

FINAL REPORT

Foundation Investigation and Design Site 16-318 Highway 416 and County Road 43 Ramp Terminal Intersection Improvements United Counties of Leeds and Grenville, Ontario *GWP 4129-18-00*

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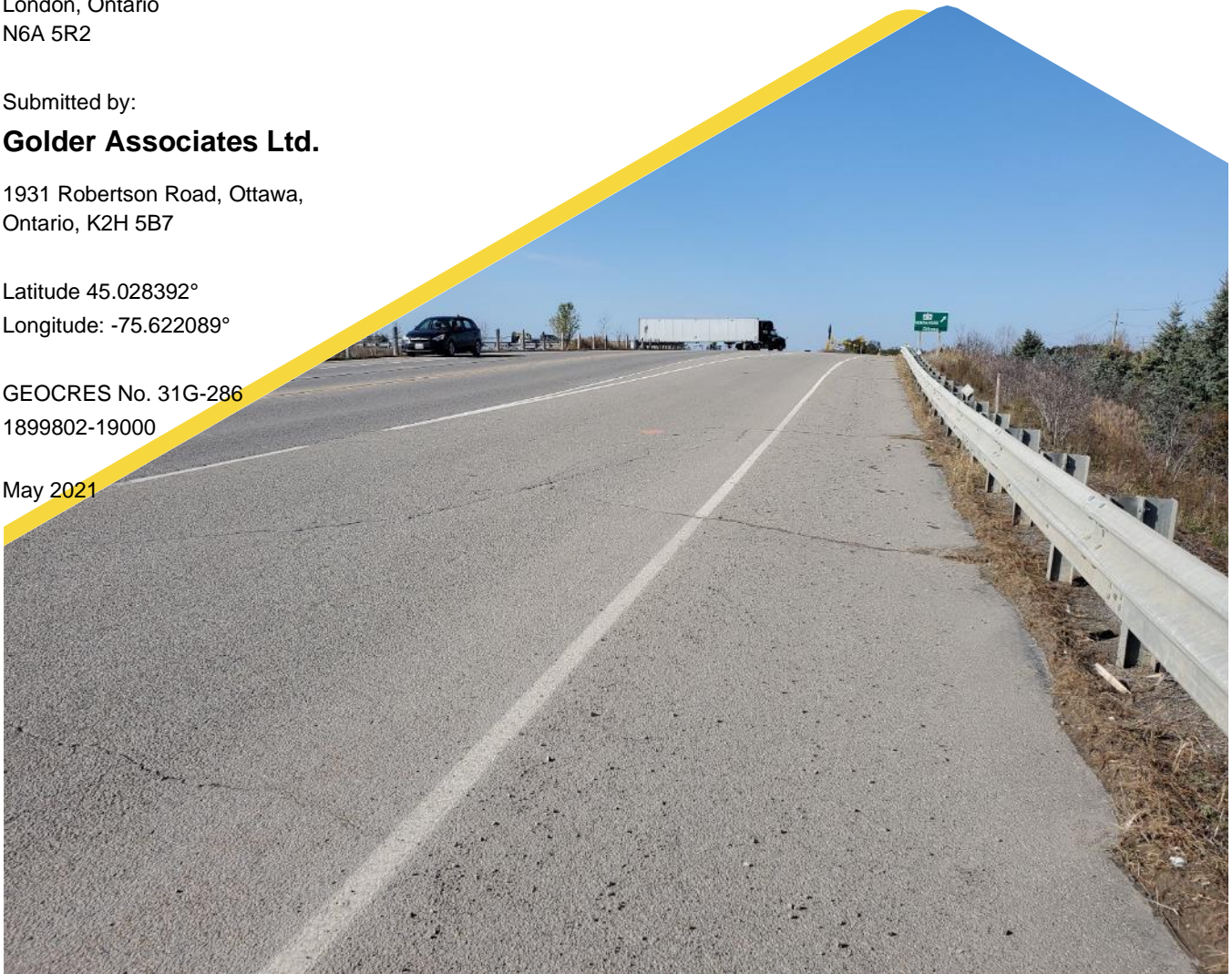
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PART A

Foundation Investigation
Site 16-318 Highway 416 and County Road 43
Ramp Terminal Intersection Improvements
United Counties of Leeds and Grenville, Ontario
GWP 4129-18-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Dillon Consulting Limited (Dillon) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the Highway 416 and County Road 43 Ramp Terminal Intersection Improvements as part of the Mega 10 Project (Purchase Order No. 4017-E-0019; GWP 4129-18-00).

This report presents the results of the foundation investigation carried out within the County Road (CR) 43 east embankment. The purpose of this foundation investigation was to assess the subsurface conditions in the existing east embankment on CR43, to evaluate causes of the settlement at the existing embankment, and to provide geotechnical input for selection of a technically preferred rehabilitation alternatives to address the existing deficiencies. The foundation investigation included drilling boreholes, installing a groundwater monitoring well, and subsequent laboratory testing on selected soil samples.

The terms of reference for the original scope of work are outlined in MTO's Work Item Order Form for Assignment 19, dated October 20, 2020. Golder's scope of work for foundation engineering services associated with the County Road 43 overpasses is contained in Appendix 2B: Work Item Quote Form of Dillon's Technical Proposal for this assignment.

The work was carried out in accordance with Golder's Project Specific Supplementary Quality Control Plan dated October 2020.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

The existing underpass (Site 16-318), which carries two lanes of CR43 over Highway 416, is located about 2 km east of the town of Kemptville within the United Counties of Leeds and Grenville, Ontario. The location of the underpass structure is shown on the Key Plan on Drawings 1 and 2. Site photographs showing the general conditions at the site are presented in Appendix D.

CR43 within the area under consideration consists of two 3.75 m wide through lanes, one each in the eastbound and westbound directions, a Highway 416 northbound onramp lane. At this location Highway 416 has a four lane cross-section with two northbound and two southbound through lanes, plus speed change lanes at the interchange ramps.

The current foundation investigation was specified by MTO to address an area of pavement distress at the existing east embankment which seems to be the result of embankment movement. The Preliminary Design Report by Dillon indicated that the pavement distress at the east embankment had been observed between approximately CR43 Stations 10+300 and 10+400. A pavement investigation was not completed as part of the preliminary design and at the time of preparing this report, maintenance history for the existing east embankment was not available.

A previous investigation was carried out for the design of the existing structure in 1991 (GEOCREC No. 31G-205). Further discussion of the subsurface conditions encountered during the previous investigation is provided in Section 3.2. Of note, at the boreholes advanced at the east embankment location, a soft to firm silty clay layer up to 6.9 m in thickness was encountered. That clay deposit was not encountered in the boreholes advanced at the proposed abutment or pier locations. Groundwater was observed to be at the ground surface at the borehole locations.

The interchange was part of the original construction of Highway 416, completed under Contract 97-17. Existing construction drawings indicate that the existing two span structure was to be supported on spread footings founded within the glacial till or on compacted granular pads placed on the glacial till.

The base plan mapping provided by Dillon for this project and the ground surface elevations at the borehole locations surveyed during the current field investigation indicate that the top of roadway elevation of CR43 in the vicinity of the distressed area ranges from Elevation 96.1 m at Station 10+325 to Elevation 94.9 m at Station 10+375. The existing east approach embankment is approximately 5 to 7 m in height above the natural ground level and the side slopes are generally oriented at about 2 horizontal to 1 vertical (2H:1V).

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, this section of Highway 416 lies within the major physiographic region known as the Edwardsburg Sand Plain.

Edwardsburg Sand Plain is characterized by sands of glaciofluvial origin that were likely well spread by wave action in the late stages of the Champlain Sea. The plain is nearly level or only slightly undulating. These deposits overlie relatively thin, commonly reworked glacial till, that in turn overlie bedrock².

Surficial geology mapping indicates the area comprises of glaciofluvial nearshore sediments of fine to medium grained sand.

This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield. Regional bedrock mapping indicates that the bedrock at this site is primarily dolomite of the Oxford Formation³.

Figures 3 through 5 provide the published geology information for the site and surrounding area.

The site falls within the Western Québec (WQ) seismic zone according to the Geological Survey of Canada. The WQ zone constitutes a large area which encompasses the urban areas of Montreal, Ottawa-Hull and Cornwall. Within the WQ zone recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. The two major earthquakes that have recently occurred in the WQ zone are the 1935 Témiscaming event, which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2, and the 1944 Cornwall-Massena event, which had a magnitude of 5.6.

¹ Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

³ Williams, D.A. Rae, A.M., and Wolf, R.R. 1984: *Paleozoic Geology of the Ottawa Area*, Southern Ontario, Ontario Geological Survey, Map P.2716. Geological Series-Preliminary Map, scale 1:50,000. Geology 1982.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

The field work for the current investigation was carried out between November 24 and December 10, 2020 and included advancing six boreholes, numbered 20-01 to 20-06. Three of the boreholes were located within the highway platform, while two were located along the north and one along the south embankment toes of slopes.

The boreholes were advanced using a combination of truck mounted (Boreholes 20-01, 20-02 and 20-5), portable rotary drill (toe of slope Boreholes 20-03 and 20-04) and track mounted (toe of slope Borehole 20-06) drilling equipment. All drilling equipment was supplied and operated by CCC Geotechnical & Environmental Drilling Limited of Ottawa, Ontario.

Traffic control required to close the driving lanes and shoulders of CR43 while carrying out the field operations was provided by Beacon Lite Limited of Ottawa, Ontario.

Soil samples were obtained using a 50 mm outer diameter split-spoon sampler in general accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). In Boreholes 20-01, 20-02, 20-05 and 20-06 soil samples were obtained at vertical sampling intervals of about 0.76 m. Soil samples from the boreholes advanced with portable drilling equipment, were obtained in continuous vertical increments of about 0.6 m using a full-weight hammer as per ASTM. Relatively undisturbed samples of the clay were also retrieved throughout the cohesive deposit using a fixed piston sampler and thin-walled Shelby tubes.

In-situ vane testing was carried out within the cohesive deposits, using an MTO N-size vane, with the reaction (torque) measured by a pair of calibrated scales, to measure the undrained shear strength of the cohesive soils. After determining the undrained shear strength, remoulded shear strengths were also measured at selected intervals.

A monitoring well was installed in Borehole 20-06, to observe the stabilised groundwater level at the site. The monitoring well consists of a 32 mm outside diameter PVC tubing with a 1.5 m long slotted tip. The groundwater level was measured in the well on January 20, 2021, and April 13, 2021.

The boreholes were backfilled with bentonite mixed with soil cuttings within the overburden. The boreholes were then capped with either asphaltic concrete cold patch or granular material, depending on the surrounding surface cover. The boreholes were backfilled in general accordance with the intent of O.Reg 903, as amended. The site conditions were restored following completion of the field work. The monitoring well has not been decommissioned so to be available for future monitoring of the groundwater levels during the design phase.

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, logged the boreholes and examined and cared for the samples. The soil samples were identified in the field, placed in labelled containers, and transported to Golder's laboratory in Ottawa for further examination and testing. Index and classification tests consisting of water content determinations, grain size distribution analyses, and Atterberg Limits testing were carried out on selected soil samples. Incremental loaded oedometer consolidations tests were carried out on three Shelby tube samples; two from the clay deposit at boreholes located within the existing embankment and one from a toe of slope borehole. The laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate at Golder's Ottawa laboratory.

The borehole locations and elevations were surveyed by Golder using a Trimble R10 GPS unit referenced to the NAD83 CSRS CBNv6-2010.0 MTM Zone 9 geodetic datum. The borehole locations, including northing and easting coordinates, CR43 Stationing, ground surface elevations, and drilled depths are summarized in Table 1.

Table 1: Borehole Location Summary

Borehole	Borehole Location	Station	NAD83 CSRS CBNv62010.0 MTM Zone 9		Ground Surface Elevation (m)	Drilled Length (m)
			Northing (m)	Easting (m)		
20-01	CR43 westbound lane	10+310	4987977.8	373996.3	96.1	14.0
20-02	CR43 westbound lane	10+375	4987967.1	374030.0	94.9	12.8
20-03	CR43 North toe of slope	10+310	4987993.6	374004.6	91.3	10.1
20-04	CR43 North toe of slope	10+375	4987978.4	374035.3	91.3	5.8
20-05	CR43 eastbound lane	10+325	4987965.5	373998.6	96.1	15.1
20-06	CR43 South toe of slope	10+325	4987944.0	373993.8	92.1	11.3

3.2 Previous Investigation (1991)

A previous investigation was carried out for the design of the existing structures in 1991. The subsurface information and results of the original investigation are contained in the report titled:

- Foundation Investigation Report for Highway 416 Underpass and Interchange at Highway 43, Kemptville, W.P. 372-89-06, Site 16-318, District 9, Kingston, dated December 1991 (GEOCRES No. 31G-205).

As part of the current assignment, previously collected subsurface information pertinent to the site was reviewed and compiled.

The boreholes from the previous investigation were advanced prior to construction of the bridge and the ground surface conditions shown may not be representative of the post-construction subsurface conditions, particularly with respect to the composition and thickness of overburden and fill. Also, the coordinates of the boreholes indicated in GEOCRES No. 31G-205 locate the proposed interchange alignment approximately 200 m south of the existing alignment.

A total of eight boreholes were advanced at the site as part of the original investigation along the then proposed CR43 bridge alignment over Highway 416. The subsurface soil, bedrock and groundwater conditions encountered in the boreholes and the results of in-situ testing from the previous investigation are given on the Record of Borehole presented in Appendix C. The borehole locations and the interpreted stratigraphic profiles projected along the proposed alignment provided on Drawing 372-89-06-A also presented in Appendix C.

In particular, Boreholes 12-7 and 12-8, were advanced along the east side of Highway 416 at the then proposed east embankment location. In general, at these borehole locations the subsurface conditions consist of peat and topsoil overlying native compact sand, overlying stiff weathered clay crust, overlying soft to firm grey clay up to about 7 m in thickness all underlain by sand and glacial till. Dolostone bedrock was found below Elevations 83.5 and 82.9 m in the Boreholes 12-1 and 12-4, for the west and east abutment locations, respectively.

Groundwater was measured by means of two standpipe piezometers in Boreholes 12-1 and 12-3 (abutment boreholes) at Elevations 91.3 m and 92.9 m respectively. At Boreholes 12-7 and 12-8 (approach embankment boreholes) the groundwater was observed to be at the ground surface at the completion of drilling at approximate Elevation of 91.3 and 90.7 m respectively.

The design guidance provided in GEOCRE 31G-205 indicated that settlements of the main approach embankments would be in the order of 10 mm, which was to primarily occur during the initial loading of the embankments. However, the anticipated receiving foundation for the embankment fill indicated in the report was the glacial till or sand strata and not the soft to firm clay encountered at the embankment boreholes.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The subsurface soil, and groundwater conditions encountered in the boreholes and the results of in-situ testing from the current investigation are given on the Record of Boreholes, presented in Appendix A. The results of the laboratory testing carried out during the investigation are presented on the Record of Borehole sheets as well as on Figures B1 to B13 in Appendix B. The borehole locations and the interpreted stratigraphic profiles projected along the centerline of CR43 and across the poor performing area are provided on Drawings 1 and 2. Plots of the measured engineering properties of the soil layers encountered at the boreholes are provided on Figure A in Appendix B.

4.2 Site Stratigraphy Overview

At the boreholes, the subsurface conditions generally consist of asphaltic concrete or topsoil surface cover, overlying fill materials, a compact native sand, overlying a stiff to very stiff weathered clay crust overlying a soft to firm clay, which in turn overlies dense sand and gravel glacial till, all underlain by dolostone bedrock.

The groundwater level was measured at the site at a depth of 0.3 m, corresponding to Elevation 91.8 m, approximately two months after installation. The most recent (spring) groundwater level was measured at the site at a depth of 0.2 m, corresponding to Elevation 91.8 m.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from observations of drilling progress and noncontinuous sampling and therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

A more detailed description of the overburden soil deposits, conditions encountered during the field investigation is provided in the following sections.

4.3 Surface Cover / Surficial Materials

Boreholes 20-01, 20-02, 20-05 were advanced through the Highway 416 pavement structure. The thickness of the asphaltic concrete pavement at ranged from 100 to 150 mm.

Topsoil with thicknesses ranging from 150 to 500 mm was encountered at surface at Boreholes 20-03, 20-04 and 20-06.

4.4 Pavement Structure and Embankment Fills

Pavement structure fill consisting predominantly of sand and gravel with varying amounts of silt was encountered below the asphaltic concrete pavement at the highway borehole locations. The top of this layer was encountered at elevations ranging from 96.0 to 94.8 m and ranges in thickness from 0.7 to 0.9 m. The measured moisture content of one sample tested was 3%. The results of grain size analysis testing carried out on a single sample of pavement structure fill are provided on Figure B1 in Appendix B.

Fill consisting predominantly of sand and silt with varying amounts of gravel was encountered below pavement structure fill at the highway boreholes and below the topsoil at the toe of slope boreholes. The top of this layer was encountered at elevations ranging from 95.3 to 95.1 m at the highway boreholes and from 91.8 to 90.8 m in the toe of slope boreholes. At the embankment boreholes the thickness of the fill ranges from 5.6 to 4.4 m, decreasing in thickness from west to east along CR43. The thickness of the fill at the toe of slope boreholes ranges from 0.4 to 1.0 m. The SPT N values ranged from 3 to greater than 100 blows per 0.3 m of penetration, but were more typically 11 to 55, indicating a compact to very dense state of compactness. The higher blow counts (i.e., greater than 100) noted on the Record of Boreholes for this layer, particularly at the highway boreholes, may have been influenced by the presence of cobbles or boulders within the fill rather than the state of compactness of the soil matrix. The measured moisture content of the samples tested ranged from 3 to 5%. The results of grain size analysis testing carried out on four samples of the fill are illustrated on Figure B2 in Appendix B.

4.5 Buried Topsoil

Buried topsoil with a thickness of 200 mm was encountered below the fill materials at Borehole 20-06. The top of this layer was encountered at Elevation 90.9 m.

4.6 Sand

Sand with a varying amount of silt was encountered beneath the fill materials at Boreholes 20-01 to 20-05 and beneath the buried topsoil at Borehole 20-06. The top of this layer was encountered at elevations ranging from 90.7 to 89.7 m and ranges in thickness from 0.9 to 1.6 m. SPT N values ranging from 7 to 36 blows per 0.3 m of penetration, but were more typically 10 to 21, indicating a compact state of compactness.

The moisture content of the samples of the sand tested ranged from 22 to 24%. The results of grain size analysis testing carried out on four samples of this material are illustrated on Figure B3 in Appendix B.

4.7 Clay

A clay deposit was encountered beneath the sand layer at all the boreholes.

The upper portion of the deposit has been weathered to a stiff crust. The top of the crust was encountered at elevations ranging from 89.4 to 88.8 m. The thickness of the crust ranges from 0.7 to 1.8 m and the SPT N values ranged from 2 to 15 blows per 0.3 m of penetration, but were more typically 4 to 7, indicating a stiff to very stiff consistency. The results of a single in-situ shear vane test carried out in weathered clay crust indicate the undrained shear strength of about 95 kPa, indicating a stiff consistency.

The moisture content of the samples of the clay crust tested ranged from 36 to 52%. The results of grain size analysis testing carried out on three samples of this material are illustrated on Figure B4 in Appendix B. The results of Atterberg Limits testing completed on four samples of the weathered crust indicate liquid limits ranging from 50 to 53, plastic limits ranging from 22 to 24 and plasticity indices ranging from 27 to 31. The Atterberg Limits analysis results are illustrated on Figure B5 in Appendix B and indicate a clay intermediate to high plasticity but more typically high plasticity (CH).

The clay below the depth of weathering is grey. The top of the grey clay was encountered at elevations ranging from 88.6 to 87.1 m. The thickness of the grey clay ranges from 2.4 to 6.8 m. Borehole 20-04 was terminated in this stratum. The SPT N values ranged from weight of rod to weight of hammer. In-situ shear vane test results indicate the undrained shear strength of the grey unweathered clay ranges from 22 to 76 kPa, but more typically 26 to 40 kPa, indicating firm consistency. Based on the ratio of the measured in-situ natural shear strength to the remolded shear strength ranging from 4 to 9, the clay is classified as medium sensitive to extra sensitive.

The moisture content of the samples of the grey clay tested ranged from 53 to 79%. The results of grain size analysis testing conducted on four samples of the grey clay are illustrated on Figure B6 in Appendix B. The results of Atterberg Limits testing completed on seven samples of the grey clay indicate liquid limits ranging from 56 to 77, plastic limits ranging from 21 to 25 and plasticity indices ranging from 33 to 52. The Atterberg Limits analysis results are illustrated on Figure B7 in Appendix B and indicate a clay with high plasticity (CH).

The calculated liquidity indices vary from 0.5 to 1.3, indicating the measured natural water content of the selected samples is generally the same or slightly greater than their liquid limit values.

Table 2 summarizes the results of the measured natural water content, and the results of Atterberg Limits testing.

Table 2: Summary of Measured Properties of the Clay Strata

Borehole	Ground Surface Elevation (m)	Sample	Test Elevation (m)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Liquidity Index
Weathered Crust								
BH20-01	96.1	11	88.2	36	50	22	27	0.5
BH20-02	94.9	9	88.5	39	52	24	28	0.6
BH20-05	96.1	11	88.2	37	52	23	29	0.5
BH20-03	91.30	6	88.0	52	53	23	31	1.0
Minimum				36	50	22	27	0.5
Maximum				52	53	24	31	1
Grey Clay								
BH20-01	96.1	13	85.9	57	65	25	40	0.8
BH20-02	94.9	11	86.2	64	62	21	41	1.0

Borehole	Ground Surface Elevation (m)	Sample	Test Elevation (m)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Liquidity Index
BH20-02	94.9	13	83.2	60	58	23	35	1.1
BH20-05	96.1	13	85.1	53	59	25	34	0.8
BH20-04	91.30	6	88.0	65	56	23	33	1.3
BH20-04	91.30	7	86.4	79	77	25	52	1.0
BH20-06	92.10	5	88.0	56	63	23	40	0.8
Minimum				53	56	21	33	0.8
Maximum				79	77	25	52	1.3

Laboratory oedometer consolidation testing was carried out on three samples of the grey clay deposit. The preconsolidation pressures were estimated from the Void Ratio versus Logarithmic Pressures plot (e - $\log \sigma'_p$) using the Casagrande and work methods. The results of the testing are provided on Figures B8 to B10 in Appendix B and are summarized in the Table 3.

The measured engineering properties of the clay deposit outlined in Section 4.7 are illustrated in the plots on Figure A provided in Appendix B.

Table 3: Summary of Consolidation Testing

Borehole	Ground Surface Elevation (m)	Sample	Test Elevation (m)	Unit Weight (kN/m ³)	e_0	σ'_p (kPa)	σ'_{vo} (kPa)	$\sigma_p - \sigma_{vo}$ (kPa)	C_c	C_r	OCR
20-01	96.1	13	85.9	16.6	1.590	200	140	60	0.79	0.180	1.42
20-02	94.9	11	86.2	16.1	1.810	166	113	53	1.27	0.013	1.47
20-04	91.30	6	88.0	16.0	1.890	125	28	97	0.91	0.028	4.33

Notes:

σ'_p	Apparent preconsolidation pressure
σ'_{vo}	Computed existing vertical effective stress
C_c	Compression index
C_r	Recompression index
e_0	Initial void ratio
OCR	Overconsolidation ratio

Based on the results of the IL oedometer consolidation testing on three of the samples from the current investigation, the cohesive soils beneath the embankment are slightly overconsolidated, with OCR values of about 1.4. Based on OCR values of greater than 4, the cohesive soils at the toe of the embankment are overconsolidated.

4.8 Clayey Silt

Clayey silt was encountered beneath the grey clay in Boreholes 20-01, 20-03 and 20-06. The top of the clayey silt layer was encountered at elevations ranging from 84.7 to 82.5 m. Borehole 20-01 was terminated in this stratum. An SPT N value of weight of hammer was measured in this layer, with the exception of one test in Borehole 20-03, which recorded a blow count of 2 per 0.3 m of penetration. In-situ shear vane test results indicate the undrained shear strength of the clayey silt ranges from 36 to 43 kPa, indicating firm consistency.

The measured moisture content of samples tested ranged from 30 to 48%. The results of grain size analysis testing carried out on two samples of this material are provided on Figure B11 in Appendix B. The results of Atterberg Limits tests completed on three samples of the clayey silt indicated liquid limit values ranging from 27 to 33, plastic limit value values ranging from 17 to 18, and plasticity indices values ranging from 10 to 15. The Atterberg Limits analysis results are provided on Figure B12 in Appendix B and indicate a clayey silt of low plasticity (CL).

4.9 Glacial Till

Glacial till was encountered below the grey clay Boreholes 20-02 and 20-05 and below the clayey silt in Boreholes 20-03 and 20-06. The glacial till generally consists of a heterogeneous mixture of cobbles and boulders within a soil matrix of sand, gravel and silt with trace amounts of clay. The top of the glacial till was encountered at elevations ranging from 81.9 to 81.2 m. s but was proven to the depths ranging from 10.1 to 15.1 m below the existing grade.

The SPT N values ranged from 7 to 57 blows per 0.3 m of penetration indicating a loose to very dense state of compactness. The measured moisture content of the sample tested was 8%. The results of grain size analysis testing carried out on a single sample of this material are provided on Figure B13 in Appendix B.

4.10 Groundwater Conditions

A monitoring well was installed in Borehole 20-06, to observe the stabilized groundwater level at the site.

Table 4 summarizes the depths and the elevations of the groundwater levels measured in the monitoring well installed at the site during the current investigation.

Table 4: Summary of Groundwater Conditions

Borehole	Screened Interval	Depth (m)	Elevation (m)	Date
19-3607	Glacial Till	0.3	91.8	January 20, 2021
		0.2	91.9	April 13, 2021

At the time of the original investigation, the groundwater level was measured at Elevations of 91.3 m and 92.9 m in Boreholes 12-1 (proposed west abutment) and 12-3 (proposed east abutment), respectively. At the time of drilling during the previous investigation, groundwater was observed at the ground surface along the alignment of the proposed east embankment.

It should be noted that groundwater levels are anticipated to fluctuate seasonally, and higher groundwater levels are expected during wet / precipitation periods of the year.

5.0 CLOSURE

This report was prepared Kenton Power, P.Eng. The report was reviewed by William Cavers, P.Eng. an Associate, Senior Geotechnical Engineer with Golder and the Designated MTO Foundations Contact for this project.

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PART B

Foundation Design
Site 16-318 Highway 416 and County Road 43
Ramp Terminal Intersection Improvements
United Counties of Leeds and Grenville, Ontario
GWP 4129-18-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This report presents the results of the foundation design recommendations for the possible rehabilitation of the existing east embankment of the Highway 416 / County Road (CR) 43 overpass structure. The purpose of this foundation investigation was to assess the subsurface conditions at the existing east embankment on CR43, to evaluate causes of the distress observed at the existing embankment, and to provide geotechnical input for selection of a technically preferred rehabilitation alternatives to address the existing deficiencies. The foundation investigation included drilling boreholes, installing a groundwater monitoring well, and subsequent laboratory testing on selected soil samples.

The foundation investigation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 Existing Conditions

CR43 within the area under consideration consists of two 3.75 m through lanes, one each in the eastbound and westbound directions, a Highway 416 northbound onramp lane. At this location Highway 416 has a four lane cross-section with two northbound and two southbound through lanes, plus speed change lanes at the interchange ramps. The Preliminary Design Report by Dillon indicated that distress at the existing CR43 east embankment was observed between approximate Stations 10+300 and 10+400, although a pavement investigation was not completed as part of the preliminary design. At the time of preparing this report maintenance history for the existing east embankment was not available.

The interchange was part of the original construction of Highway 416, completed under Contract 97-17. Available existing construction drawings indicate the existing three span structure is supported on spread footings founded within the glacial till or on compacted granular pads placed on the glacial till.

A previous investigation was carried out for the design of the existing structure in 1991, (GEOCRE 31G00-205). It should be noted that the coordinates of the original boreholes locate the proposed interchange alignment approximately 200 m south of the existing alignment. Of note, at the east embankment location the boreholes from the previous investigation encountered a soft to firm silty clay layer up to 6.9 m in thickness that was not encountered at the proposed abutment or pier locations. Groundwater was observed to be at the ground surface at these borehole locations. Boreholes advanced during the current investigation within the noted distress area of the existing east approach embankment encountered a layer of soft to firm clay layer up to 8 m in thickness beneath the embankment fill.

The base plan mapping provided by Dillon for this project and the ground surface elevations at the borehole locations surveyed during the current field investigation indicate that the top of roadway elevation of CR43 in the vicinity of the distressed area ranges from Elevation 96.1 to 94.9 m. The existing east approach embankment is approximately 5 to 7 m in height above the natural ground level constructed with side slopes that are generally oriented at about 2H:1V.

6.2 Seismic Design

6.2.1 Seismic Hazard and Importance Category

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The current seismic hazard maps (referred to as the 5th generation seismic hazard maps) were developed by the GSC and were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, it is understood that Highway 416 at this location has been given an importance category of “Major Route”.

6.2.2 Seismic Site Classification

In accordance with the CHBDC, the selection of the seismic site classification is based on the soil conditions encountered in the upper 30 m of the stratigraphy below the founding elevation. Based on the soil conditions encountered, the site is classified as a Seismic Site Class E in accordance with Table 4.1 of the CHBDC.

6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the Highway 416 / CR43 interchange (latitude 45.028 N longitude 75.622 W), the values provided in Table 5 are the reference Site Class C (reference) peak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca).

Table 5: Site Class C Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 Years (2,475-year) (g)
PGA	0.290
T ≤ 0.2 s	0.452
T = 0.5 s	0.239
T = 1.0 s	0.117
T = 2.0 s	0.055
T = 5.0 s	0.014
T ≥ 10.0 s	0.005

The values given in Table 5 are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given in Section 6.2.2 (Site Class E) in accordance with Section 4.4.3 of the CHBDC. As indicated in Section 4.4.3.3 of the CHBDC, the value of PGA_{ref} for use with Tables 4.2 to 4.9 shall be taken as 80% of the PGA for Site Class C where $S_a(0.2)/PGA$ is less than 2.0. Based on this requirement a PGA_{ref} value of 0.232g was used for the 2,475-year return period. The corresponding site-specific Site Class E seismic hazard values given in Table 6 can be used for design.

Table 6: Site Class E Spectral Values for Subject Site

Parameter	2% Probability of Exceedance in 50 Years (2,475-year) (g)
PGA	0.334
$T \leq 0.2$ s	0.533
$T = 0.5$ s	0.406
$T = 1.0$ s	0.231
$T = 2.0$ s	0.118
$T = 5.0$ s	0.032
$T \geq 10.0$ s	0.011

6.2.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”.

The liquefaction susceptibility of granular soils at these sites were evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required to trigger liquefaction.

The methodology used to assess liquefaction potential at the site is consistent with the “simplified” approach outlined in the CHBDC and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analyses were carried out using the in-situ testing data collected at the borehole locations from the current investigation.

The design groundwater level was established based on the groundwater elevations measured in the standpipe piezometers installed in Boreholes 20-06.

The CRR profile with depth was calculated at each borehole location using the parameter, $SPT (N_1)_{60cs}$, that is based on the SPT N values obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment using the simplified method indicate that the site soils are not considered to be susceptible to liquefaction during the 2,475-year design earthquake loading outlined in Section 6.2.3.

6.3 Assessment of Global Stability

6.3.1 General

The global stability for the as-constructed approach embankment was evaluated using GeoStudio 2021 Slope/W software for limit equilibrium analysis. The slope geometry used in the stability analysis was based on the topographic survey provided by Dillon. The immediate post-construction and current embankment conditions were analysed under undrained, drained and seismic design conditions. The results of the slope stability analysis are provided in Figures E1 to E4 provided in Appendix E.

The following geometry/parameters were used in the analysis:

- The embankment was constructed with maximum 2H:1V side slopes
- A seismic horizontal loading of 0.167g, equal to ½ of the site adjusted PGA value (0.334g Site Class E) was used for seismic analysis, (see Section 6.2.3)
- Soil stratigraphy based on Section B-B shown on Drawing 2, and
- Groundwater level of 91.0 m, approximately at the existing ground surface at the toes of the embankments.

The analysis looked at the following two scenarios:

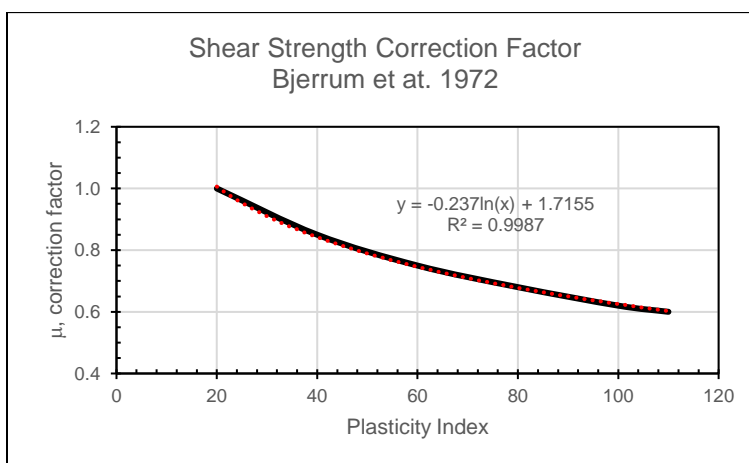
- 1) **Post-construction:** This analysis assumed the *worst-case* shear strengths at the time of construction, with the shear strength of the clay stratum based on the results of the corrected shear strength ($S_{u\text{cor}}$) values from the current toe of slope boreholes (i.e., prior to the effect of any consolidation from the embankment loading).
- 2) **Long Term:** This analysis also assumed the *worst-case*, shear strength values but assumed an increase in $S_{u\text{cor}}$ values under the embankment, as indicated at the current embankment boreholes, due to consolidation of the clay as a result of the embankment loading.

6.3.2 Corrected Shear Strength Values

As part of the analysis, the measured in-situ shear strengths were corrected, following the procedure outlined in Bjerrum et. al. 1972, using the following equation:

$$S_{u\text{cor}} = \mu S_{u\text{field}}$$

The correction factor μ varies with measured plasticity index in accordance with the relationship provided in Bjerrum et. al. 1972 and reproduced in following graph.



Graph 1: Shear Strength Correction Factor, after Bjerrum et al 1972

As shown on the Plot 1 on Figure B in Appendix E, there is a variation in the plasticity index values throughout the clay stratigraphy. Design μ values were therefore selected based on the results of the index testing carried out for the current investigation as indicated on Plots 2 and 3 on Figure B and summarized in Table 7 for the embankment and toe of slope shear strengths, respectively.

Table 7: Bjerrum et. al. 1972 Correction Factor, μ

Cohesive Stratum	Plasticity Range	μ – Embankment Boreholes	μ – Toe of Slope Boreholes
Weathered Crust (CH)	27 to 35	0.92	0.88
Grey Clay (CH)	34 to 42	0.88	0.78
Grey Clay (CL to CI)	10 to 15	1.07	1.17

The measured in-situ shear strengths ($S_{u\text{field}}$) and the resulting corrected shear strengths ($S_{u\text{cor}}$) are shown on Plots 4 and 5 respectively on Figure B. As illustrated on Plot 5, there is a marked increase in the $S_{u\text{cor}}$ values between those measured at the embankment boreholes in comparison to those beyond the toe of slope. This increase in shear strength values, which are correlated with pre-consolidation pressure, is likely due to consolidation of the clay as a result of the embankment loading.

6.3.3 Scenario 1 Stability Analysis

Input parameters for the immediate post-construction analysis provided in Table 8 are based on the measured in-situ SPT N and $S_{u\text{cor}}$ values from the toe of slope boreholes and the results of the laboratory testing from the current investigation. The design *worst-case* $S_{u\text{cor}}$ profile was used for the Scenario 1 analysis as illustrated on Plot 2 on Figure C in Appendix E.

Table 8: Geotechnical Design Parameters for Scenario 1, Post-construction Stability Analysis

Material	Stratum Bottom Depth below Embankment Fill (m)	Bulk Unit Weight (kN/m ³)	Undrained Shear Strength, $S_{u\text{cor}}$ (kPa)	Drained Analysis	
				Internal Angle of Friction, ϕ' (°)	Cohesion, c' , (kPa)
Embankment Fill	96.0 to 91.0	21	-	28	0
Sand	91.0 to 89.1	22	-	28	0
Weathered Clay Crust	89.1 to 87.6	18	80	35	7.7
Upper Grey Clay	87.6 to 85.5	17.3	20	28	7.4
Lower Grey Clay	85.5 to 81	17.3	20 to 40 increasing with depth	33	7.7
Glacial Till	81 to 79	21	-	38	0

The results of the stability analysis for the post construction Scenario 1 indicate that the existing embankment at the time of construction had an undrained factor of safety (FOS) greater than 1.8 against deep seated slope instability. The FOS increases to greater than 2 under the drained condition. At the time of construction, the embankment had a FOS of 0.9 to 1.0 against seismic instability. The results of the slope stability analysis for Scenario 1 are provided in Figures E1 to E3 in Appendix E.

6.3.4 Scenario 2 Stability Analysis

Input parameters for the analysis provided in Table 9 are based on a combination of the increased $S_{u\text{cor}}$ profile of the clay beneath the embankment (as a result of the embankment loading and resulting consolidation of the clay) and the existing $S_{u\text{cor}}$ profile at the toes of the embankment slopes.

Table 9: Geotechnical Design Parameters for Scenario 2 Stability Analysis

Material	Elevation (m)	Bulk Unit Weight (kN/m ³)	Undrained Analysis Shear Strength, $S_{u\text{cor}}$		Drained Analysis	
			Toe of Slope (kPa)	Embankment (kPa)	Internal Angle of Friction, ϕ' (°)	Cohesion, c' , (kPa)
Embankment Fill	96.0 to 91.0	21	-	-	28	0
Sand	91.0 to 89.1	22	-	-	28	0
Weathered Clay Crust	89.1 to 87.6	18	80	100	35	7.7
Upper Grey Clay	87.6 to 85.5	17.3	20	35	28	7.4
Lower Grey Clay	85.5 to 81.0	17.3	20 to 40 increasing with depth	35 to 50 increasing with depth	33	7.7
Glacial Till	81.0 to 79	21	-	-	38	0

Under the Scenario 2 conditions, the embankment has an FOS of greater than 1.3 against seismic instability. The results of the seismic slope stability analysis for Scenario 2 are provided in Figure E4 Appendix E.

Based on the results of the stability analysis it is considered that the slopes were and are stable, both immediately after construction and in its current condition, although it was marginally unstable from a seismic perspective immediately at the time of construction. Therefore, the distress currently observed at the site is likely not caused by slope instability of the soil under static or seismic loading. This assumes that the embankment slope was not affected by a seismic event at the time of completion.

6.4 Settlement Analyses

6.4.1 General

Settlement analyses were carried out using Rocscience's Settle3D 4.0 software to estimate the magnitude of settlement of the clayey deposit since the original construction and estimate the potential future settlement.

The analyses were carried out assuming the following:

- 1) **Initial Analyses:** These analyses assumed the *worst-case* and the *most likely* consolidation parameters, based on the in-situ and laboratory testing, and water levels consistent with those measured during the current investigation.
- 2) **Sensitivity Analysis:** For these analyses, the effect of lowering the groundwater level, either temporarily or permanently, on the existing effective stress of the clay layer and the estimated settlement magnitudes was investigated.

There are several limitations to the settlement analyses that have been carried out:

- MTO's Maintenance Department has confirmed that there are no settlement monitoring data or maintenance history for the existing east embankment that can be used to validate the settlement analysis. Also, there are no other settlement records available to verify the actual magnitudes of settlement.
- There is limited information on the original pavement design and previous rehabilitations specific to the embankment area. The previous rehabilitations (e.g., pad and overlay the roadway periodically to reinstate the roadway profile) could have added some minor additional loading, potentially leading to increased settlement of the normally consolidated cohesive soils. However, the magnitudes of the additional loading and any resulting settlement are unknown.
- The consolidation parameters (e.g., C_r , C_c , e_o , etc.) of the clayey deposit are not constant but have changed (or are changing) over time because of the embankment loading. However, the rate of change is not known and is not easily modelled.

Considering the limitations outlined above, the input to the analyses (including the stress levels, preconsolidation pressure profile and consolidation parameters) were estimated based on the consolidation characteristics of the deposit from the current investigation as presented on the Summary of Engineering Properties, Figure A in Appendix B.

6.4.2 Preconsolidation Pressure (σ'_p) Values

As outlined in Section 4.7, incremental loading oedometer consolidation testing was carried out on three samples of the clay sampled during the current investigation; two samples from the clay layer beneath the embankment and one sample from a borehole at the toe of slope.

Preconsolidation pressures were calculated for the clay layer beneath the embankment and beyond the toe of slope following the procedure outlined in Lerouei et al. 1983. This procedure correlates the in-situ shear strengths ($S_{u\text{field}}$) with the preconsolidation pressures based on the measured plasticity index (IP) of the soil strata according to the following equation:

$$\frac{S_{u\text{field}}}{\sigma'_p} = 0.0024(IP) + 0.2$$

As shown on Plot 3 on Figure C in Appendix F, there is a variation in the plasticity index values throughout the clay deposit. Design IP values were selected based on the results of the index testing carried out for the current investigation and those were used to obtain the appropriate $S_{u\text{field}}/\sigma'_p$ values over depth for use estimating the preconsolidation pressure values throughout the deposit based on the field vane shear strengths.

Based on the results of the IL oedometer consolidation testing on two of the samples from the current investigation, the clay deposits underlying the embankment are slightly overconsolidated, with OCR values of about 1.4. However, as shown on plots 2 to 5 on Figure D provided in Appendix F. The S_u/σ'_p ratios based on the correlation with IP indicate that portions of the clay below the embankment are normally consolidated, with the correlated σ'_p equal to or less than the existing effective overburden pressure, σ'_{vo} .

6.4.3 Settlement Analysis

The worst-case and most-likely input parameters for *Initial Analyses* where most of the cohesive soils are over-consolidated are provided in Table 10.

Table 10: Geotechnical Design Parameters for Initial Analyses Settlement Analysis

Material	Bulk Unit Weight (kN/m ³)	Elevation (m)	Preconsolidation Pressure σ'_p (kPa)		C_c	C_r	C_v (cm ² /s)	C_{vr} (cm ² /s)	C_d/C_c
			Worst Case	Most Likely					
Embankment Fill	21	96.0 to 91.0	-	-	-	-	-	-	-
Sand	22	91.0 to 89.1	-	-	-	-	-	-	-
Weathered Clay Crust	18	89.1 to 87.6	350 to 77	350 to 120	1.09	0.04	5x10 ⁻⁵	0.007	0.04
Upper Grey Clay	17.3	87.6 to 81.0	77 to 150	120 to 190	1.09	0.04	5x10 ⁻⁵	0.007	0.04
Glacial Till	21	81.0 to 79	-	-	-	-	-	-	-

6.4.4 Initial Analyses Settlement Results

Based on the existing embankment height, the assessed effective stress level, the measured groundwater levels and the correlated σ'_p profile within the cohesive deposit, the estimated total settlement for the first 30 years since construction and at 50 years (i.e., at the end of the typical rehabilitation cycle for bridge approaches) are provided in the Table 11.

Table 11: Settlement Estimates Initial Analyses

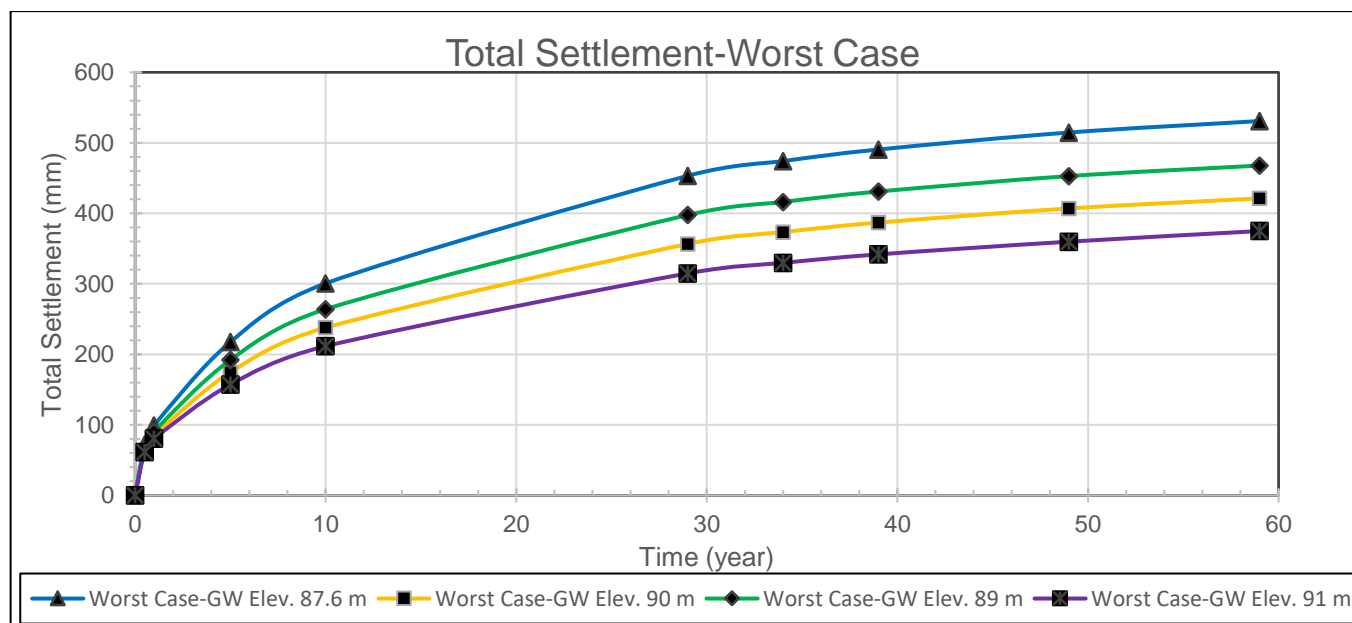
Time Period	Maximum Settlement (mm)	
	Worst-Case	Most-Likely Case
1992 to 2021	315	125
2021 to 2041	60	25

6.4.5 Sensitivity Settlement Analyses

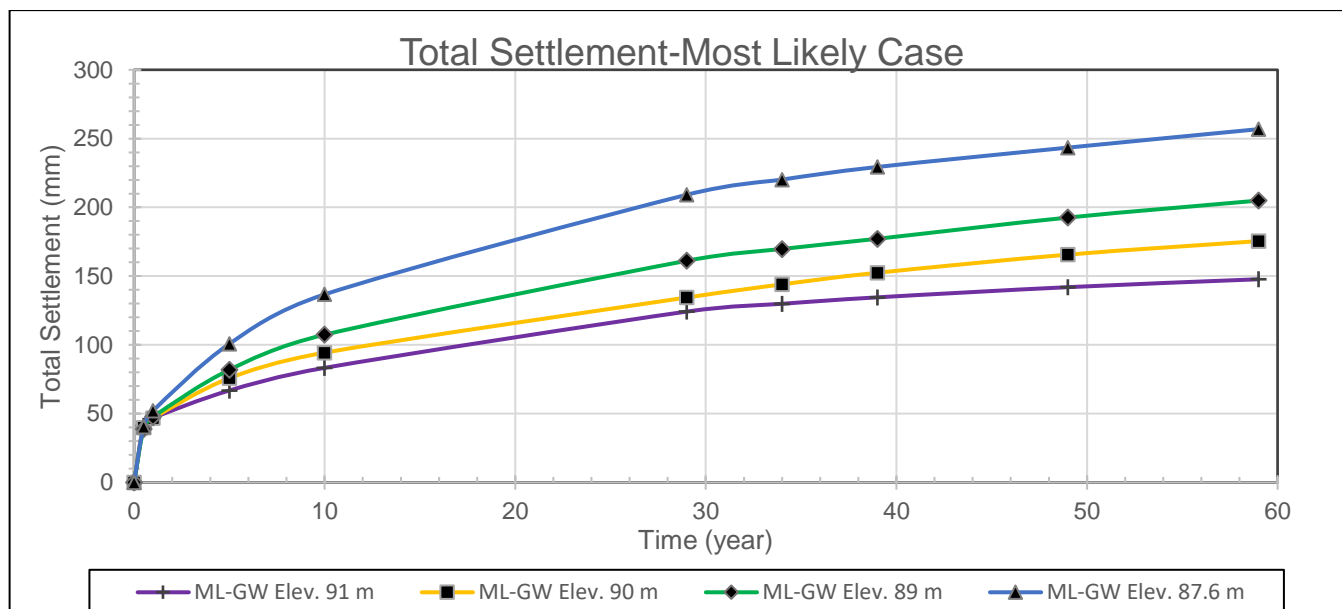
A sensitivity analysis was carried out to determine the effect of lowering the groundwater level either temporarily and/or permanently on the existing effective stress of the cohesive layer and thereby the estimated settlements, since the investigation information indicates the water level is currently above the weathered clay (which is an indicator of the historical water level).

Three groundwater regimes were analysed using the most-likely and worst-case σ'_p profiles and groundwater depths at 1.0 m, 2.0 m, and 3.4 m (top of grey clay) below existing grade.

The results of the settlement versus time results are reproduced in Graphs 2 and 3, for the *worst case* analysis parameters and the *most likely* analysis parameters, respectively:



Graph 2: The results of the settlement versus time worst case analysis parameters



Graph 3: The results of the settlement versus time most likely analysis parameters

From the above graphs it can be seen that during the 30 years the embankment has been in place, about 125 to 315 mm of total (primary consolidation plus secondary) settlement has likely occurred. In addition, there has likely been periods of increased settlement during periods with lower groundwater levels. The magnitude of the additional settlement due to temporarily lowered groundwater levels is difficult to estimate, since the above graphs assume a lowered groundwater level over the entire time period. However, lowering the groundwater level would re-initiate primary consolidation response to the increased loading and it is reasonable to assume that the increased settlement would correspond to the initial settlement at the start of loading as shown in the above graphs (i.e., may be in the order of 50 mm over a few months).

It should be noted that no historical settlement monitoring records are available at this site and therefore the total amount of settlement that has occurred since the start of construction could not be verified.

The settlement estimates provided in the graphs above would be entirely differential relative to the structure and to portions of the embankment founded on sand and/or till.

7.0 SUMMARY AND POTENTIAL MITIGATION MEASURES

The analyses carried out above indicate that the embankment had a sufficient factor of safety against static instability immediately after construction but was marginally stable or slightly unstable from a seismic perspective (assuming the current code design earthquake). However, as the clay gained in strength due to consolidation as a result of the embankment loading the factors of safety against instability, both statically and seismically, have improved and are in excess of those considered acceptable.

The settlement analyses indicate that the embankment as constructed likely exceeded the preconsolidation pressure of the underlying clay but by relatively small magnitudes. Relatively modest embankment settlement magnitudes can however only be expected if the groundwater levels at the site are relatively high and are constant over time. This is unlikely and the thickness of the weathered crust provides an indication of the potential variation in water level over time since the weathering is a result of water level variations within that upper part of

the clay. It is likely that the water levels over the embankment lifetime have been lower than the currently indicated level, which is above the surface of the weathered crust (and is at the ground surface adjacent to the embankment).

Based on the analyses completed, it seems unlikely that the current distress at the embankment is related to instability. Although the embankment had a factor of safety of less than 1 against seismic instability immediately after construction, the seismic factor of safety for the embankment has increased over time (assuming some degree of consolidation has occurred). In addition, although there have been some recent earthquakes in Ottawa or nearby areas, none have resulted in embankment failures that Golder is aware of.

It is therefore likely that the embankment distress has been caused by settlement in response to the embankment loading and may have been exacerbated by varying ground water levels. Ottawa has experienced both spring flooding (2018 and 2019) and summer droughts (2016 and 2018) within the last 5 years, and it is likely that groundwater levels have varied sufficiently during the last few years to cause additional embankment settlement that would result in the observed pavement distress.

Based on the information provided above, the following options may be considered for remediating this section of the CR43 in most to least favourable in terms of cost and feasibility. Their respective advantages and disadvantages are outlined below and are summarized in Table 12 following the text of this report.

- 1) Reconstruct the roadway pavement and then monitor and maintain as required.
- 2) Intrusive ground improvement (e.g., soil mix or stone columns), followed by reconstruction of the pavement structure.
- 3) Preload and surcharge the embankment to reduce the future settlements to within tolerable levels and then reconstruct the pavement.
- 4) Reconstruct the upper 3 m of the embankment with expanded polystyrene (EPS) to reduce the settlement magnitude sufficiently to meet the MTO paving criteria.
- 5) Full removal of the compressible clay layers and replacement with suitable granular or earth fill embankment followed by embankment reconstruction.

7.1 Option 1 Road Reconstruction, Monitor and Maintain

Option 1 would be the least cost option. This option would not address the potential for ongoing settlement at the site but would allow for the measurement of the magnitude of settlement to guide establishment of maintenance intervals. This option is feasible, as the settlement estimates indicate that most of the settlement may have occurred at the site over the past 30 years, although the potential exists for increased settlement rates and magnitudes should there be groundwater lowering (as a result of drought conditions, for example). Based on the estimates above, the settlement magnitudes are likely to be in the order of 50 mm over the next 20 years, but could be as high as about 100 mm.

The monitoring would also allow further assessment of the magnitude of the settlement and the response to groundwater levels to confirm the suspected mechanism.

This option would require the development of a settlement monitoring plan that will serve to document the magnitude and timing of the settlement going forward. Monitoring of settlement instruments on this project would be constrained by the continuous and high traffic volume and the limited periods during which access to CR43

can be obtained. By necessity, a non-intrusive system is recommended for the ground surface (i.e., pavement) monitoring, such as a Shape Accelerometer Array (SAA) which can be accessed remotely (i.e., via web-based software).

As noted above and in Section 6.4.6, lowering of the groundwater at the site either permanently or temporarily may increase the total settlement of the cohesive layer, therefore monitoring the groundwater monitoring in conjunction with settlement monitoring is recommended at this site. The groundwater monitoring should include the installation of two additional nested vibrating wire piezometers with screens sealed within the clay and embankment layers. Together with the monitoring well currently installed at the site, these instruments would provide an understanding of seasonal changes in the groundwater level at the site.

However, it should be taken under advisement that any future pad and overlay of the roadway to reinstate the roadway profile would add some minor additional loading, potentially leading to increased settlement of the normally consolidated cohesive soils.

7.2 Option 2 Intrusive Ground Improvement

The second option is to improve the clay soils under the affected part of the embankment using intrusive ground improvement techniques, such as soil mixing or stone columns.

These options have the advantage of being able to install the improvement columns from the surface which would avoid the need for deep excavations that would be required for other options (Options 4 and 5) as well as the high costs for additional traffic protection systems and dewatering. Depending on equipment size, proper construction staging would likely allow for maintaining a single lane of traffic available during construction.

These options could reduce the ongoing to settlements to within tolerable magnitudes.

7.3 Option 3 Surcharging

To address the ongoing settlement and to alleviate the distress, the existing embankment could be surcharged with additional fill and allowed to settle over a period of a few months. This would result in higher preconsolidation pressures within the clay deposit under the embankment and would reduce the magnitude of future settlements after placing the roadway back in service. However, it is likely that the roadway would need to be closed to traffic during the surcharge period and this would result in significant disruption to the surrounding communities.

7.4 Option 4 Embankment Reconstruction with EPS

To address the ongoing settlement and to alleviate the distress, the existing embankment fill, to a depth of about 3 m (this would need to be confirmed during additional detailed design) could be sub-excavated and replaced with ultra-lightweight fill such as expanded polystyrene (EPS). This would have the advantage of reducing the stresses imposed on the underlying compressible clay layers to less than the deposit's current preconsolidation pressures. Construction staging and traffic protection systems to maintain traffic and allow CR43 to remain open would be required.

7.5 Option 5 Clay Removal and Replacement

Results of the current investigation indicate that the base of the clay layer is approximately 14 m below the existing top of pavement and based on that depth material removal this option is not considered feasible.

7.6 Conclusion


From a foundation perspective, Option 1 is likely the most cost-effective option but does have some risk, recognizing that frequent monitoring and maintenance would be required to assess rideability and determine when profile corrections are required. The period between padding to restore the pavement may vary depending on the actual degree of consolidation and climate conditions

Options 2 and 4 are also considered feasible and would limit the post-construction settlement and have the lowest risk of rideability and safety concerns in the future. From a constructability and cost perspective, Option 2 is likely the preferred option should full risk reduction be desired.

8.0 CLOSURE

This report was prepared Kenton Power, P.Eng. The report was reviewed by William Cavers, P.Eng. a Senior Geotechnical Engineer with Golder and the Designated MTO Foundations Contact for this project.

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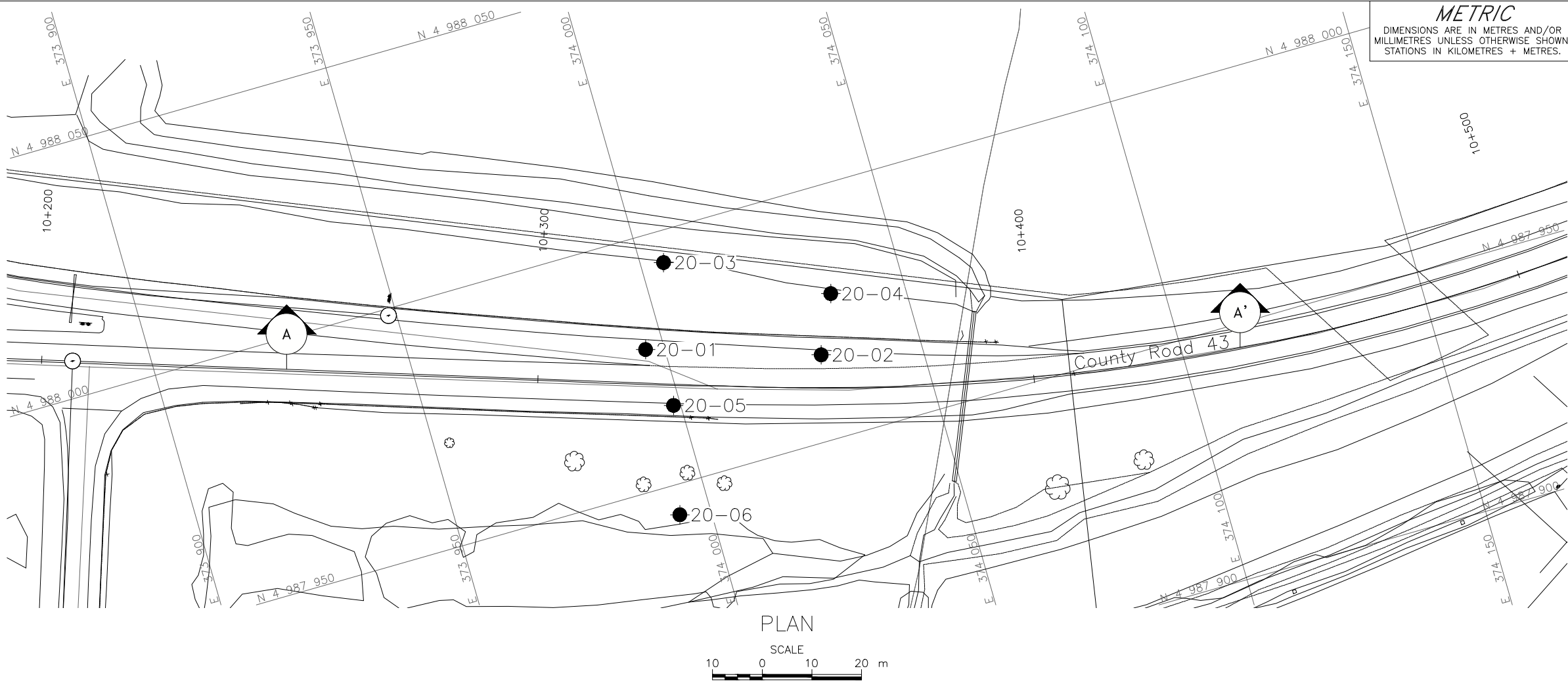
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Table 12: Comparison of Embankment Settlement Mitigation Alternatives

Embankment Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1 Re-surface, Monitor and Maintain	<ul style="list-style-type: none">■ Feasible■ Preferred option from a foundation perspective	<ul style="list-style-type: none">■ Monitoring would aid in verifying settlement estimates and timing of maintenance.■ Least disruption to users of CR43	<ul style="list-style-type: none">■ Would not address the potential for ongoing settlement■ More frequent maintenance than estimated may be required■ Possible interim safety issue, between overlays, due to settlement■ Future padding/overlaying to reinstate the roadway profile would increase loading, potentially leading to increased settlement of the normally consolidated cohesive soils.	<ul style="list-style-type: none">■ Likely least expensive option but must consider post-construction monitoring and maintenance costs	<ul style="list-style-type: none">■ Highest risk as it does not address the settlement.■ Excessive roadway settlement in the long-term
Option 2 Intrusive Ground Improvement	<ul style="list-style-type: none">■ Feasible■ Preferred reconstruction option from a foundation perspective	<ul style="list-style-type: none">■ Will reduce post-construction settlements to within tolerable limits.■ Avoids the need for deep shored excavations■ Avoids the costs for additional protection systems and dewatering during construction.■ Staging may allow for a single lane of CR43 traffic to remain operational during construction	<ul style="list-style-type: none">■ Ground improvement will require specialist sub-contractors for a relatively small footprint■ Significant impact on CR43 use during construction	<ul style="list-style-type: none">■ High costs, although potentially less costly than EPS replacement option	<ul style="list-style-type: none">■ Depending on equipment and/or improvement program CR43 could be fully closed to traffic.■ Detour or single lane will be required during construction.
Option 3 Preload and Surcharge	<ul style="list-style-type: none">■ Feasible, but not likely practical if CR 43 is to be maintained open for traffic	<ul style="list-style-type: none">■ Avoids the need for deep excavations of the remove/replace options■ Avoids the costs for additional protection systems and dewatering during construction.	<ul style="list-style-type: none">■ Closure of CR43 would be required for several months.	<ul style="list-style-type: none">■ Less costly option than intrusive ground improvement, clay replacement or EPS fill.	<ul style="list-style-type: none">■ Significant disruption to surrounding communities due to closure of CR43
Option 4 Embankment Reconstruction with EPS	<ul style="list-style-type: none">■ Feasible	<ul style="list-style-type: none">■ Limits post-construction maintenance	<ul style="list-style-type: none">■ Require subexcavation of about 3 metres of embankment fill and replacement with EPS to meet settlement requirements.■ Significant impact to CR43 use during construction	<ul style="list-style-type: none">■ High costs	<ul style="list-style-type: none">■ Low risk option■ Depending staging CR43 could be fully closed to traffic.■ Detour or single lane will be required during construction.
Option 5 Clay removal and replacement with granular or earth fill.	<ul style="list-style-type: none">■ Not feasible or practical.	<ul style="list-style-type: none">■ N/A	<ul style="list-style-type: none">■ N/A	<ul style="list-style-type: none">■ N/A	<ul style="list-style-type: none">■ N/A



CONT No.
GWP No.

HIGHWAY 416 UNDERPASS
AT COUNTY ROAD 43
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 45.028392 LONG. -75.622089

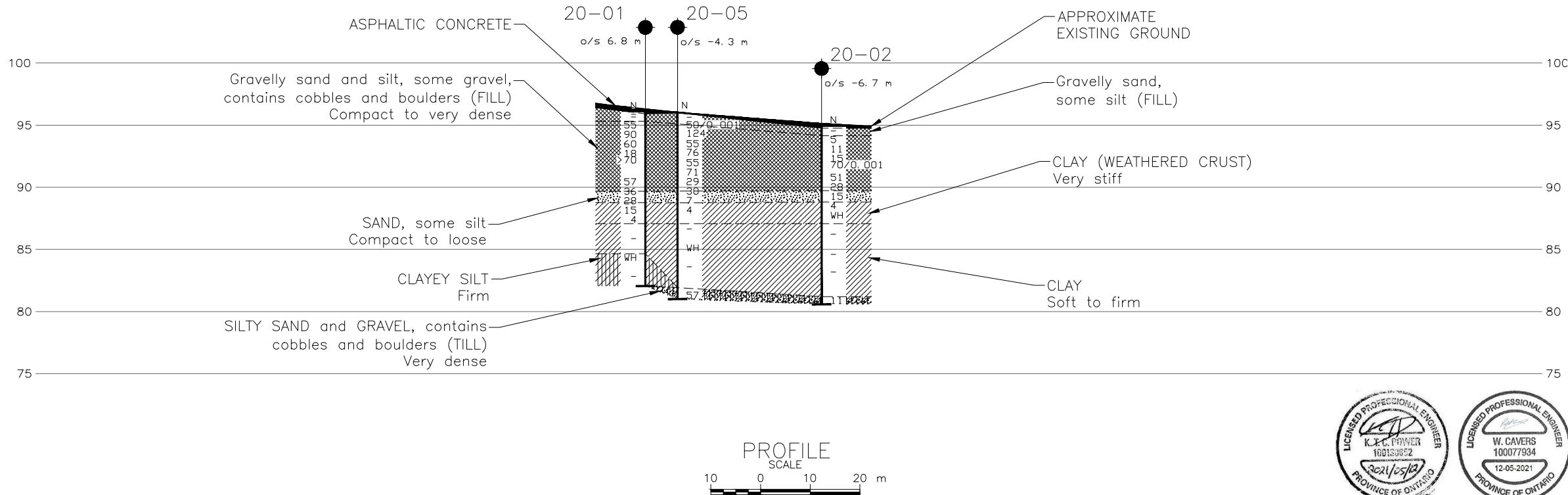
SHEET



KEY PLAN
SCALE
500 0 500 1000 m

LEGEND

- Borehole – Current Investigation
- ⬮ Seal
- ⬮ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL upon completion of drilling



BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
20-01	96.1	4987977.8	373996.3
20-02	94.9	4987967.1	374030.0
20-03	91.3	4987993.6	374004.6
20-04	91.3	4987978.4	374035.3
20-05	96.1	4987965.5	373998.6
20-06	92.1	4987944.0	373993.8

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

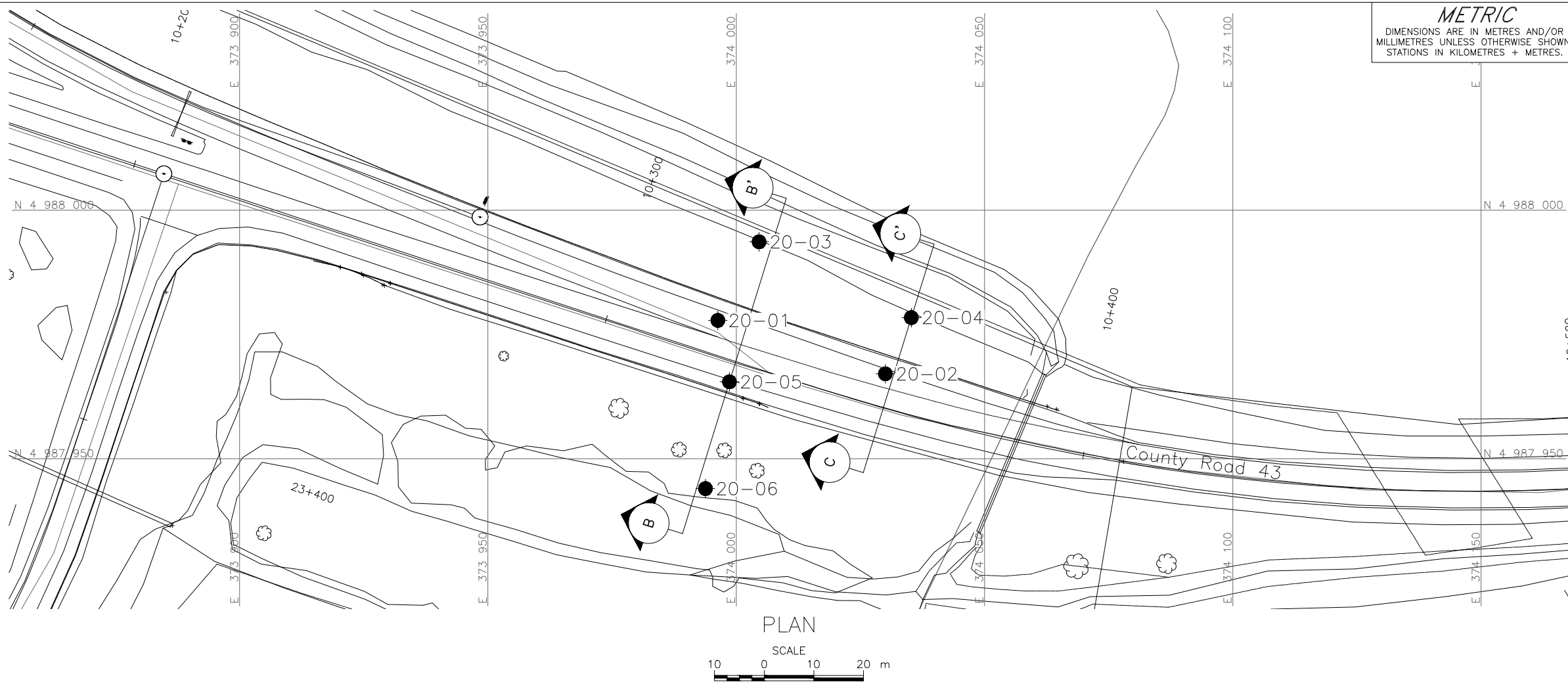
REFERENCE

Base plans provided in digital format by Dillon, drawing file no. 4017-E-0039-2.dwg, received 12 16, 2020.

NO.	DATE	BY	REVISION

Geocres No. 31G-286

HWY. 416		PROJECT NO. 1899802		DIST. EASTERN	
SUBM'D. KCP	CHKD. KCP	DATE: 5/13/2021	SITE: 19-318		
DRAWN: ZS	CHKD. KCP	APPD. WC	DWG. 1		

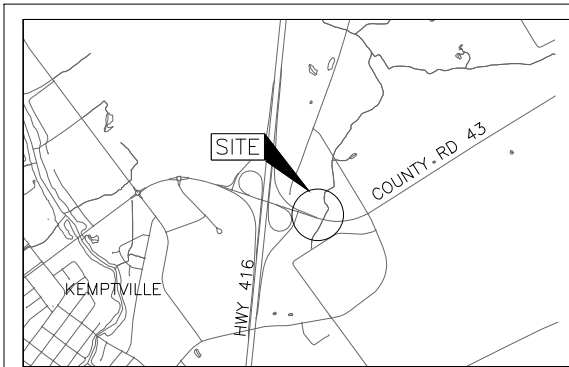


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.

HIGHWAY 416 OVERPASS
AT COUNTY ROAD 43
BOREHOLE LOCATIONS AND SOIL STRATA
LAT. 45.028392 LONG. -75.622089

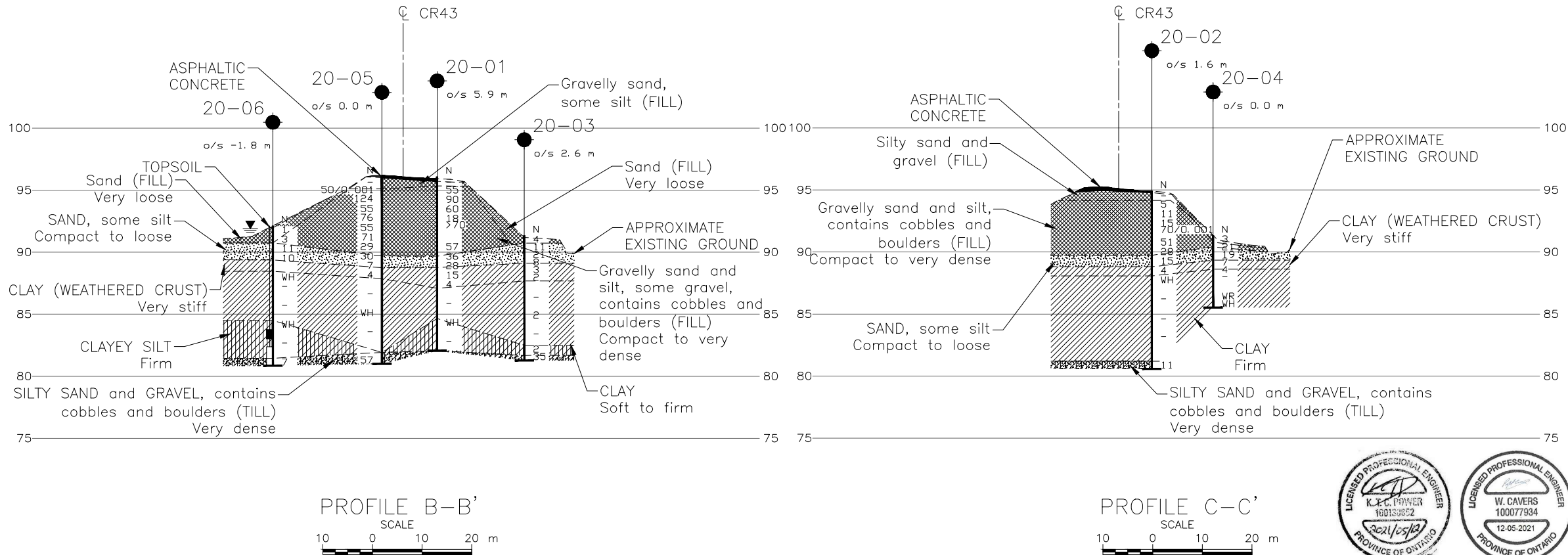
SHEET



KEY PLAN
SCALE
500 0 500 1000 m

LEGEND

- Borehole - Current Investigation
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- WL in piezometer, measured on April 13, 2021



BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
20-01	96.1	4987977.8	373996.3
20-02	94.9	4987967.1	374030.0
20-03	91.3	4987993.6	374004.6
20-04	91.3	4987978.4	374035.3
20-05	96.1	4987965.5	373998.6
20-06	92.1	4987944.0	373993.8

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

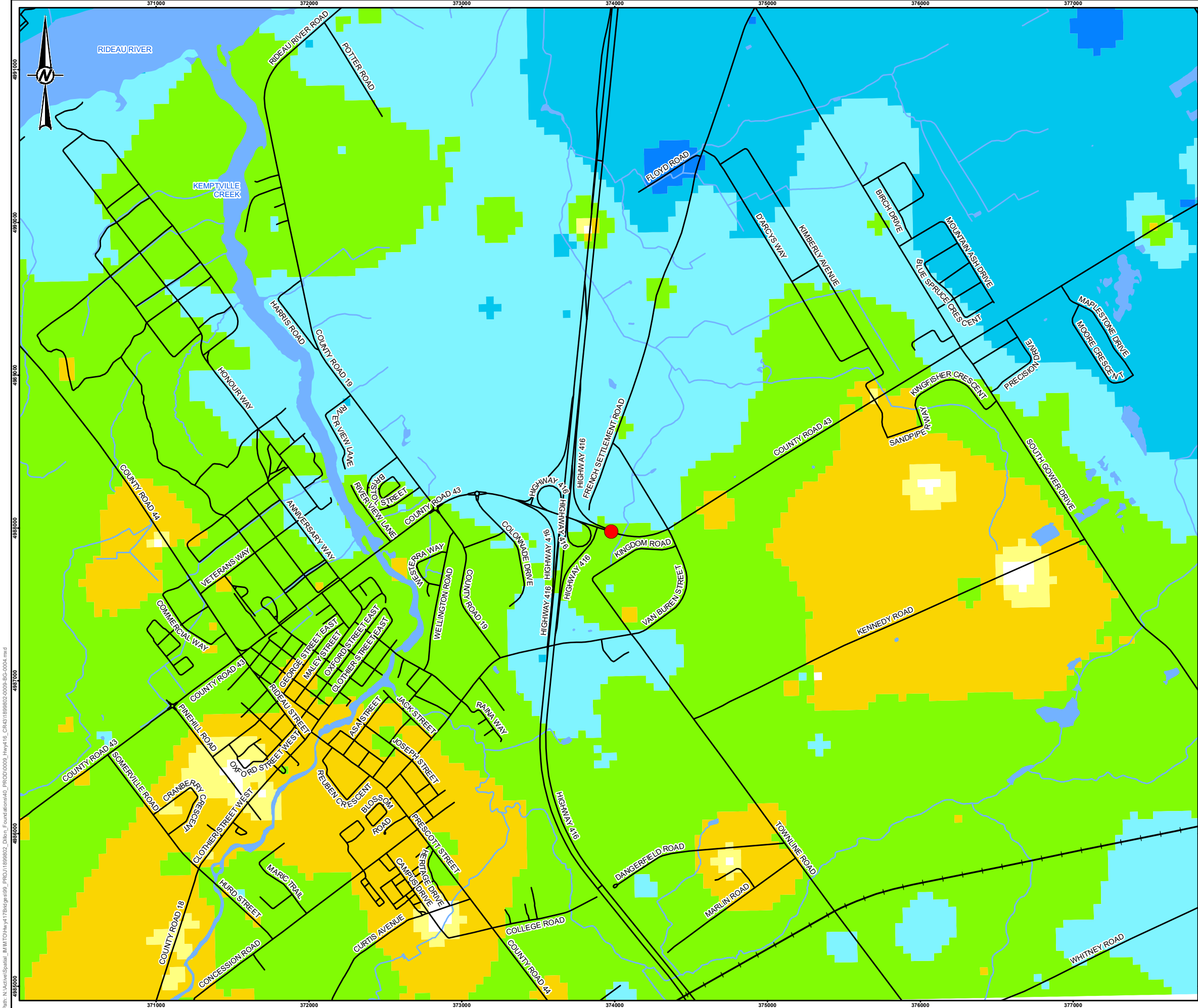
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

REFERENCE

Base plans provided in digital format by Dillon, drawing file no. 4017-E-0039-2.dwg, received 12 16, 2020.



NO.	DATE	BY	REVISION
Geocres No. 31G-286			
HWY. 416	PROJECT NO. 1899802		DIST. EASTERN
SUBM'D. KCP	CHKD. KCP	DATE: 5/13/2021	SITE: 19-318
DRAWN: ZS	CHKD. WC	APPD. WC	DWG. 2



SCALE 1:425,000

LEGEND

- SITE LOCATION
- ROADWAY
- RAILWAY
- WATERCOURSE
- WATERBODY

TREND IN DEPTH TO BEDROCK (METRES)

- 1 to 2
- 2 to 3
- 3 to 5
- 5 to 10
- 10 to 15
- 15 to 25
- 25 to 50

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)
1. 2010 BÉLANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020
3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28



CLIENT
MTO

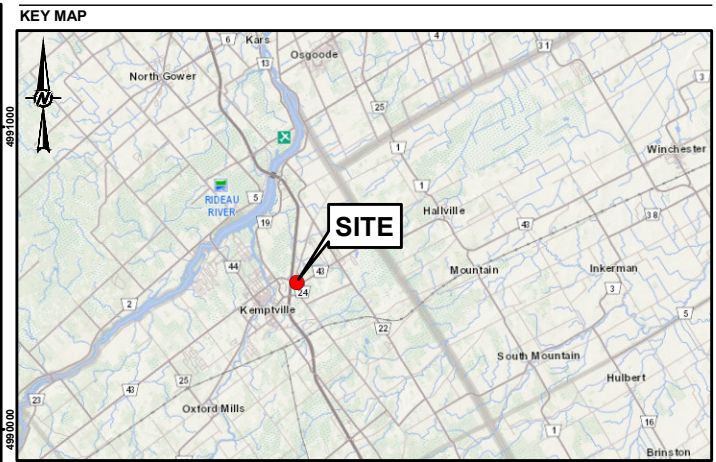
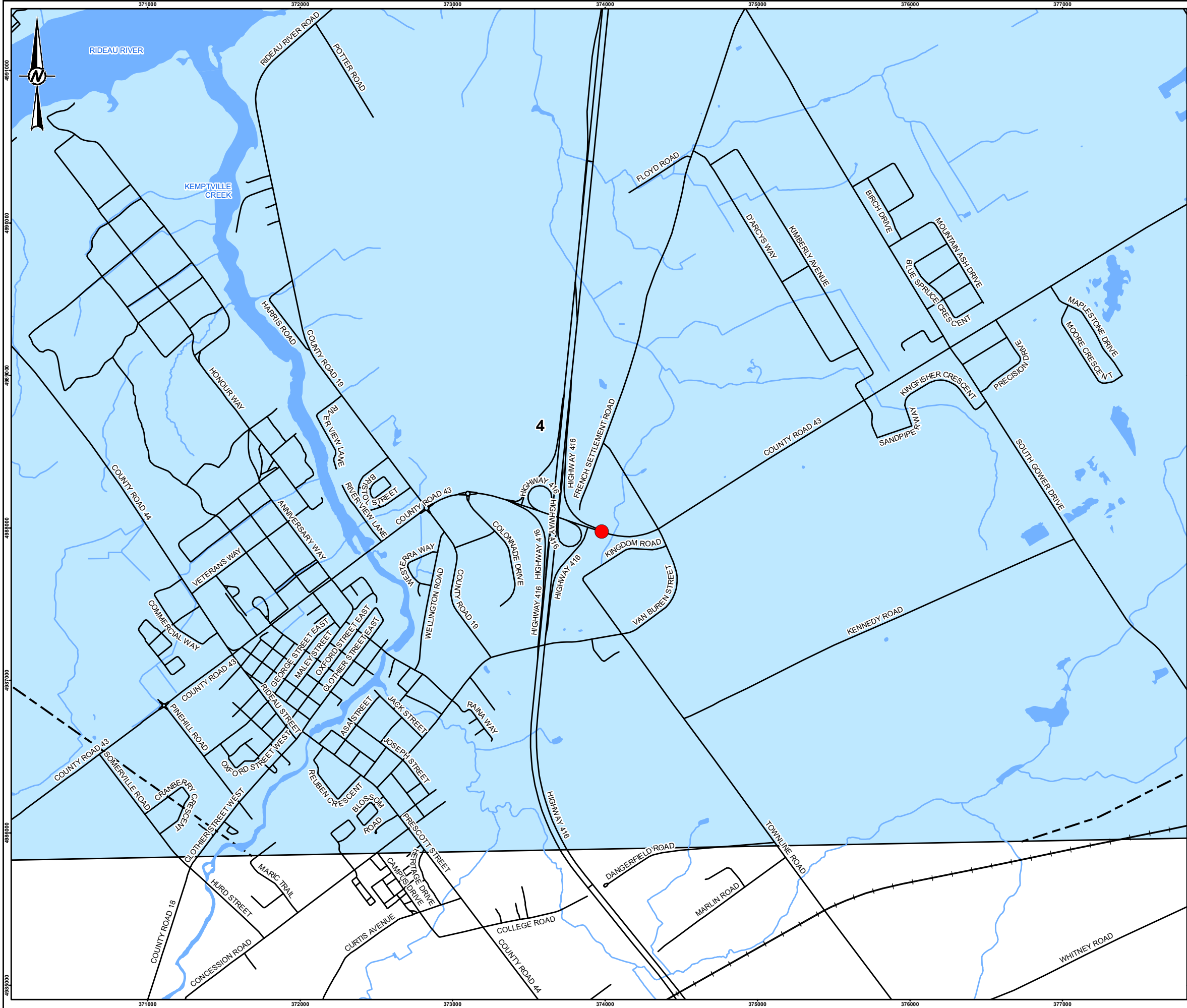
PROJECT
EAST APPROACH EMBANKMENT HIGHWAY 416 UNDERPASS AT COUNTY ROAD 43

TITLE
DEPTH TO BEDROCK SURFACE

CONSULTANT	YYYY-MM-DD	2021-05-10
DESIGNED	KCP	
PREPARED	BR	
REVIEWED	KCP	
APPROVED	WC	

GOLDER

PROJECT NO. 1899802	CONTROL 0009	REV. 0	FIGURE 4
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SCALE 1:425,000

LEGEND

- SITE LOCATION
- ROADWAY
- + RAILWAY
- WATERCOURSE
- WATERBODY
- FAULT
- 4. OXFORD FORMATION: SUBLITHOGRAPHIC TO FINE CRYSTALLINE DOLOSTONE

NOTE(S)
1. ALL LOCATIONS ARE APPROXIMATE

REFERENCE(S)
1. BÉLANGER, J. R., URBAN GEOLOGY OF THE NATIONAL CAPITAL AREA, GEOLOGICAL SURVEY OF CANADA, OPEN FILE D3256, 2001.
2. LAND INFORMATION ONTARIO (LIO) DATA PRODUCED BY GOLDER ASSOCIATES LTD. UNDER LICENCE FROM ONTARIO MINISTRY OF NATURAL RESOURCES, © QUEENS PRINTER 2020
3. PROJECTION: TRANSVERSE MERCATOR, DATUM: NAD 83, COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28




CLIENT
MTO

PROJECT
EAST APPROACH EMBANKMENT HIGHWAY 416 UNDERPASS AT COUNTY ROAD 43

TITLE
BEDROCK GEOLOGY

CONSULTANT	YYYY-MM-DD	2021-05-10
	DESIGNED	KCP
	PREPARED	BR
	REVIEWED	KCP
	APPROVED	WC

 **GOLDER**

PROJECT NO. 1899802	CONTROL 0009	REV. 0	FIGURE 5
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APPENDIX A

Lists of Abbreviations and Symbols
Current Record of Boreholes 20-01 to 20-06

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

MINISTRY OF TRANSPORTATION, ONTARIO

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>200	>8
COBBLES	Not Applicable	75 to 200	3 to 8
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
FINES	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY COMPONENTS^{1,2}

Percentage by Mass	Modifier
> 35	Use 'and' to combine primary and secondary component (<i>i.e.</i> , SAND and gravel)
> 20 to 35	Primary soil name prefixed with "gravelly, sandy" as applicable
> 10 to 20	some (<i>i.e.</i> , some sand)
≤ 10	trace (<i>i.e.</i> , trace fines)

1. Only applicable to components not described by Primary Group Name.

2. Classification of Primary Group Name based on Unified Soil Classification System (ASTM D2487) for coarse-grained soils; fine-grained soils described per current MTO Soil Classification System.

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)

An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (*q_t*), porewater pressure (*u*) and sleeve friction (*f_s*) are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); N_d:

The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC / SC	Rock core / Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample
OD / ID	Outer Diameter / Inner Diameter
HSA / SSA	Hollow-Stem Augers / Solid-Stem Augers

SOIL TESTS

w	water content
PL, w _p	plastic limit
LL, w _L	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G _s)
DS	direct shear test
GS	specific gravity
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V (FV)	field vane (LV-laboratory vane test)
Y	unit weight

1. Tests anisotropically consolidated prior to shear are shown as CAD, CAU.

COARSE-GRAINED SOILS

Compactness¹

Term	SPT 'N' (blows/0.3m) ²
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

- Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grain size. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.
- SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.

FINE-GRAINED SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N' ^{1,2} (blows/0.3m)
Very Soft	< 12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	> 200	> 30

- SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
- SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

LIST OF SYMBOLS

MINISTRY OF TRANSPORTATION, ONTARIO

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
$\ln x$	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)

σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_L or LL	liquid limit
w_P or PL	plastic limit
I_P or PI	plasticity index $= (w_L - w_P)$
NP	non-plastic
w_s	shrinkage limit
I_L	liquidity index $= (w - w_P) / I_P$
I_c	consistency index $= (w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
$C_{\alpha(e)}$	secondary compression index
C_{α}	rate of secondary compression
$C_{\alpha(e)}$	modified secondary compression index
m_v	coefficient of volume change
C_v	coefficient of consolidation (vertical direction)
C_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
c'	effective cohesion
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q or q'	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity


* Density symbol is ρ . Unit weight symbol is γ .
where $\gamma = \rho \cdot g$ (i.e., mass density multiplied by
acceleration due to gravity)

Notes: 1
2

$\tau = c' + \sigma' \tan \phi'$
shear strength = (compressive strength)/2

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTO\HWY417\BRIDGES\02_DATA\GINT\1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS

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

PROJECT 1899802-19000			RECORD OF BOREHOLE No 20-01			SHEET 2 OF 2			METRIC								
G.W.P. 4129-18-00			LOCATION N 4987977.8; E 373996.3 NAD 83 MTM ZONE 9 (LAT. 45.028418; LONG. -75.621876)			ORIGINATED BY JS											
DIST Eastern HWY 416/CR43			BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)			COMPILED BY ZS											
DATUM Geodetic			DATE November 24, 2020			CHECKED BY KCP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
84.7	(CH) CLAY Firm to stiff Grey w=LL		13	TO	-												
11.4	(CL) CLAYEY SILT Firm Grey w>LL		14	SS	WH												
82.1			15	TO	-												
14.0	End of Borehole																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTOWHY417BRIDGES02_DATA\GINTV1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS




N:\ACTIVE\SPATIAL IM\MT0\HWY417BRIDGES\02 DATA\GINT\1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS

Continued Next Page

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

PROJECT <u>1899802-19000</u>		RECORD OF BOREHOLE No 20-02		SHEET 2 OF 2		METRIC	
G.W.P. <u>4129-18-00</u>		LOCATION <u>N 4987967.1; E 374030.0 NAD 83 MTM ZONE 9 (LAT. 45.028318; LONG. -75.621450)</u>		ORIGINATED BY <u>JS</u>			
DIST <u>Eastern</u> HWY <u>416/CR43</u>		BOREHOLE TYPE <u>Power Auger, 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>November 30, 2020</u>		CHECKED BY <u>KCP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	w _p	w		w _L			
	--- CONTINUED FROM PREVIOUS PAGE ---																			
	(CH) Clay Firm Grey W>LL		12	TP	-															
								84	×		+									
				13	SS	-														
							82	×		+										
81.2																				
13.7	(SM) SILTY SAND, some clay, some gravel (TILL) Compact Brown-grey Wet		14	SS	11		81													
80.6																				
14.3	End of Borehole																			

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITO\HWY417BRIDGES\02_DATA\GINTV1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS

PROJECT		1899802-19000		RECORD OF BOREHOLE No 20-03		SHEET 1 OF 2		METRIC								
G.W.P.		4129-18-00		LOCATION		N 4987993.6; E 374004.6 NAD 83 MTM ZONE 9 (LAT. 45.028559; LONG. -75.621769)		ORIGINATED BY JS								
DIST		Eastern HWY 416/CR43		BOREHOLE TYPE		Wash Boring, BW Casing, Portable Rotary Drill, AW Casing		COMPILED BY ZS								
DATUM		Geodetic		DATE		November 28, 2020		CHECKED BY KCP								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
91.3	GROUND SURFACE															
0.0	(SP) Sand, some silt, organics present (TOPSOIL)															
0.2	Loose Dark brown Moist to wet		1	SS	4											
90.7	(SP) Sand (FILL)															
0.6	Loose Light brown Wet		2	SS	11											
	(SM) SAND, some silt Loose to compact Light brown Wet		3	SS	21											
89.1			4	SS	8											
2.2	(CH) CLAY (WEATHERED CRUST) Very stiff Grey w=LL		5	SS	3											
			6	SS	2											
87.6																
3.7	(CH) CLAY Soft to firm Grey w=LL															
			7	TO	-											
			8	SS	2											
			9	TO	-											
82.5																
8.8	(CL) CLAYEY SILT Firm Grey w>LL		10	SS	2											
81.6																
9.7			11	SS	35											

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTD\HWY417\BRIDGES\02_DATA\GINTV1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS



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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417BRIDGES\02 DATA\GINT\1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS



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

PROJECT		1899802-19000		RECORD OF BOREHOLE No 20-05		SHEET 1 OF 2		METRIC										
G.W.P.		4129-18-00		LOCATION		N 4987965.5; E 373998.6 NAD 83 MTM ZONE 9 (LAT. 45.028306; LONG. -75.621849)		ORIGINATED BY JS										
DIST		Eastern HWY 416/CR43		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY ZS										
DATUM		Geodetic		DATE		December 3, 2020		CHECKED BY KCP										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	25 50 75	kN/m ³			
96.1	0.0	GROUND SURFACE																
0.1		ASPHALTIC CONCRETE																
		(SW/GW) Sand and gravel, some silt (PAVEMENT STRUCTURE) (FILL)		1	AS	-												
		Compact Brown Moist		2	SS	50/0.00												
95.1	1.0	(SM/ML) Sand and silt, some gravel, contains cobbles and boulders (FILL) Very dense Brown moist		3	SS	124												13 39 (48)
				4	SS	55												
				5	SS	76												
				6	SS	55												
				7	SS	71												
90.9	5.2	(SM) Silty sand, some gravel, trace clay, contains cobbles and boulders (FILL) Compact Brown Moist		8	SS	29												
				9	SS	30												
89.7	6.4	(SM) SAND, some silt Compact to loose Brown to grey Wet		10	SS	7												
				11	SS	4												
88.8	7.3	(CH) CLAY (WEATHERED CRUST) Very stiff Brown and grey, mottled w>PL																0 1 39 60
87.9	8.2	(CH) CLAY Stiff to firm Grey w=LL																
				12	TP	-												

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\TOHWY417BRIDGES\02_DATA\GINTV1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS



PROJECT <u>1899802-19000</u>		RECORD OF BOREHOLE No 20-05		SHEET 2 OF 2		METRIC	
G.W.P. <u>4129-18-00</u>		LOCATION <u>N 4987965.5; E 373998.6 NAD 83 MTM ZONE 9 (LAT. 45.028306; LONG. -75.621849)</u>		ORIGINATED BY <u>JS</u>			
DIST <u>Eastern</u> HWY <u>416/CR43</u>		BOREHOLE TYPE <u>Power Auger, 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>December 3, 2020</u>		CHECKED BY <u>KCP</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _P	W	W _L		GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\IMTO\HWY417\BRIDGES\02_DATA\GINTV1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS

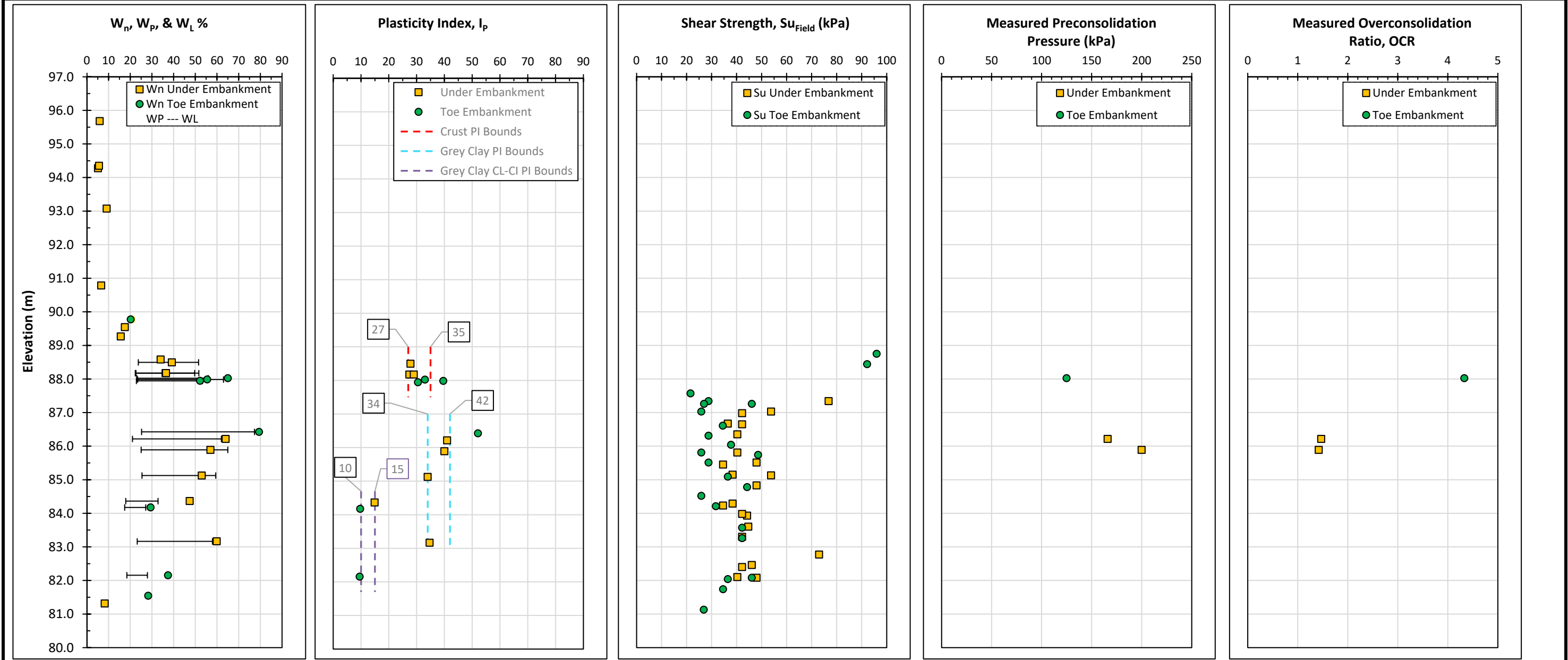
+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT		RECORD OF BOREHOLE No 20-06				SHEET 2 OF 2		METRIC									
1899802-19000		G.W.P. 4129-18-00		LOCATION N 4987944.0; E 373993.8 NAD 83 MTM ZONE 9 (LAT. 45.028113; LONG. -75.621913)		ORIGINATED BY JS											
DIST Eastern		HWY 416/CR43		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY ZS											
DATUM Geodetic		DATE December 5, 2020				CHECKED BY KCP											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
81.4	(CL) CLAYEY SILT Firm Grey w>LL						82	X	+								
10.7	(SM/GM) SILTY SAND and GRAVEL, contains cobbles and boulders (TILL) Loose Grey Wet		10	SS	7		81	X	+								
80.8	End of Borehole																
11.3	NOTE(S): 1. Water level in well screen at a depth of 0.2 m below ground surface (Elev. 91.9 m), measured on April 13, 2021.																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITO\HWY417BRIDGES\02_DATA\GINTV1899802\1899802.GPJ GAL-GTA.GDT 5/10/21 ZS

APPENDIX B

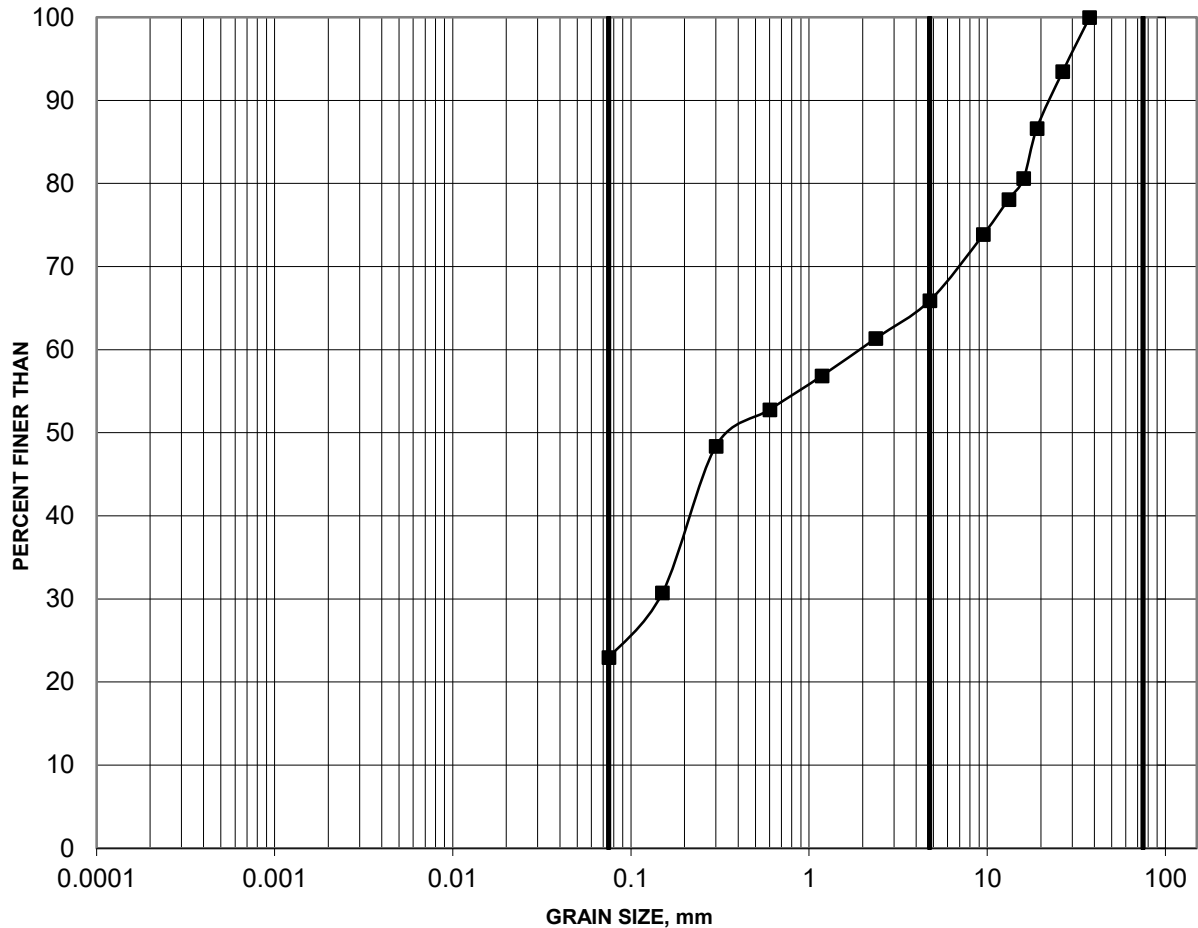
Current Laboratory Test Results
Figure A - Measured Engineering Properties
Figures B1 to B13



GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY SAND AND GRAVEL (FILL)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-01	1B	0.23-0.61	34	43	23	

Project: 1899802-19000



Created by: KCP

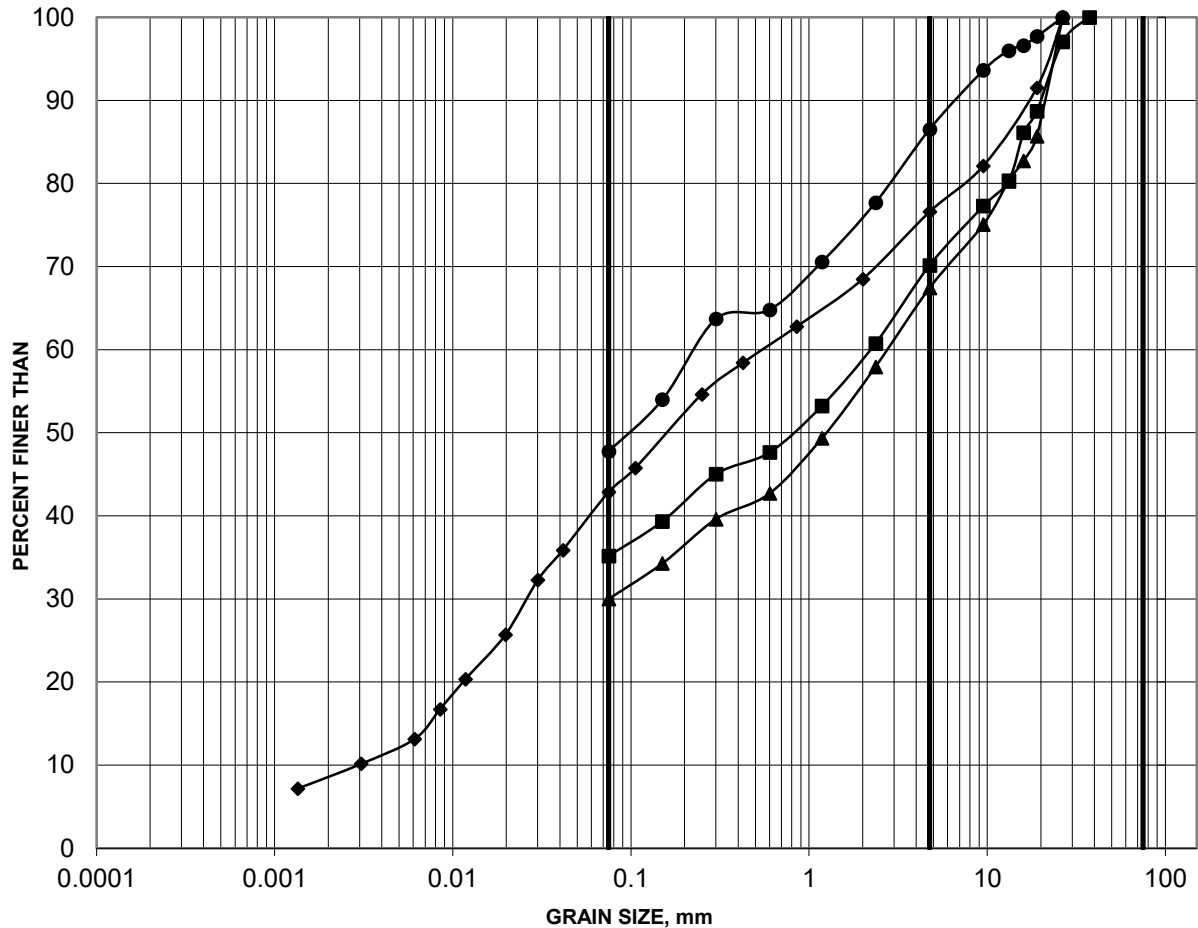
Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND AND SILT (FILL)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-01	3	1.52-2.13	30	35		35
◆	20-02	3	1.52-2.13	23	34	34	9
▲	20-02	6	3.81-4.42	33	37		30
●	20-05	3	1.52-1.98	13	39		48

Project: 1899802-19000



Created by: KCP

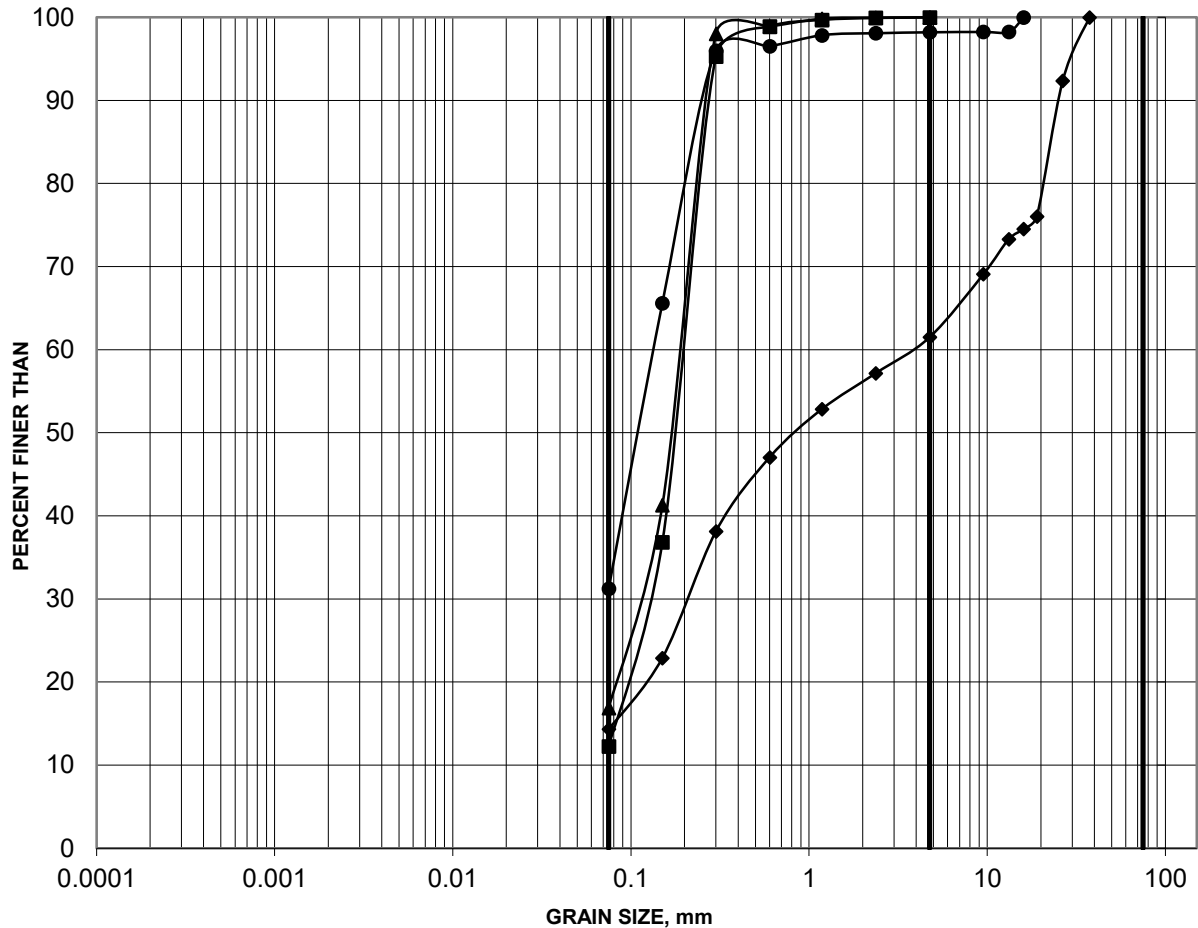
Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND TO SAND AND GRAVEL



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-01	9B	6.40-6.71	0	88	12	
◆	20-02	8	5.33-5.94	38	48	14	
▲	20-03	2	1.22-1.83	0	83	17	
●	20-04	3	1.22-1.83	2	67	31	

Project: 1899802-19000



Created by: KCP

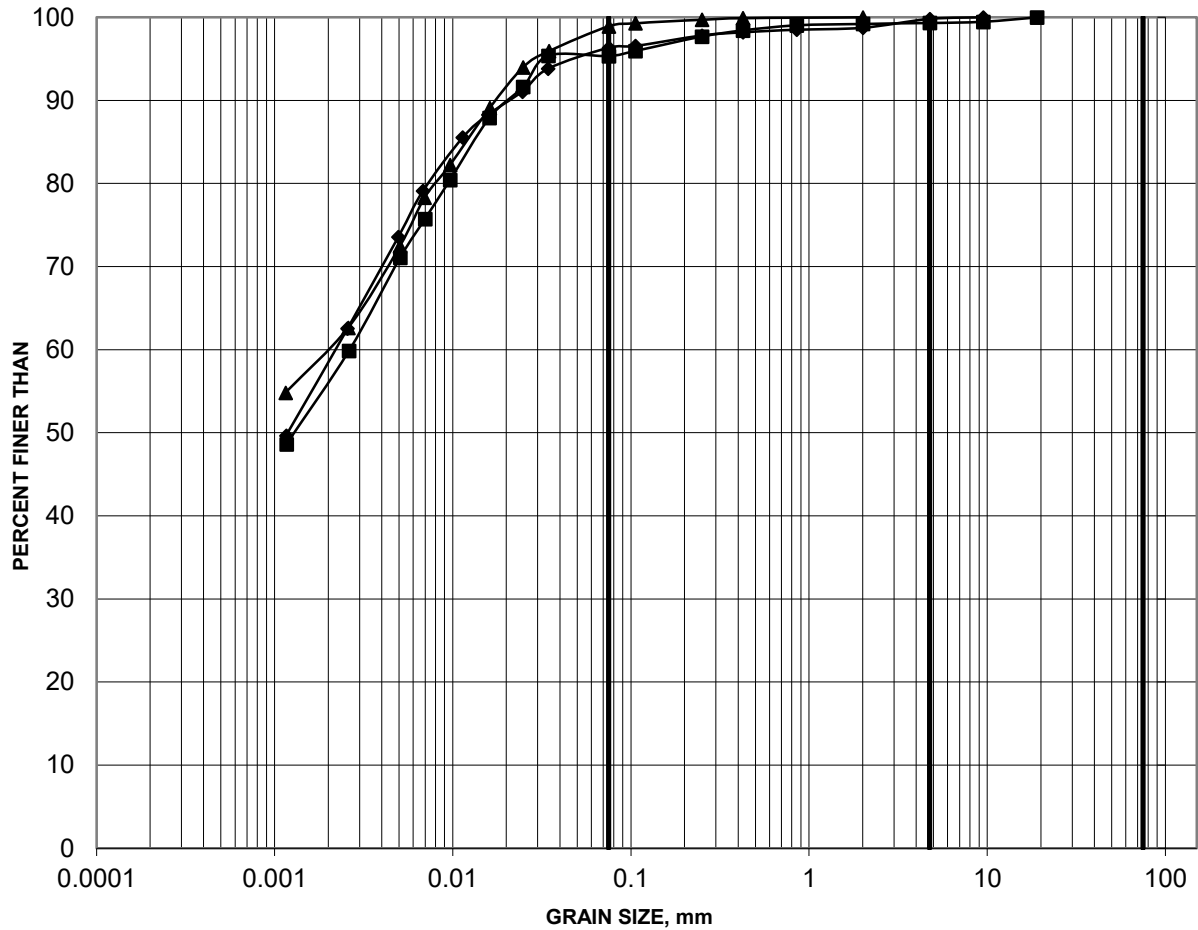
Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>

GRAIN SIZE DISTRIBUTION

FIGURE B4

WEATHERED CLAY CRUST



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-01	11	7.62-8.23	1	4	39	56
◆	20-03	6	3.05-3.66	0	4	37	59
▲	20-05	10B	7.35-7.65	0	1	39	60

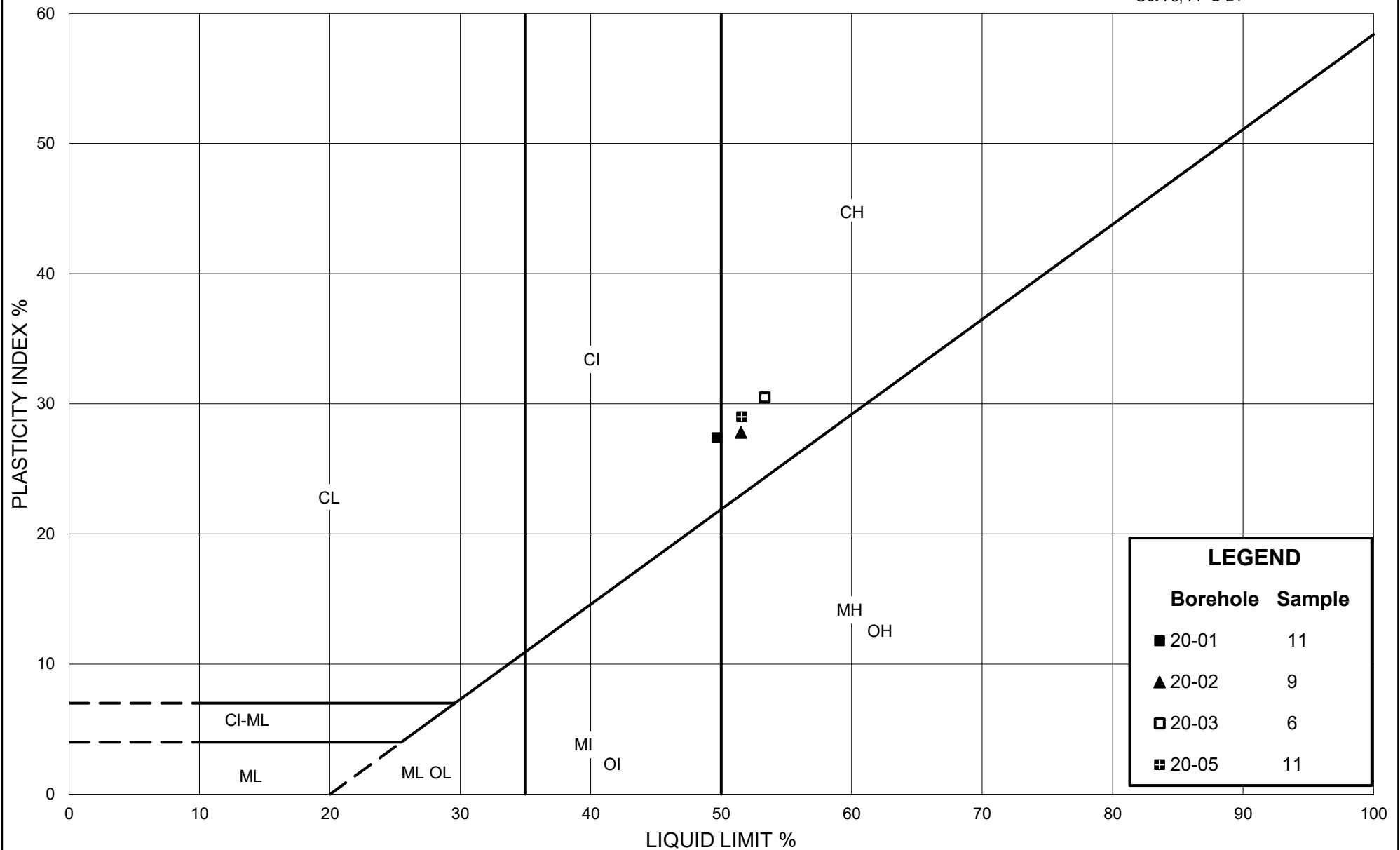
Project: 1899802-19000



Created by: KCP

Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>



Ministry of Transportation

PLASTICITY CHART WEATHERED CLAY CRUST

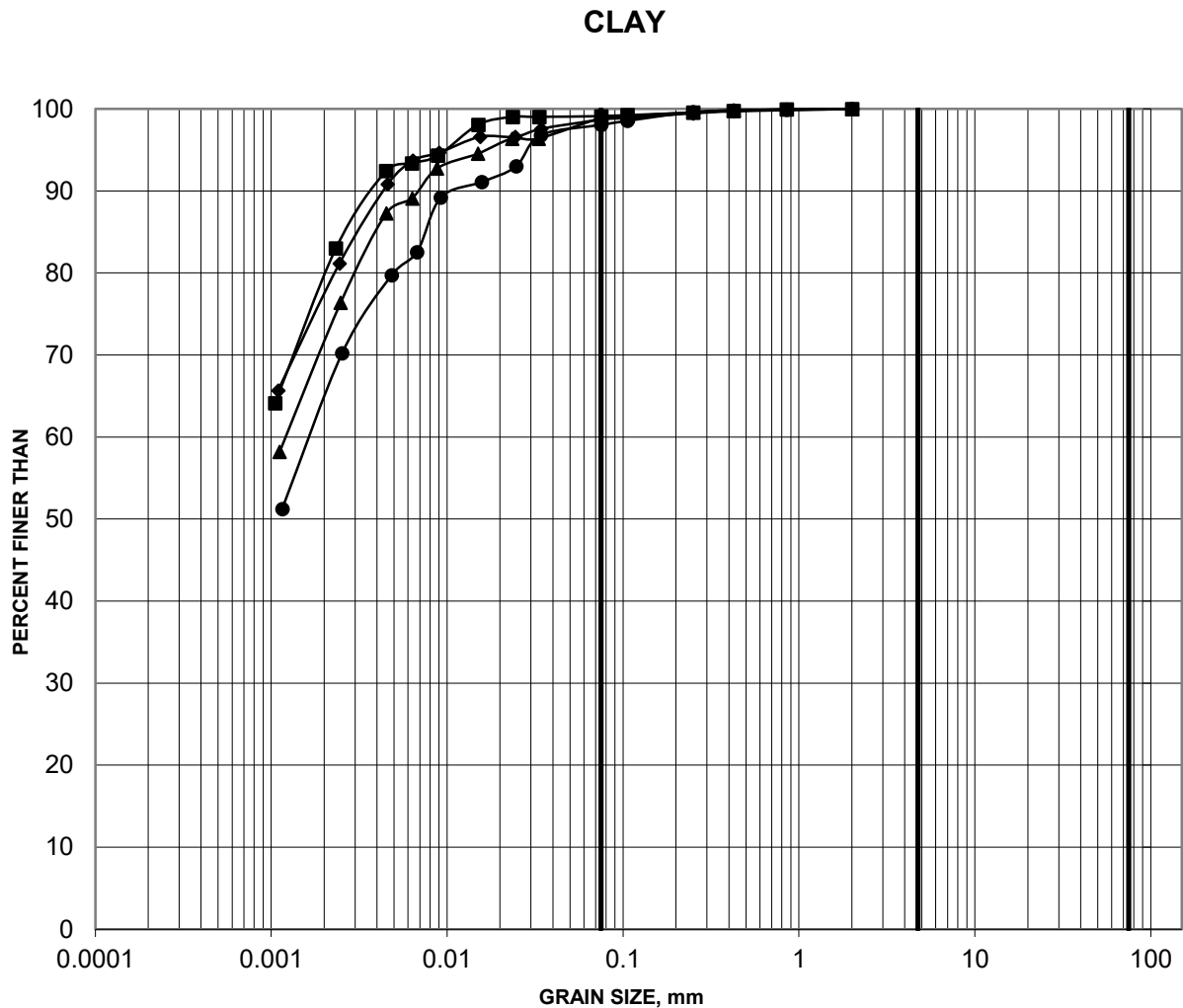
Figure: B5

Project: 1899802/19000

Created By: KCP Checked By: MI

GRAIN SIZE DISTRIBUTION

FIGURE B6



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-02	11	8.38-8.99	0	1	20	79
◆	20-04	7	4.57-5.18	0	1	22	77
▲	20-05	13	10.67-11.28	0	1	27	72
●	20-06	5	3.81-4.42	0	2	33	65

Project: 1899802-19000

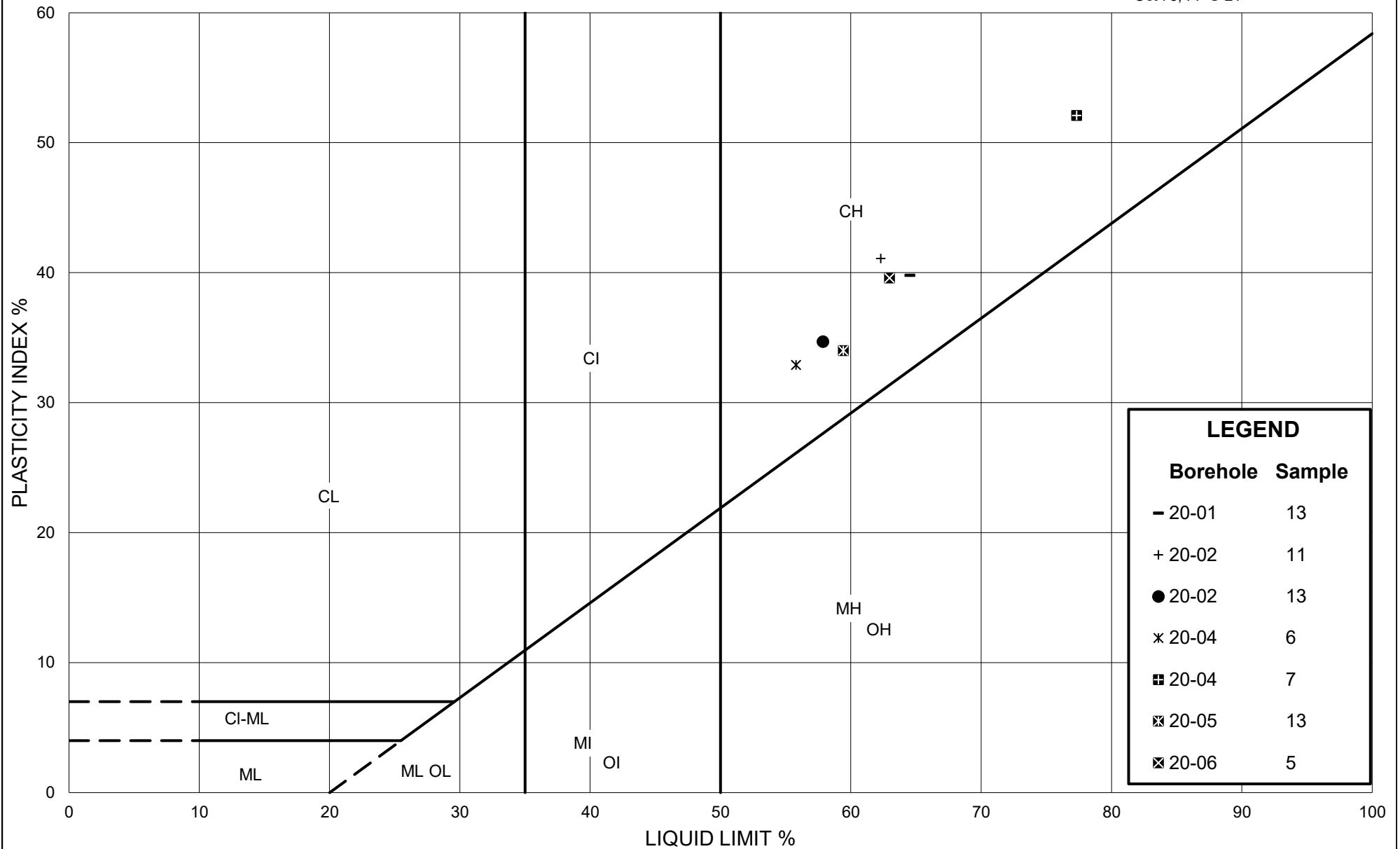


Created by: KCP

Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>

Oct 75, FF-S-21



Ministry of Transportation

PLASTICITY CHART CLAY

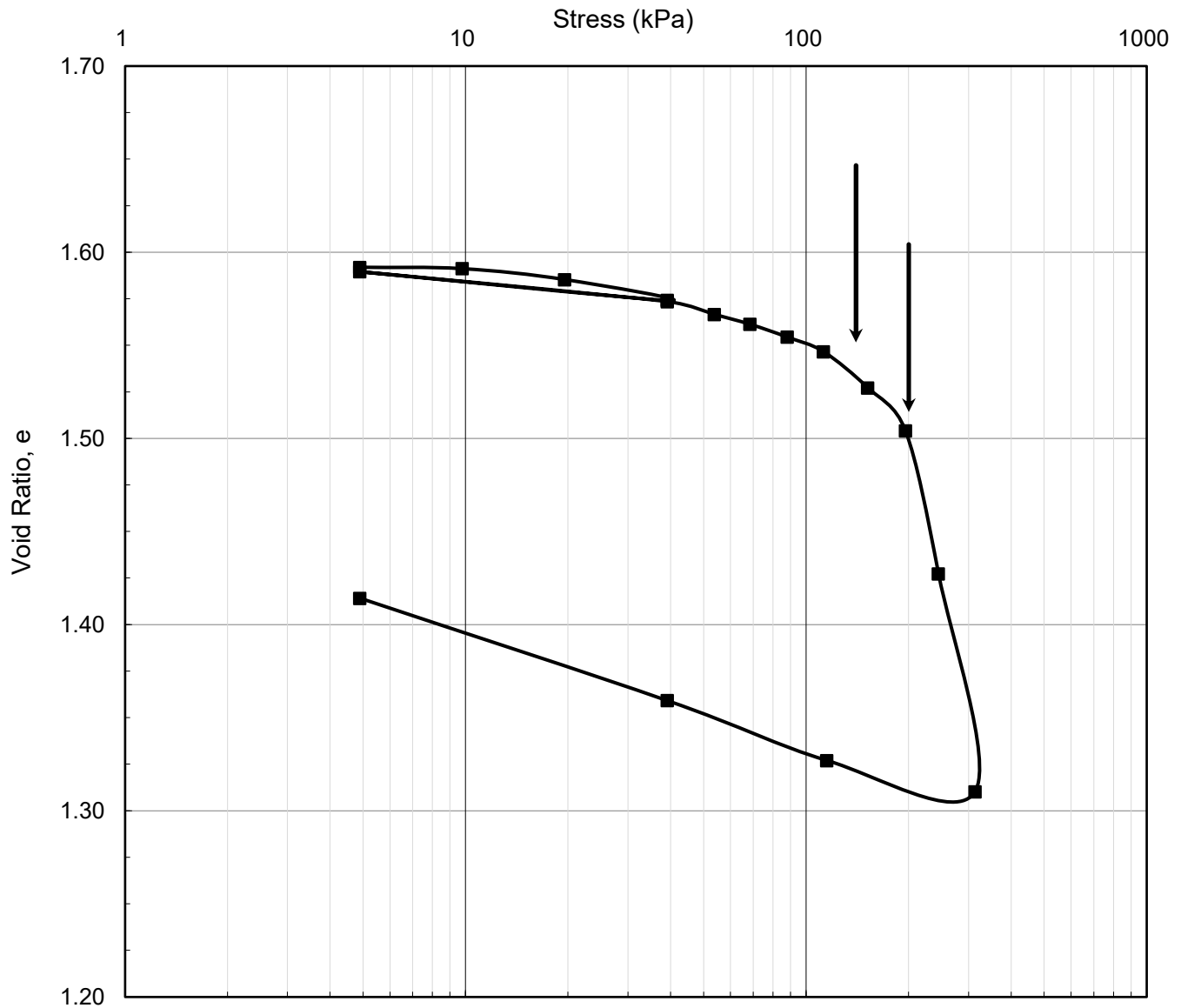
Figure: B7

Project: 1899802/19000

Created By: KCP Checked By: MI

$\sigma'_{vo} = 140 \text{ kPa}$
CALCULATED EXISTING EFFECTIVE
OVERBURDEN PRESSURE

$\sigma'_p = 200 \text{ kPa}$
MOST PROBABLE APPARENT
PRECONSOLIDATION PRESSURE



LEGEND

Borehole:	20-01	$w_i =$	57%	$S_o =$	100%	$\gamma =$	16.6 kN/m ³
Sample:	13	$w_f =$	52%	$e_o =$	1.59	$G_s =$	2.80
Depth (m):	10.36	$w_l =$	65%	$C_c =$	0.790		
Elevation (m):	96.1	$w_p =$	25%	$C_r =$	0.180		



GOLDER

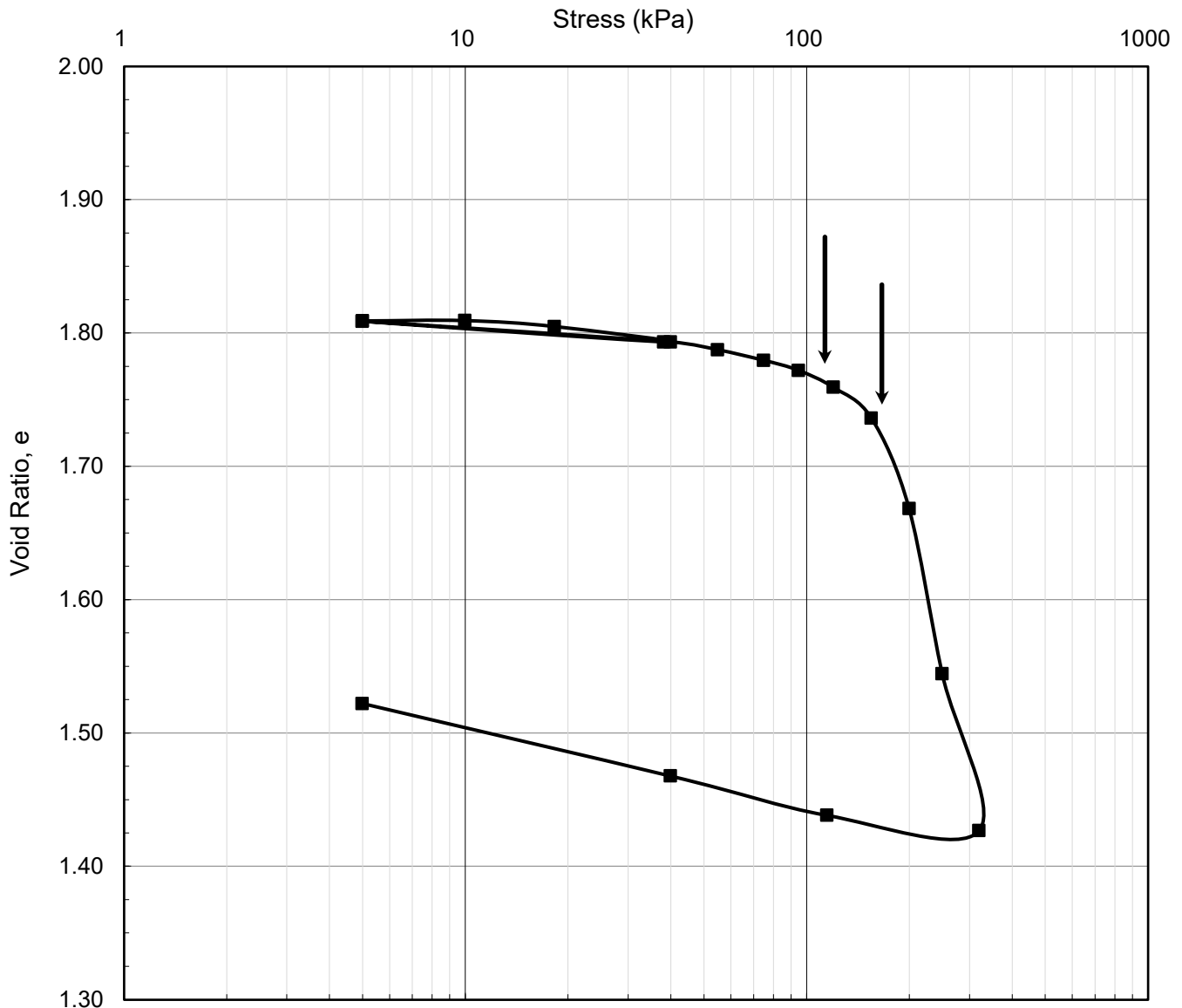
Consolidation Test Results

Void Ratio vs Stress

Figure No.	B8
Revision	1
Project No.	1899802/19000
Created By	KCP
Checked By	MI

$\sigma'_{vo} = 113 \text{ kPa}$
CALCULATED EXISTING EFFECTIVE
OVERBURDEN PRESSURE

$\sigma'_p = 166 \text{ kPa}$
MOST PROBABLE APPARENT
PRECONSOLIDATION PRESSURE



LEGEND

Borehole:	20-02	$w_i =$	64%	$S_o =$	100%	$\gamma =$	16.1 kN/m ³
Sample:	11	$w_f =$	55%	$e_o =$	1.81	$G_s =$	2.81
Depth (m):	8.86	$w_l =$	62%	$C_c =$	1.270		
Elevation (m):	94.9	$w_p =$	21%	$C_r =$	0.013		



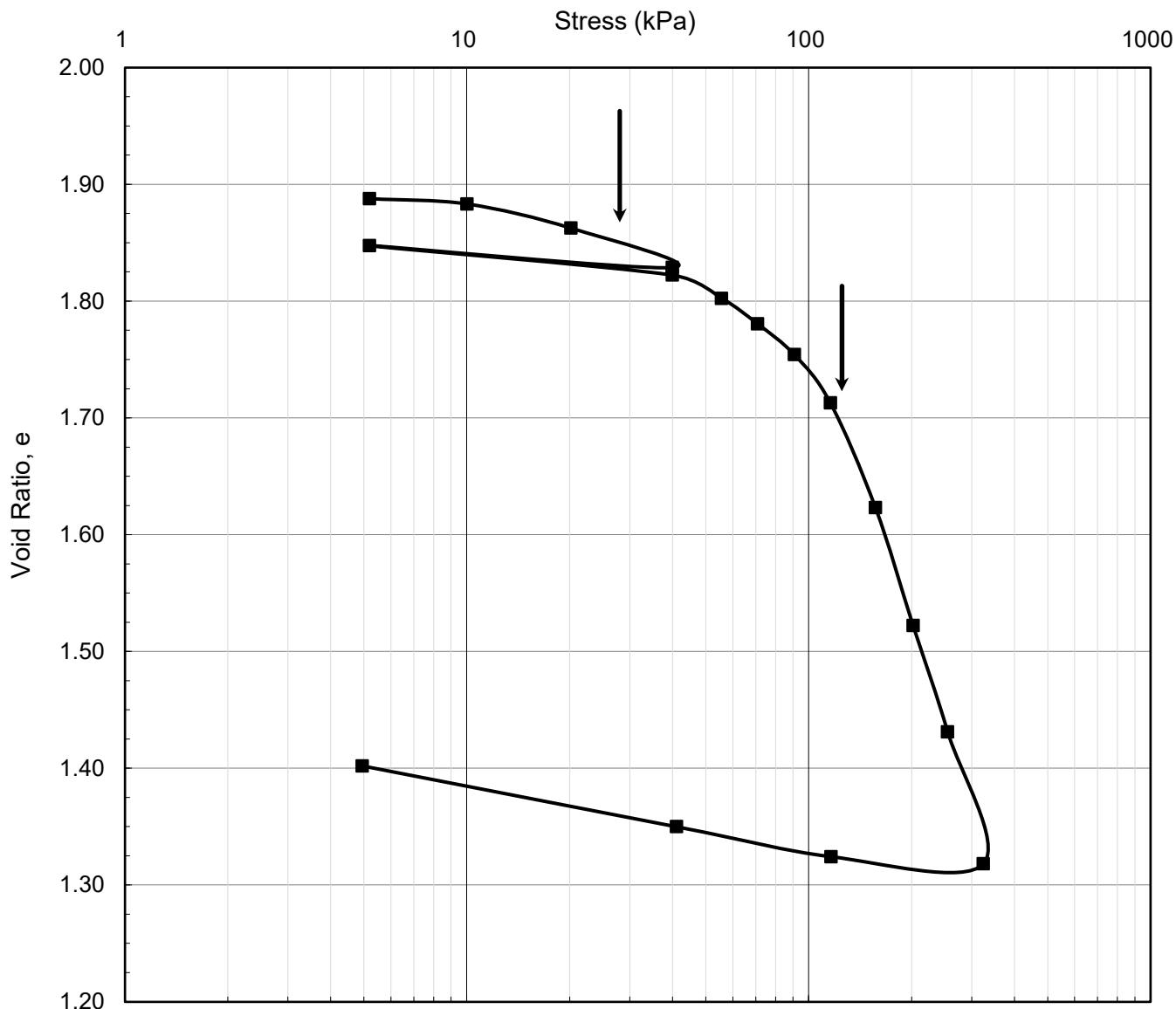
Consolidation Test Results

Void Ratio vs Stress

Figure No.	B9
Revision	1
Project No.	1899802/19000
Created By	KCP
Checked By	MI

$\sigma'_{vo} = 28 \text{ kPa}$
CALCULATED EXISTING EFFECTIVE
OVERBURDEN PRESSURE

$\sigma'_p = 125 \text{ kPa}$
MOST PROBABLE APPARENT
PRECONSOLIDATION PRESSURE



Consolidation Test Results

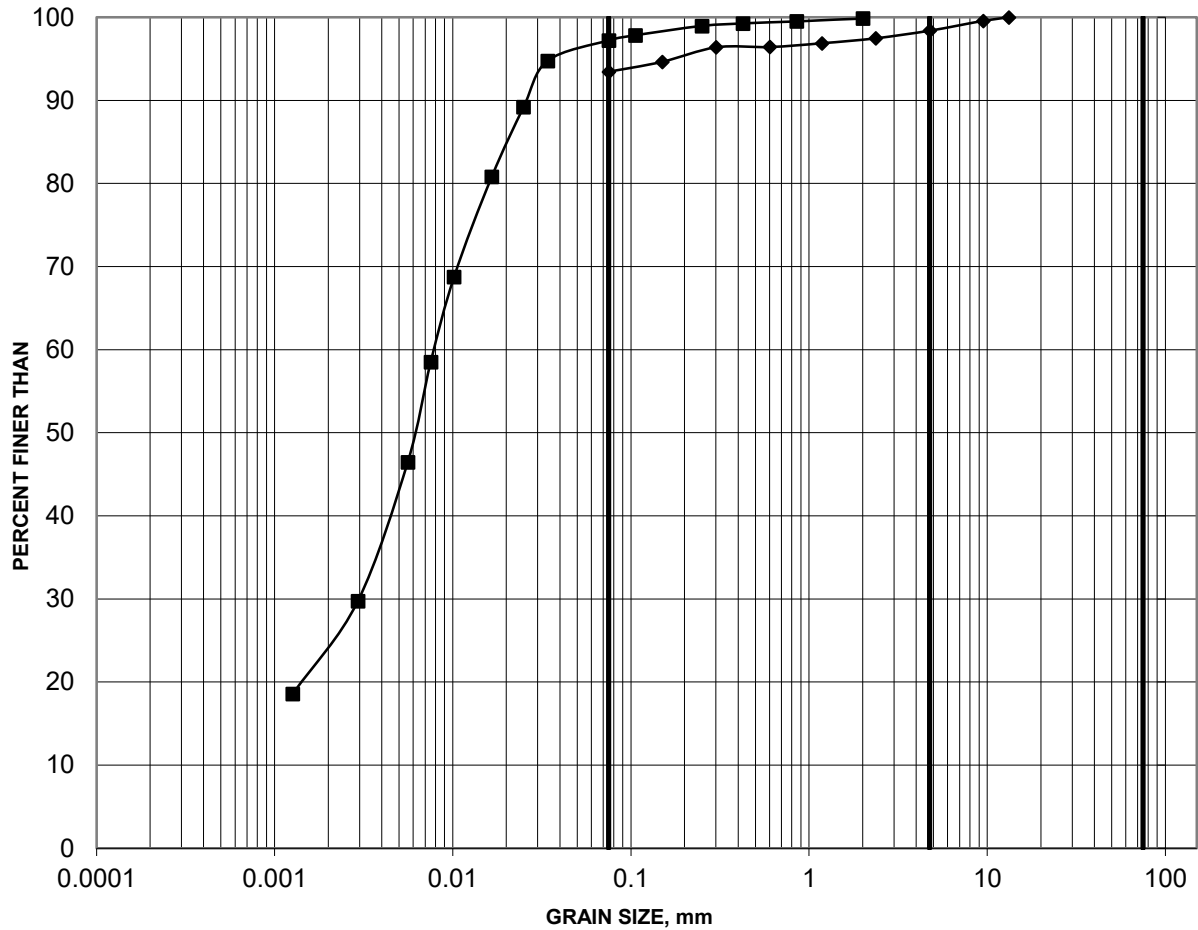
Void Ratio vs Stress

Figure No.	B10
Revision	1
Project No.	1899802/19000
Created By	KCP
Checked By	MI

GRAIN SIZE DISTRIBUTION

FIGURE B11

CLAYEY SILT



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-03	10	8.84-9.45	0	3	72	25
◆	20-03	11	9.45-10.06	2	5	93	

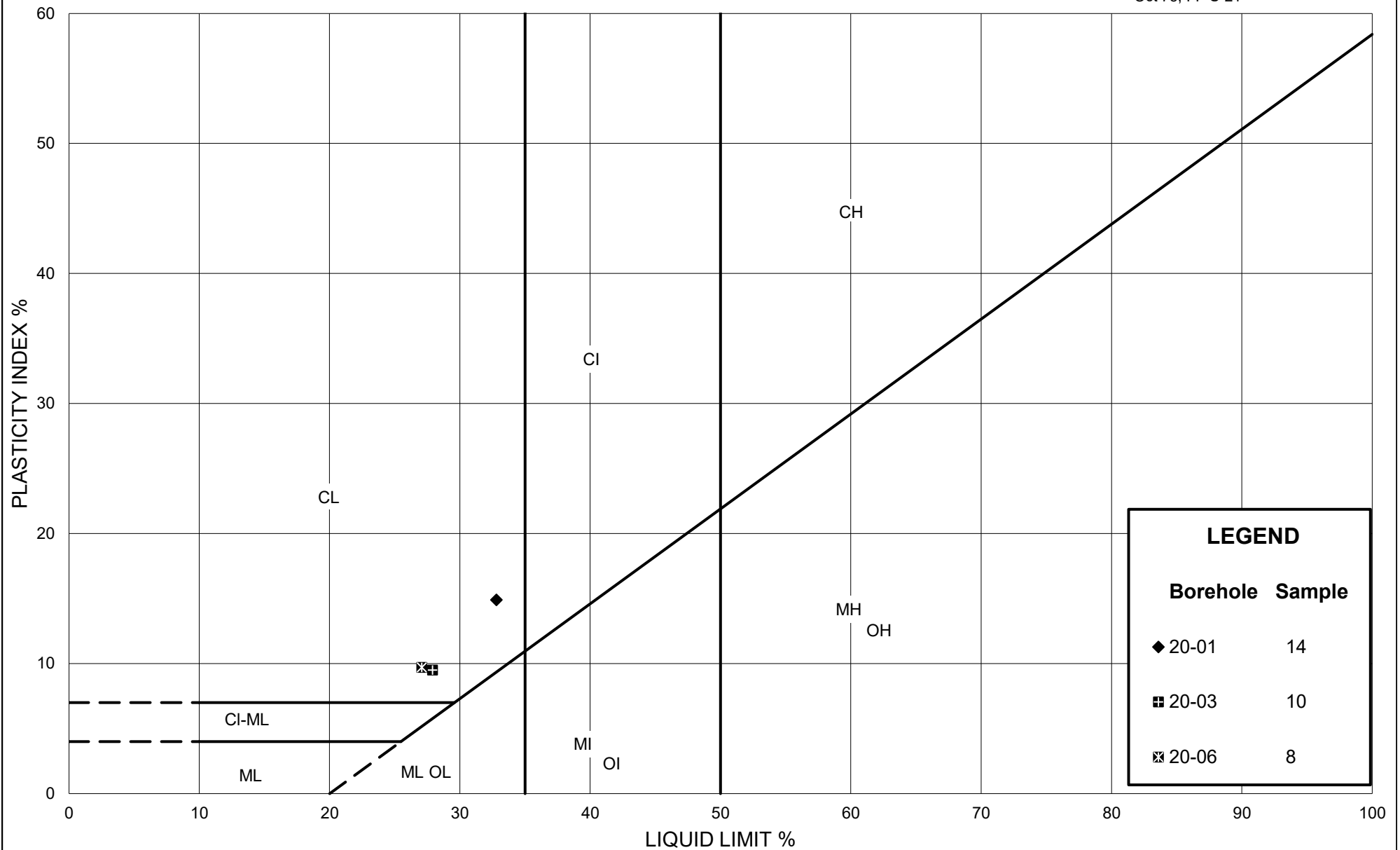
Project: 1899802-19000



Created by: KCP

Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>



Ministry of Transportation

PLASTICITY CHART

CLAYEY SILT

Figure: B12

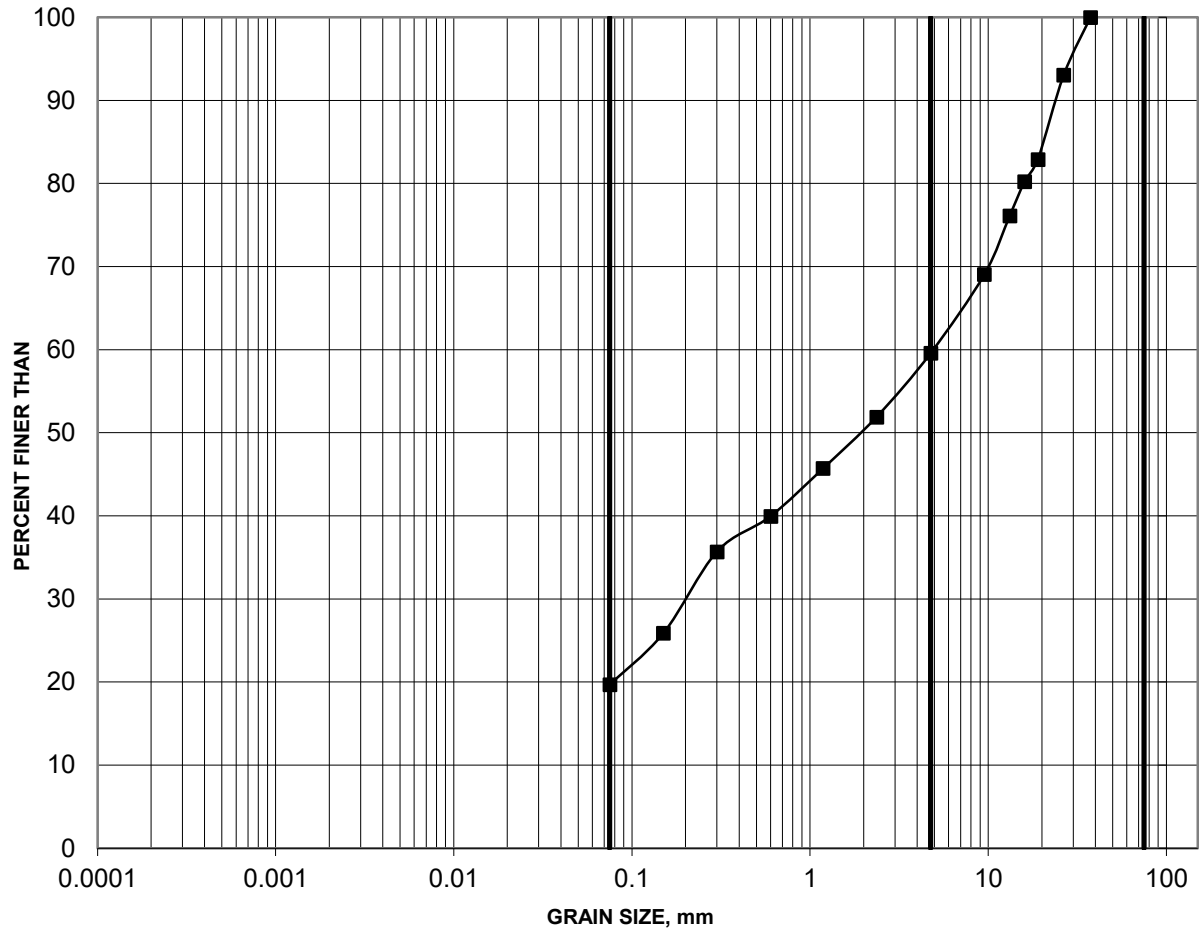
Project: 1899802/19000

Created By: KCP Checked By: MI

GRAIN SIZE DISTRIBUTION

FIGURE B13

SILTY SAND AND GRAVEL (GLACIAL TILL)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

	Borehole	Sample	Depth (m)	Constituents (%)			
				Gravel	Sand	Silt	Clay
■	20-05	15	14.48-15.09	40	40	20	

Project: 1899802-19000



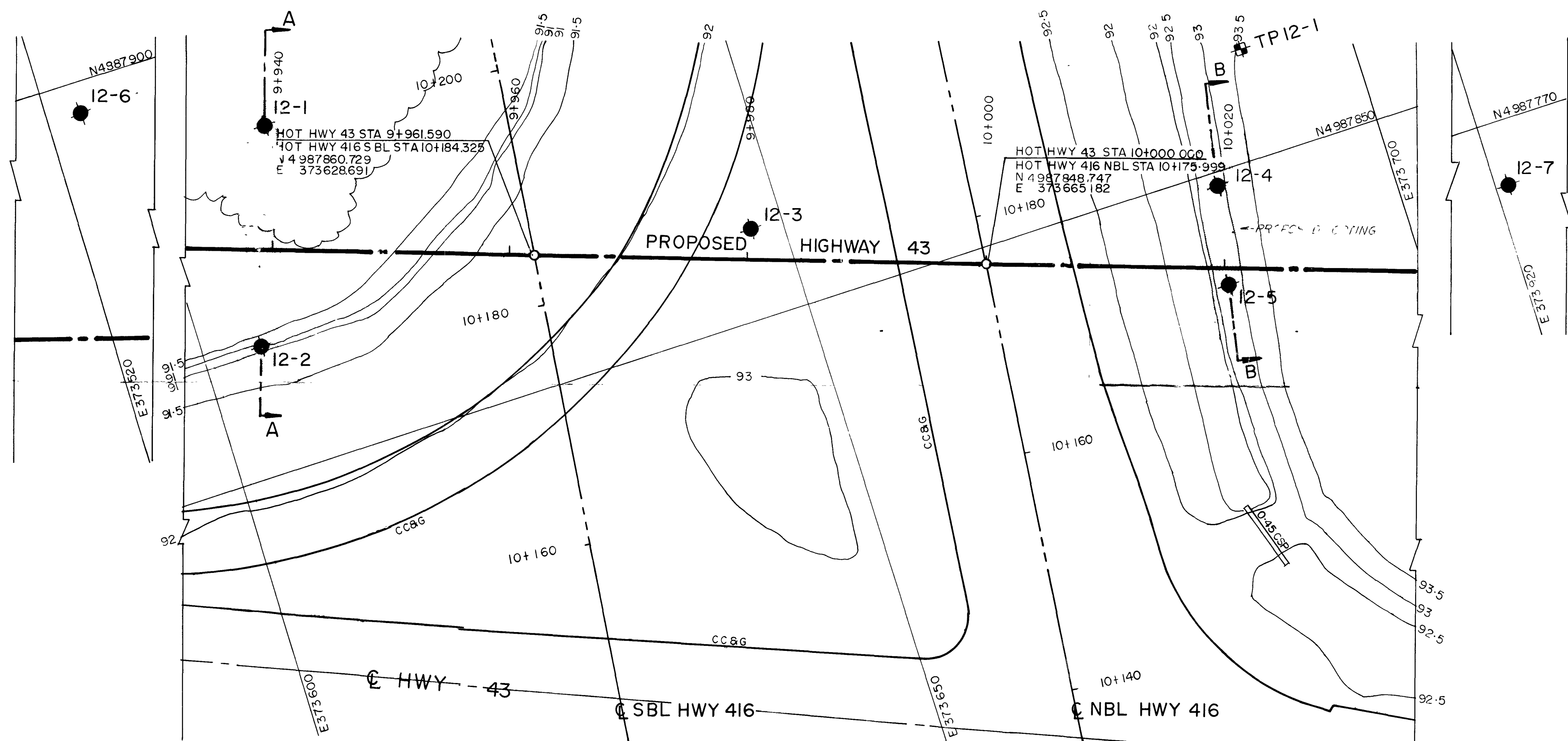
Created by: KCP

Checked by: MI

<https://golderassociates.sharepoint.com/sites/35409g/Shared Documents/Active/2018/1899802/Ph 19000/Figures/>

APPENDIX C

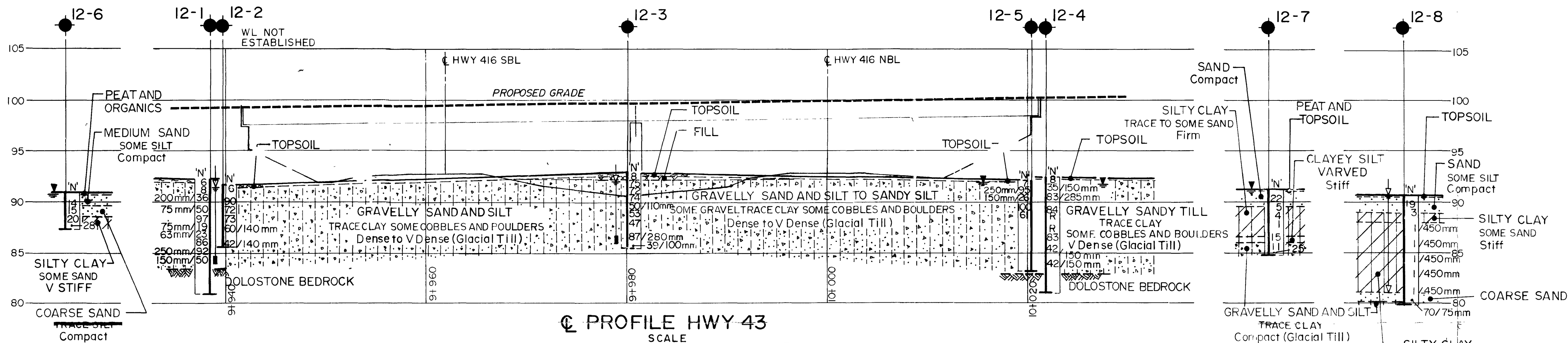
Previous Investigation GEOCREs No. 31G-205
Bore Hole Locations & Soil Strata Drawing
Record of Boreholes 12-1 to 12-8
Sheet 346 Highway 43 Underpass Bridge 12
Highway 416 General Arrangement Drawing



PLAN

SCALE

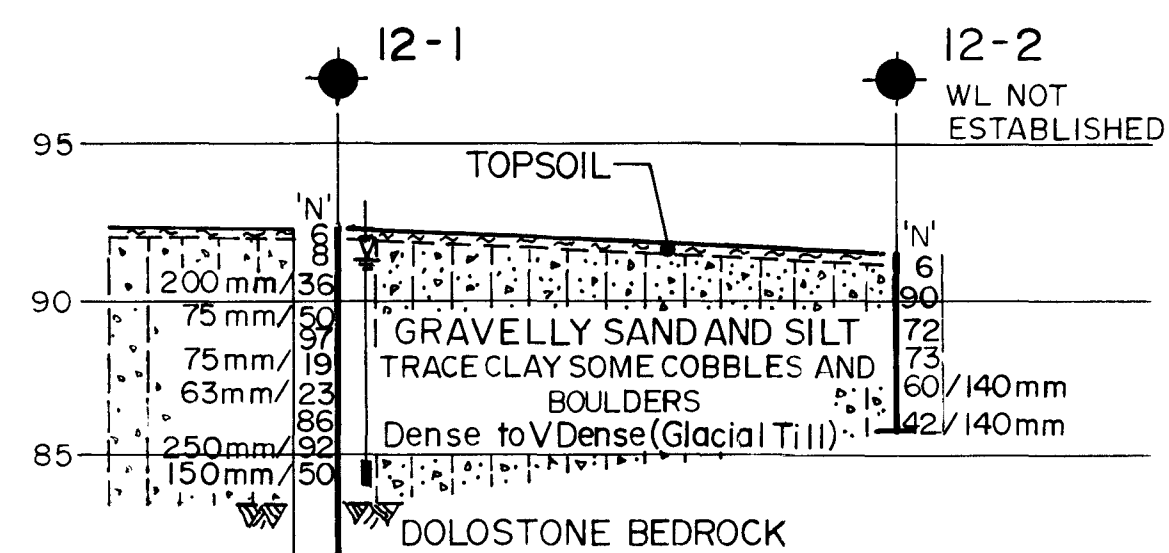
10 5 0 10 METRES



PROFILE HWY 43

SCALE

10 5 0 10 METRES

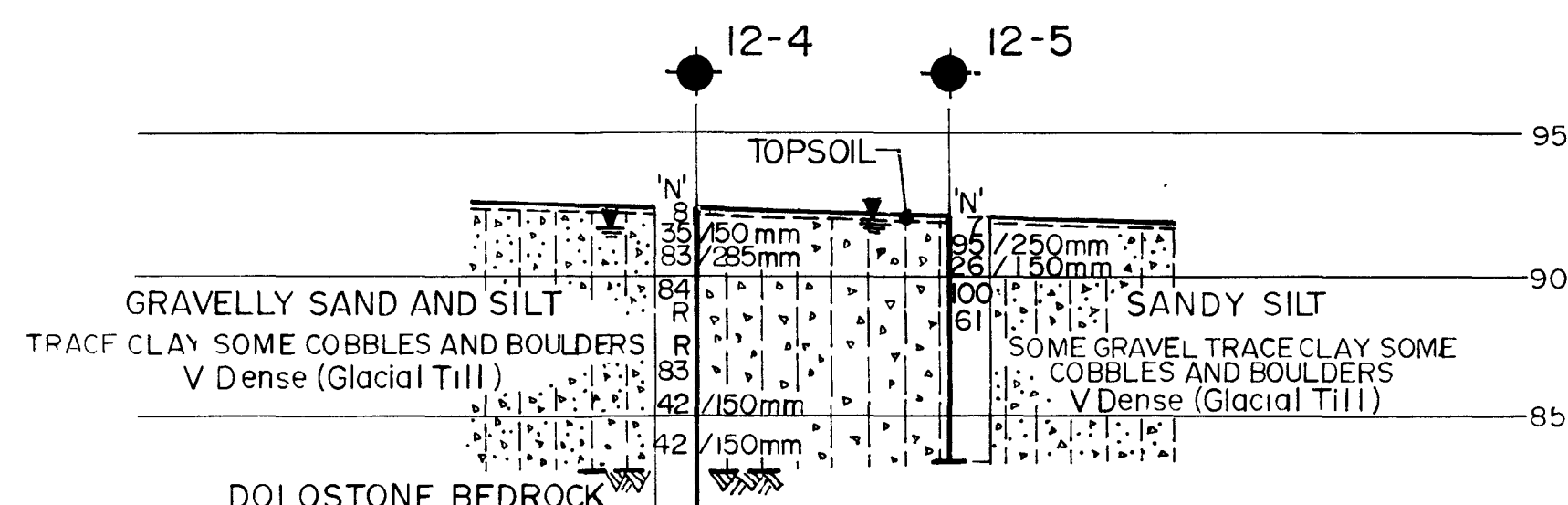


A-A

SECTIONS

SCALE

10 5 0 10 METRES



B-B

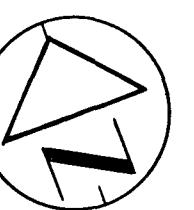
METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 372-89-06

HIGHWAY 416 UNDERPASS
AT HIGHWAY 43

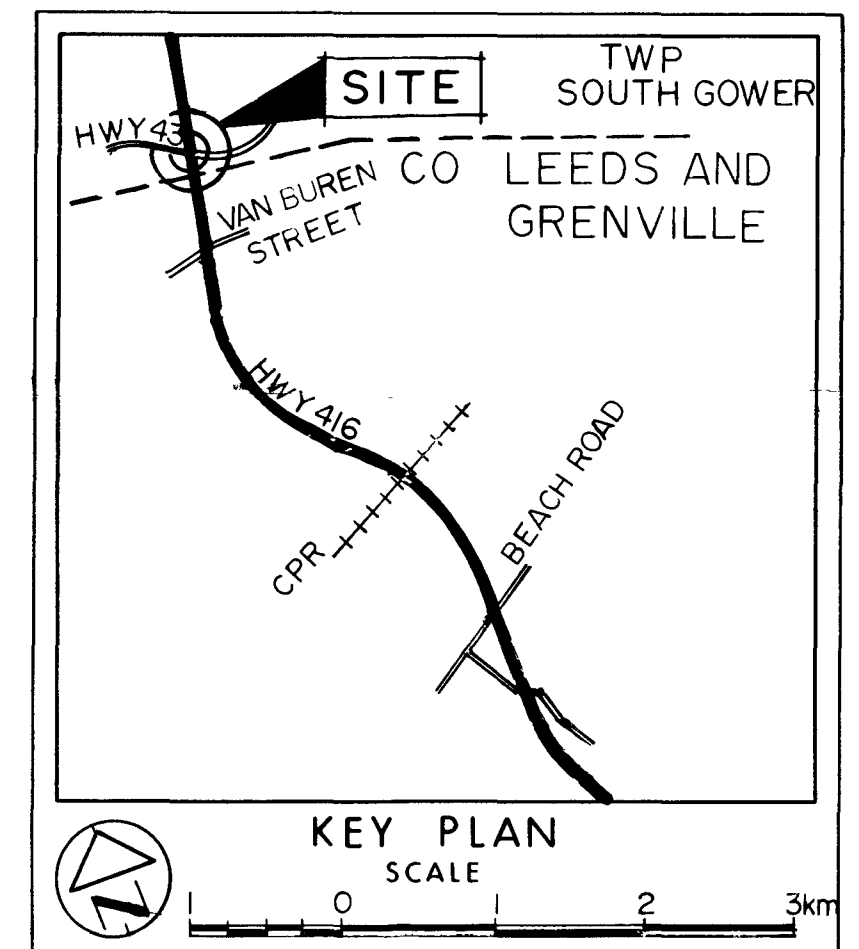
BORE HOLE LOCATIONS & SOIL STRATA



SHEET

A

GEOCON(1991) INC



LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation
- Piezometer
- Test Pit

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	92.3	4 987 878.2	373 610.0
2	91.6	4 987 860.8	373 604.3
3	93.0	4 987 857.4	373 646.6
4	92.5	4 987 848.9	373 685.8
5	92.2	4 987 840.6	373 683.6
6	91.0	4 987 897.3	373 517.2
7	92.3	4 987 764.7	373 921.0
8	90.7	4 987 734.9	374 016.4
TP12-1	93.5	4 987 858.9	373 690.8

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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.

REV	DATE	BY	DESCRIPTION

Geocres No 31

HWY No 416	SUBM'D NK	CHECKED RB	DATE 1991 08 26	DIST 9
DRAWN MZ	CHECKED NK	APPROVED	SITE 16-318	DWG 372-89-06-A

EXPLANATION OF TERMS USED IN REPORT

N VALUE THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 31mm O D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63 kg, FALLING FREELY A DISTANCE OF 0.76m FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED AVERAGE N VALUE IS DENOTED THUS \bar{N}

DYNAMIC CONE PENETRATION TEST CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O D 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND /OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm* IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	< 50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
e_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

EXPLANATION OF THE TERM

ROCK QUALITY DESIGNATION (RQD)

The description of bedrock quality for engineering purposes can be inferred from a modified core recovery logging procedure designated as RQD, developed by D.U. Deere.* This classification is based on a modified diamond drill core recovery percentage in which only the pieces of sound core over 4 inches (10 cm) long are counted as recovery. The core must be carefully examined to discount fresh irregular breaks caused by the drilling process (fresh broken pieces are fitted together and counted as one piece). The remaining fragments less than 4 inches (10 cm) length are considered to be due to very close bedding, jointing, fracturing, shearing, or weathering in the rock mass and are not counted. The procedure penalizes the rock where recovery is poor. This is appropriate because poor core recovery usually depicts poor quality rock. In the case of certain shaley sedimentary or thinly foliated metamorphic rocks, the method is not as exact as for other rock types and rock quality requires interpretation by a specialist for the particular engineering application. To minimize the occurrence of core breaks from drilling procedures RQD logging is normally run on core obtained by double or triple tube core barrels and generally of "N" size or greater.

The table below may be used as a general indicator to correlate (RQD) and rock mass quality.

RQD	DESCRIPTION OF ROCK QUALITY
90 - 100	Excellent - intact, very sound, massive
75 - 90	Good - moderately jointed or sound
50 - 75	Fair - blocky and seamy, fractured
25 - 50	Poor - shattered and very seamy or blocky, severely fractured
0 - 25	Very poor - crushed, very severely fractured

*See, for instance:

K.G. Stagg and O.C. Zienkiewicz, "Rock Mechanics in Engineering Practice". New York, Wiley, 1968, Chapter I.

RECORD OF BOREHOLE No 12-1

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,878.2 N; 373,610.0 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, Tricone, NXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE October 31, 1990 CHECKED BY I.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100								WATER CONTENT (%) 20 40 60
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE								
92.3	Ground Surface															
0.00	Silty Sand Topsoil.		1	SS	6		92									
91.9	Some roots.															
0.4	Loose Black															
	Gravelly Sand and Silt.		2	SS	8											
	Trace clay (Glacial Till)															
	Loose to Very Dense		3	SS	36/ 200											
	Brown															
89.9																
2.4	Gravelly Sand and Silt.		4	SS	50/ 75mm											
	Trace clay. Some cobbles and boulders (Glacial Till)															
	Very Dense Grey		5	SS	97											
			6	SS	19/ 75mm											
			7	SS	23/ 63mm											
			8	SS	86*											
			9	SS	92/ 250mm											
			10	SS	50/ 150mm											
83.5					Rec%											
8.8	Dolostone Unweathered, fine grained, closely to moderately bedded.		11	RC NXL	98											
	Sound Grey															
			12	RC NXL	100											
80.9																
11.4	End of Borehole															
Notes: 1. Water level in stand-pipe piezometer measured at elevation 91.3 m on Dec. 9/90. 2. 00* indicates that the quoted 'N' value is based on the first 0.3 m of penetration (full penetration not achieved) 3. Borehole advanced from 4.7 m to bedrock by Triconing inside																

+3, x5: Numbers refer to Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 12-2

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,860.8 N; 373,604.3 E ORIGINATED BY N.K.
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, Tricone COMPILED BY N.K.
DATUM Geodetic DATE November 31, 1990 CHECKED BY I.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
91.6	Ground Surface																
0.0	Silty Sand Topsoil.		1	SS	6	Not											
91.1	Some Roots and Organics					Not											
0.5	Loose Black					Not											
	Gravelly Sand and Silt.		2	SS	90		90										
	Trace clay. Some																
	cobbles and boulders		3	SS	72												
	(Glacial Till)																
	Very Dense Grey		4	SS	73		88										
			5	SS	60/ 140 mm												
85.6	End of Borehole		6	SS	42/ 140 mm		86										
	Notes:																
	1) Borehole advanced																
	from 3.10 m to																
	5.95 m by Triconing																
	inside an N casing.																

+3, x5 : Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-3

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,857.4 N; 373,646.6 E ORIGINATED BY N.K.
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, Tricone COMPILED BY N.K.
DATUM Geodetic DATE October 30, 1990 CHECKED BY I.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100								WATER CONTENT (%) 20 40 60	GR SA SI CL
								SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
93.0	Ground Surface																
0.0	Silty Sand Topsoil.																
92.6	Loose Black		1	SS	8												
0.4	Sand (Fill)																
92.4																	
0.6	Gravelly Sand and Silt. Trace clay. Some cobbles and boulders (Glacial Till).		2	SS	75*		92										
	Dense to Very Dense		3	SS	72										28 32 35 5		
			4	SS	74*												
			5	SS	50/ 110mm												
	Brown																
	Grey		6	SS	53*												
			7	SS	47												
			8	SS	87/ 280mm												

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 12-4

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,848.9 N; 373,685.8 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, Tricone, NXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE October 29, 30 and November 2, 1990 CHECKED BY I.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
								SHEAR STRENGTH kPa						
92.5	Ground Surface													
0.0	Silty Sand Topsoil													
92.2	Some roots. Black		1	SS	8									
0.3	Gravelly Sand and Silt. Trace clay. Some cobbles and boulders. (Glacial Till)		2	SS	35/ 150mm									
	Very Dense Brown		3	SS	83/ 265mm									
	Grey													
			4	SS	84									
			5	SS	R									
			6	SS	R									
			7	SS	83*									
			8	SS	42/ 150mm									
			9	SS	42/ 150mm									
82.9					RecZ									
9.6	Dolostone		10	RC NXL	100									
	Unweathered, fine grained closely to moderately bedded. Some calcite intrusions		11	RC NXL	100									
	Sound Grey		12	RC NXL	100									
80.9														
11.6	End of Borehole													
	Notes:													
	1) Water level in open borehole measured at elevation 91.69 m on October 30, 1990.													
	2) 00* indicates quoted N value based on the first 0.3 m penetration (full penetration not achieved).													
	3) Borehole advanced from 3.43 m to top of bedrock by Triconing inside N casing.													
	4) R = SPT Refusal													

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 12-5

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,840.6 N; 373,683.6 E ORIGINATED BY N.K.
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers, Tricone, NXL Rock Core COMPILED BY N.K.
 DATUM Geodetic DATE October 29 and 30, 1990 CHECKED BY I.C.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
92.2	Ground Surface																
0.0 92.0 0.2	Silty Sand Topsoil Some clay and roots Black		1	SS	7		92										
	Sandy Silt. Some gravel. Trace clay. Some cobbles and boulders (Glacial Till)		2	SS	95/ 250mm												
			3	SS	26/ 150mm												
	Very Dense Brown Grey		4	SS	100		90										
			5	SS	61												
							88										
							86										
							84										
83.3																	
8.9	End of Borehole Casing Refusal Inferred Bedrock Notes: 1) Water in open borehole measured at elevation 91.9 m on morning of Oct. 30/90. 2) Borehole advance from 3.56 m to 8.92 m by Triconing inside N casing.																

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 12-6

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,897.3 N; 373,517.2 E ORIGINATED BY GY
DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers COMPILED BY IC
DATUM Geodetic DATE February 28, 1991 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100									
								SHEAR STRENGTH kPo									
											</						

RECORD OF BOREHOLE No 12-7

METRIC

W P 372-89-06 LOCATION Co-ords: 4,987,764.7 N; 373,921.0 E
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers
 DATUM Geodetic DATE March 6, 1991
 ORIGINATED BY GY
 COMPILED BY IC
 CHECKED BY IC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
91.3	Ground Surface																
0.0	Peat and Topsoil Black																
91.0																	
0.3	Sand																
	Compact Grey		1	SS	22		90										
89.6																	
1.7	Silty Clay. Trace to Some Sand. (Weathered Crust)		2	SS	5												
	Firm Grey		3	SS	4												
			4	SS	1		88										
86.7																	
4.6	Clayey Silt (Varved)		5	SS	15												
86.0	Stiff Grey						86										
5.3	Gravelly Sand and Silt Trace clay. (Glacial Till)		6	SS	11												
84.7	Compact Grey		7	SS	25												
6.6	End of Borehole																
	Notes: 1) Water level in open borehole measured at elevation 91.3 upon completion of drilling.																

OFFICE REPORT ON SOIL EXPLORATION

+³, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 12-8

METRIC

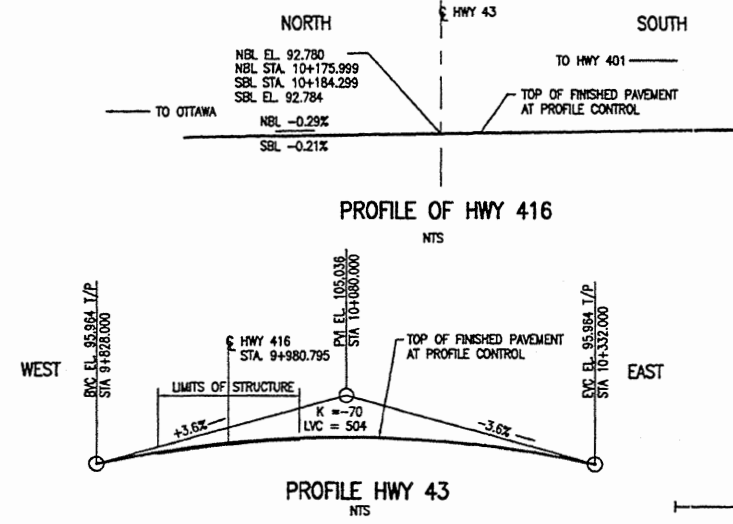
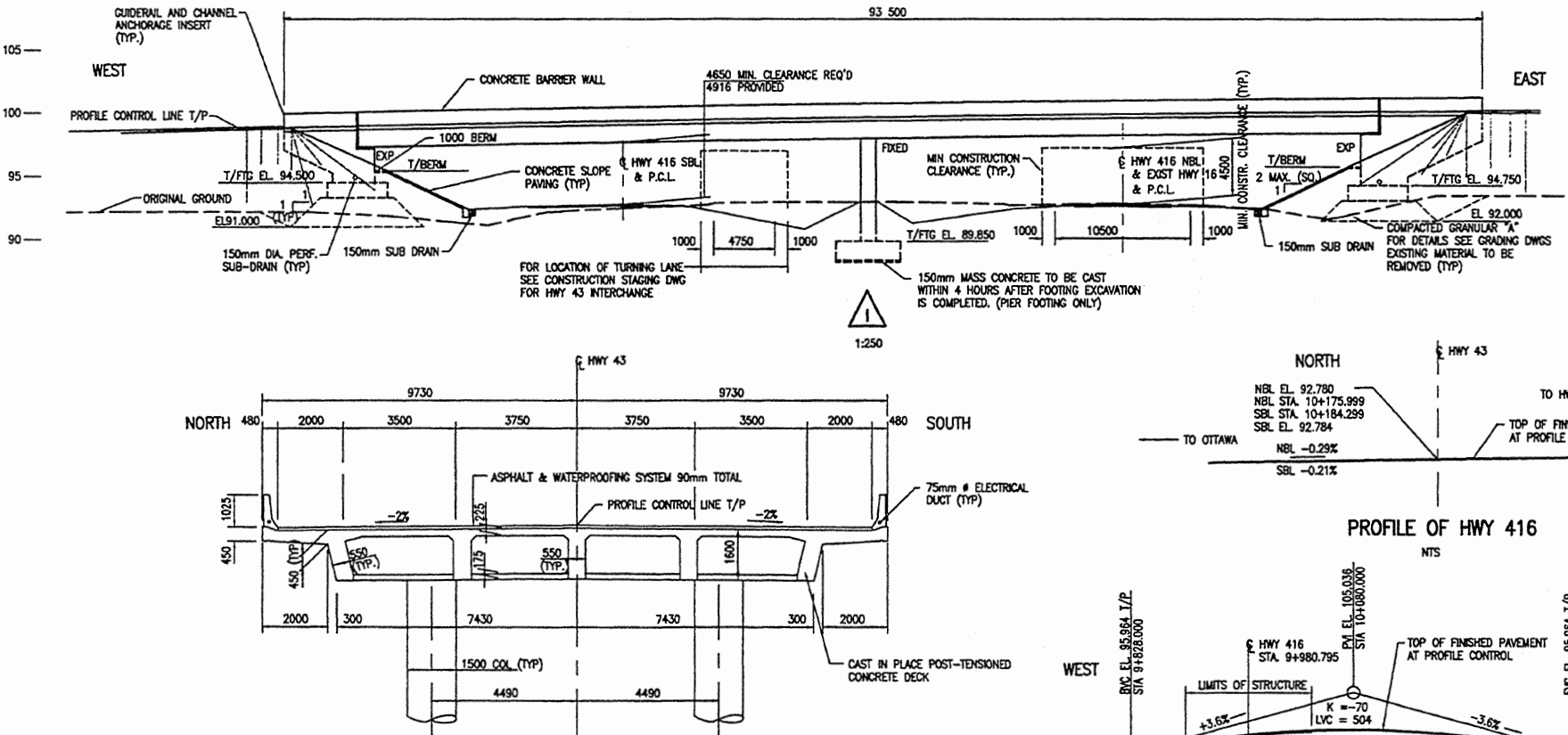
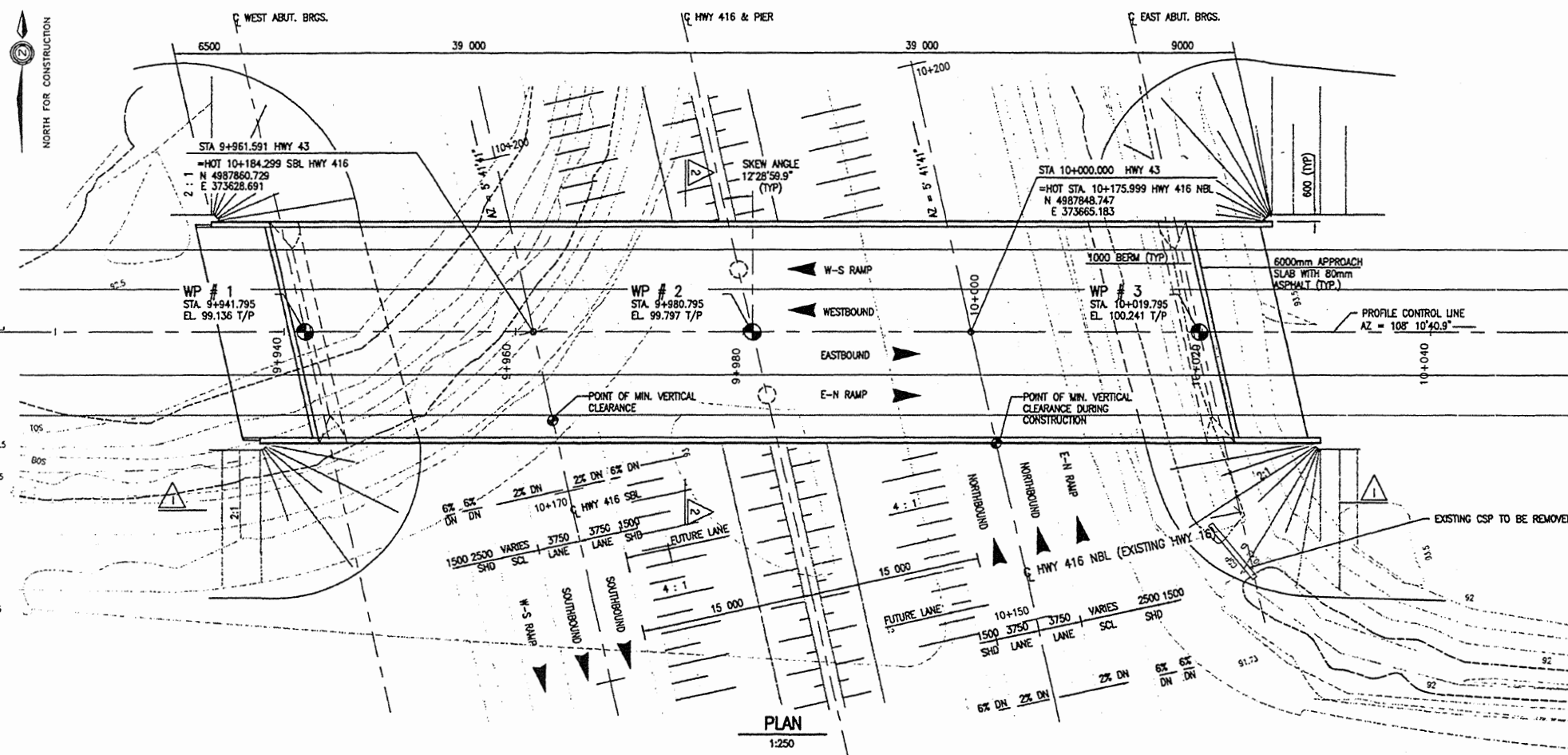
W P 372-89-06 LOCATION Co-ords: 4,987,734.9 N; 374,016.4 E ORIGINATED BY CY
 DIST 9 HWY 416 BOREHOLE TYPE Hollow Stem Augers COMPILED BY IC
 DATUM Geodetic DATE March 1, 1991 CHECKED BY IC

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100							SHEAR STRENGTH kPo
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	x LAB VANE	20 40 60			
90.7	Ground Surface														
0.0	Peat and Topsoil Black														
90.5															
0.2	Sand. Some Silt.						90							0 82 18 0	
	Compact Brown		1	SS	19										
89.0															
1.7	Silty Clay Some Sand (Weathered Crust)		2	SS	3								0 16 44 40		
	Stiff Grey						88								
88.0															
2.7	Silty Clay. Trace Sand. Silt content increases with depth.		3	SS	1/ 450mm										
	Soft to Firm Grey														
			4	SS	1/ 450mm		86								
			5	SS	1/ 450mm		84						0 7 61 32		
			6	SS	1/ 450mm		82								
81.1			7	SS	1/ 450mm										
9.6	Coarse Sand						See Note 1.								
80.0							80								
10.7															
79.9	Gravelly Sand and Silt		8	SS	70/ 75mm										
10.8	Trace Clay (Glacial Till)														
	Very Dense Grey														
	End of Borehole														

+³, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



CONTROL MONUMENT No. 010791026
27.0m EAST OF C HWY 16
38.1m NORTH OF C HWY 43
EL. 93.600

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

DIST. No. 9 HWY 416
CONT. No. 97-19
WP. No. 372-89-06

HIGHWAY 43 UNDERPASS
BRIDGE No. 12 HIGHWAY 416
GENERAL ARRANGEMENT

SHEET
346

Fenco
FENCO ENGINEERS INC.

GENERAL NOTES
CLASS OF CONCRETE
DECK AND COLUMNS 35 MPa
REMAINDER 30 MPa

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX "C" DENOTE COATED BARS.

CONSTRUCTION NOTES
THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATION & SOIL STRATA
3. FOUNDATION LAYOUT AND REINFORCEMENT
4. WEST ABUTMENT
5. WEST ABUTMENT WINGWALLS
6. EAST ABUTMENT
7. EAST ABUTMENT WINGWALLS
8. PIER
9. DECK LAYOUT AND DETAILS
10. DECK PRESTRESSING
11. LONGITUDINAL TENDONS
12. TRANSVERSE TENDONS
13. DECK REINFORCEMENT I
14. DECK REINFORCEMENT II
15. DECK REINFORCEMENT III
16. 6000mm APPROACH SLAB
17. BARRIER WALLS
18. JOINT ANCHORAGE AND ARMOURING WITH INJECTION HOSE SYSTEM-ASSEMBLY
19. JOINT ANCHORAGE AND ARMOURING WITH INJECTION HOSE SYSTEM-DETAILS
20. DETAILS OF CONCRETE SLOPE PAVING
21. STANDARD DETAILS
22. ELECTRICAL EMBEDDED WORK
23. QUANTITIES
24. QUANTITIES

LIST OF ABBREVIATIONS

- WP - WORKING POINT
- T/P - TOP OF PAVEMENT

APPLICABLE STANDARDS

OPSD 3501.00 GRANULAR BACKFILL REQUIREMENTS-ABUTMENTS
OPSD 4010.00 GUDE RAIL AND CHANNEL ANCHORAGE
OPSD 4602.00 FALSEWORK CLEARANCES



REVISIONS	DATE	BY	DESCRIPTION
DESIGN	WYC	CHK. LNC	CODE 0180C-91
DRAWN	JW	CHK. WYC	SITE 16-318
			STRUCT. SCHEME
			DWG. 1

DATE: APR 11 1997 1:30 D.M.L.S.O.

APPENDIX D

Site Photographs



Photograph 1: Station 10+200 Looking east along CR43



Photograph 2: Station 10+300 Near Borehole 20-01 looking east along County Road 43



Photograph 3: Station 10+325 looking south-east along County Road 43



Photograph 4: Station 10+325 looking west along County Road 43 towards Borehole 20-02



Photograph 5: Station 10+375 looking west along County Road 43 North Ditchline



Photograph 6: Station 10+375 looking north-west along County Road 43



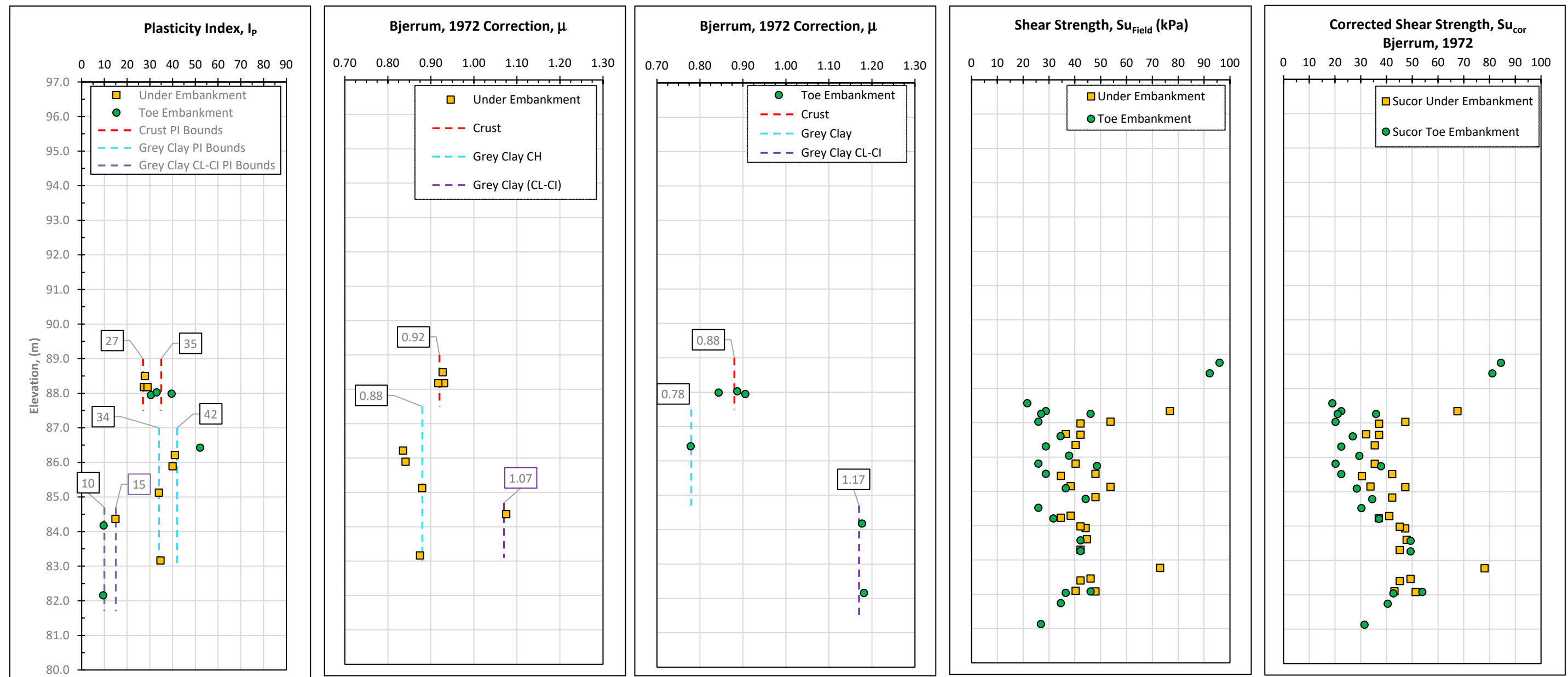
Photograph 7: Station 10+350 looking east along County Road 43



Photograph 8: Station 10+350 looking west along County Road 43 South Ditchline

APPENDIX E

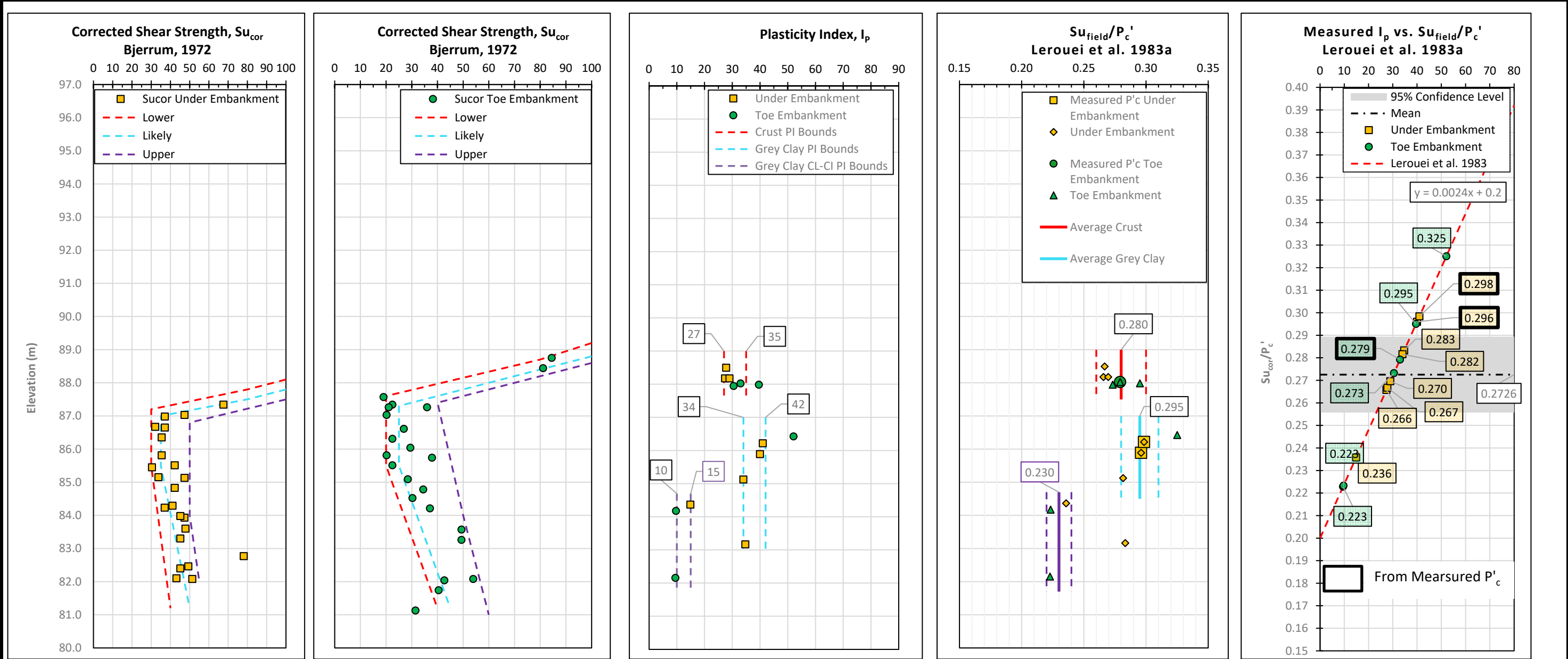
Results of Stability Analysis



Foundation Investigation and Design
Highway 416 Underpass at County Road 43
Corrected Shear Strength Bjerrum, 1972

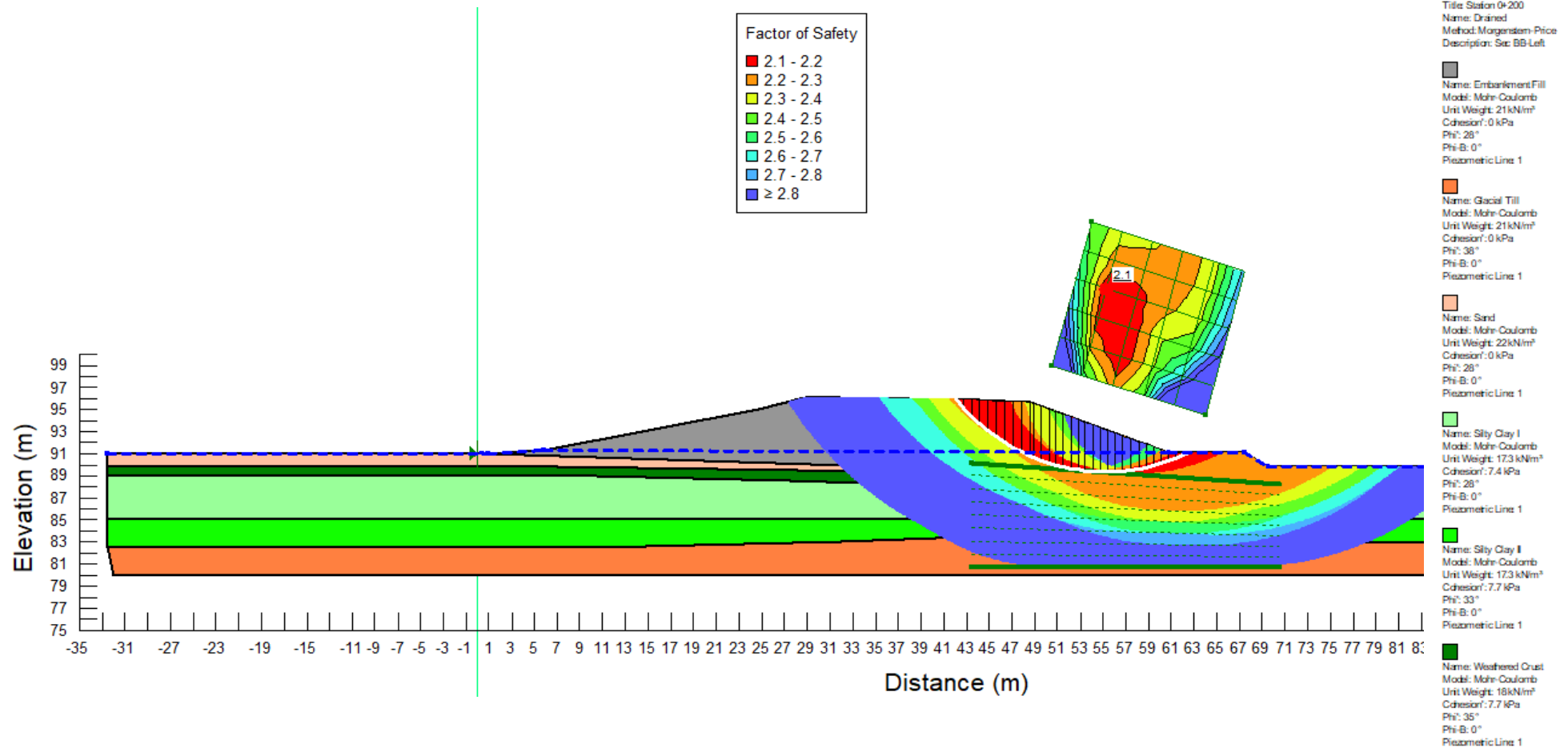
Project No.: 1899802-19000
Date: February 19, 2021
Drawn: KCP
Review: WC

FIGURE B



Section BB

Scenario 1 – Static Drained Analysis



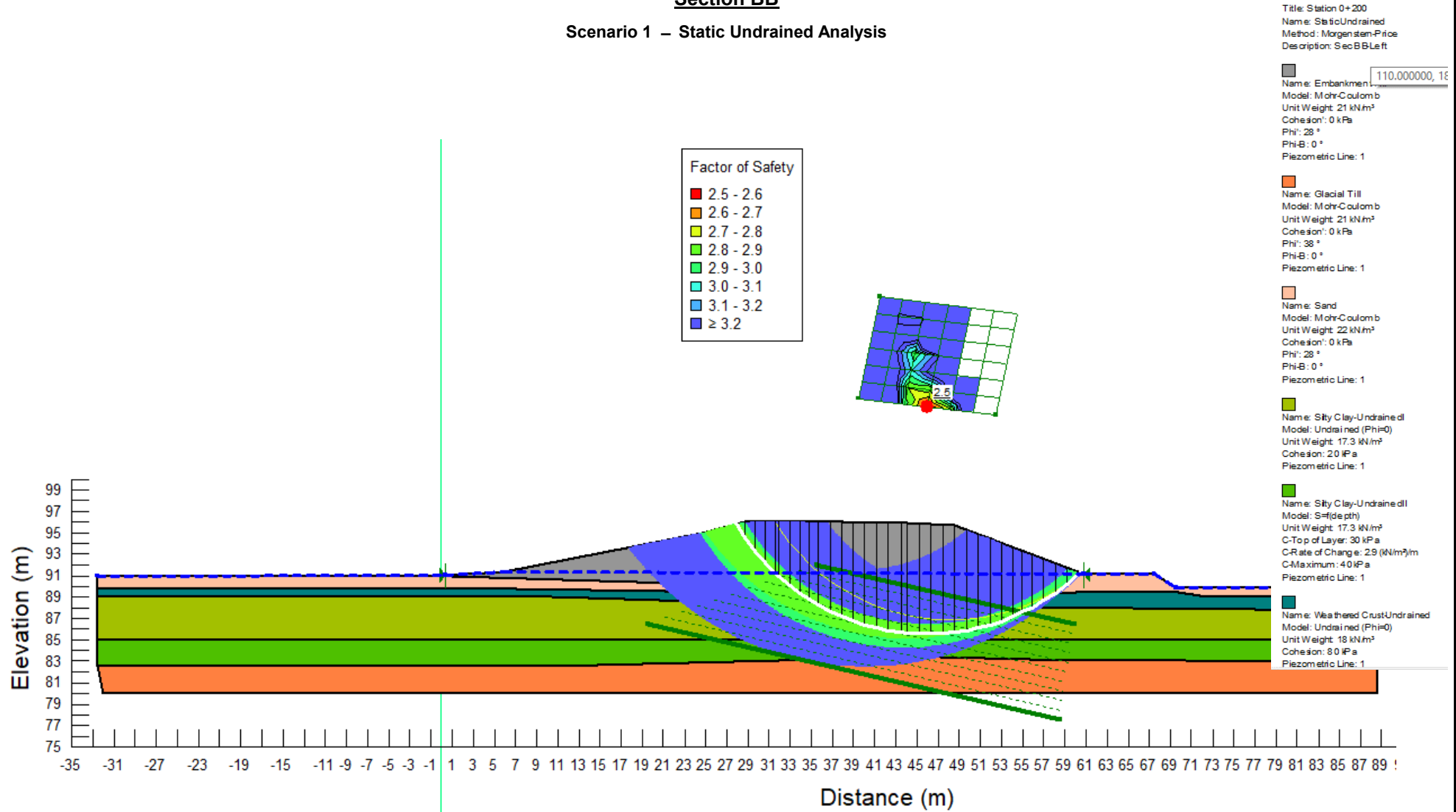
Foundation Investigation and Design
 Highway 416 Underpass at County Road 43
 Slope Stability Assessment

Project No: 1899802-19000
 Drawn: SS
 Date: March 31, 2021
 Checked: KCP
 Review: WC

FIGURE E1

Section BB

Scenario 1 – Static Undrained Analysis



Foundation Investigation and Design
Highway 416 Underpass at County Road 43
Slope Stability Assessment

Project No: 1899802-19000
Drawn: SS
Date: March 31, 2021
Checked: KCP
Review: WC

FIGURE E2

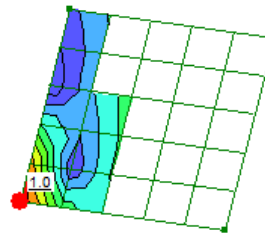
Section BB

Scenario 1 – Seismic Undrained

Title: Station 0+200
Name: Seismic
Method: Morgenstern-Price
Description: Sec BB-Left

Factor of Safety

- 1.0 - 1.1
- 1.1 - 1.2
- 1.2 - 1.3
- 1.3 - 1.4
- 1.4 - 1.5
- 1.5 - 1.6
- 1.6 - 1.7
- ≥ 1.7



Name: Embankment Fill
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 28°
Phi-B: 0°
Piezometric Line 1

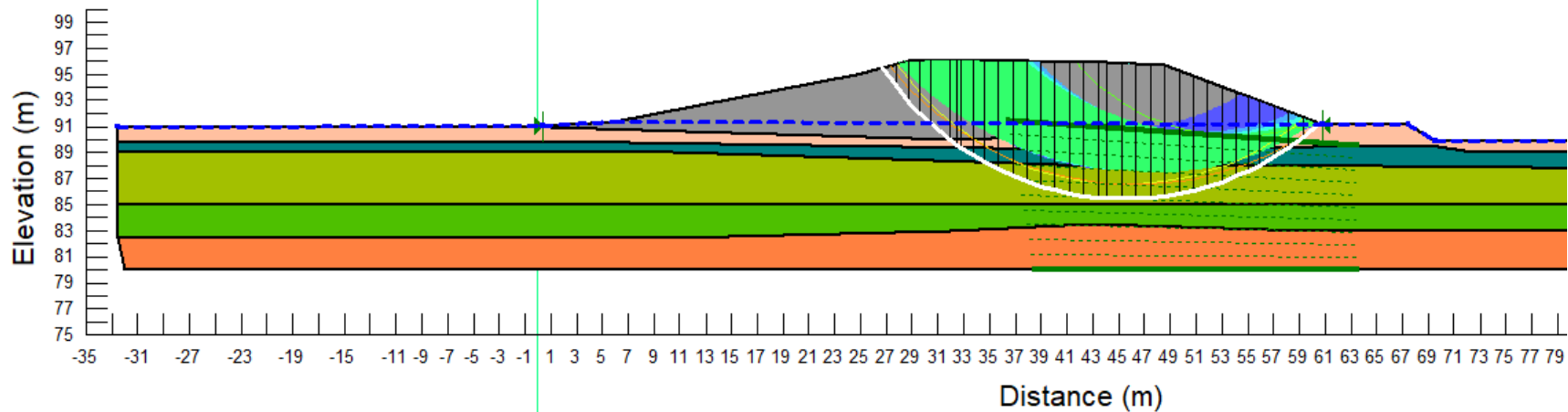
Name: Glacial Till
Model: Mohr-Coulomb
Unit Weight: 21 kN/m³
Cohesion: 0 kPa
Phi: 38°
Phi-B: 0°
Piezometric Line 1

Name: Sand
Model: Mohr-Coulomb
Unit Weight: 22 kN/m³
Cohesion: 0 kPa
Phi: 28°
Phi-B: 0°
Piezometric Line 1

Name: Silty Clay-Undrained
Model: Undrained (Phi=0)
Unit Weight: 17.3 kN/m³
Cohesion: 20 kPa
Piezometric Line 1

Name: Silty Clay-Undrained
Model: S=(depth)
Unit Weight: 17.3 kN/m³
C-Top of Layer: 30 kPa
C-Rate of Change: 2.9 (kN/m²)/m
C-Maximum: 40 kPa
Piezometric Line 1

Name: Weathered Crust-Undrained
Model: Undrained (Phi=0)
Unit Weight: 18 kN/m³
Cohesion: 80 kPa
Piezometric Line 1



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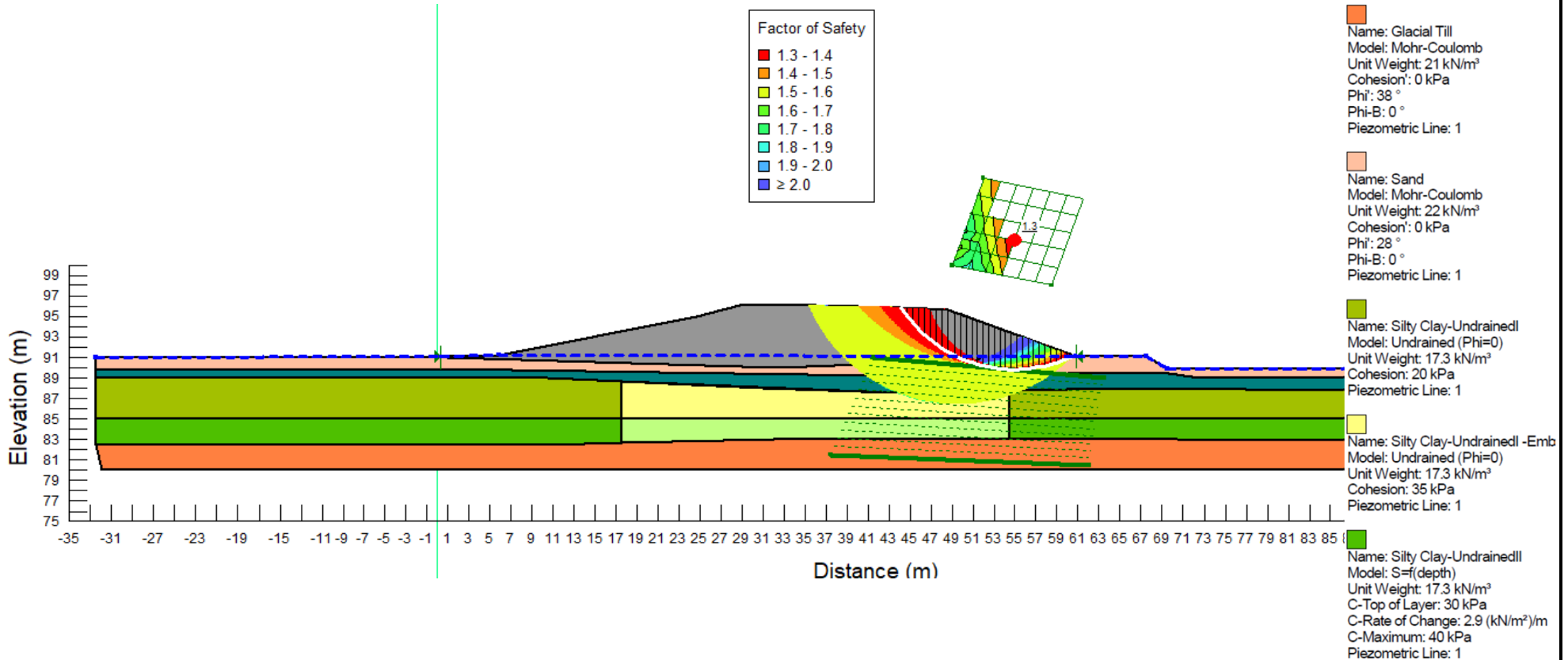
Project No: 1899802-19000
Drawn: SS
Date: March 31, 2021
Checked: KCP
Review: WC

FIGURE E3

Section BB

Scenario 2 – Seismic Undrained

Title: CR43/Hwy416
Name: Seismic_Left
Method: Morgenstern-Price
Description: Sec BB-Left



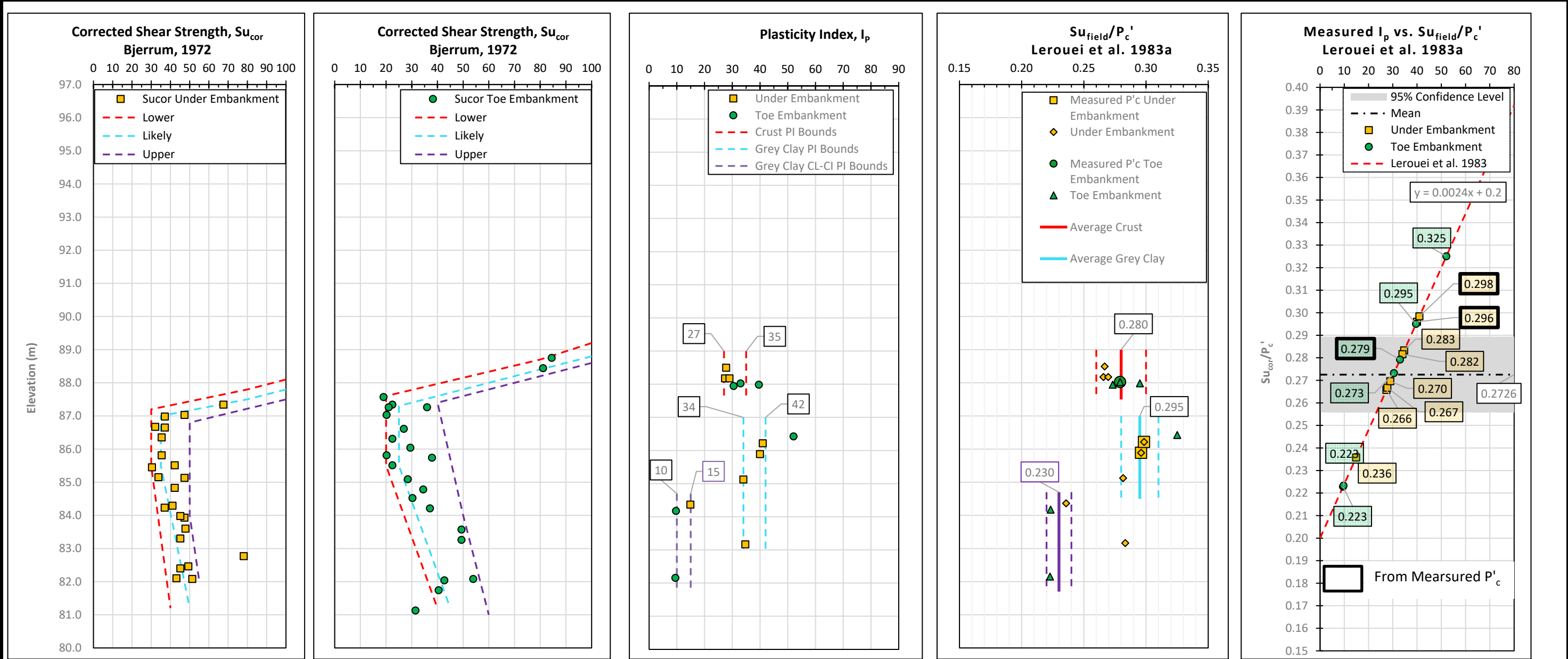
Foundation Investigation and Design
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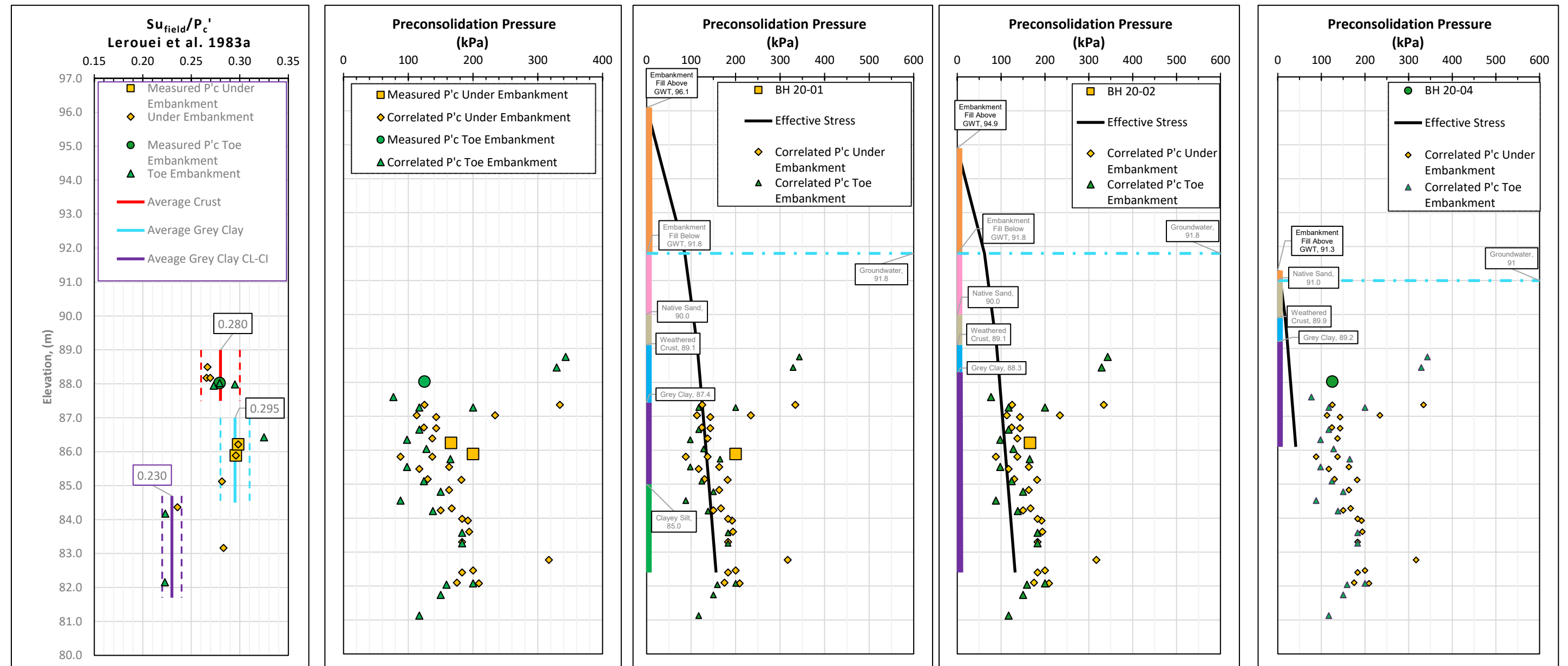
Project No: 1899802-19000
Drawn: SS
Date: March 31, 2021
Checked: KCP
Review: WC

FIGURE E4

APPENDIX F

Results of Settlement Analysis

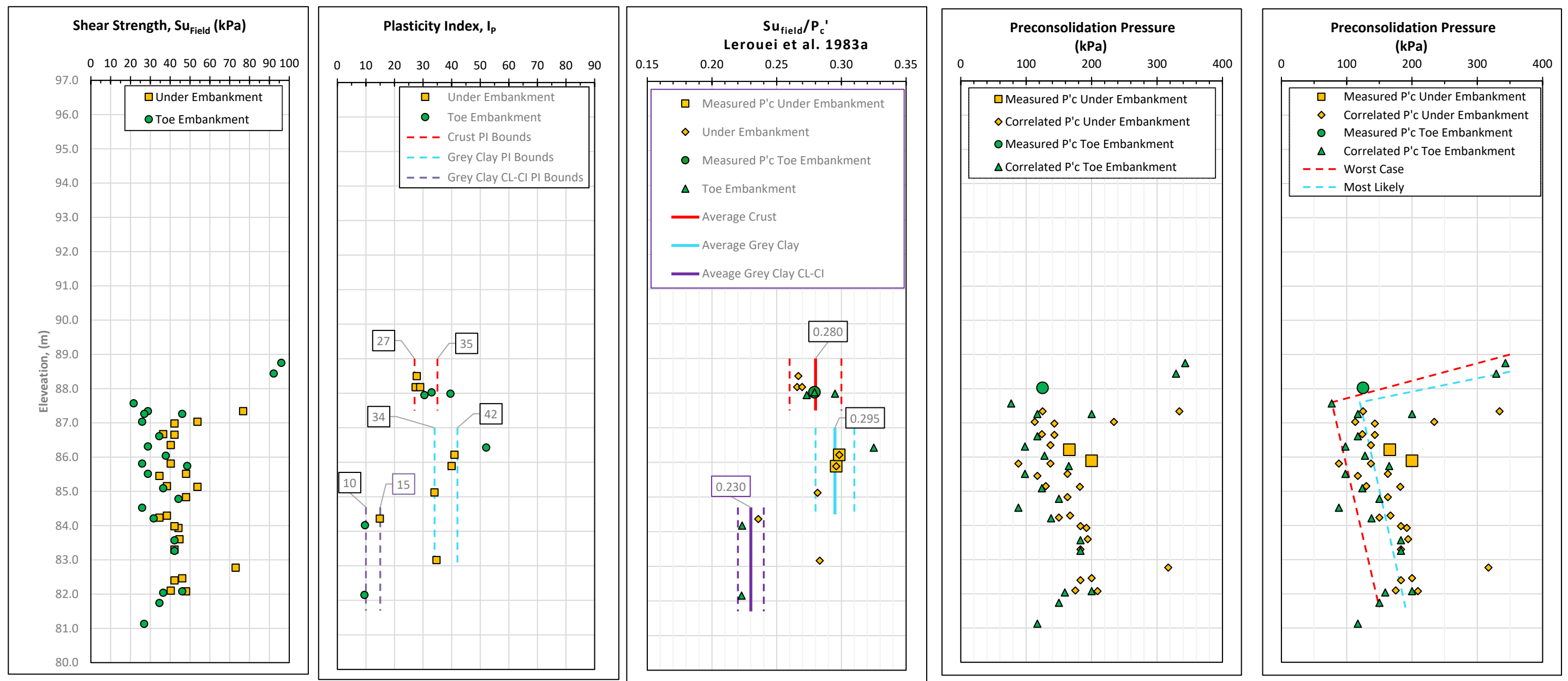




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Preconsolidation Pressure & Lerouei et al. 1983a

Project No.: 1899802-19000
Date: February 19, 2021
Drawn: KCP
Review: WC

FIGURE D





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