocity profiles can be derived from the same dispersion curve. Background information on the survey area is incorporated into the model to reduce non-uniqueness.

In spite of non-uniqueness of inversion results, a dispersion curve can reliably estimate the average velocity above a certain depth. The Vs^{30m}/Vs^{100ft} parameter is an example. It can also determine the thickness and velocity of overburden. The value of Vs^{30m} is fairly stable and can be relied on with relatively good confidence.

The following table displays the difference between Vs and Vp for common soils and rocks. Note that both Vs and Vp are higher for saturated soils compared to dry soils. Therefore, MASW analyses can be used to detect the water table where there is sufficient velocity contrast. It can also be used, in limited cases and qualitatively, to determine the bulk density of the various layers detected where the velocity varies proportionally with density.

Type of formation	P wave velocity (m/s)	S wave velocity (m/s)	Density (g/cm^3)	Density of constituent crystal (g/cm^3)
Scree, vegetal soil	300-700	100-300	1.7-2.4	-
Dry sands	400-1200	100-500	1.5-1.7	2.65 quartz
Wet sands	1500-2000	400-600	1.9-2.1	2.65 quartz
Saturated shales and clays	1100-2500	200-800	2.0-2.4	-
Marls	2000-3000	750-1500	2.1-2.6	-
Saturated shale and sand sections	1500-2200	500-750	2.1-2.4	-
Porous and saturated sandstones	2000-3500	800-1800	2.1-2.4	2.65 quartz
Limestones	3500-6000	2000-3300	2.4-2.7	2.71 calcite
Chalk	2300-2600	1100-1300	1.8-3.1	2.71 calcite
Salt	4500-5500	2500-3100	2.1-2.3	2.1 halite
Anhydrite	4000-5500	2200-3100	2.9-3.0	-
Dolomite	3500-6500	1900-3600	2.5-2.9	(Ca, Mg) $CO_3$2.8-2.9
Granite	4500-6000	2500-3300	2.5-2.7	-
Basalt	5000-6000	2800-3400	2.7-3.1	-
Gneiss	4400-5200	2700-3200	2.5-2.7	-
Coal	2200-2700	1000-1400	1.3-1.8	-
Water	1450-1500	-	1.0	-
Ice	3400-3800	1700-1900	0.9	-
Oil	1200-1250	-	0.6-0.9	-

Appendix A (Instrument Specifications) provides additional information.

4. Discussion of Results

Two (2) seismic spreads were setup and read: spread 1 in the northeast and spread 2 in the northwest as depicted in Figure 1. The spreads were aligned along the toe of the northern slope from Roseville Road. Figure 2 depicts the typical vegetation along the spreads.

Figure 2: Spread 1 on left looking east; Spread 2 on right looking east.

The following Table 1 lists the seismic shot files and shot positions along each geophone spread. It also indicates the refraction analyses P-wave velocities and depths assuming two upper layers and a lower higher seismic velocity half-space (e.g., compact/dense material). These are based on the first-arrival straight line interpretations displayed in Figure 3 through Figure 6 for each of the four end-of-spread shots. Note from Table 1 column D1+D2, interpreted higher velocity half-space appears a few metres deeper under spread 1 compared to under spread 2.

	File	File_Posn	Gnd_Posn	V1 (m/s)	V2 (m/s)	V3 (m/s)	D1 (m)	D2 (m)	D1+D2 (m)
SP1:			(Figure 1)						
east_end	**62**	75	-6						
east_end	**63**	69	0	539	1670	3204	1.1	12.3	13.4
east_end	**64**	60	9						
west_end	**65**	75	75						
west_end	**66**	69	69	1754	1785	2660	6.8	6.9	13.7
west_end	**67**	60	60						
west_end	**68**	45	45						
SP2:									
east_end	**69**	75	0						
east_end	**70**	69	6	756	1523	2264	1.1	7.1	8.2
	71	*n/a*	*n/a*						
east_end	**72**	60	15						
east_end	**73**	45	30						
west_end	**74**	81	87						
	75	69	75	520	1433	2365	2	3.6	5.6
	76	60	66						
	77	45	51						

Table 1: Seismic files and shot positions

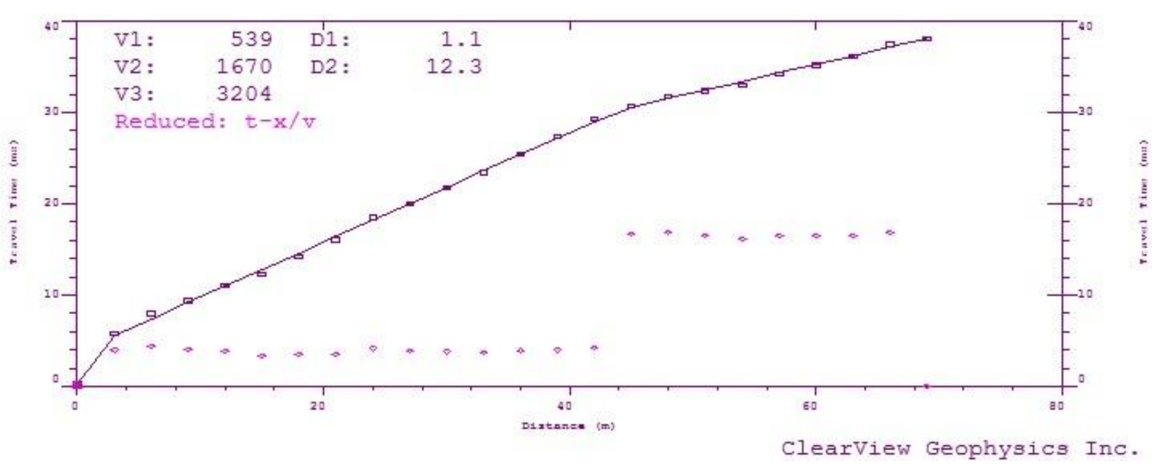


Figure 3: Spread 1 East End Shot 63.

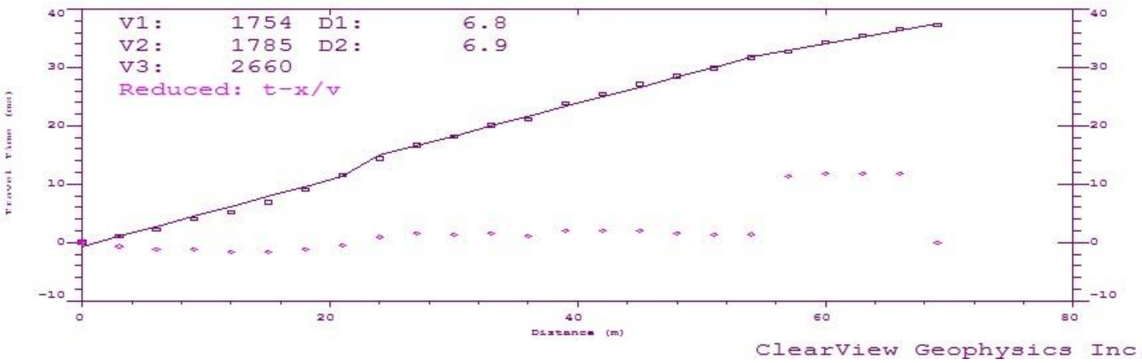


Figure 4: Spread 1 West End Shot 66.

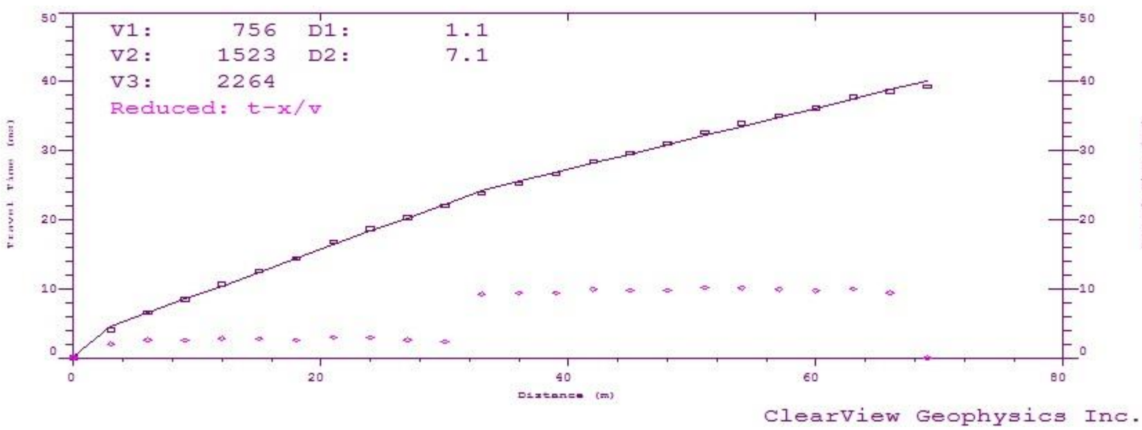


Figure 5: Spread 2 East End Shot 70.

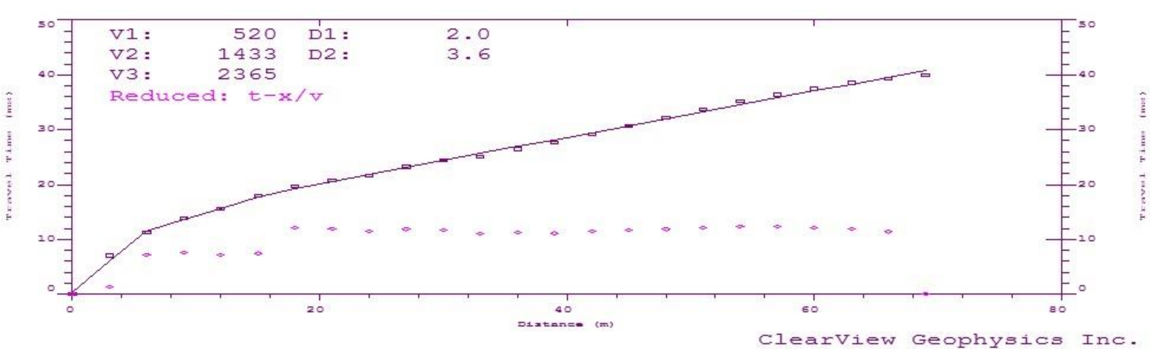


Figure 6: Spread 2 West End Shot 75.

The MASW results were 1D-modeled for each of the four grouped shots listed in Table 1. The dispersion curves and model depth profiles are presented in Figure 7 through Figure 10.

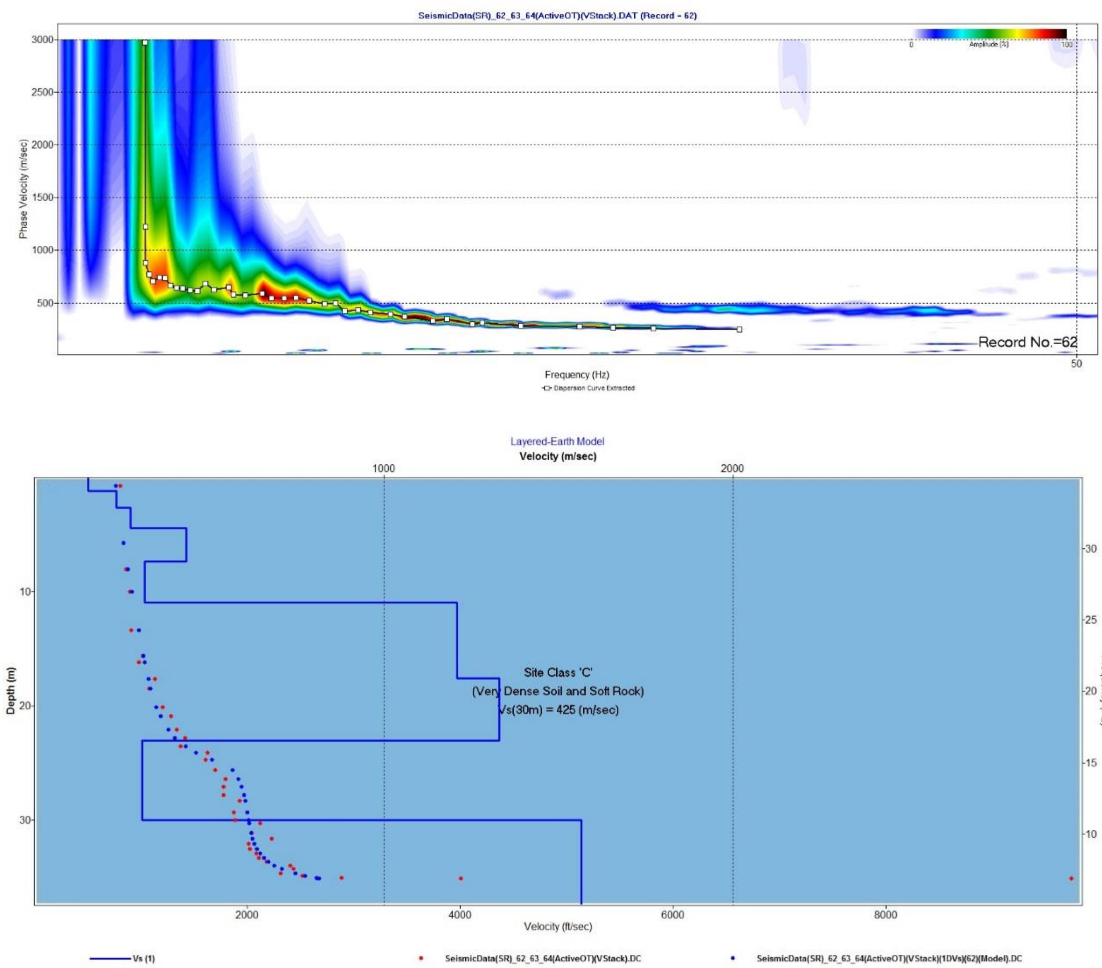


Figure 7: Spread 1 East End Stacked Shots 62, 63 & 64 (refer to Table 1).

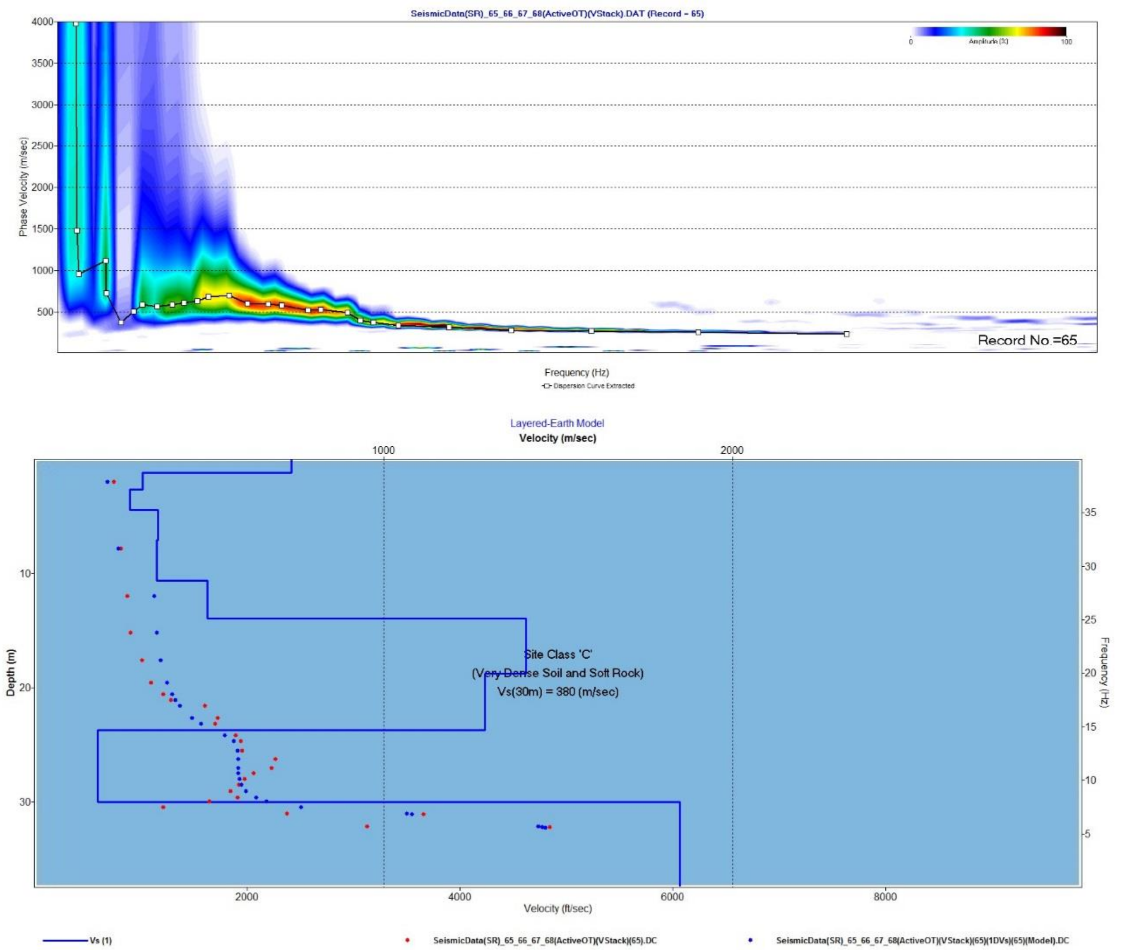


Figure 8: Spread 1 West End Stacked Shots 65, 66, 67 & 68 (refer to Table 1).

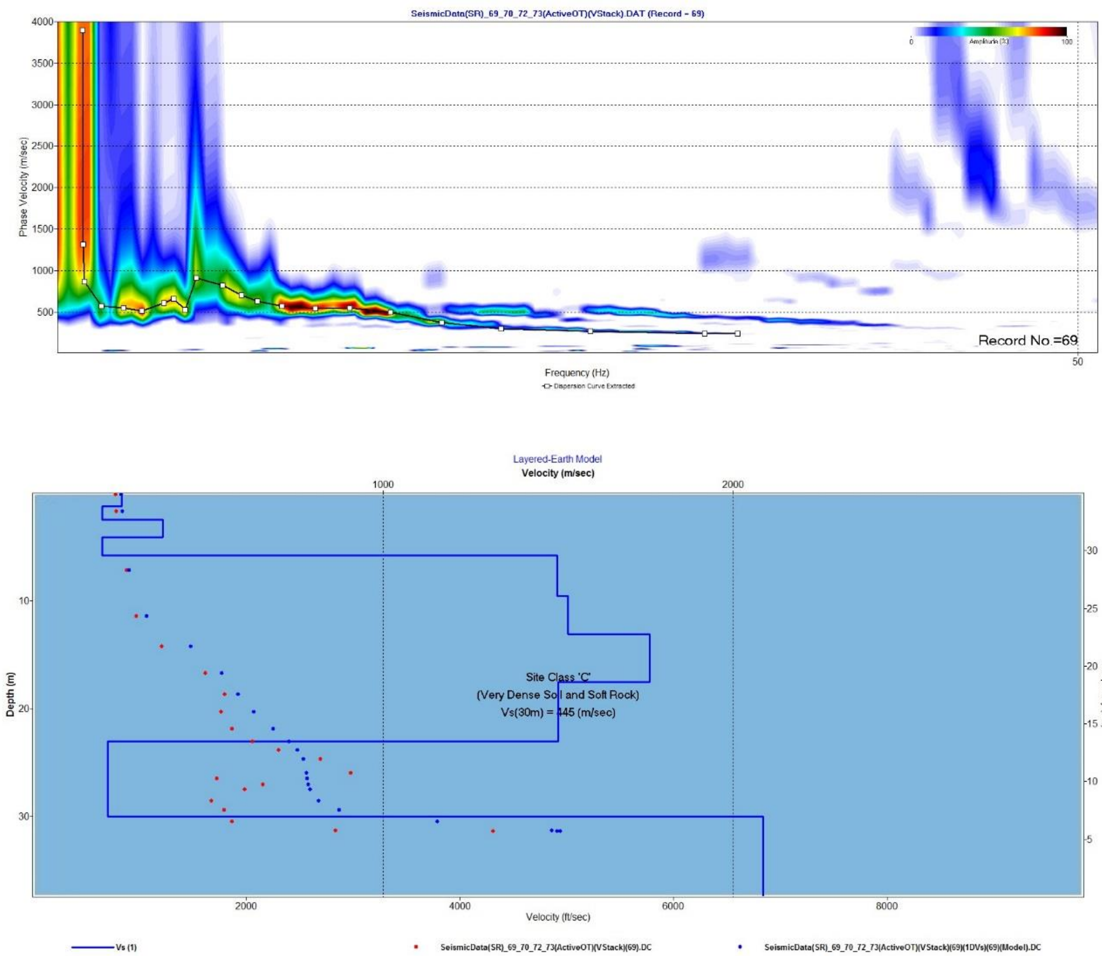


Figure 9: Spread 2 East End Stacked Shots 69, 70, 72 & 73 (refer to Table 1).

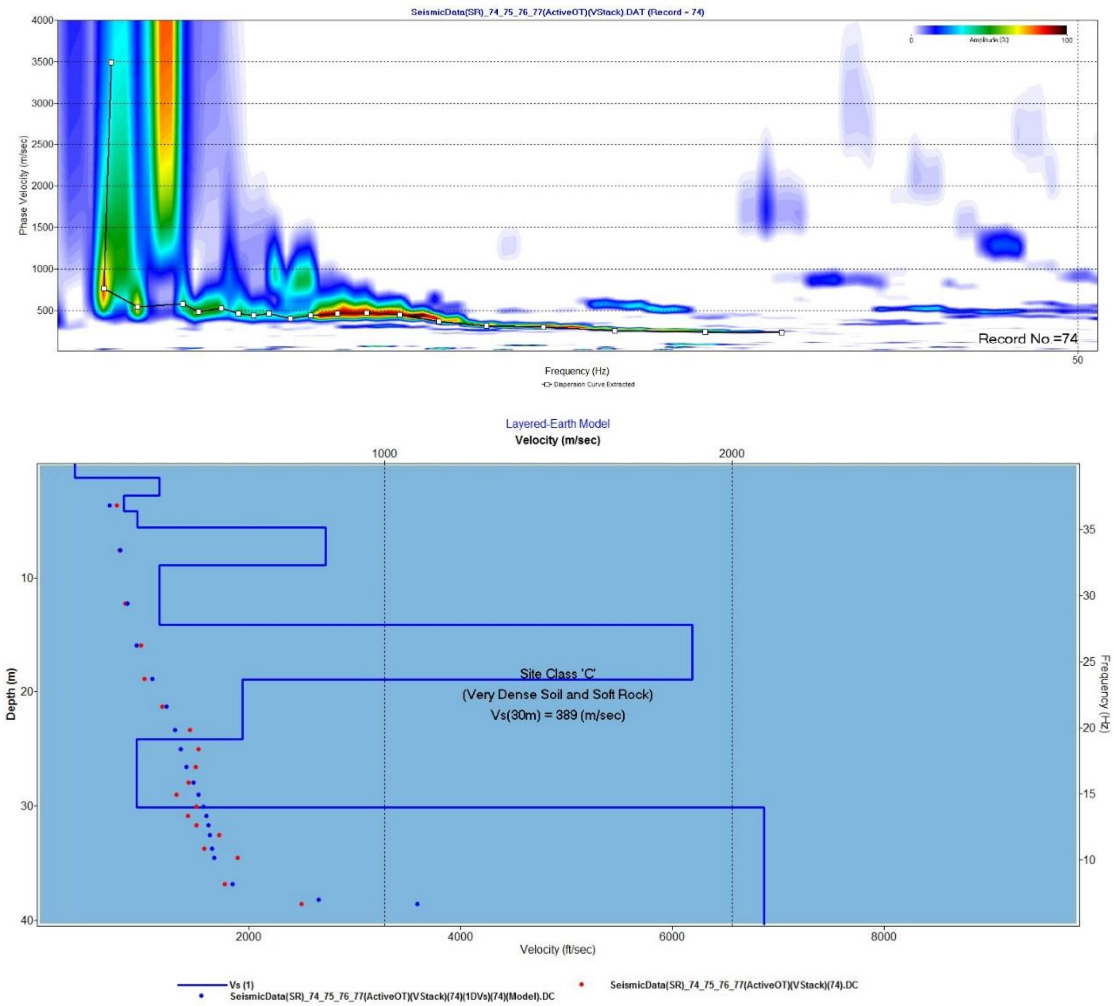


Figure 10: Spread 2 West End Stacked Shots 74, 75, 76 & 77 (refer to Table 1).

The models depicted in Figure 7 through Figure 10 show relatively high variability in soil types with depth. Vs^{30m} results are summarized as follows in Table 2.

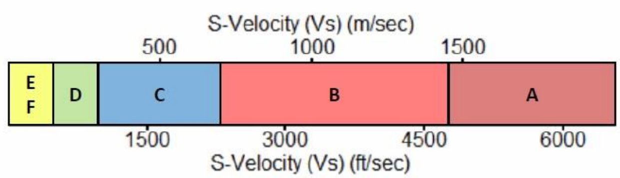
Spread 1, East End (Figure 7)	425 m/s
Spread 1, West End (Figure 8)	380 m/s
Spread 2, East End (Figure 9)	445 m/s
Spread 2, West End (Figure 10)	389 m/s

Table 2: MASW 1D Vs^{30m} Results

Appendix B presents the model results in spreadsheet format. Note that, as with the refraction analyses presented in Table 1 and Figure 3 through Figure 6, the P-wave velocities determined from the MASW analyses also indicates a more compact faster half-space material is likely deeper under spread 1 compared to under spread 2. The complete detailed model results with legends for each seismic shot are recorded in the <*.lyr> files preserved in the \\MASW_used\ directory.

Figure 11 presents a Seismic Site Classification chart. The results presented in Appendix B and Table 2 indicate the upper 30 metres of soils are generally classed '**C, Very Dense Soil and Soft Rock**'.

Seismic Site Classification ($Vs^{30\text{-}m}$ or $Vs^{100\text{-}ft}$)



NEHRP* Seismic site classification based on shear-velocity (Vs) ranges.

Site Class	S-Velocity (Vs) (ft/sec)	S-Velocity (Vs) (m/sec)
A **(Hard Rock)**	> 5,000	**> 1500**
B **(Rock)**	2,500 – 5000	**760 – 1500**
C **(Very Dense Soil and Soft Rock)**	1,200 – 2,500	**360 – 760**
D **(Stiff Soil)**	600 – 1,200	**180 – 360**
E **(Soft Clay Soil)**	< 600	**< 180**
F **(Soils Requiring Add'l Response)**	< 600, and meeting some additional conditions.	**< 180, and meeting some additional conditions.**

*** National Earthquake Hazard Reduction Program (www.nehrp.gov)**

Figure 11: Seismic Site Class Chart.

5. Conclusions

The inversion MASW model results presented in Appendix B and Table 2 were done with default layers to obtain a best fit with recorded data. The results for Vs^{30m} were consistently classed '**C, Very Dense Soil and Soft Rock**' for all shots. However, the models also depict poorly defined layering with high variability in calculated velocities within this 30-metre region.

The refraction and MASW analyses of the data indicate a faster seismic layer is likely a few metres to several metres deeper under spread 1 (i.e., east side of HWY 401) compared to under spread 2 (i.e., west side of HWY 401).

If there are any questions about the surveys or the interpretation, please do not hesitate to contact the undersigned.

Sincerely,

ClearView Geophysics Inc.

Per:

Joe Mihelcic, P.Eng., M.B.A.
Geophysicist/President

Appendix A – Instrument Specifications

Geode

Ultra-Light Exploration Seismograph

The Geodes operate from either your laptop for occasional surveys or from the StrataVisor NZ in harsh field conditions where reliability counts

The new Geode seismic recorder is the next generation of seismic recording system, combining the best of traditional seismic recorders with the flexibility and convenience of a distributed system. Place a Geode module out on the line close to the geophones and eliminate long, expensive analog cables. Never hassle with poor connections and bad roll box contacts when doing a reflection survey. And when you are not using the Geode for exploring, use it for monitoring earthquakes, quarry blasts or vibration from heavy equipment.

For light-duty applications, you can use your laptop to view, record and process your data. For more demanding applications where ruggedness and reliability are key, use the Geode in combination with StrataVisor NZ series seismographs. The StrataVisor and laptop software have the same user interface, so transition between configurations is seamless.

The Geode seismic module weighs only 8 lbs (3.6 kg) and comes with 25 m of digital interface cable. Because getting the right answer is key, the Geode is bundled with a suite of no-charge industry-standard professional software that expands your capabilities for commercial or research applications.

- Multi-purpose seismic recorder: refraction, reflection, earthquake monitoring, blast and vibration measurements
- Light-weight (8 lb/3.6 kg), in-field module connects to your laptop for small surveys or connects to StrataVisor NZ series seismographs for professional applications in harsh environments
- Available with 3 to 24 channels per module; connect more modules to build low cost distributed systems
- Data transmitted from Geode to host computer digitally, eliminating noisy, expensive analog cables
- Uses inexpensive refraction cables for local geophone connections
- 24-bit resolution, low distortion and built-in geophone and line testing
- 20 kHz bandwidth (8 to 0.02 ms sampling) allows ultra-high resolution engineering surveys or earthquake monitoring
- Bundled professional software gives quick answers: Built-in first break picker and layer assignment
  - Industry-standard refraction analysis (SIPQC)
  - New refraction tomography (Optim)
  - Reflection processing (WinSeis-Lite)
- Three year warranty
- Standby low-power mode means light batteries, long life

The StrataVisor NZ has a daylight visible color screen, built-in plotter and is weather and shock resistant

Appendix B – MASW Model Results

Spread 1, East End, Stacked Shots 62,63,64										
Layer#	Depth(m)	Thck(m)	Vs(m/s)	Vp(m/s)	Poisson	Density*	Qs*	Qp*	Vs-Lower*	Vs-Upper*
1	1.163	1.163	153.54	420.89	0.423	1.75	5	20	115.119	191.954
2	2.616	1.453	235.83	646.45	0.423	1.75	10	30	195.438	276.215
3	4.433	1.817	275.35	754.8	0.423	1.75	10	30	257.753	292.942
4	7.349	2.916	434.97	1192.36	0.423	1.75	20	50	434.974	434.974
5	10.963	3.614	315.68	700.3	0.372	2	20	50	303.577	327.775
6	17.597	6.634	1209.31	2682.75	0.372	2	50	150	925.31	1493.316
7	23.07	5.473	1331.45	2953.71	0.372	2	50	150	1022.929	1639.971
8	30	6.93	308.62	684.64	0.372	2	50	150	271.176	346.06
9	HalfSpace	N/A	1565.93	3473.89	0.372	2	75	250	1318.086	1813.781
Spread 1, West End, Stacked Shots 65, 66, 67, 68										
Layer#	Depth(m)	Thck(m)	Vs(m/s)	Vp(m/s)	Poisson	Density*	Qs*	Qp*	Vs-Lower*	Vs-Upper*
1	1.163	1.163	735.7	1671.75	0.38	1.75	5	20	17.066	1454.34
2	2.616	1.453	310.6	851.44	0.423	2	10	30	10.076	611.12
3	4.433	1.817	273.82	750.62	0.423	2	10	30	18.919	528.724
4	7.084	2.651	354.06	936.91	0.417	2.25	20	50	41.422	666.688
5	10.626	3.542	349.88	959.13	0.423	2.5	50	150	349.883	349.883
6	13.943	3.317	496.07	1245.34	0.406	2.5	50	150	491.263	500.877
7	18.778	4.835	1409.11	2804.47	0.331	2.5	50	150	70.409	2747.816
8	23.726	4.948	1291.08	2616.25	0.339	2.5	50	150	121.882	2460.284
9	30	6.274	181.86	728.97	0.467	2.5	50	150	60.083	303.633
10	HalfSpace	N/A	1848.15	3566.3	0.316	2.75	75	250	205.022	3491.274
Spread 2, East End, Stacked Shots 69, 70, 72, 73										
Layer#	Depth(m)	Thck(m)	Vs(m/s)	Vp(m/s)	Poisson	Density*	Qs*	Qp*	Vs-Lower*	Vs-Upper*
1	1.237	1.237	254.72	537.95	0.355	1.75	5	20	0	509.449
2	2.474	1.237	199.26	596.39	0.437	2	10	30	199.26	199.26
3	4.104	1.63	371.82	1019.25	0.423	2	10	30	0	743.632
4	5.791	1.687	198.63	544.5	0.423	2.25	20	50	0	397.26
5	9.542	3.751	1497.22	2979.83	0.331	2.25	20	50	0	2994.436
6	13.09	3.548	1527	3039.09	0.331	2.5	50	150	0	3053.994
7	17.526	4.436	1761.16	3422.86	0.32	2.5	50	150	0	3522.328
8	23.07	5.544	1500.82	2962.02	0.327	2.5	50	150	0	3001.647
9	30	6.93	213.64	639.43	0.437	2.5	50	150	0	427.28
10	HalfSpace	N/A	2084.32	3292.78	0.166	2.75	75	250	0	4168.646
Spread 2, West End, Stacked Shots 74, 75, 76, 77										
Layer#	Depth(m)	Thck(m)	Vs(m/s)	Vp(m/s)	Poisson	Density*	Qs*	Qp*	Vs-Lower*	Vs-Upper*
1	1.256	1.256	108.38	324.38	0.437	1.75	5	20	108.378	108.378
2	2.825	1.569	351.49	1052.02	0.437	1.75	10	30	221.946	481.034
3	4.168	1.343	249.78	747.59	0.437	1.75	10	30	139.794	359.757
4	5.599	1.431	288.16	687.13	0.393	2	20	50	175.626	400.7
5	8.896	3.297	831.07	1981.7	0.393	2	20	50	531.991	1130.15
6	14.138	5.242	352.76	841.17	0.393	2	50	150	303.387	402.134
7	18.928	4.79	1884.98	4494.87	0.393	2	50	150	1183.661	2586.301
8	24.138	5.21	591.4	1410.2	0.393	2	50	150	378.389	804.406
9	30.111	5.972	285.5	680.78	0.393	2	50	150	207.545	363.459
10	HalfSpace	N/A	2092.02	4988.47	0.393	2	75	250	1258.274	2925.775

*Note: Densities are in gram per cubic centimeters (gm/cc).

Qs and Qp: Quality (Q) factors for S and P waves, respectively (...not used for dispersion calculation)

Vs-Lower and Vs-Upper, if assigned, indicate lower and upper limits of 99 percent (%) confidence in solution.

APPENDIX E

E.1 FIGURE E1: GEOTECHNICAL MODEL (NORTH SIDE OF BRIDGE)

E.2 FIGURE E2: GEOTECHNICAL MODEL (SOUTH SIDE OF BRIDGE)

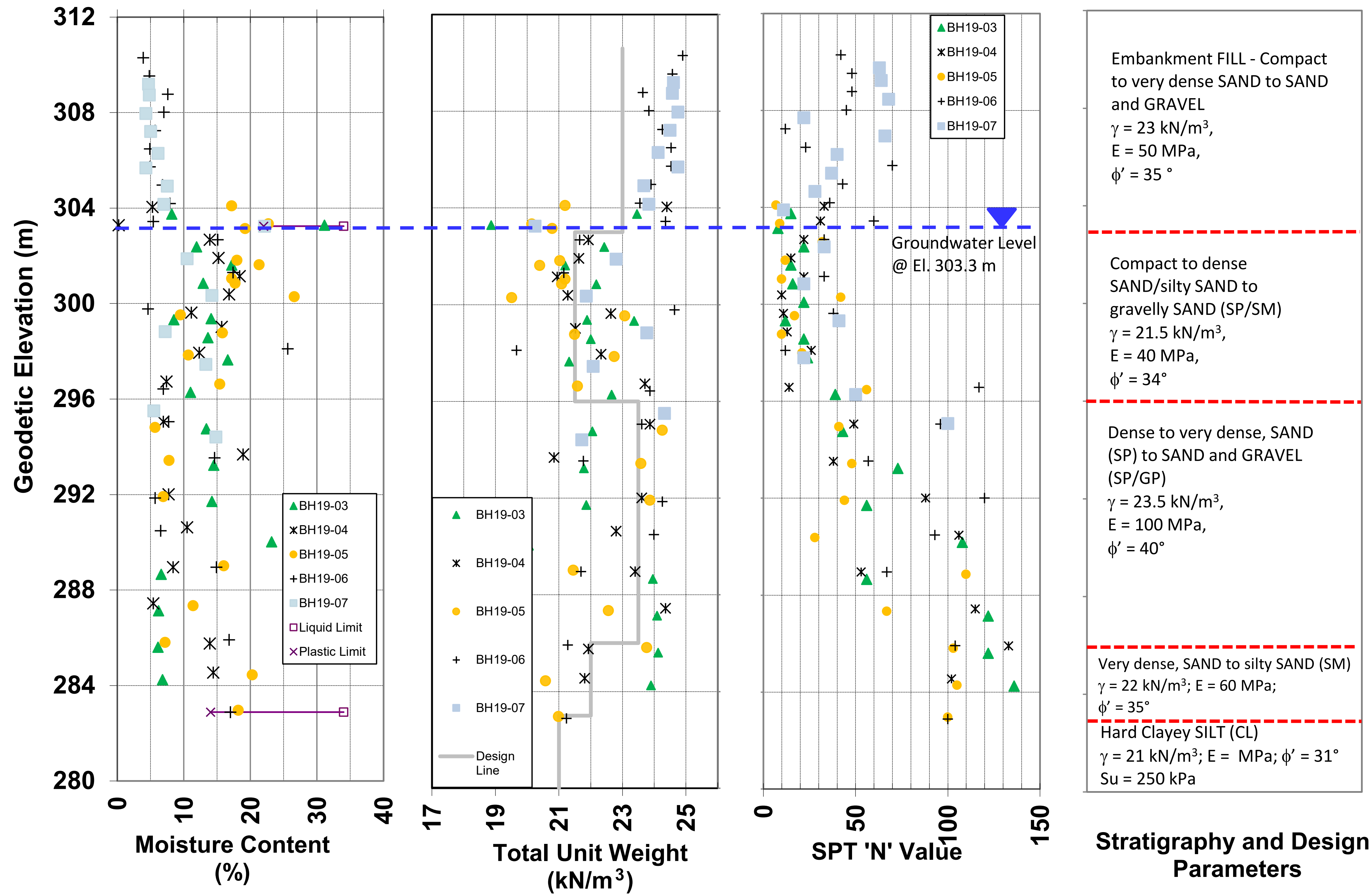
E.3 FIGURE E3: STATIC SLOPE STABILITY ANALYSIS (SOUTH ABUTMENT)

E.4 FIGURE E4: STATIC SLOPE STABILITY ANALYSIS (SOUTH APPROACH EMBANKMENT - 2H:1V SIDESLOPE)

E.5 FIGURE E5: SETTLEMENT ANALYSIS RESULTS - EXISTING EMBANKMENT FILL CONFIGURATION

E.6 FIGURE E6: SETTLEMENT ANALYSIS RESULTS – FINAL WIDENED EMBANKMENT FILL CONFIGURATION

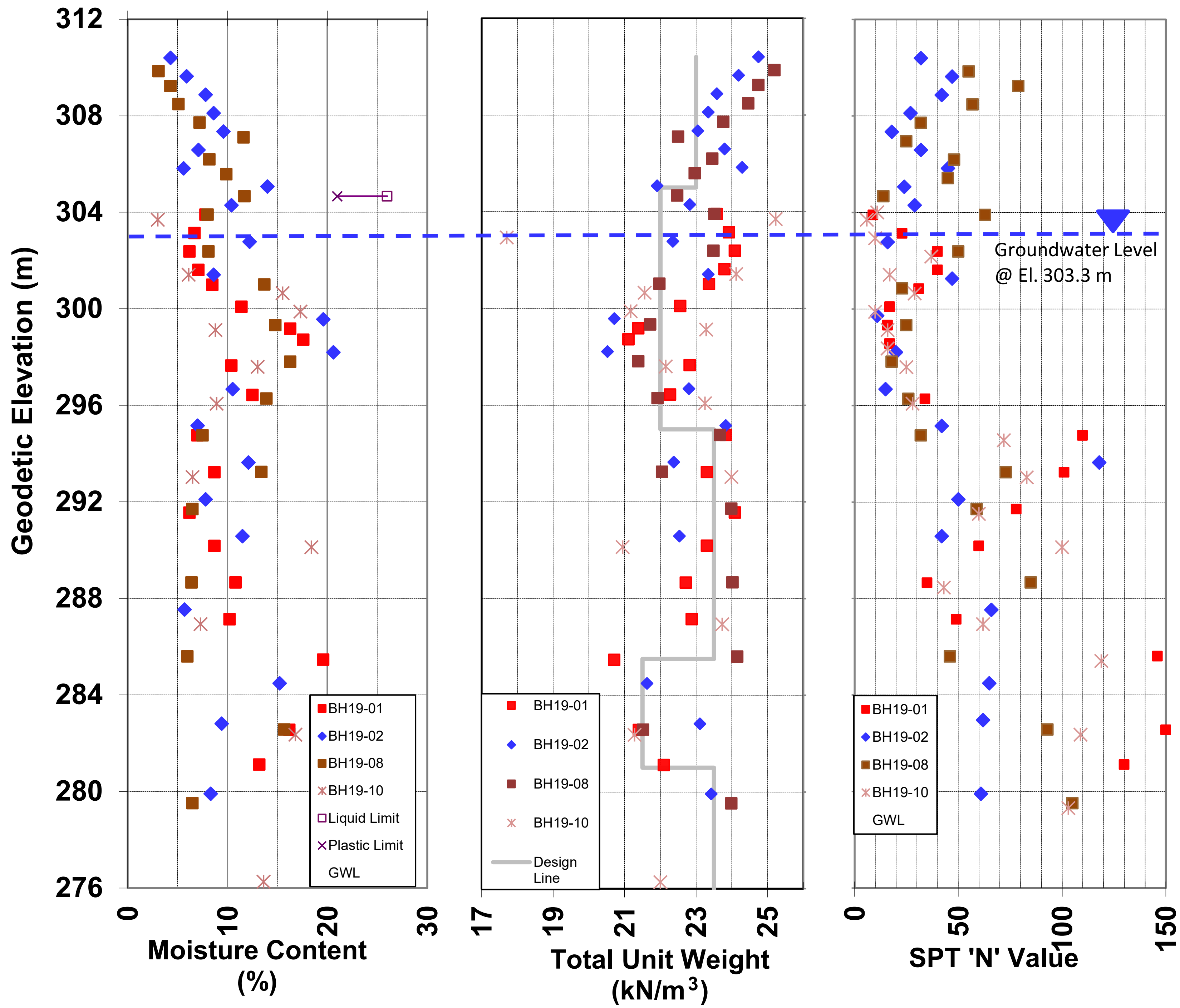
E.7 TABLE E1 TO E8: LOAD INTENSITY P (KN/M) VS LATERAL DEFLECTION Y (M) DATA POINTS



Stantec Consulting Ltd.

Geotechnical Model - North Side of Bridge
Hwy 401 Roseville Road

Figure E1



Embankment FILL
Above 305 m: Compact to very dense SAND to SAND and GRAVEL
γ = 23.0 kN/m^3, E = 50 MPa,
ϕ' = 35°
Below 305 m: Very stiff to hard, gravelly CLAYEY SILT with sand
γ = 22.0 kN/m^3, E = 25 MPa,
ϕ' = 32°, Su = 100 kPa

Compact to dense, SAND and GRAVEL (SP/GP) to SAND/Silty SAND (SP/SM)
γ = 22 kN/m^3,
E = 40 MPa,
ϕ' = 34 °

Dense to very dense, SAND and GRAVEL (SP/GP) to SAND/Gravelly SAND (SP)
γ = 23.5 kN/m^3,
E = 100 MPa,
ϕ' = 40°

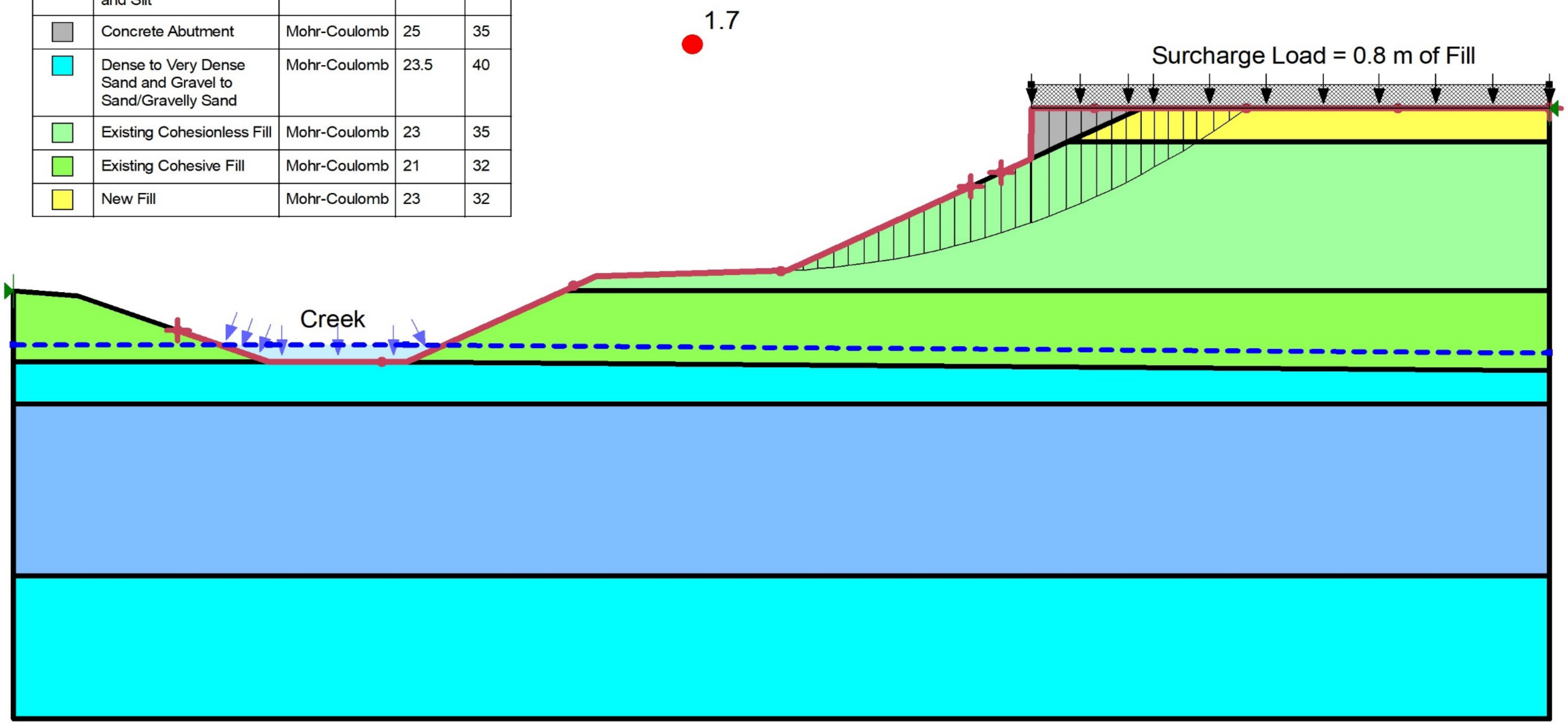
Very dense, SAND to SAND and SILT (SP to SM)
γ = 23.5 kN/m^3, E = 60 MPa;
ϕ' = 35°

Very dense, SAND/Gravelly SAND (SP) to Sandy GRAVEL (GP)
γ = 23.5 kN/m^3; E = 100 MPa;
ϕ' = 38°

Stratigraphy and Design Parameters

Stantec Consulting Ltd.	**Geotechnical Model - South Side of Bridge Hwy 401 Roseville Road**	**Figure E2**

Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	Compact Sand to Sand and Silt	Mohr-Coulomb	22.5	34
	Concrete Abutment	Mohr-Coulomb	25	35
	Dense to Very Dense Sand and Gravel to Sand/Gravelly Sand	Mohr-Coulomb	23.5	40
	Existing Cohesionless Fill	Mohr-Coulomb	23	35
	Existing Cohesive Fill	Mohr-Coulomb	21	32
	New Fill	Mohr-Coulomb	23	32



Stantec

Static Slope Stability Analysis

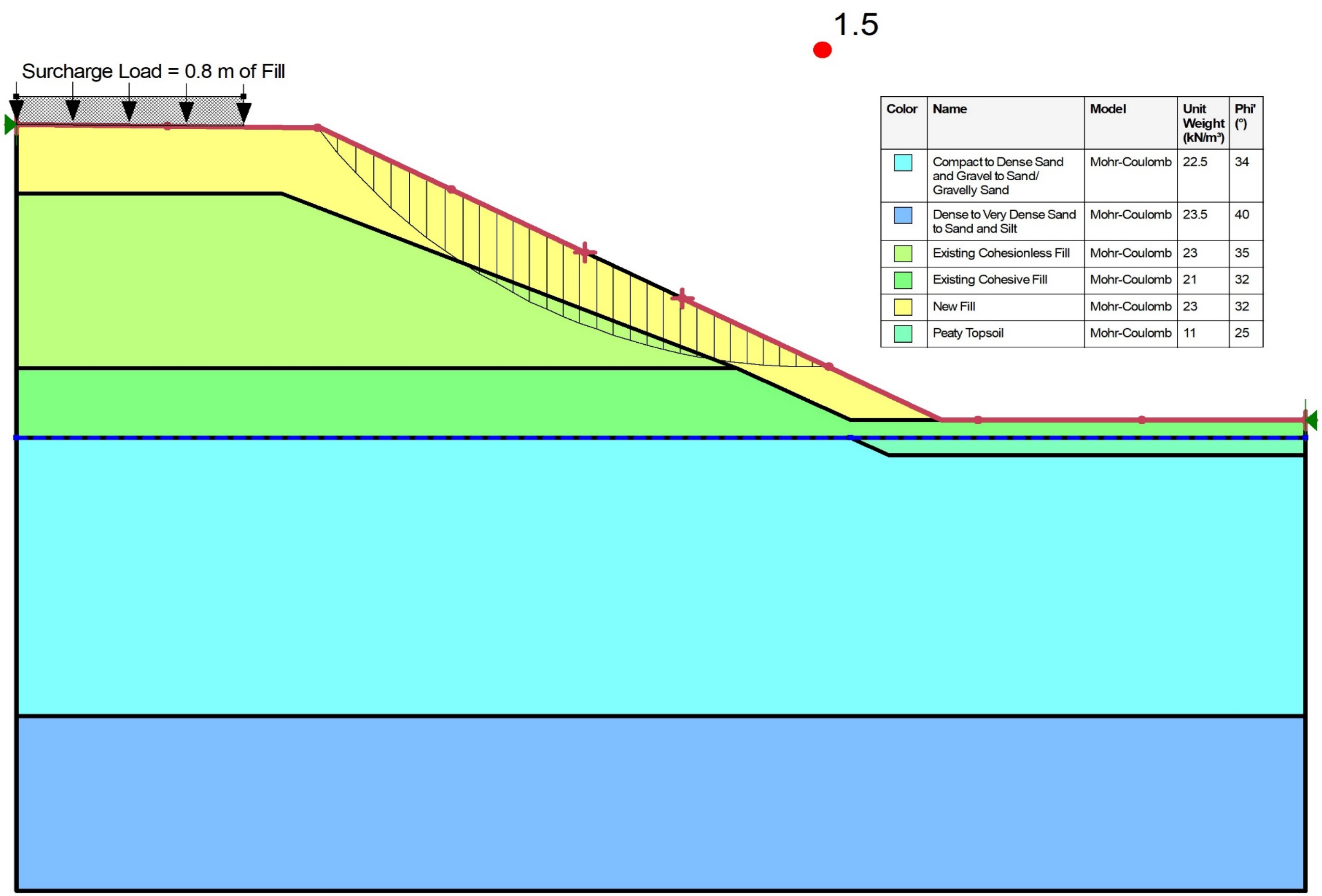
South Abutment

Highway 401 Underpass at Roseville Road

Figure E3

Project No. 165001107

GWP No. 3204-16-00



Color	Name	Model	Unit Weight (kN/m³)	Phi' (°)
	Compact to Dense Sand and Gravel to Sand/ Gravelly Sand	Mohr-Coulomb	22.5	34
	Dense to Very Dense Sand to Sand and Silt	Mohr-Coulomb	23.5	40
	Existing Cohesionless Fill	Mohr-Coulomb	23	35
	Existing Cohesive Fill	Mohr-Coulomb	21	32
	New Fill	Mohr-Coulomb	23	32
	Peaty Topsoil	Mohr-Coulomb	11	25

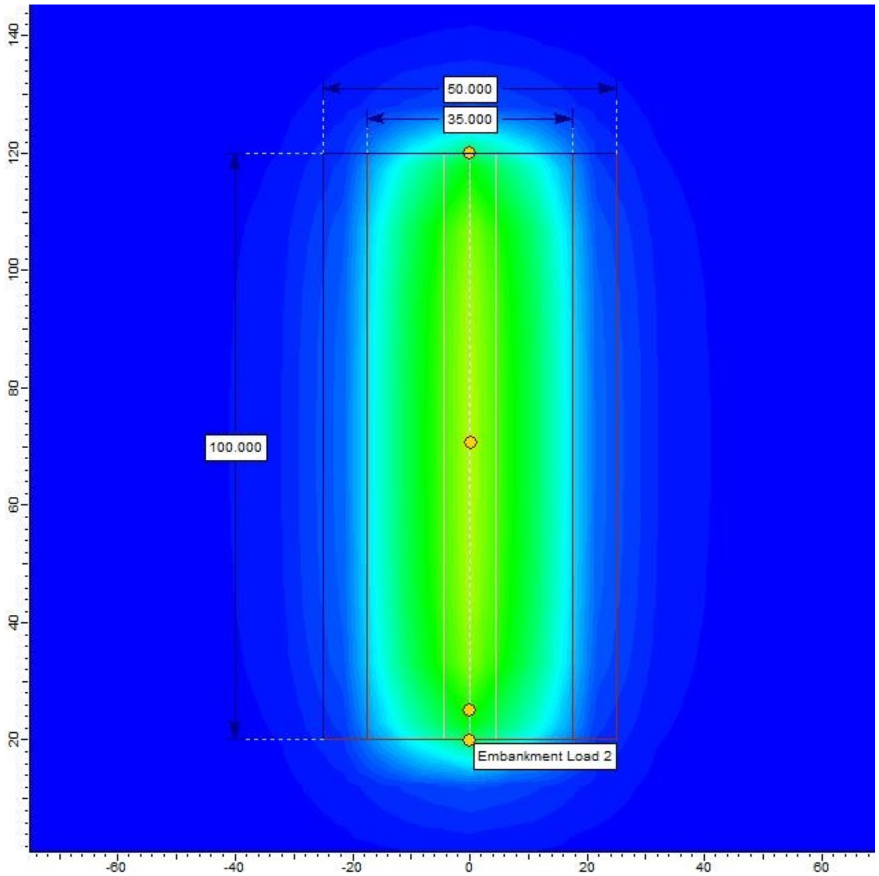
Stantec

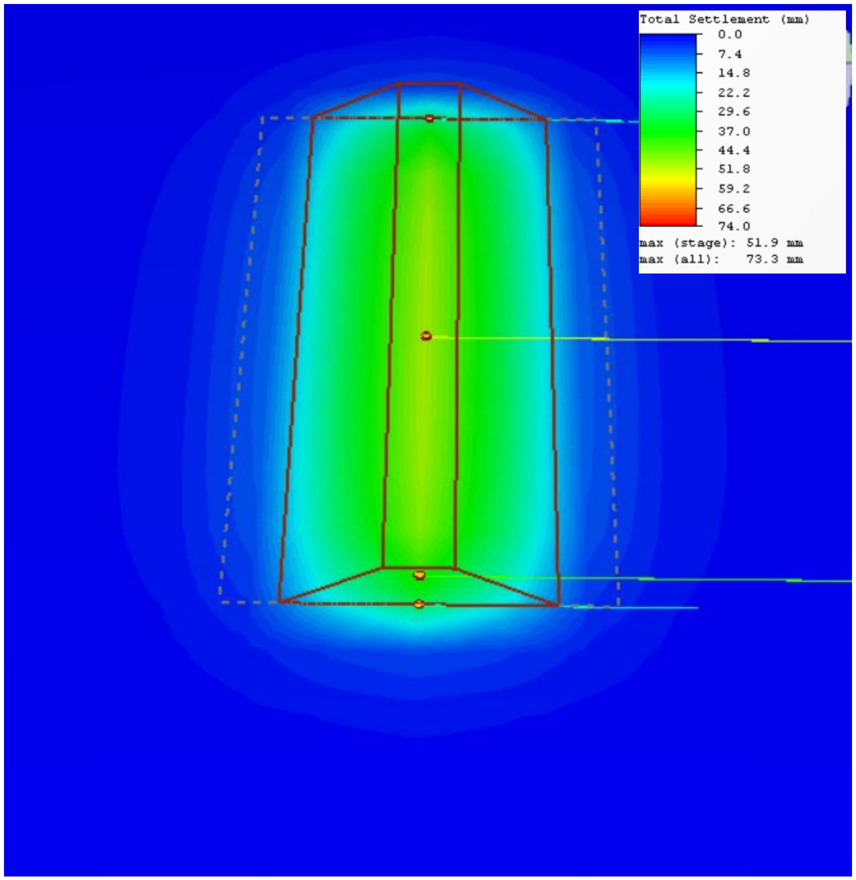
Static Slope Stability Analysis
South Approach Embankment - 2H:1V Sideslope
Highway 401 Underpass at Roseville Road

Figure E4

Project No. 165001107

GWP No. 3204-16-00



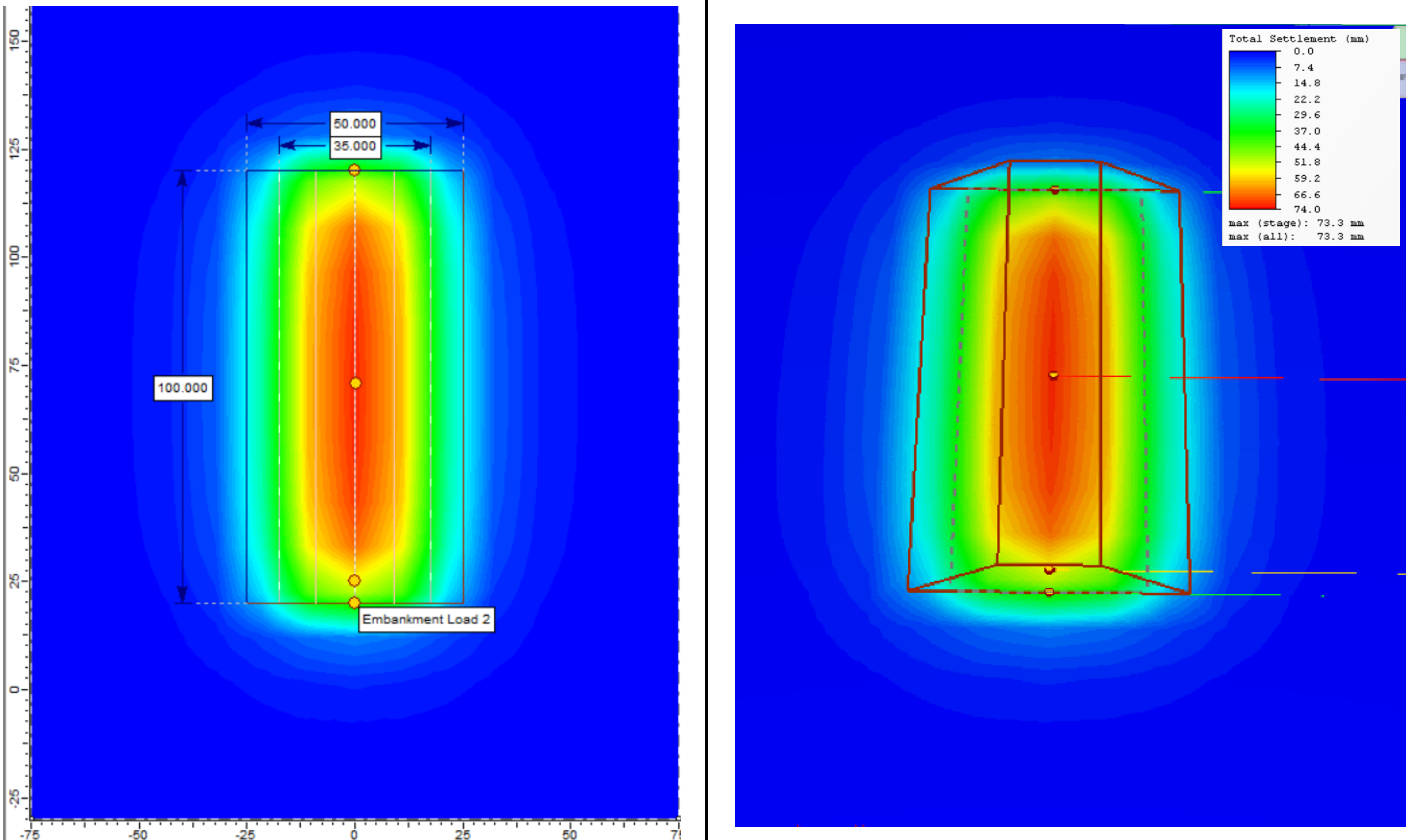


Stantec

Settlement Analysis Results
Existing Embankment Fill Configuration
Roseville Road Underpass Replacement

Figure E-5

Project No. 165001107



Stantec

Settlement Analysis Results
Final Widened Embankment Fill Configuration
Roseville Road Underpass Replacement

Figure E-6

Project No. 165001107

Table E1: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for North Abutment - HP 310 x 110

Depth Below Abutment Wall (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.00147	0.00295	0.00442	0.0059	0.00737	0.00884	0.01032	0.01179	0.01327	0.01474	0.01621	0.01769	0.01916	0.02064	0.02211	0.023585
	P	0	17.611	30.9062	38.9678	43.2235	45.3067	46.2887	46.7435	46.9522	47.0477	47.0914	47.1113	47.1203	47.1245	47.1264	47.1272	47.1276
1.0	Y	0	0.00236	0.00472	0.00707	0.00943	0.01179	0.01415	0.01651	0.01887	0.02122	0.02358	0.02594	0.0283	0.03066	0.03302	0.03537	0.037732
	P	0	43.5788	76.4782	96.4266	106.958	112.112	114.543	115.668	116.184	116.421	116.529	116.578	116.6	116.611	116.615	116.617	116.6183
2.0	Y	0	0.00337	0.00675	0.01012	0.0135	0.01687	0.02025	0.02362	0.02699	0.03037	0.03374	0.03712	0.04049	0.04387	0.04724	0.05062	0.05399
	P	0	103.926	182.384	229.957	255.07	267.364	273.159	275.842	277.075	277.638	277.895	278.013	278.066	278.091	278.102	278.107	278.1093
3.0	Y	0	0.00441	0.00881	0.01322	0.01763	0.02203	0.02644	0.03084	0.03525	0.03966	0.04406	0.04847	0.05288	0.05728	0.06169	0.0661	0.070503
	P	0	189.998	333.435	420.407	466.321	488.796	499.39	504.296	506.549	507.579	508.049	508.264	508.362	508.407	508.427	508.436	508.4405
4.0	Y	0	0.00263	0.00526	0.00789	0.01052	0.01315	0.01578	0.0184	0.02103	0.02366	0.02629	0.02892	0.03155	0.03418	0.03681	0.03944	0.042068
	P	0	282.712	496.143	625.556	693.874	727.316	743.081	750.38	753.732	755.265	755.965	756.285	756.43	756.497	756.527	756.541	756.5473
5.0	Y	0	0.00272	0.00544	0.00817	0.01089	0.01361	0.01633	0.01905	0.02177	0.0245	0.02722	0.02994	0.03266	0.03538	0.03811	0.04083	0.043549
	P	0	551.121	967.184	1219.46	1352.64	1417.83	1448.57	1462.8	1469.33	1472.32	1473.68	1474.31	1474.59	1474.72	1474.78	1474.81	1474.818
6.0	Y	0	0.00326	0.00652	0.00978	0.01305	0.01631	0.01957	0.02283	0.02609	0.02935	0.03261	0.03587	0.03914	0.0424	0.04566	0.04892	0.05218
	P	0	723.356	1269.45	1600.57	1775.37	1860.93	1901.27	1919.95	1928.52	1932.45	1934.24	1935.05	1935.43	1935.6	1935.67	1935.71	1935.726
7.0	Y	0	0.00312	0.00624	0.00936	0.01248	0.01559	0.01871	0.02183	0.02495	0.02807	0.03119	0.03431	0.03743	0.04055	0.04366	0.04678	0.049902
	P	0	736.497	1292.51	1629.64	1807.62	1894.74	1935.81	1954.83	1963.56	1967.55	1969.37	1970.21	1970.59	1970.76	1970.84	1970.87	1970.891
8.0	Y	0	0.00297	0.00594	0.0089	0.01187	0.01484	0.01781	0.02077	0.02374	0.02671	0.02968	0.03265	0.03561	0.03858	0.04155	0.04452	0.047484
	P	0	794.252	1393.86	1757.44	1949.37	2043.32	2087.61	2108.12	2117.54	2121.84	2123.81	2124.71	2125.12	2125.3	2125.39	2125.43	2125.445
9.0	Y	0	0.00285	0.0057	0.00855	0.01139	0.01424	0.01709	0.01994	0.02279	0.02564	0.02848	0.03133	0.03418	0.03703	0.03988	0.04273	0.045575
	P	0	852.007	1495.22	1885.23	2091.12	2191.9	2239.41	2261.41	2271.51	2276.13	2278.24	2279.21	2279.65	2279.85	2279.94	2279.98	2279.998
10.0	Y	0	0.00275	0.0055	0.00826	0.01101	0.01376	0.01651	0.01926	0.02201	0.02477	0.02752	0.03027	0.03302	0.03577	0.03853	0.04128	0.04403
	P	0	909.761	1596.58	2013.03	2232.87	2340.49	2391.22	2414.71	2425.49	2430.43	2432.68	2433.71	2434.18	2434.39	2434.49	2434.53	2434.552
11.0	Y	0	0.00267	0.00534	0.00802	0.01069	0.01336	0.01603	0.0187	0.02138	0.02405	0.02672	0.02939	0.03207	0.03474	0.03741	0.04008	0.042753
	P	0	967.516	1697.93	2140.82	2374.62	2489.07	2543.02	2568	2579.47	2584.72	2587.11	2588.21	2588.71	2588.93	2589.04	2589.08	2589.106
12.0	Y	0	0.00261	0.00521	0.00782	0.01042	0.01303	0.01563	0.01824	0.02084	0.02345	0.02605	0.02866	0.03126	0.03387	0.03647	0.03908	0.041681
	P	0	1025.27	1799.29	2268.61	2516.37	2637.65	2694.82	2721.3	2733.45	2739.01	2741.55	2742.71	2743.24	2743.48	2743.59	2743.64	2743.66
13.0	Y	0	0.00246	0.00491	0.00737	0.00982	0.01228	0.01473	0.01719	0.01964	0.0221	0.02455	0.02701	0.02946	0.03192	0.03437	0.03683	0.039284
	P	0	1595.01	2799.14	3529.27	3914.7	4103.38	4192.32	4233.5	4252.41	4261.06	4265.01	4266.81	4267.64	4268.01	4268.18	4268.26	4268.294
14.0	Y	0	0.00261	0.00522	0.00783	0.01045	0.01306	0.01567	0.01828	0.02089	0.0235	0.02612	0.02873	0.03134	0.03395	0.03656	0.03917	0.041786
	P	0	2525.64	4432.34	5588.47	6198.79	6497.55	6638.39	6703.6	6733.54	6747.24	6753.49	6756.34	6757.65	6758.24	6758.51	6758.63	6758.689
15.0	Y	0	0.00258	0.00516	0.00775	0.01033	0.01291	0.01549	0.01807	0.02065	0.02324	0.02582	0.0284	0.03098	0.03356	0.03615	0.03873	0.041309
	P	0	2668.97	4683.87	5905.61	6550.57	6866.28	7015.11	7084.02	7115.67	7130.14	7136.75	7139.76	7141.14	7141.77	7142.05	7142.18	7142.243
16.0	Y	0	0.00256	0.00511	0.00767	0.01022	0.01278	0.01533	0.01789	0.02044	0.023	0.02556	0.02811	0.03067	0.03322	0.03578	0.03833	0.040889
	P	0	2812.3	4935.41	6222.75	6902.35	7235.02	7391.84	7464.45	7497.79	7513.04	7520.01	7523.19	7524.63	7525.3	7525.6	7525.73	7525.797

17.0	Y	0	0.00253	0.00506	0.0076	0.01013	0.01266	0.01519	0.01773	0.02026	0.02279	0.02532	0.02786	0.03039	0.03292	0.03545	0.03799	0.040518
	P	0	2955.62	5186.94	6539.9	7254.13	7603.75	7768.57	7844.88	7879.92	7895.95	7903.27	7906.61	7908.13	7908.82	7909.14	7909.29	7909.351
18.0	Y	0	0.00251	0.00502	0.00753	0.01005	0.01256	0.01507	0.01758	0.02009	0.0226	0.02512	0.02763	0.03014	0.03265	0.03516	0.03767	0.040186
	P	0	3098.95	5438.48	6857.04	7605.91	7972.49	8145.29	8225.31	8262.05	8278.85	8286.53	8290.03	8291.62	8292.35	8292.68	8292.84	8292.905
19.0	Y	0	0.00249	0.00499	0.00748	0.00997	0.01247	0.01496	0.01745	0.01994	0.02244	0.02493	0.02742	0.02992	0.03241	0.0349	0.0374	0.039889
	P	0	3242.28	5690.01	7174.19	7957.69	8341.22	8522.02	8605.73	8644.17	8661.76	8669.79	8673.45	8675.12	8675.88	8676.23	8676.39	8676.459
20.0	Y	0	0.00248	0.00495	0.00743	0.00991	0.01238	0.01486	0.01733	0.01981	0.02229	0.02476	0.02724	0.02972	0.03219	0.03467	0.03714	0.03962
	P	0	3385.61	5941.55	7491.33	8309.47	8709.96	8898.75	8986.16	9026.3	9044.66	9053.05	9056.87	9058.61	9059.41	9059.77	9059.94	9060.014
21.0	Y	0	0.00246	0.00492	0.00738	0.00984	0.01231	0.01477	0.01723	0.01969	0.02215	0.02461	0.02707	0.02953	0.03199	0.03445	0.03692	0.039377
	P	0	3528.94	6193.08	7808.47	8661.25	9078.69	9275.47	9366.59	9408.43	9427.57	9436.3	9440.29	9442.11	9442.94	9443.32	9443.49	9443.568
22.0	Y	0	0.00245	0.00489	0.00734	0.00979	0.01224	0.01468	0.01713	0.01958	0.02202	0.02447	0.02692	0.02937	0.03181	0.03426	0.03671	0.039155
	P	0	3672.27	6444.61	8125.62	9013.03	9447.43	9652.2	9747.02	9790.56	9810.47	9819.56	9823.71	9825.6	9826.47	9826.86	9827.04	9827.122
23.0	Y	0	0.00227	0.00455	0.00682	0.00909	0.01136	0.01364	0.01591	0.01818	0.02046	0.02273	0.025	0.02727	0.02955	0.03182	0.03409	0.036366
	P	0	2728.41	4788.2	6037.14	6696.47	7019.21	7171.36	7241.8	7274.15	7288.95	7295.7	7298.79	7300.19	7300.83	7301.12	7301.26	7301.319
24.0	Y	0	0.00226	0.00452	0.00678	0.00904	0.0113	0.01357	0.01583	0.01809	0.02035	0.02261	0.02487	0.02713	0.02939	0.03165	0.03391	0.036173
	P	0	2039.21	3578.7	4512.16	5004.94	5246.16	5359.87	5412.52	5436.7	5447.76	5452.81	5455.11	5456.16	5456.64	5456.86	5456.96	5457.007
25.0	Y	0	0.00225	0.00449	0.00674	0.00898	0.01123	0.01347	0.01572	0.01796	0.02021	0.02245	0.0247	0.02694	0.02919	0.03143	0.03368	0.035921
	P	0	2107.64	3698.78	4663.56	5172.87	5422.19	5539.71	5594.13	5619.12	5630.55	5635.77	5638.15	5639.24	5639.73	5639.96	5640.06	5640.107

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Table E2: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for North Abutment - HP 360 x 108

Depth Below Abutment Wall (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.00184	0.00368	0.00552	0.00736	0.0092	0.01104	0.01288	0.01472	0.01657	0.01841	0.02025	0.02209	0.02393	0.02577	0.02761	0.02945
	P	0	21.9901	38.5913	48.6574	53.9714	56.5726	57.7988	58.3666	58.6273	58.7466	58.801	58.8259	58.8372	58.8424	58.8447	58.8458	58.8463
1.0	Y	0	0.00244	0.00488	0.00732	0.00976	0.0122	0.01464	0.01708	0.01951	0.02195	0.02439	0.02683	0.02927	0.03171	0.03415	0.03659	0.039029
	P	0	45.0767	79.1069	99.741	110.634	115.966	118.48	119.643	120.178	120.422	120.534	120.585	120.608	120.619	120.624	120.626	120.6267
2.0	Y	0	0.00345	0.00691	0.01036	0.01381	0.01726	0.02072	0.02417	0.02762	0.03108	0.03453	0.03798	0.04143	0.04489	0.04834	0.05179	0.055245
	P	0	106.343	186.626	235.305	261.003	273.583	279.513	282.258	283.519	284.096	284.359	284.479	284.534	284.559	284.57	284.576	284.578
3.0	Y	0	0.00448	0.00897	0.01345	0.01794	0.02242	0.0269	0.03139	0.03587	0.04035	0.04484	0.04932	0.05381	0.05829	0.06277	0.06726	0.071741
	P	0	193.335	339.291	427.791	474.511	497.381	508.162	513.154	515.446	516.494	516.973	517.191	517.291	517.336	517.357	517.367	517.3709
4.0	Y	0	0.00269	0.00538	0.00807	0.01077	0.01346	0.01615	0.01884	0.02153	0.02422	0.02691	0.02961	0.0323	0.03499	0.03768	0.04037	0.043062
	P	0	289.392	507.866	640.337	710.269	744.502	760.639	768.111	771.542	773.111	773.828	774.155	774.304	774.372	774.403	774.417	774.4236
5.0	Y	0	0.00277	0.00554	0.00831	0.01108	0.01385	0.01662	0.01939	0.02217	0.02494	0.02771	0.03048	0.03325	0.03602	0.03879	0.04156	0.044331
	P	0	561.022	984.561	1241.37	1376.94	1443.31	1474.59	1489.08	1495.73	1498.77	1500.16	1500.79	1501.08	1501.22	1501.28	1501.3	1501.315
6.0	Y	0	0.00365	0.0073	0.01095	0.0146	0.01825	0.0219	0.02555	0.0292	0.03285	0.0365	0.04015	0.0438	0.04745	0.0511	0.05475	0.058397
	P	0	809.538	1420.69	1791.26	1986.89	2082.65	2127.79	2148.69	2158.29	2162.68	2164.69	2165.6	2166.02	2166.21	2166.29	2166.33	2166.352
7.0	Y	0	0.00372	0.00745	0.01117	0.01489	0.01861	0.02234	0.02606	0.02978	0.0335	0.03723	0.04095	0.04467	0.04839	0.05212	0.05584	0.059561
	P	0	879.045	1542.67	1945.06	2157.48	2261.46	2310.48	2333.18	2343.6	2348.37	2350.54	2351.54	2351.99	2352.2	2352.29	2352.33	2352.353
8.0	Y	0	0.00354	0.00708	0.01063	0.01417	0.01771	0.02125	0.0248	0.02834	0.03188	0.03542	0.03896	0.04251	0.04605	0.04959	0.05313	0.056675
	P	0	947.978	1663.64	2097.59	2326.67	2438.8	2491.67	2516.14	2527.38	2532.52	2534.87	2535.94	2536.43	2536.65	2536.75	2536.8	2536.821
9.0	Y	0	0.0034	0.0068	0.0102	0.0136	0.017	0.0204	0.0238	0.0272	0.0306	0.034	0.0374	0.0408	0.0442	0.0476	0.051	0.054396
	P	0	1016.91	1784.62	2250.11	2495.85	2616.14	2672.85	2699.11	2711.16	2716.68	2719.2	2720.34	2720.87	2721.11	2721.22	2721.27	2721.288
10.0	Y	0	0.00328	0.00657	0.00985	0.01314	0.01642	0.01971	0.02299	0.02628	0.02956	0.03284	0.03613	0.03941	0.0427	0.04598	0.04927	0.052552
	P	0	1085.84	1905.59	2402.64	2665.04	2793.48	2854.03	2882.07	2894.94	2900.83	2903.52	2904.75	2905.31	2905.56	2905.68	2905.73	2905.756
11.0	Y	0	0.00319	0.00638	0.00957	0.01276	0.01595	0.01914	0.02232	0.02551	0.0287	0.03189	0.03508	0.03827	0.04146	0.04465	0.04784	0.051028
	P	0	1154.78	2026.56	2555.17	2834.23	2970.82	3035.22	3065.03	3078.72	3084.99	3087.85	3089.15	3089.75	3090.02	3090.14	3090.2	3090.223
12.0	Y	0	0.00311	0.00622	0.00933	0.01244	0.01555	0.01866	0.02176	0.02487	0.02798	0.03109	0.0342	0.03731	0.04042	0.04353	0.04664	0.049748
	P	0	1223.71	2147.54	2707.7	3003.41	3148.16	3216.4	3248	3262.51	3269.14	3272.17	3273.55	3274.18	3274.47	3274.6	3274.66	3274.69
13.0	Y	0	0.00293	0.00586	0.00879	0.01172	0.01465	0.01758	0.02051	0.02344	0.02637	0.0293	0.03224	0.03517	0.0381	0.04103	0.04396	0.046887
	P	0	1903.72	3340.91	4212.35	4672.39	4897.58	5003.74	5052.89	5075.46	5085.78	5090.5	5092.65	5093.63	5094.08	5094.28	5094.37	5094.416
14.0	Y	0	0.00312	0.00623	0.00935	0.01247	0.01559	0.0187	0.02182	0.02494	0.02805	0.03117	0.03429	0.03741	0.04052	0.04364	0.04676	0.049874
	P	0	3014.47	5290.21	6670.1	7398.56	7755.14	7923.23	8001.07	8036.81	8053.15	8060.62	8064.02	8065.58	8066.28	8066.61	8066.75	8066.822
15.0	Y	0	0.00308	0.00616	0.00924	0.01233	0.01541	0.01849	0.02157	0.02465	0.02773	0.03082	0.0339	0.03698	0.04006	0.04314	0.04622	0.049304
	P	0	3185.54	5590.43	7048.63	7818.42	8195.24	8372.88	8455.13	8492.89	8510.17	8518.06	8521.65	8523.3	8524.04	8524.39	8524.54	8524.612
16.0	Y	0	0.00305	0.0061	0.00915	0.0122	0.01525	0.0183	0.02135	0.0244	0.02745	0.0305	0.03355	0.0366	0.03965	0.0427	0.04575	0.048803
	P	0	3356.61	5890.65	7427.16	8238.29	8635.34	8822.52	8909.18	8948.98	8967.18	8975.49	8979.29	8981.02	8981.8	8982.16	8982.33	8982.403

17.0	Y	0	0.00302	0.00604	0.00907	0.01209	0.01511	0.01813	0.02116	0.02418	0.0272	0.03022	0.03325	0.03627	0.03929	0.04231	0.04534	0.04836
	P	0	3527.68	6190.87	7805.68	8658.16	9075.45	9272.16	9363.24	9405.07	9424.2	9432.93	9436.92	9438.74	9439.56	9439.94	9440.11	9440.193
18.0	Y	0	0.003	0.006	0.00899	0.01199	0.01499	0.01799	0.02098	0.02398	0.02698	0.02998	0.03298	0.03597	0.03897	0.04197	0.04497	0.047964
	P	0	3698.75	6491.09	8184.21	9078.02	9515.55	9721.8	9817.3	9861.15	9881.21	9890.37	9894.55	9896.46	9897.32	9897.72	9897.9	9897.984
19.0	Y	0	0.00298	0.00595	0.00893	0.0119	0.01488	0.01785	0.02083	0.0238	0.02678	0.02976	0.03273	0.03571	0.03868	0.04166	0.04463	0.047609
	P	0	3869.82	6791.3	8562.74	9497.89	9955.65	10171.4	10271.4	10317.2	10338.2	10347.8	10352.2	10354.2	10355.1	10355.5	10355.7	10355.77
20.0	Y	0	0.00296	0.00591	0.00887	0.01182	0.01478	0.01773	0.02069	0.02364	0.0266	0.02956	0.03251	0.03547	0.03842	0.04138	0.04433	0.047289
	P	0	4040.89	7091.52	8941.27	9917.76	10395.8	10621.1	10725.4	10773.3	10795.2	10805.2	10809.8	10811.9	10812.8	10813.3	10813.5	10813.56
21.0	Y	0	0.00294	0.00587	0.00881	0.01175	0.01469	0.01762	0.02056	0.0235	0.02644	0.02937	0.03231	0.03525	0.03819	0.04112	0.04406	0.046998
	P	0	4211.96	7391.74	9319.79	10337.6	10835.9	11070.7	11179.5	11229.4	11252.3	11262.7	11267.4	11269.6	11270.6	11271.1	11271.3	11271.36
22.0	Y	0	0.00292	0.00584	0.00876	0.01168	0.0146	0.01752	0.02045	0.02337	0.02629	0.02921	0.03213	0.03505	0.03797	0.04089	0.04381	0.046733
	P	0	4383.03	7691.96	9698.32	10757.5	11276	11520.4	11633.5	11685.5	11709.3	11720.1	11725.1	11727.3	11728.4	11728.8	11729	11729.15
23.0	Y	0	0.00271	0.00543	0.00814	0.01085	0.01356	0.01628	0.01899	0.0217	0.02442	0.02713	0.02984	0.03255	0.03527	0.03798	0.04069	0.043405
	P	0	3256.49	5714.94	7205.62	7992.56	8377.77	8559.36	8643.44	8682.05	8699.71	8707.77	8711.45	8713.13	8713.9	8714.25	8714.4	8714.477
24.0	Y	0	0.0027	0.0054	0.0081	0.01079	0.01349	0.01619	0.01889	0.02159	0.02429	0.02698	0.02968	0.03238	0.03508	0.03778	0.04048	0.043175
	P	0	2433.9	4271.35	5385.48	5973.64	6261.55	6397.27	6460.11	6488.97	6502.17	6508.19	6510.94	6512.2	6512.77	6513.03	6513.15	6513.201
25.0	Y	0	0.00268	0.00536	0.00804	0.01072	0.0134	0.01608	0.01876	0.02144	0.02412	0.0268	0.02948	0.03216	0.03483	0.03751	0.04019	0.042873
	P	0	2515.57	4414.67	5566.18	6174.08	6471.64	6611.92	6676.87	6706.69	6720.33	6726.56	6729.41	6730.7	6731.29	6731.56	6731.69	6731.741

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Table E3: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for North Pier - HP 310 x 110

Depth Below Pier Cap (m)	Curve Points																	
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.0012	0.00239	0.00359	0.00478	0.00598	0.00717	0.00837	0.00956	0.01076	0.01195	0.01315	0.01434	0.01554	0.01673	0.01793	0.019125
	P	0	75.2681	132.091	166.545	184.734	193.638	197.835	199.778	200.67	201.079	201.265	201.35	201.389	201.407	201.415	201.418	201.42
1.0	Y	0	0.00151	0.00303	0.00454	0.00606	0.00757	0.00909	0.0106	0.01212	0.01363	0.01515	0.01666	0.01818	0.01969	0.02121	0.02272	0.024239
	P	0	143.098	251.129	316.633	351.213	368.14	376.12	379.814	381.511	382.287	382.641	382.803	382.877	382.91	382.926	382.933	382.9358
2.0	Y	0	0.00184	0.00369	0.00553	0.00738	0.00922	0.01107	0.01291	0.01476	0.0166	0.01845	0.02029	0.02214	0.02398	0.02583	0.02767	0.029517
	P	0	232.337	407.738	514.092	570.236	597.72	610.675	616.674	619.429	620.689	621.264	621.526	621.646	621.701	621.726	621.737	621.7422
3.0	Y	0	0.00218	0.00436	0.00654	0.00871	0.01089	0.01307	0.01525	0.01743	0.01961	0.02179	0.02397	0.02614	0.02832	0.0305	0.03268	0.034859
	P	0	342.985	601.918	758.921	841.804	882.376	901.502	910.358	914.424	916.284	917.133	917.521	917.698	917.778	917.815	917.832	917.8393
4.0	Y	0	0.00213	0.00426	0.00639	0.00852	0.01065	0.01278	0.01491	0.01704	0.01917	0.0213	0.02343	0.02556	0.02768	0.02981	0.03194	0.034074
	P	0	402.309	706.028	890.187	987.406	1035	1057.43	1067.82	1072.59	1074.77	1075.76	1076.22	1076.43	1076.52	1076.56	1076.58	1076.592
5.0	Y	0	0.00209	0.00417	0.00626	0.00835	0.01044	0.01252	0.01461	0.0167	0.01879	0.02087	0.02296	0.02505	0.02714	0.02922	0.03131	0.033399
	P	0	460.064	807.384	1017.98	1129.16	1183.58	1209.23	1221.11	1226.57	1229.06	1230.2	1230.72	1230.96	1231.06	1231.11	1231.14	1231.146
6.0	Y	0	0.00198	0.00396	0.00594	0.00792	0.0099	0.01189	0.01387	0.01585	0.01783	0.01981	0.02179	0.02377	0.02575	0.02773	0.02971	0.031696
	P	0	762.608	1338.33	1687.42	1871.71	1961.92	2004.44	2024.13	2033.17	2037.31	2039.2	2040.06	2040.45	2040.63	2040.71	2040.75	2040.766
7.0	Y	0	0.00214	0.00427	0.00641	0.00855	0.01068	0.01282	0.01495	0.01709	0.01923	0.02136	0.0235	0.02564	0.02777	0.02991	0.03205	0.034182
	P	0	1282.36	2250.47	2837.48	3147.37	3299.06	3370.56	3403.67	3418.88	3425.83	3429.01	3430.46	3431.12	3431.42	3431.56	3431.62	3431.647
8.0	Y	0	0.00214	0.00428	0.00641	0.00855	0.01069	0.01283	0.01496	0.0171	0.01924	0.02138	0.02351	0.02565	0.02779	0.02993	0.03206	0.034203
	P	0	1425.69	2502	3154.62	3499.15	3667.79	3747.29	3784.1	3801.01	3808.74	3812.27	3813.88	3814.61	3814.95	3815.1	3815.17	3815.201
9.0	Y	0	0.00214	0.00428	0.00642	0.00855	0.01069	0.01283	0.01497	0.01711	0.01925	0.02139	0.02353	0.02566	0.0278	0.02994	0.03208	0.034219
	P	0	1569.02	2753.54	3471.77	3850.93	4036.53	4124.02	4164.53	4183.13	4191.64	4195.53	4197.3	4198.11	4198.48	4198.64	4198.72	4198.755
10.0	Y	0	0.00214	0.00428	0.00642	0.00856	0.0107	0.01284	0.01498	0.01712	0.01926	0.0214	0.02354	0.02567	0.02781	0.02995	0.03209	0.034233
	P	0	1712.35	3005.07	3788.91	4202.71	4405.26	4500.75	4544.96	4565.26	4574.55	4578.79	4580.72	4581.6	4582	4582.19	4582.27	4582.31
11.0	Y	0	0.00214	0.00428	0.00642	0.00856	0.0107	0.01284	0.01498	0.01712	0.01926	0.0214	0.02354	0.02568	0.02782	0.02996	0.0321	0.034245
	P	0	1855.68	3256.61	4106.06	4554.49	4773.99	4877.47	4925.39	4947.39	4957.45	4962.04	4964.14	4965.1	4965.53	4965.73	4965.82	4965.864
12.0	Y	0	0.00214	0.00428	0.00642	0.00856	0.0107	0.01285	0.01499	0.01713	0.01927	0.02141	0.02355	0.02569	0.02783	0.02997	0.03211	0.034255
	P	0	1999.01	3508.14	4423.2	4906.27	5142.73	5254.2	5305.81	5329.51	5340.35	5345.3	5347.56	5348.59	5349.06	5349.28	5349.37	5349.418
13.0	Y	0	0.00214	0.00428	0.00642	0.00857	0.01071	0.01285	0.01499	0.01713	0.01927	0.02141	0.02356	0.0257	0.02784	0.02998	0.03212	0.034263
	P	0	2142.34	3759.68	4740.34	5258.05	5511.46	5630.93	5686.24	5711.64	5723.26	5728.56	5730.98	5732.09	5732.59	5732.82	5732.92	5732.972
14.0	Y	0	0.00214	0.00428	0.00643	0.00857	0.01071	0.01285	0.01499	0.01714	0.01928	0.02142	0.02356	0.0257	0.02785	0.02999	0.03213	0.034271
	P	0	2285.67	4011.21	5057.49	5609.83	5880.2	6007.65	6066.67	6093.77	6106.16	6111.82	6114.4	6115.58	6116.12	6116.36	6116.48	6116.526
15.0	Y	0	0.00214	0.00428	0.00643	0.00857	0.01071	0.01285	0.015	0.01714	0.01928	0.02142	0.02357	0.02571	0.02785	0.02999	0.03214	0.034278
	P	0	2429	4262.74	5374.63	5961.61	6248.93	6384.38	6447.1	6475.89	6489.07	6495.08	6497.83	6499.08	6499.65	6499.91	6500.03	6500.08
16.0	Y	0	0.002	0.004	0.006	0.008	0.01	0.012	0.014	0.016	0.018	0.02	0.02201	0.02401	0.02601	0.02801	0.03001	0.032008
	P	0	1839.39	3228.01	4070	4514.49	4732.08	4834.65	4882.14	4903.95	4913.92	4918.47	4920.55	4921.5	4921.93	4922.13	4922.22	4922.261

17.0	Y	0	0.002	0.00399	0.00599	0.00799	0.00999	0.01198	0.01398	0.01598	0.01798	0.01997	0.02197	0.02397	0.02596	0.02796	0.02996	0.031956
	P	0	1397.05	2451.74	3091.25	3428.85	3594.11	3672.01	3708.09	3724.65	3732.22	3735.68	3737.26	3737.98	3738.31	3738.46	3738.53	3738.56
18.0	Y	0	0.00199	0.00398	0.00597	0.00796	0.00995	0.01194	0.01393	0.01592	0.01791	0.0199	0.02189	0.02388	0.02587	0.02786	0.02985	0.031845
	P	0	1465.48	2571.82	3242.65	3596.79	3770.14	3851.86	3889.69	3907.07	3915.02	3918.64	3920.3	3921.06	3921.4	3921.56	3921.63	3921.661

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Table E4: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for North Pier - HP 360x108

Depth Below Pier Cap (m)	Curve Points		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.00123	0.00246	0.00369	0.00493	0.00616	0.00739	0.00862	0.00985	0.01108	0.01231	0.01355	0.01478	0.01601	0.01724	0.01847	0.019704
	P	0	77.5469	136.09	171.588	190.327	199.5	203.824	205.826	206.746	207.166	207.358	207.446	207.486	207.504	207.513	207.516	207.518
1.0	Y	0	0.00155	0.00309	0.00464	0.00619	0.00774	0.00928	0.01083	0.01238	0.01393	0.01547	0.01702	0.01857	0.02011	0.02166	0.02321	0.024756
	P	0	146.145	256.476	323.375	358.692	375.979	384.129	387.902	389.635	390.427	390.789	390.954	391.03	391.064	391.08	391.087	391.09
2.0	Y	0	0.00188	0.00375	0.00563	0.0075	0.00938	0.01125	0.01313	0.015	0.01688	0.01875	0.02063	0.0225	0.02438	0.02625	0.02813	0.030001
	P	0	236.153	414.434	522.534	579.601	607.535	620.704	626.801	629.601	630.882	631.466	631.733	631.855	631.91	631.936	631.947	631.9526
3.0	Y	0	0.00221	0.00442	0.00662	0.00883	0.01104	0.01325	0.01545	0.01766	0.01987	0.02208	0.02429	0.02649	0.0287	0.03091	0.03312	0.035325
	P	0	347.569	609.962	769.064	853.055	894.169	913.55	922.524	926.645	928.53	929.39	929.783	929.962	930.044	930.081	930.098	930.1057
4.0	Y	0	0.00254	0.00508	0.00763	0.01017	0.01271	0.01525	0.01779	0.02033	0.02288	0.02542	0.02796	0.0305	0.03304	0.03558	0.03813	0.040669
	P	0	480.175	842.679	1062.48	1178.52	1235.32	1262.09	1274.49	1280.18	1282.79	1283.98	1284.52	1284.77	1284.88	1284.93	1284.95	1284.965
5.0	Y	0	0.00249	0.00498	0.00747	0.00997	0.01246	0.01495	0.01744	0.01993	0.02242	0.02491	0.02741	0.0299	0.03239	0.03488	0.03737	0.039863
	P	0	549.108	963.652	1215.01	1347.7	1412.66	1443.28	1457.45	1463.96	1466.94	1468.3	1468.92	1469.21	1469.33	1469.39	1469.42	1469.433
6.0	Y	0	0.00236	0.00473	0.00709	0.00946	0.01182	0.01419	0.01655	0.01892	0.02128	0.02364	0.02601	0.02837	0.03074	0.0331	0.03547	0.03783
	P	0	910.21	1597.36	2014.02	2233.97	2341.64	2392.4	2415.9	2426.69	2431.63	2433.88	2434.91	2435.38	2435.59	2435.69	2435.73	2435.753
7.0	Y	0	0.00255	0.0051	0.00765	0.0102	0.01275	0.0153	0.01785	0.0204	0.02295	0.0255	0.02805	0.0306	0.03315	0.0357	0.03825	0.040798
	P	0	1530.56	2686.04	3386.67	3756.53	3937.58	4022.93	4062.45	4080.6	4088.9	4092.69	4094.42	4095.2	4095.56	4095.73	4095.8	4095.837
8.0	Y	0	0.00255	0.0051	0.00765	0.01021	0.01276	0.01531	0.01786	0.02041	0.02296	0.02551	0.02807	0.03062	0.03317	0.03572	0.03827	0.040822
	P	0	1701.63	2986.26	3765.2	4176.4	4377.69	4472.57	4516.51	4536.68	4545.91	4550.13	4552.05	4552.92	4553.32	4553.51	4553.59	4553.627
9.0	Y	0	0.00255	0.00511	0.00766	0.01021	0.01276	0.01532	0.01787	0.02042	0.02297	0.02553	0.02808	0.03063	0.03318	0.03574	0.03829	0.040842
	P	0	1872.7	3286.48	4143.72	4596.27	4817.79	4922.22	4970.57	4992.77	5002.93	5007.56	5009.68	5010.64	5011.08	5011.28	5011.38	5011.418
10.0	Y	0	0.00255	0.00511	0.00766	0.01021	0.01277	0.01532	0.01788	0.02043	0.02298	0.02554	0.02809	0.03064	0.0332	0.03575	0.03831	0.040859
	P	0	2043.77	3586.7	4522.25	5016.13	5257.89	5371.86	5424.63	5448.86	5459.94	5465	5467.31	5468.36	5468.84	5469.06	5469.16	5469.208
11.0	Y	0	0.00255	0.00511	0.00766	0.01022	0.01277	0.01533	0.01788	0.02044	0.02299	0.02555	0.0281	0.03065	0.03321	0.03576	0.03832	0.040873
	P	0	2214.84	3886.92	4900.78	5436	5697.99	5821.5	5878.69	5904.94	5916.96	5922.44	5924.94	5926.08	5926.6	5926.84	5926.95	5926.999
12.0	Y	0	0.00256	0.00511	0.00767	0.01022	0.01278	0.01533	0.01789	0.02044	0.023	0.02555	0.02811	0.03066	0.03322	0.03577	0.03833	0.040885
	P	0	2385.91	4187.14	5279.3	5855.87	6138.1	6271.14	6332.74	6361.03	6373.97	6379.88	6382.57	6383.8	6384.36	6384.62	6384.74	6384.789
13.0	Y	0	0.00256	0.00511	0.00767	0.01022	0.01278	0.01534	0.01789	0.02045	0.023	0.02556	0.02812	0.03067	0.03323	0.03578	0.03834	0.040895
	P	0	2556.99	4487.36	5657.83	6275.73	6578.2	6720.78	6786.8	6817.12	6830.98	6837.32	6840.21	6841.52	6842.12	6842.4	6842.52	6842.579
14.0	Y	0	0.00256	0.00511	0.00767	0.01023	0.01278	0.01534	0.0179	0.02045	0.02301	0.02557	0.02812	0.03068	0.03323	0.03579	0.03835	0.040904
	P	0	2728.06	4787.57	6036.36	6695.6	7018.3	7170.43	7240.86	7273.21	7288	7294.75	7297.84	7299.24	7299.88	7300.18	7300.31	7300.37
15.0	Y	0	0.00256	0.00511	0.00767	0.01023	0.01279	0.01534	0.0179	0.02046	0.02301	0.02557	0.02813	0.03068	0.03324	0.0358	0.03836	0.040912
	P	0	2899.13	5087.79	6414.88	7115.46	7458.4	7620.07	7694.92	7729.29	7745.01	7752.19	7755.47	7756.96	7757.64	7757.95	7758.1	7758.16
16.0	Y	0	0.00239	0.00478	0.00716	0.00955	0.01194	0.01433	0.01671	0.0191	0.02149	0.02388	0.02626	0.02865	0.03104	0.03343	0.03582	0.038203
	P	0	2195.4	3852.79	4857.75	5388.27	5647.96	5770.38	5827.07	5853.1	5865	5870.44	5872.92	5874.05	5874.56	5874.8	5874.91	5874.956

17.0	Y	0	0.00238	0.00477	0.00715	0.00954	0.01192	0.0143	0.01669	0.01907	0.02145	0.02384	0.02622	0.02861	0.03099	0.03337	0.03576	0.038141
	P	0	1667.45	2926.27	3689.56	4092.5	4289.75	4382.73	4425.78	4445.55	4454.59	4458.72	4460.6	4461.46	4461.85	4462.03	4462.11	4462.152
18.0	Y	0	0.00238	0.00475	0.00713	0.0095	0.01188	0.01425	0.01663	0.019	0.02138	0.02376	0.02613	0.02851	0.03088	0.03326	0.03563	0.038008
	P	0	1749.12	3069.59	3870.26	4292.94	4499.84	4597.38	4642.54	4663.28	4672.76	4677.09	4679.07	4679.97	4680.38	4680.57	4680.65	4680.692

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Project No. 159000204
Roseville Road Underpass Replacement, Hwy 401, Township of North Dumfries, ON

Table E5: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for South Pier - HP 310 x 110

Depth Below Pier Cap (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.00121	0.00243	0.00364	0.00485	0.00606	0.00728	0.00849	0.0097	0.01091	0.01213	0.01334	0.01455	0.01576	0.01698	0.01819	0.0194
	P	0	76.3526	133.994	168.945	187.396	196.428	200.685	202.657	203.562	203.976	204.165	204.251	204.291	204.309	204.317	204.321	204.3223
1.0	Y	0	0.00155	0.0031	0.00464	0.00619	0.00774	0.00929	0.01083	0.01238	0.01393	0.01548	0.01702	0.01857	0.02012	0.02167	0.02321	0.024762
	P	0	146.182	256.541	323.457	358.782	376.074	384.226	388	389.733	390.526	390.888	391.053	391.128	391.163	391.178	391.185	391.1887
2.0	Y	0	0.00189	0.00378	0.00568	0.00757	0.00946	0.01135	0.01325	0.01514	0.01703	0.01892	0.02082	0.02271	0.0246	0.02649	0.02839	0.030279
	P	0	238.336	418.265	527.364	584.958	613.151	626.441	632.595	635.421	636.713	637.304	637.573	637.696	637.752	637.777	637.789	637.7941
3.0	Y	0	0.0022	0.0045	0.00672	0.00896	0.01121	0.01345	0.01569	0.01793	0.02017	0.02241	0.02465	0.02689	0.02913	0.03138	0.03362	0.035858
	P	0	352.813	619.165	780.667	865.925	907.659	927.333	936.442	940.625	942.539	943.412	943.811	943.993	944.075	944.113	944.131	944.1384
4.0	Y	0	0.00219	0.00439	0.00658	0.00878	0.01097	0.01317	0.01536	0.01756	0.01975	0.02195	0.02414	0.02634	0.02853	0.03073	0.03292	0.035119
	P	0	414.65	727.685	917.494	1017.69	1066.74	1089.87	1100.57	1105.49	1107.74	1108.76	1109.23	1109.45	1109.54	1109.59	1109.61	1109.617
5.0	Y	0	0.00215	0.00431	0.00646	0.00862	0.01077	0.01293	0.01508	0.01724	0.01939	0.02155	0.0237	0.02586	0.02801	0.03016	0.03232	0.034474
	P	0	474.873	833.373	1050.75	1165.5	1221.68	1248.16	1260.42	1266.05	1268.62	1269.8	1270.33	1270.58	1270.69	1270.74	1270.76	1270.775
6.0	Y	0	0.00212	0.00425	0.00637	0.0085	0.01062	0.01275	0.01487	0.017	0.01912	0.02124	0.02337	0.02549	0.02762	0.02974	0.03187	0.03399
	P	0	535.096	939.061	1184	1313.31	1376.61	1406.45	1420.26	1426.61	1429.51	1430.83	1431.44	1431.71	1431.84	1431.9	1431.92	1431.934
7.0	Y	0	0.00202	0.00405	0.00607	0.0081	0.01012	0.01215	0.01417	0.0162	0.01822	0.02024	0.02227	0.02429	0.02632	0.02834	0.03037	0.032391
	P	0	876.745	1538.64	1939.97	2151.84	2255.55	2304.44	2327.08	2337.47	2342.22	2344.4	2345.39	2345.84	2346.04	2346.14	2346.18	2346.2
8.0	Y	0	0.00219	0.00437	0.00656	0.00875	0.01093	0.01312	0.01531	0.01749	0.01968	0.02187	0.02405	0.02624	0.02842	0.03061	0.0328	0.034984
	P	0	1458.27	2559.17	3226.7	3579.1	3751.59	3832.91	3870.56	3887.85	3895.76	3899.37	3901.02	3901.77	3902.11	3902.27	3902.34	3902.373
9.0	Y	0	0.00219	0.00438	0.00657	0.00876	0.01095	0.01314	0.01533	0.01752	0.01971	0.0219	0.0241	0.02629	0.02848	0.03067	0.03286	0.035048
	P	0	1607.03	2820.23	3555.86	3944.2	4134.3	4223.91	4265.4	4284.45	4293.17	4297.15	4298.96	4299.79	4300.17	4300.34	4300.42	4300.455
10.0	Y	0	0.00219	0.00439	0.00658	0.00878	0.01097	0.01316	0.01536	0.01755	0.01974	0.02194	0.02413	0.02633	0.02852	0.03071	0.03291	0.035101
	P	0	1755.78	3081.3	3885.02	4309.31	4517	4614.91	4660.24	4681.06	4690.58	4694.92	4696.91	4697.81	4698.23	4698.41	4698.5	4698.538
11.0	Y	0	0.0022	0.00439	0.00659	0.00879	0.01098	0.01318	0.01538	0.01757	0.01977	0.02197	0.02416	0.02636	0.02856	0.03075	0.03295	0.035146
	P	0	1904.54	3342.36	4214.17	4674.41	4899.7	5005.9	5055.08	5077.66	5087.98	5092.7	5094.85	5095.83	5096.28	5096.49	5096.58	5096.621
12.0	Y	0	0.0022	0.0044	0.0066	0.0088	0.011	0.01319	0.01539	0.01759	0.01979	0.02199	0.02419	0.02639	0.02859	0.03079	0.03299	0.035185
	P	0	2053.3	3603.42	4543.33	5039.52	5282.4	5396.9	5449.91	5474.26	5485.39	5490.48	5492.8	5493.86	5494.34	5494.56	5494.66	5494.704
13.0	Y	0	0.0022	0.0044	0.0066	0.0088	0.01101	0.01321	0.01541	0.01761	0.01981	0.02201	0.02421	0.02641	0.02862	0.03082	0.03302	0.035218
	P	0	2202.06	3864.48	4872.49	5404.62	5665.1	5787.9	5844.75	5870.86	5882.8	5888.25	5890.74	5891.88	5892.39	5892.63	5892.74	5892.786
14.0	Y	0	0.0022	0.00441	0.00661	0.00881	0.01101	0.01322	0.01542	0.01762	0.01983	0.02203	0.02423	0.02644	0.02864	0.03084	0.03304	0.035248
	P	0	2350.82	4125.54	5201.65	5769.73	6047.8	6178.89	6239.59	6267.46	6280.21	6286.03	6288.69	6289.9	6290.45	6290.7	6290.82	6290.869
15.0	Y	0	0.0022	0.00441	0.00661	0.00882	0.01102	0.01323	0.01543	0.01764	0.01984	0.02205	0.02425	0.02646	0.02866	0.03086	0.03307	0.035274
	P	0	2499.58	4386.61	5530.8	6134.83	6430.51	6569.89	6634.43	6664.06	6677.62	6683.81	6686.63	6687.92	6688.51	6688.77	6688.9	6688.952
16.0	Y	0	0.00221	0.00441	0.00662	0.00882	0.01103	0.01324	0.01544	0.01765	0.01985	0.02206	0.02427	0.02647	0.02868	0.03088	0.03309	0.035297
	P	0	2648.34	4647.67	5859.96	6499.94	6813.21	6960.89	7029.27	7060.66	7075.03	7081.58	7084.58	7085.94	7086.56	7086.85	7086.98	7087.034

17.0	Y	0	0.00206	0.00411	0.00617	0.00823	0.01029	0.01234	0.0144	0.01646	0.01852	0.02057	0.02263	0.02469	0.02675	0.0288	0.03086	0.032918
	P	0	1439.12	2525.56	3184.32	3532.09	3702.32	3782.57	3819.73	3836.79	3844.6	3848.16	3849.79	3850.53	3850.87	3851.02	3851.09	3851.122
18.0	Y	0	0.00204	0.00409	0.00613	0.00817	0.01022	0.01226	0.01431	0.01635	0.01839	0.02044	0.02248	0.02452	0.02657	0.02861	0.03065	0.032698
	P	0	1504.73	2640.72	3329.52	3693.14	3871.14	3955.04	3993.9	4011.74	4019.9	4023.62	4025.32	4026.1	4026.45	4026.61	4026.69	4026.719

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Table E6: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for South Pier - HP 360 x 108

Depth Below Pier Cap (m)	Curve Points																	
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.00125	0.0025	0.00375	0.005	0.00625	0.0075	0.00874	0.00999	0.01124	0.01249	0.01374	0.01499	0.01624	0.01749	0.01874	0.019987
	P	0	78.6643	138.051	174.06	193.069	202.375	206.761	208.792	209.725	210.152	210.346	210.435	210.476	210.494	210.503	210.506	210.5082
1.0	Y	0	0.00158	0.00316	0.00474	0.00632	0.0079	0.00948	0.01106	0.01264	0.01423	0.01581	0.01739	0.01897	0.02055	0.02213	0.02371	0.025289
	P	0	149.295	262.004	330.345	366.422	384.082	392.407	396.262	398.032	398.842	399.211	399.38	399.457	399.492	399.508	399.515	399.5187
2.0	Y	0	0.00192	0.00385	0.00577	0.00769	0.00962	0.01154	0.01346	0.01539	0.01731	0.01924	0.02116	0.02308	0.02501	0.02693	0.02885	0.030776
	P	0	242.25	425.133	536.025	594.565	623.22	636.729	642.984	645.856	647.17	647.769	648.043	648.168	648.225	648.251	648.263	648.268
3.0	Y	0	0.0023	0.0045	0.00681	0.00908	0.01136	0.01363	0.0159	0.01817	0.02044	0.02271	0.02498	0.02725	0.02952	0.03179	0.03407	0.036337
	P	0	357.528	627.44	791.1	877.497	919.79	939.726	948.957	953.196	955.135	956.02	956.424	956.609	956.693	956.731	956.748	956.7563
4.0	Y	0	0.00262	0.00524	0.00786	0.01048	0.0131	0.01572	0.01834	0.02096	0.02358	0.0262	0.02882	0.03144	0.03406	0.03668	0.0393	0.041916
	P	0	494.905	868.528	1095.07	1214.67	1273.21	1300.81	1313.59	1319.45	1322.14	1323.36	1323.92	1324.18	1324.29	1324.35	1324.37	1324.381
5.0	Y	0	0.00257	0.00514	0.00771	0.01029	0.01286	0.01543	0.018	0.02057	0.02314	0.02572	0.02829	0.03086	0.03343	0.036	0.03857	0.041146
	P	0	566.784	994.671	1254.12	1391.08	1458.13	1489.73	1504.37	1511.09	1514.16	1515.57	1516.21	1516.5	1516.63	1516.69	1516.72	1516.732
6.0	Y	0	0.00254	0.00507	0.00761	0.01014	0.01268	0.01521	0.01775	0.02028	0.02282	0.02536	0.02789	0.03043	0.03296	0.0355	0.03803	0.040569
	P	0	638.663	1120.81	1413.17	1567.5	1643.05	1678.66	1695.15	1702.72	1706.19	1707.77	1708.49	1708.82	1708.97	1709.04	1709.07	1709.083
7.0	Y	0	0.00242	0.00483	0.00725	0.00966	0.01208	0.0145	0.01691	0.01933	0.02175	0.02416	0.02658	0.02899	0.03141	0.03383	0.03624	0.03866
	P	0	1046.44	1836.44	2315.45	2568.32	2692.11	2750.46	2777.48	2789.88	2795.56	2798.15	2799.33	2799.87	2800.12	2800.23	2800.28	2800.303
8.0	Y	0	0.00261	0.00522	0.00783	0.01044	0.01305	0.01566	0.01827	0.02088	0.02349	0.0261	0.02871	0.03132	0.03393	0.03654	0.03915	0.041755
	P	0	1740.51	3054.49	3851.23	4271.82	4477.71	4574.77	4619.7	4640.34	4649.78	4654.09	4656.05	4656.95	4657.36	4657.55	4657.63	4657.671
9.0	Y	0	0.00261	0.00523	0.00784	0.01046	0.01307	0.01569	0.0183	0.02092	0.02353	0.02614	0.02876	0.03137	0.03399	0.0366	0.03922	0.041831
	P	0	1918.06	3366.09	4244.09	4707.59	4934.48	5041.44	5090.96	5113.7	5124.1	5128.85	5131.02	5132.01	5132.46	5132.67	5132.76	5132.802
10.0	Y	0	0.00262	0.00524	0.00786	0.01047	0.01309	0.01571	0.01833	0.02095	0.02357	0.02618	0.0288	0.03142	0.03404	0.03666	0.03928	0.041895
	P	0	2095.61	3677.68	4636.96	5143.36	5391.26	5508.11	5562.22	5587.07	5598.43	5603.62	5605.99	5607.07	5607.56	5607.78	5607.89	5607.933
11.0	Y	0	0.00262	0.00524	0.00787	0.01049	0.01311	0.01573	0.01835	0.02097	0.0236	0.02622	0.02884	0.03146	0.03408	0.03671	0.03933	0.041949
	P	0	2273.16	3989.27	5029.82	5579.13	5848.03	5974.79	6033.48	6060.43	6072.76	6078.38	6080.95	6082.12	6082.66	6082.9	6083.01	6083.064
12.0	Y	0	0.00262	0.00525	0.00787	0.0105	0.01312	0.01575	0.01837	0.021	0.02362	0.02625	0.02887	0.0315	0.03412	0.03675	0.03937	0.041995
	P	0	2450.71	4300.86	5422.69	6014.91	6304.8	6441.46	6504.74	6533.79	6547.08	6553.15	6555.92	6557.18	6557.76	6558.02	6558.14	6558.195
13.0	Y	0	0.00263	0.00525	0.00788	0.01051	0.01314	0.01576	0.01839	0.02102	0.02364	0.02627	0.0289	0.03153	0.03415	0.03678	0.03941	0.042035
	P	0	2628.26	4612.45	5815.55	6450.68	6761.57	6908.13	6975.99	7007.15	7021.41	7027.92	7030.89	7032.24	7032.86	7033.14	7033.27	7033.326
14.0	Y	0	0.00263	0.00526	0.00789	0.01052	0.01315	0.01578	0.01841	0.02103	0.02366	0.02629	0.02892	0.03155	0.03418	0.03681	0.03944	0.04207
	P	0	2805.82	4924.04	6208.42	6886.45	7218.35	7374.81	7447.25	7480.52	7495.73	7502.68	7505.85	7507.3	7507.96	7508.26	7508.39	7508.456
15.0	Y	0	0.00263	0.00526	0.00789	0.01053	0.01316	0.01579	0.01842	0.02105	0.02368	0.02631	0.02894	0.03158	0.03421	0.03684	0.03947	0.042101
	P	0	2983.37	5235.63	6601.28	7322.22	7675.12	7841.48	7918.51	7953.88	7970.06	7977.45	7980.82	7982.35	7983.06	7983.38	7983.52	7983.587
16.0	Y	0	0.00263	0.00527	0.0079	0.01053	0.01317	0.0158	0.01843	0.02106	0.0237	0.02633	0.02896	0.0316	0.03423	0.03686	0.0395	0.042128
	P	0	3160.92	5547.22	6994.15	7757.99	8131.89	8308.16	8389.77	8427.24	8444.39	8452.21	8455.78	8457.41	8458.15	8458.49	8458.65	8458.718

17.0	Y	0	0.00246	0.00491	0.00737	0.00982	0.01228	0.01473	0.01719	0.01964	0.0221	0.02456	0.02701	0.02947	0.03192	0.03438	0.03683	0.039289
	P	0	1717.65	3014.38	3800.65	4215.72	4418.9	4514.68	4559.03	4579.4	4588.71	4592.96	4594.91	4595.79	4596.19	4596.38	4596.46	4596.5
18.0	Y	0	0.00244	0.00488	0.00732	0.00976	0.0122	0.01464	0.01707	0.01951	0.02195	0.02439	0.02683	0.02927	0.03171	0.03415	0.03659	0.039027
	P	0	1795.97	3151.82	3973.94	4407.94	4620.39	4720.54	4766.91	4788.2	4797.94	4802.39	4804.42	4805.34	4805.76	4805.96	4806.04	4806.083

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Table E7: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for South Abutment - HP 310 x 110

Depth Below Abutment Wall (m)	Curve Points																	
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.001474061	0.002948122	0.00442	0.0059	0.00737	0.00884	0.01032	0.01179	0.01327	0.01474	0.01621	0.01769	0.01916	0.02064	0.02211	0.023585
	P	0	17.610989	30.906224	38.9678	43.2235	45.3067	46.2887	46.7435	46.9522	47.0477	47.0914	47.1113	47.1203	47.1245	47.1264	47.1272	47.1276
1.0	Y	0	0.002358248	0.004716496	0.00707	0.00943	0.01179	0.01415	0.01651	0.01887	0.02122	0.02358	0.02594	0.0283	0.03066	0.03302	0.03537	0.037732
	P	0	43.578796	76.47816	96.4266	106.958	112.112	114.543	115.668	116.184	116.421	116.529	116.578	116.6	116.611	116.615	116.617	116.6183
2.0	Y	0	0.003374345	0.00674869	0.01012	0.0135	0.01687	0.02025	0.02362	0.02699	0.03037	0.03374	0.03712	0.04049	0.04387	0.04724	0.05062	0.05399
	P	0	103.92594	182.38377	229.957	255.07	267.364	273.159	275.842	277.075	277.638	277.895	278.013	278.066	278.091	278.102	278.107	278.1093
3.0	Y	0	0.000001	0.000024	0.00012	0.00038	0.00092	0.0019	0.00353	0.00602	0.00964	0.0147	0.02152	0.03047	0.04197	0.05646	0.0744	0.093
	P	0	18.6	37.2	55.8	74.4	93	111.6	130.2	148.8	167.4	186	204.6	223.2	241.8	260.4	279	279
4.0	Y	0	1.46963E-06	2.35141E-05	0.00012	0.00038	0.00092	0.0019	0.00353	0.00602	0.00964	0.0147	0.02152	0.03047	0.04197	0.05646	0.0744	0.093
	P	0	18.6	37.2	55.8	74.4	93	111.6	130.2	148.8	167.4	186	204.6	223.2	241.8	260.4	279	279
5.0	Y	0	0.001896936	0.003793872	0.00569	0.00759	0.00948	0.01138	0.01328	0.01518	0.01707	0.01897	0.02087	0.02276	0.02466	0.02656	0.02845	0.030351
	P	0	328.49259	576.48468	726.854	806.235	845.093	863.41	871.892	875.786	877.568	878.381	878.752	878.922	878.999	879.034	879.05	879.0573
6.0	Y	0	0.002316493	0.004632986	0.00695	0.00927	0.01158	0.0139	0.01622	0.01853	0.02085	0.02316	0.02548	0.0278	0.03011	0.03243	0.03475	0.037064
	P	0	474.08324	831.9875	1049	1163.57	1219.64	1246.08	1258.32	1263.94	1266.51	1267.69	1268.22	1268.47	1268.58	1268.63	1268.65	1268.663
7.0	Y	0	0.002721246	0.005442493	0.00816	0.01088	0.01361	0.01633	0.01905	0.02177	0.02449	0.02721	0.02993	0.03265	0.03538	0.0381	0.04082	0.04354
	P	0	642.59799	1127.7207	1421.87	1577.16	1653.17	1689.01	1705.6	1713.22	1716.7	1718.29	1719.02	1719.35	1719.5	1719.57	1719.6	1719.614
8.0	Y	0	0.002805448	0.005610895	0.00842	0.01122	0.01403	0.01683	0.01964	0.02244	0.02525	0.02805	0.03086	0.03367	0.03647	0.03928	0.04208	0.044887
	P	0	750.81222	1317.6302	1661.32	1842.75	1931.57	1973.44	1992.82	2001.72	2005.79	2007.65	2008.5	2008.89	2009.07	2009.15	2009.18	2009.199
9.0	Y	0	0.002711477	0.005422953	0.00813	0.01085	0.01356	0.01627	0.01898	0.02169	0.0244	0.02711	0.02983	0.03254	0.03525	0.03796	0.04067	0.043384
	P	0	811.03516	1423.3178	1794.57	1990.56	2086.5	2131.73	2152.67	2162.28	2166.68	2168.69	2169.6	2170.02	2170.21	2170.3	2170.34	2170.358
10.0	Y	0	0.002635405	0.005270809	0.00791	0.01054	0.01318	0.01581	0.01845	0.02108	0.02372	0.02635	0.02899	0.03162	0.03426	0.0369	0.03953	0.042166
	P	0	871.25809	1529.0054	1927.83	2138.37	2241.43	2290.02	2312.51	2322.84	2327.57	2329.72	2330.71	2331.16	2331.36	2331.45	2331.5	2331.516
11.0	Y	0	0.002572563	0.005145125	0.00772	0.01029	0.01286	0.01544	0.01801	0.02058	0.02315	0.02573	0.0283	0.03087	0.03344	0.03602	0.03859	0.041161
	P	0	931.48103	1634.6931	2061.08	2286.18	2396.36	2448.31	2472.36	2483.4	2488.45	2490.76	2491.81	2492.29	2492.51	2492.61	2492.65	2492.675
12.0	Y	0	0.002519775	0.005039551	0.00756	0.01008	0.0126	0.01512	0.01764	0.02016	0.02268	0.0252	0.02772	0.03024	0.03276	0.03528	0.0378	0.040316
	P	0	991.70397	1740.3807	2194.34	2433.99	2551.3	2606.6	2632.2	2643.96	2649.34	2651.79	2652.91	2653.42	2653.66	2653.76	2653.81	2653.833
13.0	Y	0	0.00238475	0.004769501	0.00715	0.00954	0.01192	0.01431	0.01669	0.01908	0.02146	0.02385	0.02623	0.02862	0.031	0.03339	0.03577	0.038156
	P	0	1549.2074	2718.7657	3427.92	3802.29	3985.55	4071.94	4111.94	4130.31	4138.71	4142.54	4144.29	4145.09	4145.46	4145.62	4145.7	4145.732
14.0	Y	0	0.002546532	0.005093065	0.00764	0.01019	0.01273	0.01528	0.01783	0.02037	0.02292	0.02547	0.02801	0.03056	0.0331	0.03565	0.0382	0.040745
	P	0	2462.6585	4321.8171	5449.11	6044.22	6335.53	6472.85	6536.44	6565.64	6578.99	6585.09	6587.87	6589.14	6589.72	6589.98	6590.1	6590.157
15.0	Y	0	0.002526141	0.005052281	0.00758	0.0101	0.01263	0.01516	0.01768	0.02021	0.02274	0.02526	0.02779	0.03031	0.03284	0.03537	0.03789	0.040418
	P	0	2611.4169	4582.8793	5778.27	6409.33	6718.23	6863.85	6931.28	6962.24	6976.4	6982.87	6985.82	6987.16	6987.77	6988.05	6988.18	6988.24
16.0	Y	0	0.002508221	0.005016441	0.00752	0.01003	0.01254	0.01505	0.01756	0.02007	0.02257	0.02508	0.02759	0.0301	0.03261	0.03512	0.03762	0.040132
	P	0	2760.1754	4843.9414	6107.43	6774.43	7100.93	7254.85	7326.11	7358.84	7373.81	7380.64	7383.76	7385.18	7385.83	7386.13	7386.26	7386.323
17.0	Y	0	0.002492349	0.004984698	0.00748	0.00997	0.01246	0.01495	0.01745	0.01994	0.02243	0.02492	0.02742	0.02991	0.0324	0.03489	0.03739	0.039878
	P	0	2908.9339	5105.0036	6436.59	7139.54	7483.63	7645.84	7720.95	7755.44	7771.22	7778.42	7781.7	7783.2	7783.89	7784.2	7784.34	7784.405

18.0	Y	0	0.002478193	0.004956385	0.00743	0.00991	0.01239	0.01487	0.01735	0.01983	0.0223	0.02478	0.02726	0.02974	0.03222	0.03469	0.03717	0.039651
	P	0	3057.6923	5366.0657	6765.74	7504.64	7866.34	8036.84	8115.79	8152.04	8168.62	8176.19	8179.65	8181.22	8181.94	8182.27	8182.42	8182.488
19.0	Y	0	0.002465489	0.004930977	0.0074	0.00986	0.01233	0.01479	0.01726	0.01972	0.02219	0.02465	0.02712	0.02959	0.03205	0.03452	0.03698	0.039448
	P	0	3206.4508	5627.1278	7094.9	7869.75	8249.04	8427.84	8510.63	8548.64	8566.03	8573.97	8577.59	8579.25	8580	8580.34	8580.5	8580.571
20.0	Y	0	0.002454024	0.004908048	0.00736	0.00982	0.01227	0.01472	0.01718	0.01963	0.02209	0.02454	0.02699	0.02945	0.0319	0.03436	0.03681	0.039264
	P	0	3355.2093	5888.19	7424.06	8234.85	8631.74	8818.84	8905.47	8945.24	8963.44	8971.75	8975.54	8977.27	8978.06	8978.41	8978.58	8978.654
21.0	Y	0	0.002443626	0.004887251	0.00733	0.00977	0.01222	0.01466	0.01711	0.01955	0.02199	0.02444	0.02688	0.02932	0.03177	0.03421	0.03665	0.039098
	P	0	3503.9677	6149.2521	7753.21	8599.96	9014.44	9209.83	9300.3	9341.85	9360.85	9369.52	9373.48	9375.29	9376.11	9376.49	9376.66	9376.736
22.0	Y	0	0.002434152	0.004868303	0.0073	0.00974	0.01217	0.0146	0.01704	0.01947	0.02191	0.02434	0.02678	0.02921	0.03164	0.03408	0.03651	0.038946
	P	0	3652.7262	6410.3143	8082.37	8965.06	9397.14	9600.83	9695.14	9738.45	9758.26	9767.3	9771.43	9773.31	9774.17	9774.56	9774.74	9774.819
23.0	Y	0	0.002263045	0.004526089	0.00679	0.00905	0.01132	0.01358	0.01584	0.0181	0.02037	0.02263	0.02489	0.02716	0.02942	0.03168	0.03395	0.036209
	P	0	1957.8925	3435.9833	4332.22	4805.35	5036.95	5146.13	5196.68	5219.89	5230.51	5235.36	5237.57	5238.58	5239.04	5239.25	5239.34	5239.387
24.0	Y	0	0.002243425	0.00448685	0.00673	0.00897	0.01122	0.01346	0.0157	0.01795	0.02019	0.02243	0.02468	0.02692	0.02916	0.03141	0.03365	0.035895
	P	0	2023.5108	3551.1395	4477.41	4966.4	5205.76	5318.6	5370.84	5394.83	5405.81	5410.82	5413.1	5414.15	5414.62	5414.84	5414.94	5414.983

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

Table E8: Load Intensity p (kN/m) vs Lateral Deflection y (m) Data Points for South Abutment - HP 360 x 108

Depth Below Abutment Wall (m)		Curve Points																
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0.0	Y	0	0.001840599	0.003681199	0.00552	0.00736	0.0092	0.01104	0.01288	0.01472	0.01657	0.01841	0.02025	0.02209	0.02393	0.02577	0.02761	0.02945
	P	0	21.990118	38.591332	48.6574	53.9714	56.5726	57.7988	58.3666	58.6273	58.7466	58.801	58.8259	58.8372	58.8424	58.8447	58.8458	58.8463
1.0	Y	0	0.002439306	0.004878611	0.00732	0.00976	0.0122	0.01464	0.01708	0.01951	0.02195	0.02439	0.02683	0.02927	0.03171	0.03415	0.03659	0.039029
	P	0	45.076683	79.106861	99.741	110.634	115.966	118.48	119.643	120.178	120.422	120.534	120.585	120.608	120.619	120.624	120.626	120.6267
2.0	Y	0	0.00345283	0.006905661	0.01036	0.01381	0.01726	0.02072	0.02417	0.02762	0.03108	0.03453	0.03798	0.04143	0.04489	0.04834	0.05179	0.055245
	P	0	106.3432	186.62591	235.305	261.003	273.583	279.513	282.258	283.519	284.096	284.359	284.479	284.534	284.559	284.57	284.576	284.578
3.0	Y	0	0.000002	0.000028	0.00014	0.00045	0.0011	0.00227	0.00421	0.00718	0.01151	0.01754	0.02568	0.03637	0.0501	0.06738	0.0888	0.111
	P	0	20.115049	40.230099	60.3451	80.4602	100.575	120.69	140.805	160.92	181.035	201.15	221.266	241.381	261.496	281.611	301.726	301.7257
4.0	Y	0	1.75407E-06	2.80652E-05	0.00014	0.00045	0.0011	0.00227	0.00421	0.00718	0.01151	0.01754	0.02568	0.03637	0.0501	0.06738	0.0888	0.111
	P	0	22.2	44.4	66.6	88.8	111	133.2	155.4	177.6	199.8	222	244.2	266.4	288.6	310.8	333	333
5.0	Y	0	0.002017445	0.00403489	0.00605	0.00807	0.01009	0.0121	0.01412	0.01614	0.01816	0.02017	0.02219	0.02421	0.02623	0.02824	0.03026	0.032279
	P	0	349.36114	613.10774	773.03	857.454	898.78	918.261	927.282	931.423	933.318	934.183	934.578	934.758	934.84	934.877	934.894	934.9022
6.0	Y	0	0.002434882	0.004869764	0.0073	0.00974	0.01217	0.01461	0.01704	0.01948	0.02191	0.02435	0.02678	0.02922	0.03165	0.03409	0.03652	0.038958
	P	0	498.31222	874.50791	1102.61	1223.03	1281.98	1309.76	1322.63	1328.54	1331.24	1332.47	1333.04	1333.29	1333.41	1333.46	1333.49	1333.5
7.0	Y	0	0.002837586	0.005675172	0.00851	0.01135	0.01419	0.01703	0.01986	0.0227	0.02554	0.02838	0.03121	0.03405	0.03689	0.03973	0.04256	0.045401
	P	0	670.07056	1175.9334	1482.66	1644.59	1723.85	1761.21	1778.52	1786.46	1790.09	1791.75	1792.51	1792.85	1793.01	1793.08	1793.12	1793.132
8.0	Y	0	0.003230119	0.006460238	0.00969	0.01292	0.01615	0.01938	0.02261	0.02584	0.02907	0.0323	0.03553	0.03876	0.04199	0.04522	0.04845	0.051682
	P	0	864.46547	1517.0848	1912.8	2121.7	2223.96	2272.16	2294.48	2304.73	2309.42	2311.56	2312.54	2312.98	2313.18	2313.28	2313.32	2313.339
9.0	Y	0	0.003236278	0.006472557	0.00971	0.01295	0.01618	0.01942	0.02265	0.02589	0.02913	0.03236	0.0356	0.03884	0.04207	0.04531	0.04854	0.05178
	P	0	968.0097	1698.7987	2141.91	2375.83	2490.34	2544.32	2569.31	2580.79	2586.04	2588.43	2589.53	2590.03	2590.25	2590.36	2590.41	2590.427
10.0	Y	0	0.003145483	0.006290966	0.00944	0.01258	0.01573	0.01887	0.02202	0.02516	0.02831	0.03145	0.0346	0.03775	0.04089	0.04404	0.04718	0.050328
	P	0	1039.8887	1824.942	2300.96	2552.25	2675.26	2733.24	2760.09	2772.42	2778.06	2780.64	2781.81	2782.35	2782.59	2782.7	2782.75	2782.777
11.0	Y	0	0.003070478	0.006140956	0.00921	0.01228	0.01535	0.01842	0.02149	0.02456	0.02763	0.0307	0.03378	0.03685	0.03992	0.04299	0.04606	0.049128
	P	0	1111.7677	1951.0853	2460	2728.66	2860.18	2922.17	2950.88	2964.06	2970.09	2972.84	2974.1	2974.67	2974.93	2975.05	2975.1	2975.128
12.0	Y	0	0.003007474	0.006014947	0.00902	0.01203	0.01504	0.01804	0.02105	0.02406	0.02707	0.03007	0.03308	0.03609	0.0391	0.0421	0.04511	0.04812
	P	0	1183.6467	2077.2286	2619.05	2905.08	3045.09	3111.1	3141.66	3155.69	3162.11	3165.04	3166.38	3166.99	3167.27	3167.39	3167.45	3167.479
13.0	Y	0	0.00278269	0.005565381	0.00835	0.01113	0.01391	0.0167	0.01948	0.02226	0.02504	0.02783	0.03061	0.03339	0.03617	0.03896	0.04174	0.044523
	P	0	1807.7215	3172.4423	3999.94	4436.78	4650.61	4751.42	4798.09	4819.52	4829.33	4833.8	4835.85	4836.78	4837.2	4837.4	4837.48	4837.524
14.0	Y	0	0.002924322	0.005848644	0.00877	0.0117	0.01462	0.01755	0.02047	0.02339	0.02632	0.02924	0.03217	0.03509	0.03802	0.04094	0.04386	0.046789
	P	0	2828.0053	4962.9788	6257.52	6940.91	7275.43	7433.13	7506.15	7539.68	7555.01	7562.02	7565.21	7566.67	7567.33	7567.64	7567.77	7567.838
15.0	Y	0	0.003015071	0.006030142	0.00905	0.01206	0.01508	0.01809	0.02111	0.02412	0.02714	0.03015	0.03317	0.03618	0.0392	0.04221	0.04523	0.048241
	P	0	3116.8525	5469.8882	6896.65	7649.84	8018.53	8192.34	8272.81	8309.77	8326.67	8334.39	8337.91	8339.51	8340.25	8340.58	8340.73	8340.803
16.0	Y	0	0.002993683	0.005987366	0.00898	0.01197	0.01497	0.01796	0.02096	0.02395	0.02694	0.02994	0.03293	0.03592	0.03892	0.04191	0.04491	0.047899
	P	0	3294.4029	5781.4785	7289.51	8085.61	8475.31	8659.01	8744.07	8783.13	8801	8809.15	8812.87	8814.57	8815.35	8815.7	8815.86	8815.934
17.0	Y	0	0.002974739	0.005949478	0.00892	0.0119	0.01487	0.01785	0.02082	0.0238	0.02677	0.02975	0.03272	0.0357	0.03867	0.04165	0.04462	0.047596
	P	0	3471.9533	6093.0688	7682.38	8521.38	8932.08	9125.69	9215.33	9256.49	9275.32	9283.92	9287.84	9289.63	9290.45	9290.82	9290.99	9291.065

18.0	Y	0	0.002957843	0.005915686	0.00887	0.01183	0.01479	0.01775	0.0207	0.02366	0.02662	0.02958	0.03254	0.03549	0.03845	0.04141	0.04437	0.047325
	P	0	3649.5038	6404.6591	8075.24	8957.15	9388.85	9592.36	9686.59	9729.86	9749.65	9758.68	9762.81	9764.69	9765.55	9765.94	9766.11	9766.196
19.0	Y	0	0.00294268	0.00588536	0.00883	0.01177	0.01471	0.01766	0.0206	0.02354	0.02648	0.02943	0.03237	0.03531	0.03825	0.0412	0.04414	0.047083
	P	0	3827.0542	6716.2494	8468.11	9392.92	9845.63	10059	10157.8	10203.2	10224	10233.4	10237.8	10239.7	10240.6	10241.1	10241.2	10241.33
20.0	Y	0	0.002928996	0.005857992	0.00879	0.01172	0.01464	0.01757	0.0205	0.02343	0.02636	0.02929	0.03222	0.03515	0.03808	0.04101	0.04393	0.046864
	P	0	4004.6046	7027.8397	8860.97	9828.69	10302.4	10525.7	10629.1	10676.6	10698.3	10708.2	10712.7	10714.8	10715.7	10716.2	10716.4	10716.46
21.0	Y	0	0.002916585	0.005833171	0.00875	0.01167	0.01458	0.0175	0.02042	0.02333	0.02625	0.02917	0.03208	0.035	0.03792	0.04083	0.04375	0.046665
	P	0	4182.1551	7339.43	9253.84	10264.5	10759.2	10992.4	11100.4	11149.9	11172.6	11183	11187.7	11189.9	11190.8	11191.3	11191.5	11191.59
22.0	Y	0	0.002905278	0.005810555	0.00872	0.01162	0.01453	0.01743	0.02034	0.02324	0.02615	0.02905	0.03196	0.03486	0.03777	0.04067	0.04358	0.046484
	P	0	4359.7055	7651.0203	9646.7	10700.2	11215.9	11459.1	11571.6	11623.3	11647	11657.7	11662.7	11664.9	11665.9	11666.4	11666.6	11666.72
23.0	Y	0	0.002701053	0.005402106	0.0081	0.0108	0.01351	0.01621	0.01891	0.02161	0.02431	0.02701	0.02971	0.03241	0.03511	0.03781	0.04052	0.043217
	P	0	2336.8395	4101.0124	5170.71	5735.42	6011.84	6142.15	6202.49	6230.19	6242.87	6248.65	6251.29	6252.5	6253.04	6253.3	6253.41	6253.462
24.0	Y	0	0.002677636	0.005355273	0.00803	0.01071	0.01339	0.01607	0.01874	0.02142	0.0241	0.02678	0.02945	0.03213	0.03481	0.03749	0.04016	0.042842
	P	0	2415.1581	4238.4568	5344.01	5927.64	6213.33	6348	6410.36	6439	6452.09	6458.07	6460.8	6462.05	6462.61	6462.87	6462.99	6463.045

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The table presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m). The p-y points can be used for the structural design of the pile in response to lateral loads. Where spring spacings of less than 1.0 m are proposed, the tabulated “p” values are to be multiplied by the actual spring spacing; i.e. by 0.25 for 0.25 m spacings.

APPENDIX F

F.1 2015 NATIONAL BUILDING CODE SEISMIC HAZARD CALCULATION

2015 National Building Code Seismic Hazard Calculation

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

Site: 43.351N 80.416W **User File Reference:** Roseville Road Bridge at Hwy 2019-08-28 20:14 UT

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)	0.111	0.062	0.036	0.010
Sa (0.1)	0.145	0.085	0.052	0.015
Sa (0.2)	0.131	0.079	0.050	0.017
Sa (0.3)	0.105	0.065	0.043	0.015
Sa (0.5)	0.080	0.051	0.034	0.012
Sa (1.0)	0.046	0.029	0.019	0.006
Sa (2.0)	0.023	0.015	0.009	0.002
Sa (5.0)	0.006	0.003	0.002	0.000
Sa (10.0)	0.002	0.001	0.001	0.000
PGA (g)	0.080	0.047	0.029	0.008
PGV (m/s)	0.064	0.039	0.024	0.007

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

References

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

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

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

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

APPENDIX G

G.1 SAMPLE NSSPS

HIGH-STRAIN DYNAMIC TESTING, DEEP FOUNDATIONS – Item No.

Special Provision

Amendment to OPSS 903, April 2016

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International:**

D 4945-17 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the strain and acceleration response and integrity of deep foundations (herein referred to as piles) after applying an impact force and measuring the performance of the drive assembly system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.07 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Piling Contractor personnel; however, testing shall be performed under the direction of a Professional Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the analysis and test results shall be submitted to the Contract Administrator on the same day as the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

a) Pile ultimate resistance in axial compression, and pile integrity.

b) Calculated driving stresses.

c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator within 5 Calendar Days of the field testing. The final report and or supporting test forms shall be prepared in accordance with ASTM D4945 and shall include, but not be limited to, the following:

a) Results of pile ultimate resistance in axial compression (including a summary of the toe and shaft resistances), and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.

b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.

c) An appendix shall be included containing the following documents:

    i. Pile installation record

    ii. Reference subsurface information (borehole record)

    iii. Pile location drawing

    iv. Initial calibration check by the test computer unit

    v. Test set up geometry

The report shall be signed and sealed by two Professional Engineers of the testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

903.07 CONSTRUCTION

903.07.02.07 Monitoring Driven Piles

903.07.02.07.03 Driving to a Specified Ultimate Resistance

903.07.02.07.03.01 General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at the end of initial drive (EOID). If the specified ultimate resistance is not achieved, retap/restrike should be performed after sufficient time has passed to allow soil setup. The soil setup requirements are specified elsewhere in the Contract.

The results of the high-strain dynamic tests shall be submitted to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the specified ultimate resistance has been achieved.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing

903.07.02.07.04 .01 Wave Equation Analysis

Prior to mobilizing piling equipment to the site, a WEAP analysis must be performed by the Piling Contractor to demonstrate the potential for the proposed piling equipment to achieve the specified ultimate resistance as stipulated by the Contract.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure. The results of the analysis shall be reviewed by a Foundations Engineer retained by the Contract Administrator.

903.07.02.07.04 .02 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by a Professional Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and have a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test. After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945-17.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for approval.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group/foundation unit, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out as outlined in Clause **903.07.02.07.03.01** if the specified ultimate resistance is not achieved at EOID. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.10 BASIS OF PAYMENT

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

903.10.04 High-Strain Dynamic Testing, Deep Foundations - Item

Payment for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

Buried Soils Containing Organic Matter - Item No.

Special Provision

1.0 SCOPE

This Special Provision identifies requirements for the supply of construction equipment and implementation of procedures to address the presence of buried soil layers containing organic matter at the site that could impact the design of temporary falsework systems.

2.0 CONSTRUCTION

The boreholes advanced at the site identified the presence of buried layers of topsoil and soils containing organic matter. The application of loads at ground surface from falsework systems could result in compression/settlement of soils containing organic matter. This must be accounted for in the design of the falsework systems.

The Contractor is advised that appropriate equipment and construction procedures will be required to ascertain if soil containing organic matter is present within the influence zone of the foundations for temporary support/falsework systems used in the construction of the bridge.

The contractor shall implement measures and/or design falsework support systems to address the presence of organic materials and prevent unacceptable settlement of the temporary support systems.

3.0 BASIS OF PAYMENT

Payment at the Contract price for the appropriate tender items associated with temporary falsework support systems shall include full compensation for all labour, equipment and materials to complete the work.

v:\01216\active\other_pc_projects\165001107\05_report_deliv\deliverables\report\nssps and notice to contractors\165001107 nssp-buried soils containing organic matter.docx

<u>Presence of Existing Piles</u> - Item No.

Special Provision

1.0 SCOPE

This Special Provision identifies requirements for the survey of existing pile locations and orientations and supply of construction equipment and implementation of procedures to remove existing piles that conflict with new pile foundation locations.

2.0 CONSTRUCTION

H-Piles, including inclined/battered piles, supporting the existing bridge pier foundations are present in the area of the new pier foundations. The design drawings for the existing bridge identify the tip elevation of the existing H-Piles is approximately Elevation 295.6 m.

The Contract Documents specify the partial removal of the existing pier pile caps to facilitate the new construction works. The locations of the existing bridge pier pile caps that are specified to remain in place and the locations of the existing piles in the portions of the pile caps specified to be partially removed shall be surveyed immediately upon completion of concrete pier removal works to confirm that the remaining foundation elements and existing pile locations will not interfere with installation of the new piles for the replacement bridge. The Contractor shall provide the Contractor Administrator with the survey information immediately upon completion of the excavation and locating of the existing piles.

The Contractor shall monitor the conditions during pile driving. If the existing piles are encountered during pile driving, the Contract Administrator should be notified immediately. The Contractor shall make available suitable equipment to extract the new H-Piles by vibratory extraction methods and relocate them as identified by the Contract Adminstrator.

3.0 BASIS OF PAYMENT

Payment at the Contract price for the appropriate tender items associated with structure removals, pile installation and temporary roadway protection system installation shall include full compensation for all labour, equipment and materials to complete the work.

<u>Obstructions</u> - Item No.

Special Provision

1.0 SCOPE

This Special Provision identifies requirements for the supply of construction equipment and implementation of procedures to address obstructions, including cobbles and/or boulders and the existing concrete bridge foundations, present at the site that could impact excavations and/or the installation of deep foundations and/or shoring elements.

2.0 CONSTRUCTION

Cobbles and boulders were identified within the fill materials and native soil deposits at the site. Buried concrete structures associated with the foundations of the existing bridge (i.e. pile caps at pier locations) are also present at the site. Cobbles, boulders and concrete foundations may obstruct excavation activities and the installation of piles and temporary roadway protection systems.

The Contractor is advised that appropriate equipment and construction procedures will be required to penetrate through or remove obstructions, such as concrete, and cobbles and boulders, to permit excavations and installation of deep foundation elements and shoring elements.

The Contractor is also advised that the installation of sheet piles or driven soldier piles could be obstructed and the piles could be damaged as a result of encountering obstructions. Multiple attempts at driving sheet piles or soldier piles should be anticipated.

The removal of cobbles and boulders from excavations may lead to undermining of materials in the sidewalls of excavations. The contractor shall implement appropriate measures to prevent instability of any undermined materials.

3.0 BASIS OF PAYMENT

Payment at the Contract price for the appropriate tender items associated with excavations, pile installation and temporary roadway protection systems shall include full compensation for all labour, equipment and materials to complete the work.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN 0003 March 8, 2018

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work. The dewatering system shall be designed and operated to lower the groundwater level to at least 0.5 m below the founding level for the pier pile caps, to allow excavation and foundation construction in dry conditions.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 2 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517. The design Engineer and design-checking Engineer shall have a minimum of 5 years of experience in designing systems of similar nature and scope to the required work.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, utilities, and structures, within a distance of 100 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website https://www.ontario.ca/environment-and-energy/map-well-records or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete all work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

* Fill in the design storm return period according to MTO Drainage Design Standard TW-1.

** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

<u>CSP FOR INTEGRAL ABUTMENT</u> - Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the installation of the corrugated steel pipes (CSPs) at integral abutments.

2.0 REFERENCES

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

Ontario Provincial Standards Specifications, Material

OPSS 1801 Corrugated Steel Pipe Products

CSA Standards

G164-M92 (R2003) Hot Dip Galvanizing of Irregularly Shaped Articles

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Submission Requirements

The Contractor shall submit to the Contract Administrator 3 sets of Working Drawings, for information purposes only prior to making the submission. An Engineer's seal and signature shall be affixed on the Working Drawings verifying that the drawings are consistent with the Contract Drawings.

Where multi-discipline engineering work is depicted on the same Working Drawing and a single Engineer is unable to seal and sign the Working Drawing for all aspects of the work, the drawing shall be sealed and signed by as many additional Engineers as necessary.

The Contractor shall have a copy of the submitted Working Drawings on site at all times.

Working Drawings shall include the following as a minimum:

a) Layout and elevations of the CSPs;
b) Location of reference points, and location of the centroid of each pile with respect to the reference points;
c) Construction sequence and details;
d) Source of the sand fill, and description of placing method and equipment; and
e) Location and details of all temporary bracing and spacers for the piles and CSPs.

5.0 MATERIALS

5.01 Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CAN/CSA G164.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.

5.02 Sand Fill

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1.

Table 1
Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 mm	# 30	80 % to 100 %
420 mm	# 40	40 % to 80 %
250 mm	# 60	5 % to 25 %
150 mm	# 100	0 % to 6 %

The sand fill shall be uniformly graded, free-flowing and have a moisture content less than or equal to the optimum moisture content.

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION

7.01 General

The sequence of construction shall be in accordance with the Working Drawings and as follows, unless otherwise approved:

a) Auger down to underside of CSP.
b) Place CSP into hole. Backfill annular space around CSP.
c) Drive piles through CSP.
d) Place loose sand into the CSP.
e) Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeter of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	± 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation.	± 10 mm

The sand fill shall completely fill the volume between the CSP and the pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage or displace the CSP.

7.02 Management of Excess Materials

Management of excess materials shall be according to the Contract Documents.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

10.01 CSP for Integral Abutment – Item

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment, and Material to do the work.