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REPORT ON

Foundation Investigation and Design County Road 2/34 Underpass Replacement Highway 401, Site No. 31-232 Lancaster, Ontario W.P. 4013-11-01

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REPORT



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Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0	INTRODUCTION.....	1
2.0	SITE DESCRIPTION AND GEOLOGY.....	2
2.1	General.....	2
2.2	Regional Geological Conditions.....	2
2.3	Existing Structure.....	2
3.0	INVESTIGATION PROCEDURES.....	4
4.0	SITE STRATIGRAPHY.....	6
4.1	General.....	6
4.2	Topsoil and Peat.....	6
4.3	Fill.....	6
4.4	Silty Sand.....	7
4.5	Clay.....	7
4.6	Till.....	8
4.7	Sandy Silt, Gravelly Sand, and Sand and Gravel.....	8
4.8	Auger Refusal and Bedrock.....	9
4.9	Groundwater Conditions.....	10
5.0	CLOSURE.....	11
6.0	DISCUSSION AND ENGINEERING RECOMMENDATIONS.....	12
6.1	General.....	12
6.2	Seismic Design.....	12
6.2.1	Site Seismicity and Importance Category.....	12
6.2.2	Seismic Site Classification.....	13
6.2.3	Spectral Response Values and Seismic Performance Category.....	13
6.3	Bridge Foundations.....	14
6.3.1	Feasibility of Integral Abutments.....	15
6.3.2	Consequence and Site Understanding Classification.....	15
6.4	Driven Steel Pipe (Tube) Pile or Driven Steel H-Pile Foundations.....	15



6.4.1	Founding Elevations and Pile Driving.....	15
6.4.2	Axial Geotechnical Resistance.....	16
6.4.3	Downdrag Load (Negative Skin Friction).....	17
6.4.4	Lateral Geotechnical Resistance	17
6.4.5	Lateral Soil-Structure Interaction Springs.....	18
6.5	Caisson Foundations	18
6.5.1	Founding Elevations and Caisson Installation.....	18
6.5.2	Axial Geotechnical Resistance.....	19
6.5.3	Downdrag Load (Negative Skin Friction).....	19
6.5.4	Lateral Geotechnical Resistance	19
6.5.5	Lateral Soil-Structure Interaction Springs.....	19
6.6	Lateral Earth Pressures for Design.....	20
6.6.1	Lightweight Fill Embankment	20
6.6.2	Granular Fill Embankment	20
6.6.2.1	Static Lateral Earth Pressures for Design.....	21
6.6.2.2	Seismic Lateral Earth Pressures for Design	22
6.7	Approach Embankments	23
6.7.1	Geocres Review of Existing Embankments	23
6.7.2	New Embankment Construction.....	23
6.7.3	Approach Embankment Settlement	24
6.7.4	Expanded Polystyrene Lightweight Fill Embankment Construction	26
6.7.5	Global Stability	28
6.8	Construction Considerations.....	29
6.8.1	Existing Utilities	29
6.8.2	Open-Cut Excavations	29
6.8.3	Temporary Protection Systems.....	30
6.8.4	Groundwater and Surface Water Control	30
7.0	CLOSURE.....	31

Table 1	Comparison of Foundation Alternatives
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DRAWINGS

Drawing 1	Structure Replacement, County Road 2/34 Underpass - Borehole Locations and Soil Strata
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APPENDICES

APPENDIX A Borehole Records

Lists of Abbreviations and Symbols
Lithological and Geotechnical Rock Description Terminology
Records of Boreholes 16-1 to 16-9, 16-5B and 17-10

APPENDIX B Laboratory Test Results

Figure B1 Plasticity Chart –Silty Clay to Clay (Fill)
Figure B2 Grain Size Distribution Test Results – Clay (Weathered Crust)
Figure B3 Plasticity Chart – Clay (Weathered Crust)
Figure B4 Grain Size Distribution Test Results – Clay
Figure B5 Plasticity Chart – Clay
Figures B6 to B8 Consolidation Test Results
Figure B9 Grain Size Distribution Test Results – Silty Sand (Till)
Figure B10 Grain Size Distribution Test Results – Gravelly Sand (Till)
Figure B11 Grain Size Distribution Test Results – Sand and Gravel
Figure B12 Summary of Laboratory Compressive Strength Testing - Point Load Testing
Figure B13 Summary of Laboratory Compressive Strength Testing - Unconfined Compression Tests
Figure B14 Summary of Engineering Properties
Figures B15 to B17 Bedrock Core Photos

APPENDIX C Record of Boreholes, Previous Investigations

Records of Previous Boreholes 889-1 to 889-5 (Geocres No. 31G00-144)
Records of Previous Boreholes 1 to 4 (Geocres No. 31G00-145)

APPENDIX D Results of MASW Testing

APPENDIX E Non-Standard Special Provisions

Vibration Monitoring
CSP for Integral Abutments
Expanded Polystyrene Embankment

APPENDIX F Results of Analysis

Figure F1 Settlement History 1962 - 1984
Figure F2 Anticipated Settlement for Proposed Granular Fill Embankment
Figure F3 Results of Slope Stability Analysis
Figure F4/F5 P-Y Curves for H-Piles

APPENDIX G Technical Memorandum – Embankment Design Alternatives



PART A

FOUNDATION INVESTIGATION REPORT
COUNTY ROAD 2/34 UNDERPASS REPLACEMENT
HIGHWAY 401, SITE 31-232
LANCASTER, ONTARIO
W.P. 4013-11-01



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 associated with numerous culvert and bridge rehabilitations and/or replacements at various locations in the Eastern Region of Ontario as part of the 23 Structures MEGA 3 project.

This foundation investigation report addresses the proposed replacement of the existing County Road 2/34 Bridge (Site No. 31-232) over Highway 401 which is located in Lancaster, Ontario (WP 4013-11-01), as shown on the figure below.



The purpose of this foundation investigation was to assess the subsurface conditions in the area of the proposed bridge and associated approach embankment areas to provide information for the design of the bridge replacement. The foundation investigation included drilling boreholes and installing monitoring wells, as well as carrying out in-situ testing (including surface-wave velocity testing) and laboratory testing on selected soil samples.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated August 2012. In addition, Golder's letter dated July 19, 2016 described the work plan for additional foundation and engineering services for detail design.

The work was carried out in accordance with Golder's Quality Control Plan dated December 2012.



2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

The County Road 2/34 Underpass is located along Highway 401 in Lancaster, Ontario. The existing bridge (Site No. 31-232) is located at about Station 27+600 on Highway 401 (see Key Plan in Drawing 1).

It is understood that the preferred arrangement for detail design is to replace the existing bridge on a new alignment immediately west of the existing, and tying into the existing ramps.

Previous investigations were conducted for the design of the original/existing bridge. The results of those investigations are contained in the following reports:

- Report on Foundation Investigation, Proposed Crossing, Highway No. 401 and Highway No. 2, 1-1/2 Miles South of Lancaster, Township of Charlottenburg, District No. 9, Bridge No. 11, WP 108-59 (Geocres 31G00-144), by H.G. Acres & Company Limited, dated 1960.
- Report of Foundation for Proposed Underpass Bridge, HWY #401 and HWY #2 at Lancaster (Geocres No. 31G00-145), by the Department of Highways, Ontario, dated 1955.

2.2 Regional Geological Conditions

As delineated in *The Physiography of Southern Ontario*¹, this section of Highway 401 lies within the major physiographic region known as the Lancaster Flats.

The Lancaster Flats region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock. This region is underlain by a series of sedimentary rocks, consisting of limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield.

The soft and compressible Leda clay deposit that exist at this site is known to underlie a large portion of Highway 401, from about Cornwall and extending eastwards beyond the Québec border, plays a critical role on the foundation design.

2.3 Existing Structure

The existing bridge carries two lanes of traffic of County Road 2/34 over the four-lane and median-divided Highway 401.

The bridge consists of a two-span cast-in-place concrete box girder structure with abutments and central pier founded on piles. The existing structure is aligned approximately northwest to southeast and is about 67 m long and 12 m wide. It is understood that the structure was built in 1963.

The natural ground surface is relatively flat and varies from about Elevation 48 to 49 m north and south of Highway 401.

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.



The County Road 2/34 embankments are approximately 7 to 8 m in height above the natural ground level. The County Road 2/34 embankment side slopes are oriented at about 2 horizontal to 1 vertical (2H:1V). Based on visual observation at the time of the site investigation, the existing embankment side slopes appear to be performing satisfactorily, though they may have been regraded over the years.

The existing embankment loading over the deep sensitive and compressible clay deposit has led to very large settlements of the embankments since the original construction. These settlements resulted in the abutments tilting backwards due to settlement of the approach embankments, with the greatest settlements occurring some 25 to 50 m behind the abutments.



3.0 INVESTIGATION PROCEDURES

The subsurface investigation for the proposed underpass bridge replacement was carried out in two phases.

The first phase of the investigation was carried out between September 19 and 28, 2016 at which time ten boreholes (numbered 16-1 to 16-9, inclusively, and 16-5B) were advanced. The boreholes for the first phase were advanced using 108 mm inside diameter (I.D.) continuous-flight hollow-stem augers on a truck-mounted drill rig (CME 850), supplied and operated by CCC Geotechnical Environmental Drilling Ltd. of Ottawa, Ontario.

A second phase was completed on May 8, 2017, when one additional borehole (numbered 17-10) was advanced in the area of the north approach embankment to obtain an additional shear strength profile. The boreholes for the second stage were advanced using 108 mm inside diameter (I.D.) continuous-flight hollow-stem augers on a track-mounted drill rig (CME 550), supplied and operated by George Downing Estate Drilling of Hawkesbury, Ontario.

The boreholes were advanced at the locations shown on Drawing 1, as outlined below.

- Boreholes 16-1, 16-2, and 16-3 were advanced at about the proposed foundation locations for the central pier, north abutment, and south abutment, respectively. These boreholes were advanced to depths of about 12.6 to 13.9 m below the existing ground surface in the overburden. Upon encountering split spoon or auger refusal, the boreholes were advanced into the bedrock to final depths of about 15.6 to 17.5 m (i.e., Elevation 31.1 to 32.8 m) using rotary diamond drilling techniques while retrieving NQ sized bedrock core.
- Boreholes 16-4 and 16-5 were located within the proposed north and south approach embankments, respectively. The boreholes were advanced to depths of about 12.9 to 14.0 m (Elevations 35.2 to 34.4 m) below the existing ground surface in the overburden. Borehole 16-4 was terminated upon auger refusal. Borehole 16-5 was terminated in the compact glacial till.
- Borehole 16-5B was advanced immediately adjacent to 16-5 to a depth of about 7.7 m below the existing ground surface (i.e., Elevation 40.4) for the installation of a monitoring well and to retrieve two relatively undisturbed 73 mm diameter thin-walled Shelby tube samples of the clay using a fixed piston sampler.
- Four additional boreholes were advanced within the median of Highway 401 for providing additional subsurface information for roadway protection. Boreholes 16-6, and 16-7 were located to the west of the central pier and Boreholes 16-8 and 16-9 were located to the east of the central pier. The boreholes were advanced to a depth of about 7.3 m (i.e., Elevation 41.1 to 41.3 m) below the existing ground surface using a 105 mm inside diameter continuous-flight hollow-stem augers 5 m into the bedrock using HQ-size coring.
- Borehole 17-10 was advanced in the area of the proposed north abutment, just north of Borehole 16-2. This borehole was advanced to a depth of about 11.3 m below the existing ground surface (i.e., Elevation 37.3 m).

Soil samples in the boreholes were generally obtained at vertical intervals of about 0.60 and 0.76 m, using a 50 mm outer diameter split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing, using an MTO “N”-size vane was carried out to measure the undrained shear strength of the cohesive soils encountered at the site.

Monitoring wells were installed in Boreholes 16-1 and 16-5B to monitor the groundwater level at the site. The wells consist of 50 mm inside diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed by a section of bentonite pellet backfill. The water levels in the standpipe piezometers were measured about 8 months after installation, on May 26, 2017.



FOUNDATION REPORT SITE NO. 31-232
COUNTY ROAD 2/34 UNDERPASS REPLACEMENT AT HWY 401

The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock, except as indicated previously for the two monitoring wells. The site conditions were restored following completion of work.

The field work was supervised by members of Golder's technical and engineering staff, who located the boreholes, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratories in Ottawa and Mississauga for further examination. Index and classification tests consisting of grain size distribution, Atterberg limits, organic content and water content testing were carried out on selected soil samples at the Golder Ottawa laboratory. Three consolidation tests were performed on selected Shelby tube samples from boreholes 16-3 and 16-5B at Golder's Mississauga laboratory. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

Prior to drilling, the borehole locations were staked and surveyed by Golder personnel using a Trimble R8 GPS unit. The boreholes and locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole depth (m)
BH16-1	Central Pier, Highway 401 Median	5000079.4	226388.0	48.5	17.5
BH16-2	North Abutment	5000109.7	226374.7	48.8	16.9
BH16-3	South Abutment	5000049.9	226406.2	48.4	15.6
BH16-4	North Approach Embankment	5000127.4	226361.5	48.4	14.0
BH16-5	South Approach Embankment	5000033.5	226414.2	48.1	12.9
BH16-5B	South Approach Embankment	5000033.5	226414.2	48.1	7.7
BH16-6	Median, West of Pier	5000055.0	226344.1	48.6	7.3
BH16-7	Median, West of Pier	5000068.1	226365.7	48.4	7.3
BH16-8	Median, East of Pier	5000109.9	226436.1	48.5	7.3
BH16-9	Median, East of Pier	5000123.4	226457.1	48.4	7.3
BH17-10	North Abutment	5000111.7	226370.4	48.6	11.3

In addition to the borehole investigation, shear wave velocity profiling at the site was completed using the Multichannel Analysis of Surface Waves (MASW) technique and was conducted on May 23, 2013 by Golder's geophysics personnel. A series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. A 9.9 kg sledge hammer and 45 kg weight drop were used as the seismic source. The source locations were offset at distances of 5, 10, 15, and 20 m off the end and collinear with the geophone array.



4.0 SITE STRATIGRAPHY

4.1 General

As part of the current subsurface investigation, eleven boreholes were advanced within or near the limits of the proposed realignment of the County Road 2/34 Underpass. The borehole locations from the current and previous investigations at the site are shown on Drawing 1. The interpreted stratigraphic profile projected along the proposed County Road 2/34 centreline is also shown on Drawing 1.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from observations of drilling progress and from non-continuous 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.

The Record of Borehole sheets from the current investigation are presented in Appendix A. The results of the laboratory testing carried out during the current investigation are presented on the Record of Boreholes sheets and on Figures B1 to B14 in Appendix B. Photos of the bedrock core from the current investigation are presented on Figures B15 to B17 in Appendix B. The Record of Borehole sheets from the previous investigations at the site are provided for reference in Appendix C. The MASW test results and report are presented in Appendix D and include the calculated shear wave velocity profile measured from the field testing and a graphical representation of the shear wave velocity profile with depth.

In general, the subsurface conditions at the site consist of a surficial layer of fill and/or topsoil with some localized peat underlain by a thick layer of clay. The clay is underlain by relatively thin deposits of glacial till and/or sand and gravel over limestone bedrock.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. In the following discussion, emphasis is placed on the subsurface conditions from the boreholes advanced during the current investigation, which are in general agreement with the Geocres information. The Geocres information is referenced herein only in regard to the clay parameters in Section 4.5 and the bedrock surface elevation in Section 4.8.

4.2 Topsoil and Peat

Topsoil exists at ground surface at Borehole 16-5, about 200 mm in thickness.

Underlying the fill material at Boreholes 16-2 and 16-3, a thin layer of amorphous peat was encountered at depths of 0.8 and 1.4 m (i.e., Elevation 47.4 and 47.6 m) about 0.1 and 0.3 m thick, respectively.

4.3 Fill

Fill was encountered at ground surface at all boreholes with the exception of Borehole 16-5. Fill materials generally consist of silty clay and clayey silt with variable amounts of sand and gravel, with some isolated area of sand and gravel fill.

The fill extends to depths ranging from about 0.8 to 2.1 m. SPT 'N' values obtained within this material generally range from about 6 to 44 blows per 0.3 m of penetration, indicating a loose to dense state of packing.



The results of Atterberg limit testing carried out on one sample of the clay fill are summarized on Figure B1 and indicate a plasticity index value of 52 percent and a liquid limit value of 72 percent, reflecting a clay of high plasticity. The measured water contents of the fill ranges from approximately 5 to 27 percent.

4.4 Silty Sand

In Borehole16-3, a 0.2 m thick localized layer of silty sand was encountered below the peat at a depth of about 1.1 m.

4.5 Clay

The surficial materials are underlain by a thick deposit of sensitive clay. The clay deposit, where fully penetrated in Boreholes 16-1 to 16-5, inclusive, and 17-10 extends to depths of about 9.9 to 11.4 m and varies from about 8.8 to 9.9 m in thickness.

The upper portion of the clay has been weathered to form a grey brown crust. The thickness of the crust ranges from about 1.0 to 2.9 m and extends to a depth of about 2.1 to 3.6 m. Standard penetration tests carried out within the weathered crust gave 'N' values ranging from "Weight of Hammer" to about 17 blows per 0.3 m of penetration. In situ shear vane testing carried out where possible within this deposit measured undrained shear strengths of 94 to 118 kPa, indicating a stiff to very stiff consistency.

Grain size distribution testing was carried out on one sample of the weathered clay, the results of which are provided on Figure B2. The results of Atterberg limit testing carried out on five samples of the weathered clay are summarized on Figure B3 and indicate plasticity index values generally ranging from 44 to 78 percent and liquid limit values ranging from 67 to 100 percent, reflecting a clay of high plasticity. The measured water content of the weathered clay ranges from approximately 29 to 52 percent.

Standard penetration tests carried out within the unweathered portion of the deposit (below the crust) gave 'N' values ranging from "Weight of Hammer" to about 2 blows per 0.3 m of penetration. In situ shear vane testing carried out within this deposit measured undrained shear strengths ranging from about 19 to 65 kPa, indicating a soft to stiff consistency, however the deposit was found to be more generally firm. The calculated average sensitivity ratio based on remoulded shear strengths in this deposit is about 6, indicating a sensitive material.

Grain size distribution testing was carried out on one sample of the unweathered clay, the results of which are provided on Figure B4. The results of Atterberg limit testing carried out on ten samples of the unweathered clay are shown on Figure B5 and gave plasticity index values ranging from about 34 to 56 percent and liquid limit values ranging from about 59 to 78 percent, respectively, indicating a high plasticity clay. The measured water contents of the unweathered portion of the deposit were between about 58 to 82 percent.

Oedometer consolidation testing was carried out on three samples of clay, the results of which are provided on Figures B6 to B8, inclusive. The consolidation test results are summarized in the table below and indicate that the clay is slightly overconsolidated, with a preconsolidation pressure of about 100 to 140 kPa and overconsolidation ratio of 1.4 to 1.8.



Borehole/Sample Number	Sample Depth/Elevation (m)	Unit Weight (kN/m ³)	σ_p' (kP)	σ_{vo}' (kP)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e _o	OCR
16-5B / 1	5.7 / 42.4	15.2	100	60	40	1.23	0.039	2.21	1.7
16-5B / 2	7.6 / 40.5	16.3	100	70	30	0.81	0.020	1.71	1.4
16-3 / 8	8.9 / 39.5	15.9	140	80	60	2.09	0.017	1.86	1.8

Notes:

σ_p'	-	Apparent preconsolidation pressure
σ_{vo}'	-	Computed existing vertical effective stress
Cc	-	Compression index
Cr	-	Recompression index
e _o	-	Initial void ratio
OCR	-	Overconsolidation ratio

A summary of engineering properties for the clay deposit is included in Figure B14, which includes the parameters calculated/measured within the clay during both the current and past Geocres investigations.

4.6 Till

A deposit of glacial till was encountered directly beneath the clay at Boreholes 16-2 and 17-10, and below the sandy silt deposit at Borehole 16-4 at depths of about 10.5 to 11.3 m below the existing ground surface. The till generally consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of silty sand.

The till was fully penetrated at Boreholes 16-2 and 16-4 and about 2.3 and 2.7 m in thickness, respectively. The till was not fully penetrated at Borehole 17-10 but was proven to extend to a depth of about 11.3 m below the ground surface (i.e., Elevation 37.3 m). Standard penetration test 'N' values of "Weight of Hammer" to "in excess of 50" blows per 0.3 m of penetration were measured in the glacial till, indicating a very loose to very dense state of packing, although the higher 'N' values could reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix. More generally, the till was found to be compact, with 'N' values between about 12 and 18.

The measured water contents of five samples of till were between about 8 and 12 percent. Grain size distribution testing was carried out on one sample of the till, the results of which are provided on Figure B9. The sample was, however, retrieved using a 50 mm diameter sampler and therefore the results do not reflect the larger gravel, cobble and boulder content of the deposit.

4.7 Sandy Silt, Gravelly Sand, and Sand and Gravel

A localized thin deposit of sandy silt was encountered beneath the clay at Borehole 16-4. The sandy silt was about 0.9 m in thickness and extends to about 11.3 m depth below the existing ground surface (i.e., Elevation 37.1 m). A standard penetration test carried out in the sandy silt deposit gave an 'N' value of 3 blows per 0.3 m of penetration, indicating a very loose state of packing. The measured water contents of one sample of sandy silt was 28 percent.



Deposits of sand and gravel and gravelly sand were encountered directly beneath the clay at Boreholes 16-1, 16-3, and 16-5 and below the glacial till at Borehole 16-2 at depths of about 9.9 to 13.0 m below the existing ground surface (i.e., Elevations 35.9 to 38.3 m). The sand and gravel deposits were fully penetrated at most of the borehole locations, with the exception of Borehole 16-5, where the borehole was terminated within the deposit. The sand and gravel generally ranged from about 0.9 to 2.5 m in thickness, and was at least 3.0 m thick at Borehole 16-5. Standard penetration test 'N' values of 12 to "in excess of 50" blows per 0.3 m of penetration were measured in the deposit, indicating a compact to very dense state of packing, although some of the higher 'N' values reflect the bedrock surface or the presence of cobbles and boulders, rather than the state of packing of the soil matrix. The measured water contents of three samples of sand and gravel were between about 8 and 14 percent and the measured water contents of five samples of gravelly sand were between about 10 and 20 percent. Grain size distribution testing was carried out on two samples of the gravelly sand and two samples of the sand and gravel, the results of which are provided on Figure B10 and B11, respectively.

4.8 Auger Refusal and Bedrock

Refusal to augering was encountered at Borehole 16-4 at about 14.0 m depth (Elevation 34.4 m).

Bedrock was encountered beneath the till and sand and gravel deposits at Boreholes 16-1 to 16-3, inclusive, at depths ranging from about 12.6 to 13.9 m (i.e., Elevations ranging from 34.9 to 35.9 m). The bedrock was cored between 3.0 and 4.1 m using a NQ drill bit and rods. Photos of the bedrock core are shown on Figures B15 to B17 in Appendix B, following the laboratory test results.

The following table summarizes the bedrock surface or refusal depths and elevations as encountered at the borehole locations during the current and previous Geocres investigations at the site. Only the previous bedrock surface information is included where bedrock was proven by coring.

Borehole Number	Borehole Location with respect to Bridge Structure	Existing Ground Surface Elevation (m)	Depth to Bedrock/Refusal (m)	Bedrock Surface/Refusal Elevation (m)
16-1	Central Pier, Highway 401 Median	48.5	13.4	35.1
16-2	North Abutment	48.8	13.9	34.9
16-3	South Abutment	48.4	12.6	35.9
16-4	North Approach Embankment	48.4	14.0	34.4 ¹
889-2	North Abutment	47.6	12.5	35.1
889-3	North Approach Embankment	47.5	14.3	33.2
889-4	South Abutment	47.6	12.2	35.4
889-5	South Approach Embankment	47.7	12.2	35.5
BH-2	North Abutment	47.5	12.9	34.6
BH-4	South Abutment	47.5	12.9	34.6

Note 1: Refusal to auger advancement.



The bedrock encountered in these boreholes consist of fresh, thinly to thickly bedded, grey, fine grained limestone with occasional shale interbeds. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples during the current investigation typically ranged from about 85 to 100 percent, indicating a good to excellent quality rock. The upper 0.9 m of bedrock cored at Borehole 16-1 had an RQD of about 33 percent, however the lower value was likely due to a noted vertical fracture.

The results of compressive strength point load testing carried out on six bedrock core samples ranged from about 35 to 157 MPa, as shown on Figure B12. The results of unconfined compressive strength testing carried out on two bedrock core samples were about 107 and 131 MPa, as shown on Figure B13. The results indicate a medium strong to very strong rock.

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

4.9 Groundwater Conditions

Monitoring wells were installed in Boreholes 16-1 and 16-5B. The water levels measured in the wells are summarized in the following table:

Borehole	Ground Surface Elevation (m)	Water Level Depth (m)	Water Level Elevation (m)	Date
16-1	48.5	1.1	47.4	July 29, 2017
16-5B	48.1	0.3	47.8	July 29, 2017

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.



5.0 CLOSURE

This Foundation Design Report was prepared by Ms. Kim Lesage, P.Eng., and was reviewed by Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project.

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MTO Designated Contact



KM/KSL/FJH/mvrd

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PART B

FOUNDATION DESIGN REPORT
COUNTY ROAD 2/34 UNDERPASS REPLACEMENT
SITE 31-232
HIGHWAY 401
LANCASTER, ONTARIO
W.P. 4013-11-01



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed replacement of the existing County Road 2/34 Bridge (Site No. 31-232) over Highway 401 in Lancaster, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the detail design of the foundations for the replacement structure. It is understood that the bridge is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC). In accordance with Section 4.4.2 of the CHBDC, we understand that the proposed bridge structure has an importance category of *other* bridge.

Where comments are made on construction, they are provided to highlight those aspects that could affect the detail design of the project, and for which special provisions may be required in the contract documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge is shown in plan on Drawing 1 and consists of a two-lane, two-span cast-in-place concrete box girder structure with abutments and central pier founded on piles. The existing structure is aligned approximately northwest to southeast and is about 67 m long and 12 m wide. It is understood that the existing structure, built in 1963, is to be replaced with a two lane, two-span structure with a shift in the alignment to west of the existing structure. The new underpass will be founded on integral abutments located behind (and to the west of) the existing abutment foundation footprints, resulting in a longer bridge. The proposed pavement grades at the new structure will be up to about 0.5 m higher than the existing pavement grades.

The existing embankment loading over the deep sensitive and compressible clay deposit has led to very large settlements of the embankments since the original construction. These settlements resulted in the abutments of the existing structure tilting backwards towards the approach embankments.

6.2 Seismic Design

6.2.1 Site Seismicity and Importance Category

The site falls within the Western Québec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ includes 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. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower but there still exists the potential for significant earthquake events to be generated.

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 GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.



6.2.2 Seismic Site Classification

Analysis of Surface Waves (MASW) geophysical testing was carried out at the proposed staging area location to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The shear wave velocities measured are presented in a technical memorandum (see results in Appendix D) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy is 227 m/s. Based on these results and using Table 4.1 of the CHBDC, it is considered that a Site Class of D would be applicable for the design of the structure.

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 bridge (latitude 45.136 and longitude 74.497), the following are the reference Site Class C (reference) peak seismic hazard values based on the 5th generation seismic hazard maps published by the GSC.

Seismic Hazard Values for Reference Ground Condition Site Class C

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.379
PGV (m/s)	0.258
S (0.2) (g)	0.596
S (0.5) (g)	0.313
S (1.0) (g)	0.150
S (2.0) (g)	0.069
S (5.0) (g)	0.018
S (10.0) (g)	0.006

The values given above 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 D) in accordance with Section 4.4.3.3 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 percent of the PGA for Site Class C where $Sa(0.2)/PGA$ is less than 2.0. Based on this requirement a PGA_{ref} value of 0.303 for the 2,475 year return was used.

The corresponding site-specific seismic hazard values given in the table below can be used for design.



Seismic Hazard Values for Reference Ground Condition Site Class D

Seismic Hazard Values	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.374
PGV (m/s)	0.310
S(0.2) (g)	0.595
S(0.5) (g)	0.375
S(1.0) (g)	0.196
S(2.0) (g)	0.094
S(5.0) (g)	0.025
S(10.0) (g)	0.008

The fundamental period of the structure is expected to be greater than 0.5 s, which, in consideration of its *other* importance category and the site-specific seismic hazard values given above, would indicate that the bridge structure falls in Seismic Performance Category 2 in accordance with Table 4.10 of the CHBDC. Based on this Seismic Performance Category and the *regular* geometry of the bridge, it is understood that the structure will be designed using a “force-based approach” as defined in the CHBDC.

6.3 Bridge Foundations

Based on the subsurface conditions, only deep foundation options have been considered for the replacement of the existing County Road 2/34 underpass, as shallow foundations would not provide sufficient bearing resistances or acceptable settlement performance for the structure.

A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the limestone bedrock are feasible for support of the replacement bridge structure. This option would provide high geotechnical resistances and minimal post-construction settlements; in addition, this option would permit the use of integral abutments. The use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which may contain cobbles and boulders) and seating onto the limestone bedrock.
- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles driven to refusal on the limestone bedrock could also be considered as a deep foundation option for support of the abutments and central pier. This foundation option would have similar advantages to steel H-piles in terms of high geotechnical resistances and minimal settlements. This option may also permit the use of integral abutments. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation if cobbles and/or boulders are encountered within the till or sand and gravel deposits during driving.



- **Caissons:** Caissons deriving their support from bearing within the limestone bedrock are also feasible for this site. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground loss from potential flowing clay or water-bearing cohesionless layers below the clay during construction. In addition, the caissons would have to be socketed at least nominally into the bedrock to permit cleaning of the caisson bases, and such sockets would have to be advanced by rock coring and/or chisel drilling into the medium strong to very strong limestone bedrock. This foundation option is considered feasible at the pier.

Based on the above considerations, the preferred options from a geotechnical/foundations perspective is to support the abutments and centre pier on steel H-piles driven to found on the bedrock for the bridge replacement. Due to the space restrictions in the median, the central pier may also be founded on caissons.

6.3.1 Feasibility of Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and expansion bearings at abutments.

The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Integral abutments are not recommended for sites where the soil is susceptible to liquefaction, slip failure, sloughing or boiling. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site would similarly not be considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35°; and abutment wall heights less than 6 m without a retained soil system.

From a foundation perspective, integral abutments are considered feasible at this location.

6.3.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed underpass structure and foundation system may be classified as having medium to large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.6 below.

6.4 Driven Steel Pipe (Tube) Pile or Driven Steel H-Pile Foundations

6.4.1 Founding Elevations and Pile Driving

The abutments and central pier for the replacement bridge may be supported on steel H-piles or steel pipe piles driven to found on the limestone bedrock. Based on the geotechnical investigations carried out at the site, the following pile tip elevations are recommended for design of piles:



Foundation Element	Borehole Numbers	Bedrock Surface / Pile Tip Elevation
North Abutment	16-2, 889-2, BH-2	34.6 – 35.1
Central pier	16-1	35.1
South Abutment	16-3, 889-4	35.4 – 35.8

The pile caps should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

Based on the results of the field investigations, occasional cobbles and boulders are expected in the sand and gravel and till deposits. Therefore each pile should be reinforced at the tip with suitable driving points (such as Titus Standard 'H' Points for H-piles or Titus Open Cutting Shoe for pipe piles, or equivalent) to reduce the potential for damage to the piles during driving through soils that may contain boulders, in accordance with OPSS.PROV 903 (*Deep Foundations*). For steel pipe piles the driving shoes should be in accordance with OPSD 3001.100 Type II (*Steel Tube Pile Driving Shoe*).

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A Non-Standard Special Provision (NSSP) for vibration monitoring should be included in the contract documents and has been included in Appendix E of this report. A maximum peak particle velocity of 100 mm/s is recommended at the existing structure. The piles further from the existing structure should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the pile driving criteria for the remaining piles.

The piles for the new abutments and pier are not expected to be in conflict with the vertical or battered piles supporting the existing structure

6.4.2 Axial Geotechnical Resistance

For design of HP 310 x 110 piles driven to bedrock at the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 2,000 kN (geotechnical resistance factor of 0.5). For closed-end, concrete-filled, 324 mm diameter steel pipe piles (with 13 mm wall thickness) driven to bedrock, the factored axial resistance at ULS may also be taken as 2,000 kN. The factored ultimate geotechnical axial resistances (ULS) for piles driven on the limestone bedrock are higher than the above values which are limited by the structural resistances for the piles.

Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*). The drawings should incorporate the appropriate note stating that the piles should be equipped with suitable driving points and should be driven to bedrock. For piles driven to refusal on bedrock, and as described in OPSS.PROV 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.



The pile termination or set criteria for H-piles will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known.

6.4.3 Downdrag Load (Negative Skin Friction)

Since there is no grade raise proposed at the pier, no downdrag forces are anticipated on piles supporting the pier.

The placement of earth fill for the new embankments, adjacent to and over the existing embankments, will raise the effective stress level in the clay deposit which underlies the site. This increase in stress could lead to elevated settlement of the underlying clay deposit and corresponding downdrag loads on the piles at the abutments, which could in turn reduce the available capacity of the piles.

These downdrag loads (or negative skin friction) will need to be taken into account during the design of the piles supporting the bridge abutments. No downdrag loads would be expected at the piers.

The downdrag loads could vary depending on the selected embankment fill material, on the sequence of construction, and on the underside of pile cap elevation. In calculating the magnitude of the downdrag force, the method described in the Canadian Foundation Engineering Manual was considered. Assuming an underside of the pile cap of about Elevation 50 m, the unfactored downdrag load acting on a single HP 310 x 110 pile, over the length of pile is estimated to be between 600 and 900 kN. The unfactored downdrag load acting on a single 324 mm diameter steel pipe piles would be slightly lower. If the settlements are mitigated as discussed further in Section 6.7.3, the downdrag loads at the abutments would be greatly reduced. If EPS lightweight fill is used to construct the embankments, particularly in the area of influence of the abutments, no downdrag loads would be expected on the piles.

The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the CHBDC.

6.4.4 Lateral Geotechnical Resistance

To accommodate the movements associated with integral abutments, a sand-filled corrugated steel pipe (CSP), 0.6 m in diameter and 3 m in length, is typically provided extending below the underside of the pile cap. An NSSP for the supply and installation of CSP's should be included in the contract documents and a sample has been included in Appendix E of this report. The grading of the sand backfill in the CSP is also given in the NSSP.

The ULS *geotechnical* resistance to lateral loading may be calculated using passive earth pressure theory, assuming that it acts over the the pile shaft to a depth equal to six pile diameters below the underside of the pile cap and an equivalent width equal to three pile diameters.

The ULS lateral resistance of a pile group may be estimated as the of the sum of the individual pile resistances across the width of the pile group in plan, perpendicular to the direction of the applied lateral force, and a depth of six pile diameters below the underside of the pile cap.

The ULS resistances obtained using the above parameters represent unfactored values; in accordance with the CHBDC, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.



6.4.5 Lateral Soil-Structure Interaction Springs

An assessment of the lateral performance of the proposed bridge abutments and pier foundations may be carried out considering the effects of lateral loading on the structure. The lateral response of the pile foundations can be analysed considering the soil-structure interaction between the pile(s) and the surrounding soils using the load transfer method. From a geotechnical perspective, the lateral load-displacement behaviour of the soils can be modeled using P-y curves (CFEM, 2006).

P-y curves representing the non-linear response of the soils and bedrock under lateral loading from the pile foundations have been generated using the commercially available program LPILE (version 2016) produced by ENSOFT Inc. The P-y curves have been calculated considering the static loading option for the lateral soil models. The family of P-y curves calculated at 0.5 m to 1.0 m depth increments are presented in tabular format and shown graphically in the attached Appendix F, for: (i) a single HP310x110 steel pile at abutments and (ii) a single HP310x110 steel pile at the pier.

The P-y curves presented in Appendix F are for a single pile and do not include any effects of group action. Group action for lateral loading must be considered when the pile spacing in a group in the direction of loading is less than seven (7) pile diameters. These 'Group Effects' can be incorporated into the analysis using a method that modifies the single pile P-y curve(s) by a reduction factor termed a 'P-Multiplier'. Generalized P-Multipliers (or reduction factors) for a range of pile spacing's and loading directions are provided in Section C6.11.3.4 of the CHBDC.

6.5 Caisson Foundations

6.5.1 Founding Elevations and Caisson Installation

Alternatively, support of the central pier may be provided by caisson foundations.

For design purposes, the following bedrock surface elevations should be considered:

Foundation Element	Borehole Numbers	Bedrock Surface Elevation (m)
Central Pier	16-1	35.1

The native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could "flow" into the auger hole during drilled shaft installation if left unsupported. Furthermore, there are water-bearing cohesionless layers within the sand and gravel and glacial till deposits. The use of a temporary or permanent liner or casing will therefore be required in order to advance the drilled shafts with minimal loss of ground. Casing installation through the bouldery glacial till and sand and gravel deposits may be difficult. Churn drilling techniques could be required.

Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is nominally socketed into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy material at depth, could flow under the casings, at the interface with the bedrock. The casing should be extended so that it is "seated" a minimum of 300 mm into the bedrock.

Alternatively, the caisson excavations could be cleaned using methods such as airlifting prior to concreting, and tremie concreting techniques may be required for placing concrete. A minimum caisson diameter of 0.9 m is recommended, to facilitate inspection.



If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.7 m for frost protection purposes, per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

Similar to pile installation, vibration monitoring should be carried out during caisson installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A NSSP for vibration monitoring should be included in the contract documents and a sample has been included in Appendix E of this report. A maximum peak particle velocity of 100 mm/s is recommended at the existing abutments. The caissons further from the existing structure should be installed first, in order to check the vibration level at the existing structures and, if necessary, alter the installation method for the remaining caissons.

6.5.2 Axial Geotechnical Resistance

End-bearing resistance may be considered in design provided that the base of each caisson is thoroughly cleaned of any cuttings or other material. The unfactored geotechnical end-bearing resistance at ULS can be taken as 5 MPa.

SLS resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement will be greater than the factored axial geotechnical resistance at ULS.

End bearing for the caisson relies solely on the quality of the rock surface at the base of the excavation. As such, it is imperative that the rock surface be adequately cleaned of loose soils, rock, and debris prior to construction of the caisson.

6.5.3 Downdrag Load (Negative Skin Friction)

The placement of granular embankment fill would raise the effective stress level in the clay deposit, leading to some consolidation of the deposit. As discussed previously, this condition would result in downdrag forces on caissons. Since there is no grade raise proposed at the pier, no downdrag forces are anticipated on caissons at the pier.

6.5.4 Lateral Geotechnical Resistance

The ULS geotechnical resistance to lateral loading may be calculated using passive earth pressure theory as outlined in Section C6.11.2.2.1 of the Commentary to the CHBDC.

The ULS lateral resistance of a caisson group may be estimated as the sum of the individual caisson resistance across the force of a caisson group, perpendicular to the direction of the applied lateral force.

In accordance with Section 6.9.1 of the CHBCD, a resistance factor of 0.5 is to be applied in calculating the horizontal resistance.

The ULS lateral resistance calculated would provide a limit on the lateral geotechnical resistance offered using the p-y curves below for the upper 6 diameters of the caisson.

6.5.5 Lateral Soil-Structure Interaction Springs

Lateral soil-structure interaction springs for caisson foundations can be generated if caisson foundations are selected for support of the pier.



6.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

6.6.1 Lightweight Fill Embankment

As discussed later in Section 6.7.3 of this report, the use of expanded polystyrene (EPS) lightweight embankment fill is proposed as one option for mitigating the potential embankment and roadway settlements due to compression of the underlying silty clay deposit. In regards to the lateral earth pressures, the low unit weight and relatively high mechanical strength characteristics of the EPS blocks (in comparison to soil) will alter the design of lateral earth pressures. For design purposes, the EPS could be assumed to have a unit weight of 1 kN/m^3 ; this low unit weight should be considered in the calculation of the vertical stress level in the underlying material, and thus the horizontal lateral pressure applied to the abutment wall. Furthermore, because the EPS blocks would hold a vertical face without support, the lateral earth pressure applied by the EPS itself could be quite minor, resulting only from the resistance to lateral expansion of the material under vertical loading (i.e., from the 'poisson' effect), which is however difficult to quantify (and highly dependent on how tightly fitting the EPS blocks are placed against the abutment). It is therefore conventional practice that the lateral earth pressures from the EPS can be neglected. Where the backfill is relied upon to provide passive resistance to the abutment, the contribution of the EPS itself should also be neglected, but the effect of the lower unit weight and lower vertical stress level must be considered in assessing the passive resistance from the underlying backfill.

A geosynthetic sheet drainage layer, such as Mira Drain, should be provided behind the abutment wall, adjacent to the EPS (as opposed to granular layer) to minimize lateral earth pressures on the abutments and additional loading on the compressible clay soils.

6.6.2 Granular Fill Embankment

The following recommendations are made concerning the design of the walls if granular backfill is used:

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.



- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.7 m behind the back of the wall (Case (a) on Figure C6.20 of the *Commentary* to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the *Commentary* to the CHBDC).

6.6.2.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0

- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.



- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6.2.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.6.2, above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.
- Seismic Active Pressure Coefficients, K_{AE}

	Design Earthquake	Site PGA	Granular A	Granular B Type II	SSM
Yielding Wall	2,475 Yr	0.374	0.39	0.39	0.46
Non-Yielding Wall	2,475 Yr	0.374	0.55	0.55	0.66

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d , (kPa);

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m³), as given previously;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).



6.7 Approach Embankments

It is understood that the alignment of the underpass structure will be shifted to the west (immediately adjacent to the existing alignment), and therefore a widening/new embankment will be required about 8 m in height and about 15 to 20 m in width at the crest to accommodate this shift.

In general, the surficial soils at the location of the proposed County Road 2/34 Underpass alignment consist of a surficial layer of fill and/or topsoil (with some localized peat) underlain by a thick compressible clay deposit. The clay is underlain by relatively thin deposits of glacial till and/or sand and gravel over limestone bedrock.

6.7.1 Geocres Review of Existing Embankments

As part of the available correspondence in Geocres 31G00-144, it is understood that a settlement instrumentation program was recommended and carried out as part of the original bridge construction at the site in light of the significant settlements expected. The design recommendations required that the embankments be constructed one year in advance of the bridge to allow for some of the settlements to take place. However this requirement was not followed during construction.

The results of the settlement monitoring between 1962 and 1984 showed that the construction of the approach embankments led to total settlements (i.e., primary and secondary) of over 1.5 m, as shown on Figure F1 in Appendix F. Furthermore, the Geocres information indicates horizontal movements between the bridge deck and the abutments in 1967 to be about 125 mm at the north abutment and about 75 mm at the south abutment.

The large settlements were anticipated, however the lateral abutment movements towards the center of the approach embankment settlement were not anticipated.

6.7.2 New Embankment Construction

The topsoil and peat are compressible soils that are expected to experience settlement under increased load. It is recommended that all the topsoil and peat, as well as any organic matter and softened/loosened soils (including surficial fill) present within the footprint of the embankment widening be stripped prior to placement of the new embankment fill. The topsoil, peat, and surficial fill material should be stripped to the underlying clay.

The new embankment fill associated with the grade raise and widening for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (Earth Excavation and Grading) and OPSS.PROV 501 (Compacting). The use of EPS lightweight embankment fill is discussed further in Section 6.7.3.

Benching of the existing County Road 2/34 embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

The sensitive clay subgrade that will be exposed within the new embankment footprints will be susceptible to disturbance and degradation on exposure to water and construction traffic. Following fill and topsoil removal, travelling over clay the subgrade soils should be minimized to limit the disturbance.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil (OPSS 802 – *Topsoil*) and seeding (OPSS.PROV 804 – *Seed and Cover*) or pegged sod (OPSS 803 – *Sodding*) is recommended as soon as practicable after construction of the embankments.



6.7.3 Approach Embankment Settlement

As discussed in Section 6.7.1, the existing embankment loading over the thick sensitive and compressible clay deposit has led to very large settlements of the embankments since the original construction. These settlements resulted in the abutments tilting backwards towards the approach embankments. Settlement of the existing embankments has likely fully occurred over time since the original construction and the clay deposit below the existing roadway embankment are most likely now normally consolidated (i.e., cannot take on any additional load without overstressing and causing unacceptable settlements).

If conventional earth fill or granular fill is used for the new 8 m high embankments, settlement of the approach embankments will occur as a result of compression of the new embankment fill itself but, more significantly due to consolidation of the clay deposit underlying the approach embankments. Additional settlements would also occur beneath the existing embankments within the zone of influence of the new embankment fill.

The potential settlement of the underlying clay deposit is much more significant than the potential compression of the fill. In order to estimate the magnitude of settlement of the clay underlying the approach embankments, analyses were carried out using the commercially-available 'Settle-3D' software. These analyses were carried out using the preconsolidation pressure profile and parameters presented on the Summary of Engineering Properties, Figure B14.

The coefficient of consolidation of the surficial weathered clay crust, typically being a fissured soil and being stressed within its re-compression limits, is relatively high. Therefore the subgrade settlements resulting from compression of the weathered clay crust would be expected to occur quite rapidly, much will occur during embankment construction, and would not be expected to noticeably exceed the compression of the embankment fill itself. The greatest compression will occur for the underlying unweathered clay. The potential compression of the weathered crust was therefore neglected in these analyses.

The calculated ultimate effective stress levels in the *unweathered* clay would exceed the deposit's preconsolidation pressure if it is built with granular embankment fill to full height. The consolidation settlements would therefore occur in the 'virgin' compression range and be significant in magnitude. Pore water would need to be expelled for these settlements to occur and therefore, due to the low hydraulic conductivity of the clay, considerable time could be needed for these settlements to complete.

Based on the indicated embankment heights as well as the assessed existing stress level and preconsolidation pressure profile within the clay deposit, and considering the impacts from the existing embankments, the calculated primary consolidation settlements are estimated to be in the order of 0.8 m (at the location of greatest settlement in the transverse direction).

In the longer term, these settlements would increase beyond the estimates given above due to secondary compression (i.e., creep) of the deposit. It is expected that, over a period of 20 years following construction (the likely approximate time until the first repaving, when the profile could be corrected) secondary compression could increase these settlements by between 50 and 75 mm. Over a 50 year time frame, the anticipated total settlement (i.e., primary consolidation plus secondary compression) could be in the order of about 1.0 m.

The results of the settlement analysis for the proposed granular fill embankment is shown on Figure F2 in Appendix F. The results show the maximum settlements that are expected if granular embankment fill is used to construct the new embankment to full height. These settlements would also be entirely differential relative to the



structure (which would be supported on deep foundations on bedrock) and differential in the transverse direction between the existing embankment and the west crest of the new embankment.

The calculated settlements are therefore considered to be excessive and would have a negative impact on the roadway performance. The calculated settlement values exceed the usual values accepted by MTO for the approaches to bridges for non-freeways, as shown in the following table:

Distance from Abutment (m)	Tolerable Settlement (mm)
0 to 20	25
20 to 50	50
50 to 75	100
>75	200

The differential settlement rate transversely across the top of the roadway surface also needs to be limited to 100H:1V for non-freeways. The new roadway will be about 14 m wide and therefore the design should limit the differential settlements to a maximum of 140 mm in the transverse direction.

These tolerable settlements are based on roadway performance criteria and are therefore applicable only to the life-span of a pavement; at each pavement rehabilitation, the roadway profile could be corrected and any differential settlement eliminated. That pavement life-span is typically taken to be 15 to 20 years. These criteria are also only applicable to the situation where settlement-sensitive services/utilities are not present beneath the embankment. Where such services are present, the tolerable settlement over the full life-span of the utility needs to be considered.

Significant settlement of the clay and approach embankments would likely also impact the feasibility of integral abutments.

Given the significant magnitude of anticipated settlements, and their continuous/long-term nature, it is considered that periodic re-paving to correct for the settlement is not a feasible option for addressing/mitigating the settlement effects. Subexcavation of the clay would also not be feasible due to its thickness. The following feasible options were therefore considered for mitigating the anticipated settlements:

- 1) Lightweight Fill: Lightweight fill materials such as expanded polystyrene (EPS) could be used for the embankment construction, thereby reducing the stress increase on the compressible clay deposit to a level so that the settlements, within each specified distance from the abutment, would be within acceptable tolerances at paving time for the geotechnical criterion.
- 2) Preloading with Wick Drains together with Lightweight Fill: The new embankment areas could be preloaded, in part, and allowed to settle in advance of the roadway being paved or put into service over the new approach embankments. Wick-drains would be used to decrease the preload time and the magnitude of post-construction creep settlements. Due to the sensitive nature of the clay and consolidation characteristics, the preload height would have to be limited and the use of some EPS would still be required for this option to be feasible.



- 3) Rigid Inclusions: The installation of Rigid Inclusions (RI) is another alternative for mitigating settlements beneath embankments. RI's constructed of ready-mix concrete installed within the clay soil using specialty equipment would be suitable for this site. Rigid Inclusions could be installed in the clay deposit, up to original ground surface, to transfer the stress from the embankment loads down to the glacial till or bedrock. A Load Transfer Platform (LTP) created using granular material and geogrid, and/or concrete would be constructed above the Rigid Inclusions (i.e., beneath the embankment) to transfer the embankment loads to the columns.
- 4) Deep Soil Mixing: Deep soil mixing is another alternative for mitigating settlements beneath embankments. Deep soil mixing consists of in situ mechanical mixing of the native soil through a process that breaks down the soil without extraction while injecting a stabilizing agent in the mix at low pressure to

The advantages, disadvantages, relative costs, and risks associated with these four options are provided in a technical memorandum in Appendix G.

Based on discussions with the design team and the MTO following the preparation of the technical memorandum in Appendix G, it is preferred to use ground improvement (i.e., rigid inclusions or deep soil mixing) at the site and then to construct the embankments for the new alignment using granular fill. The selection of the ground improvement method will be the responsibility of the contractor, who will obtain a proprietary design for this project by a ground improvement specialty contractor.

Further details on the construction of the embankments using EPS lightweight fill are provided in the following section, although it is understood that this is not the preferred alternative for embankment design.

6.7.4 Expanded Polystyrene Lightweight Fill Embankment Construction

The stress and settlement analyses indicate that the total thickness of conventional embankment fill (including the pavement structure) needs to be limited if significant post-construction settlements are to be avoided. Given the required thickness of material needed for the pavement structure (about 1.0 m), the protective concrete slab over the EPS (discussed below), and a granular working/levelling pad for placement of the EPS, it is considered that only minimal additional embankment fill could be placed to maintain the clay below its preconsolidation pressure. Therefore the entire thickness of the embankment (under the pavement structure) would need to consist of EPS for minimal settlements of the approach embankments to occur, as in the area adjacent to the pile supported abutments.

However, if time allows for about a 1 year preload period, some thickness of granular embankment fill away from the abutment areas may be placed as a preload (i.e., to allow for some of the primary settlement to occur in advance of the roadway being paved or put into service) over the new approach embankments. This option could lessen the thickness of EPS required for embankment construction. Some of the existing embankment material would nonetheless need to be removed adjacent to the new embankments (i.e., steepening of the existing side slope adjacent to the new embankment).

It is important to limit these settlements not only to avoid roadway distortion, but also to avoid important differential settlements between the approach embankments and the bridge structure (which will be supported on deep foundations, and therefore not subjected to these consolidation settlements). It is considered that the post-construction settlements be restricted to the tolerable amounts accepted by MTO, provided in the table in Section 6.7.3. These criteria would restrict the thickness of earth fill 'preload' used for the embankment construction, as follows.



FOUNDATION REPORT SITE NO. 31-232
COUNTY ROAD 2/34 UNDERPASS REPLACEMENT AT HWY 401

Distance from Abutment (m)	Allowable Amount of Granular Fill ¹ (m)
0 to 20	0.0
20 to 50	0.5
50 to 75	1.0
>75	1.5

Note ¹: In addition to the pavement structure, expected to be about 1 m thick.

Even these limited thicknesses of granular embankment fill would likely bring the ultimate stress level in the underlying clay deposit to near or over the preconsolidation pressure. A settlement monitoring program will need to be implemented to monitor the settlements prior to, during, and following the 'preload' placement. Settlement monitoring would provide an indication that the settlements are occurring as anticipated and to determine if the granular fill heights have to be altered in consideration of the 1 m pavement structure thickness to be placed above the EPS. Additional guidelines for the settlement monitoring can be prepared if this alternative is adopted and should be included in the contract documents.

Prior to the placement of the EPS, the existing embankment side slopes should be cut down (i.e., unloaded) prior to the new embankment construction. This unloading would allow for the placement of the new pavement structure without bringing the ultimate stress level in the underlying clay deposit to over its current preconsolidation pressure and thus minimize settlements. Roadway protection, to the native ground, would likely be required for the 'unloading' of the existing side slopes.

The EPS will need to be covered with a concrete slab to protect it from being overstressed by the traffic loads; overstressing of the EPS could lead to rutting of the pavement surface. A concrete slab thickness of 125 mm with reinforcing that is typical for the protective slab.

A suitable lightweight fill type would be EPS22 in accordance with ASTM D6817-11, or equivalent.

The EPS is potentially soluble in hydrocarbons. To guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision), it is general practice to cover the outside surface of the EPS with 10 mil polyethylene sheeting.

The blocks beneath the side slopes can step up to match the 2H:1V side slope and, once covered with the polyethylene sheeting, can then be covered with soil.

A 0.3 m thick layer of OPSS Granular A or Granular B Type II would be appropriate as a levelling pad beneath the EPS.

At the end of the EPS, a transition treatment will be required to avoid differential frost heaving at the edge of the EPS (between the insulated conditions above the EPS and the potentially frost susceptible fill or subgrade beyond the EPS). The longitudinal treatment could consist of the sub-excavation of the subgrade soils beneath the end of the EPS to a depth of 1.7 m below pavement grade (the design frost depth), replacement with compacted Granular B Type II, and the construction of a longitudinal 20H:1V frost taper beyond the edge of the EPS, in a manner similar to OPSD 205.060. That detail could however be modified such that the taper beneath the EPS itself could be made at a slope of 2H:1V.



An NSSP providing additional information on the EPS material and its placement as well as the concrete protective slab should be included in the contract documents. A sample NSSP is provided in Appendix E.

6.7.5 Global Stability

Static and seismic slope stability analyses of the proposed embankments (for the condition of being constructed using conventional earth fills) were carried out with the commercially available SLOPE-W software (produced by Geo-Studio 2007), using the soil parameters given in the following table.

Soil Stratum	Bulk Unit Weight (kN/m ³)	Shear Strength Parameters	
		Angle of Internal Friction (°)	Undrained Shear Strength (kPa)
Embankment Fill	21.5	32	-
Weathered Crust	17.1	-	100
Grey Clay	15.5	-	21-32
Glacial Till	19.0	Impenetrable	

The unit weights of the weathered clay crust and the unweathered silty clay were inferred from the measured water content data for these deposits, as shown on Figure B14.

The mobilized/available undrained shear strength of the unweathered clay (C_u) was inferred from the results of the in situ vane testing as well as from the results of the laboratory oedometer consolidation testing. The in situ vane test results were corrected based on the material's plasticity using Bjerrum's relationship $C_u = \mu S_u$, where μ is a function of the soil's plasticity index.

Due to the significant difference in shear strength and stress-strain response of the sensitive clay versus the underlying glacial till deposit, the failure surface for a deep-seated instability of the embankment is unlikely to penetrate the glacial till and that strata was therefore treated as being impenetrable by the failure surface.

The analyses were carried out for undrained (i.e., short-term) conditions. Undrained conditions represent the critical condition experienced during and immediately following construction of the embankments. With time, the excess pore water pressures generated in the clay deposit as a result of the loading would dissipate and 'drained' conditions would exist, with a higher factor of safety against instability. A minimum factor of safety of 1.3 is considered acceptable against undrained deep-seated embankment instability.

The stability of the embankments was also evaluated under seismic loading conditions. The minimum factor of safety value that is typically required against instability during a seismic event is 1.1. A horizontal seismic coefficient of 0.185 was used for the analyses. This value is based on the peak horizontal ground acceleration for the site provided in Section 6.2.3 (with half that value being used, per standard practice), considering the potential amplification of the seismic ground motions that could occur through the clay deposit.

The results of the stability analyses indicate that, even with appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the up to 8 m high embankments with side slopes at 2H:1V will *not* have an acceptable factor of safety against deep-seated rotational instability for the undrained static or for seismic conditions. It should be noted that any failure would occur on the west side of the new embankments and not through the existing embankments towards the east where the clay has had over 50 years to consolidate and gain strength.



The results indicate that embankments up to about 7 m in height would have an acceptable factor of safety. Following placement of the 7 m high embankment, it would be necessary to allow the pore pressures to dissipate and the clay to consolidate and gain strength before the remaining amount is placed (i.e., staged construction). Several months would be required before the subsequent lift is placed (and maybe years due to the properties of the clay at the site).

Consideration could also be given to constructing the embankments with temporarily flatter side slopes or using temporary stabilizing mid-height berms. Our analysis show that 3 m high and 15 m wide berms, placed adjacent to the west embankments, would provide an acceptable solution, as shown on Figure F3 in Appendix F.

Nonetheless, the results of the *seismic* slope stability analyses indicate that, even with staged construction or stabilizing berms, the up to 8 m high embankments will *not* have an acceptable factor of safety against deep-seated rotational instability (i.e., at least 1.1). It should also be noted that the low factors of safety for the seismic loading condition (which are too low, even if staged construction is used) are based on relatively conventional analyses. It is possible that more sophisticated analyses (based on the potential displacements) might indicate acceptable seismic performance.

If the embankment were to be constructed (almost entirely) with lightweight fill material (such as EPS), to avoid excessive settlements, the weight of the embankment would be much less and there would therefore also be an adequate factor of safety against instability.

The embankment stability assessment can be confirmed once the embankment materials and construction details have been selected. If Options 3 or 4 are selected, the slope stability analysis would have to be reviewed/carried out following additional consultation with the suppliers since these are both proprietary systems.

6.8 Construction Considerations

The following sections identify construction considerations that may impact the future design and construction.

6.8.1 Existing Utilities

There are several utilities in the area of the proposed bridge construction, particularly a buried Bell fibre optic utility located beneath the south embankment, behind the proposed abutment. If the settlements discussed in Section 6.7.3 will not be mitigated by the use of lightweight fill or alternate settlement mitigation measure, the impact of the potential settlements to the existing utilities will need to be considered. The impact should be limited to a distance of about 3 m (in plan) from the footprint of the new embankments.

6.8.2 Open-Cut Excavations

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

Only minimal excavations are anticipated for this project for subgrade preparation, and the potential removal of the existing structure. Some excavation of the existing embankments will likely also be carried out following construction of the new roadway. Excavations will be made through the existing fill, surficial topsoil and peat, and native weathered clay deposit. No excavations are anticipated in the underlying unweathered clay. The groundwater level is indicated to be at a depth of about 0.2 to 1.1 m (i.e., about Elevation 47.4 to 47.9 m). The soils at the site are generally classified as Type 3 soils according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V). Flatter side slopes may be required where peat is encountered (i.e., 3H:1V side slope for Type 4 soils), however the thickness of peat observed in the boreholes was quite limited.



6.8.3 Temporary Protection Systems

It is anticipated that temporary roadway protection will be required along CR 2/34, adjacent to the new alignment, to permit construction of the new abutments. For the pier footing construction, temporary excavation support may be required in the median, adjacent to the driving lanes of Highway 401. It is considered that the temporary support system could consist of internally braced soldier piles and lagging or steel sheet piling.

The design of the shoring will be entirely the responsibility of the contractor. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation. Traffic loading should be included as a surcharge.

6.8.4 Groundwater and Surface Water Control

The groundwater level at the site is about 0.2 to 1.1 m below existing ground surface (i.e., Elevation 47.4 to 47.9 m). Only minimal excavations are anticipated for the construction of the new structure, which will likely involve minimal groundwater and surface water control. It should be possible to handle groundwater inflows by pumping from well filtered sumps established in the floor of the excavations. Surface water should be directed away from the excavations.



7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Kim Lesage, P.Eng with technical input from Mr. Murty Devata, P.Eng., a senior consultant. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.

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FOUNDATION REPORT SITE NO. 31-232
COUNTY ROAD 2/34 UNDERPASS REPLACEMENT AT HWY 401

Table 1 – Comparison of Foundation Alternatives

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Steel H-piles driven to bedrock	<ul style="list-style-type: none"> Feasible for support of bridge replacement Preferred 	<ul style="list-style-type: none"> High geotechnical resistances and negligible settlement Allows for integral abutment construction 	<ul style="list-style-type: none"> Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles “hanging up” in the glacial till/sand and gravel deposits and lower geotechnical resistances Pre-augering or additional piles may be required Temporary protection systems may be required at the central pier Negative skin friction (downdrag) loads must be considered in design 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Low risk of driven H-piles “hanging up” in glacial till or sand and gravel deposits
Steel pipe (tube) piles, driven to found in bedrock	<ul style="list-style-type: none"> Feasible, but not preferred 	<ul style="list-style-type: none"> Higher geotechnical resistances and negligible settlement Allows for semi-integral and potentially integral abutment configuration 	<ul style="list-style-type: none"> Potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in some piles “hanging up” in the glacial till/sand and gravel deposits and lower geotechnical resistances Pre-augering or additional piles may be required Temporary protection systems may be required at the central pier 	<ul style="list-style-type: none"> Moderate cost 	<ul style="list-style-type: none"> Slightly greater risk than for steel H-piles of pipe piles “hanging up” in glacial till or sand and gravel deposits
Caissons founded on or socketed into bedrock	<ul style="list-style-type: none"> Feasible May be preferred for central pier 	<ul style="list-style-type: none"> Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system High geotechnical resistances and negligible settlement Allows for semi-integral abutment configuration 	<ul style="list-style-type: none"> Permanent casings required to construct caissons Possibility of encountering cobbles or boulders during augering Coring or churn drilling may be required to form nominal socket in bedrock 	<ul style="list-style-type: none"> Moderate to High 	



APPENDIX A

Borehole Records

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 16-1 to 16-9, 16-5B and 17-10



LIST OF SYMBOLS

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 = $(w_l - w_p)$
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_α	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 = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
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	$(\sigma_1 - \sigma_3)/2$ 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 $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. 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.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; 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

Piezo-Cone Penetration Test (CPT)

A 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 (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils Consistency

	c_u, s_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w _p	plastic limit
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
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	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-1				SHEET 2 OF 3		METRIC								
G.W.P. 4013-11-01		LOCATION N 5000079.4; E 226388.0 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG										
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core				COMPILED BY JM										
DATUM Geodetic		DATE September 19 and 20, 2016				CHECKED BY KSL										
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 ---																
35.1	Gravelly SAND, some silt, trace clay, contains cobbles and boulders Compact to dense Grey Wet		12	SS	14											
13.4	Limestone (BEDROCK) Bedrock cored from 13.4 m depth to 17.5 m depth. For coring details see Record of Drillhole 16-1.		13	SS	50/0.2											
			1	RC	REC 32.5%											RQD = 32.5%
			2	RC	REC 100%											RQD = 97%
			3	RC	REC 100%											RQD = 90%
31.1	END OF BOREHOLE															
17.5	Note: 1. Water level in piezometer at a depth of 1.1 m below ground surface (Elev. 47.4 m) on July 29, 2017.															

PROJECT: 12-1121-0193-1140

RECORD OF DRILLHOLE: 16-1

SHEET 3 OF 3

LOCATION: N 5000079.4 ;E 226388.0

DRILLING DATE: September 19 and 20, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
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DEPTH SCALE



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PROJECT 12-1121-0193-1140			RECORD OF BOREHOLE No 16-2			SHEET 2 OF 3			METRIC								
G.W.P. 4013-11-01			LOCATION N 5000109.7; E 226374.7 MTM ZONE (LAT. ; LONG.)			ORIGINATED BY DG											
DIST Eastern HWY 401			BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core			COMPILED BY JM											
DATUM Geodetic			DATE September 23 and 26, 2016			CHECKED BY KSL											
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					
35.9	Silty SAND, some gravel, trace clay (TILL) Compact Grey Wet		13	SS	18												
13.0	SAND and GRAVEL, some silt to silty Compact to dense Grey Wet		14	SS	25												
34.9			15	SS	50/0.15												
13.9	Limestone (BEDROCK) Bedrock cored from 13.9 m depth to 16.9 m depth. For coring details see Record of Drillhole 16-2.		1	RC	REC 100%												RQD = 85%
			2	RC	REC 100%												RQD = 98%
31.9																	
16.9	END OF BOREHOLE Note: 1. Water level at a depth of 3.8 m below ground surface (Elev. 45.0 m) upon completion of drilling.																

PROJECT: 12-1121-0193-1140

RECORD OF DRILLHOLE: 16-2

SHEET 3 OF 3

LOCATION: N 5000109.7 ;E 226374.7

DRILLING DATE: September 23 and 26, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 850

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY												FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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DEPTH SCALE

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Continued Next Page

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

PROJECT <u>12-1121-0193-1140</u>		RECORD OF BOREHOLE No 16-3		SHEET 2 OF 3		METRIC	
G.W.P. <u>4013-11-01</u>		LOCATION <u>N 5000049.9; E 226406.2 MTM ZONE (LAT. ; LONG.)</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>401</u>		BOREHOLE TYPE <u>Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>September 28, 2016</u>		CHECKED BY <u>KSL</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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					w _p w w _L							
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100					20 40 60								
35.9			13	SS	50/0.3		36													
12.6	Limestone (BEDROCK) Bedrock cored from 12.6 m depth to 15.6 m depth. For coring details see Record of Drillhole 16-3.		1	RC	REC 100%		35											RQD = 86%		
			2	RC	REC 100%		34											RQD = 92%		
32.8	END OF BOREHOLE						33													
15.6	Note: 1. Water level at a depth of 5.0 m below ground surface (Elev. 43.4 m) upon completion of drilling.																			

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SHEET 3 OF 3

DATUM: Geodetic

DRILLING CONTRACTOR: CCC

[illegible]




DEPTH SCALE

1 : 60

LOGGED: DG


CHECKED:

PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-4		SHEET 1 OF 2		METRIC	
G.W.P. 4013-11-01		LOCATION N 5000127.4; E 226361.5 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG	
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)				COMPILED BY JM	
DATUM Geodetic		DATE September 22, 2016				CHECKED BY KSL	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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									WATER CONTENT (%)		
48.4	GROUND SURFACE																	
-0.0	Sandy silt (TOPSOIL/FILL)		1	SS	10													
0.1	Dark brown																	
	Moist																	
	Silty clay, some sand and gravel, contains rootlets (FILL)																	
	Grey-brown																	
47.5	Moist																	
0.9	CLAY (Weathered Crust)		2	SS	17													
	Stiff to very stiff																	
	Grey-brown																	
	Moist																	
			3	SS	9													
			4	SS	5													
			5	SS	1													
44.8	CLAY																	
3.6	Firm																	
	Grey with black organic mottling																	
	Wet																	
				6	SS	WH												
				7	SS	WH												
			8	SS	WH													
													</					

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

PROJECT 12-1121-0193-1140			RECORD OF BOREHOLE No 16-4			SHEET 2 OF 2			METRIC															
G.W.P. 4013-11-01			LOCATION N 5000127.4; E 226361.5 MTM ZONE (LAT. ; LONG.)			ORIGINATED BY DG																		
DIST Eastern HWY 401			BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)			COMPILED BY JM																		
DATUM Geodetic			DATE September 22, 2016			CHECKED BY KSL																		
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS			ELEVATION SCALE			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																			
--- CONTINUED FROM PREVIOUS PAGE ---																								
	Silty SAND, some gravel, trace clay, contains cobbles and boulders (TILL) Compact Grey Wet		12	SS	12																			
			13	SS	12																			
34.7																								
34.4	SAND and GRAVEL, some silt, trace clay (TILL) Very dense Wet		14	SS	50/0.2																			
14.0	END OF BOREHOLE AUGER REFUSAL																							
Note: 1. Water level at a depth of 4.2 m below ground surface (Elev. 44.2 m) upon completion of drilling.																								

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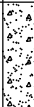
PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-5		SHEET 1 OF 2		METRIC	
G.W.P. 4013-11-01		LOCATION N 5000033.5; E 226414.2 MTM ZONE (LAT. ; LONG.)		ORIGINATED BY DG			
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE September 27, 2016		CHECKED BY KSL			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT		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		W _p	W			W _L
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)					
48.1	GROUND SURFACE						20 40 60 80 100							
0.0	Silty sand (TOPSOIL)													
0.2	Dark brown Moist													
47.3	CLAY, contains rootlets (Weathered Crust)		1	SS	10									
0.8	Stiff to very stiff Grey-brown Moist		2	SS	7									
	CLAY (Weathered Crust)													
	Stiff to very stiff Grey-brown Moist		3	SS	6									
			4	SS	1									
45.1														
3.1	CLAY Firm Grey Wet		5	SS	WH									
43.5														
4.6	CLAY Firm Grey with black organic mottling Wet		6	SS	WH									
			7	SS	WH									
			8	SS	WH									
39.0														
9.1	CLAY Firm Grey Wet		9	SS	WH									
38.2														
9.9	Gravelly SAND, some silt, trace clay Compact Grey Wet		10	SS	15									
			11	SS	14									
			12	SS	17									

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+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-5				SHEET 2 OF 2		METRIC								
G.W.P. 4013-11-01		LOCATION N 5000033.5; E 226414.2 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG										
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)				COMPILED BY JM										
DATUM Geodetic		DATE September 27, 2016				CHECKED BY KSL										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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					WATER CONTENT (%)			
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60				
35.2	Gravelly SAND, some silt, trace clay Compact Grey Wet		13	SS	30		36								o	
12.9	END OF BOREHOLE Note: 1. Water level at a depth of 2.3 m below ground surface (Elev. 45.8 m) upon completion of drilling.															

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

PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-6		SHEET 1 OF 1		METRIC	
G.W.P. 4013-11-01		LOCATION N 5000055.0; E 226344.1 MTM ZONE (LAT. ; LONG.)		ORIGINATED BY DG			
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE September 21, 2016		CHECKED BY KSL			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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
								20	40	60	80	100	20	40	60					
48.6	GROUND SURFACE																			
0.0	Silty sand (TOPSOIL/FILL)																			
0.2	Dark brown Moist		1	SS	7															
	Silty clay, trace gravel, contains rootlets and organic matter (FILL)																			
	Brown to dark grey-brown Moist		2	SS	44															
			3	SS	11															
46.5	CLAY (Weathered Crust)																			
2.1	Stiff to very stiff Grey-brown Moist		4	SS	4															
			5	SS	WH															
45.0	CLAY																			
3.6	Firm Grey with black organic mottling Wet																			
			6	SS	WH															
			7	SS	WH															
41.3	END OF BOREHOLE																			
7.3	Note: 1. Water level at a depth of 5.5 m below ground surface (Elev. 43.1 m) upon completion of drilling.																			



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

PROJECT 12-1121-0193-1140		RECORD OF BOREHOLE No 16-8				SHEET 1 OF 1		METRIC									
G.W.P. 4013-11-01		LOCATION N 5000109.9; E 226436.1 MTM ZONE (LAT. ; LONG.)				ORIGINATED BY DG											
DIST Eastern HWY 401		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)				COMPILED BY JM											
DATUM Geodetic		DATE September 21, 2016				CHECKED BY KSL											
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									
48.5	GROUND SURFACE							20	40	60	80	100					
0.0	Silty clay, contains rootlets (FILL) Grey-brown Moist		1	SS	6												
47.9	Silty clay, some gravel (FILL) Grey-brown Moist																
0.6																	
47.4	Silty sand (FILL) Loose Grey-brown Moist		2	SS	16												
1.1																	
46.5	CLAY (Weathered Crust) Stiff to very stiff Grey-brown Moist		3	SS	8												
2.0																	
44.9	CLAY Firm Grey with black organic mottling Wet		4	SS	4												
3.6																	
			5	SS	2												
			6	SS	WH												
			7	SS	WH												
41.2	END OF BOREHOLE																
7.3	Note: 1. Water level at a depth of 6.3 m below ground surface (Elev. 42.2 m) upon completion of drilling.																

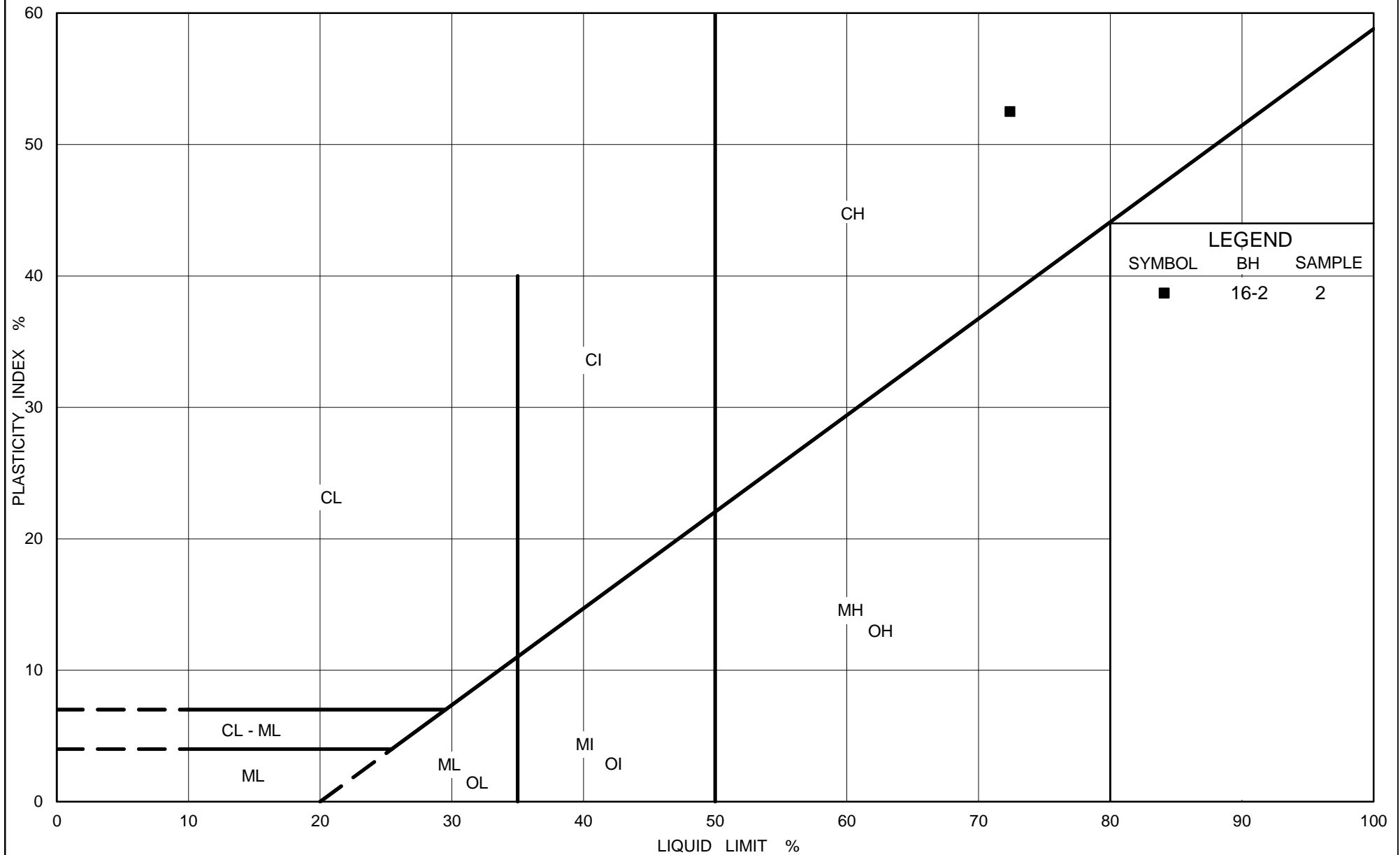
PROJECT		12-1121-0193-1140		RECORD OF BOREHOLE No 16-9		SHEET 1 OF 1		METRIC							
G.W.P.		4013-11-01		LOCATION		N 5000123.4; E 226457.1 MTM ZONE (LAT. ; LONG.)		ORIGINATED BY							
DIST		Eastern HWY 401		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY							
DATUM		Geodetic		DATE		September 21, 2016		CHECKED BY							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		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									
48.4	0.0	GROUND SURFACE													
47.8	0.6	Silty sand (TOPSOIL/FILL) Dark brown Moist		1	SS	13									
		Silty clay, trace gravel, contains rootlets (FILL) Dark brown Moist		2	SS	8									
46.7	1.7	Sandy silt (FILL) Loose Dark grey to grey Moist		3	SS	6									
		CLAY (WEATHERED CRUST) Stiff to very stiff Grey-brown Moist		4	SS	4									
45.1	3.3	CLAY Firm Grey with black organic mottling Wet		5	SS	WH									
				6	SS	WH									
				7	SS	WH									
41.1	7.3	END OF BOREHOLE													
		Note: 1. Water level at a depth of 5.5 m below ground surface (Elev. 42.9 m) upon completion of drilling.													



APPENDIX B

Laboratory Test Results

Figure B1	Plasticity Chart – Silty Clay to Clay (Fill)
Figure B2	Grain Size Distribution Test Results – Clay (Weathered Crust)
Figure B3	Plasticity Chart – Clay (Weathered Crust)
Figure B4	Grain Size Distribution Test Results - Clay
Figure B5	Plasticity Chart – Clay
Figure B6 to B8	Consolidation Test Results
Figure B9	Grain Size Distribution Test Results – Silty Sand (Till)
Figure B10	Grain Size Distribution Test Results – Gravelly Sand (Till)
Figure B11	Grain Size Distribution Test Results – Sand and Gravel
Figure B12	Summary of Laboratory Compressive Strength Testing – Point Load Testing
Figure B13	Summary of Laboratory Compressive Strength Testing – Unconfined Compression Tests
Figure B14	Summary of Engineering Properties
Figure B15 to B17	Bedrock Core Photos



Ontario

Ministry of Transportation

PLASTICITY CHART SILTY CLAY to CLAY (Fill)

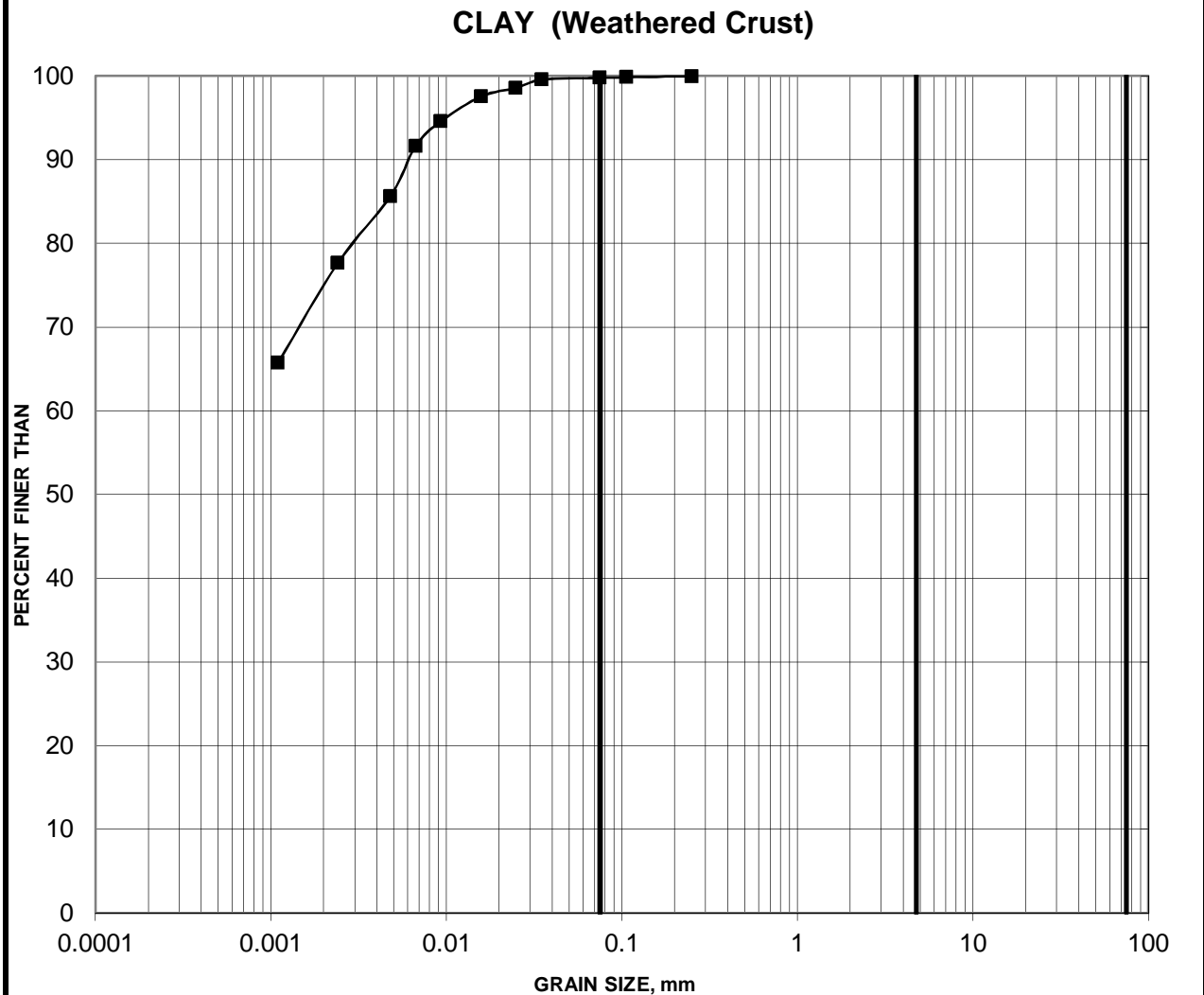
FIG No. B1

Project No. 12-1121-0193 /1140

Compiled By : MI Checked By : CNM

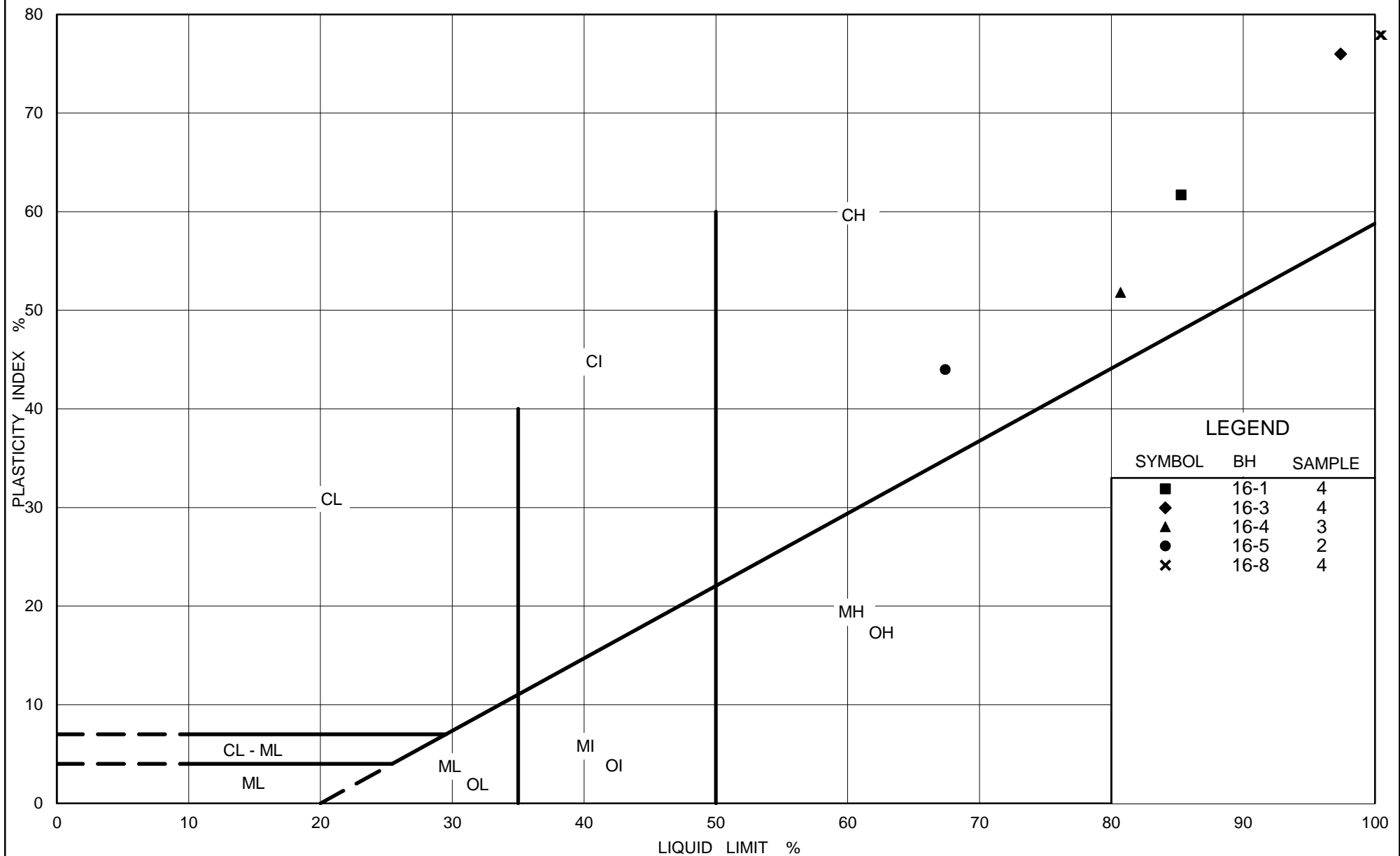
GRAIN SIZE DISTRIBUTION

FIGURE B2



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

Borehole	Sample	Depth (m)
16-2	5	2.29-2.90



Ontario

Ministry of Transportation

PLASTICITY CHART CLAY (Weathered Crust)

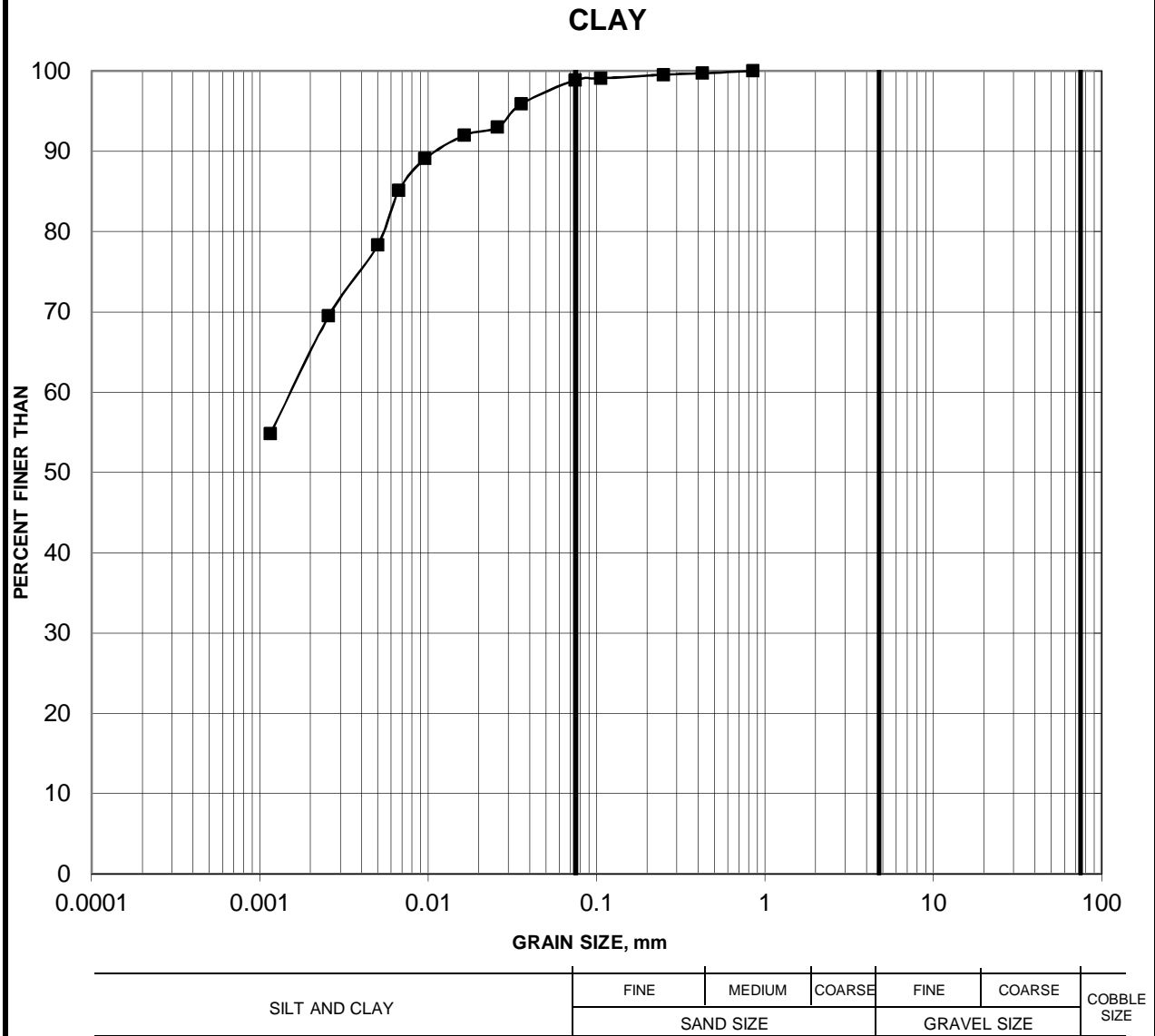
FIG No. B3

Project No. 12-1121-0193 /1140

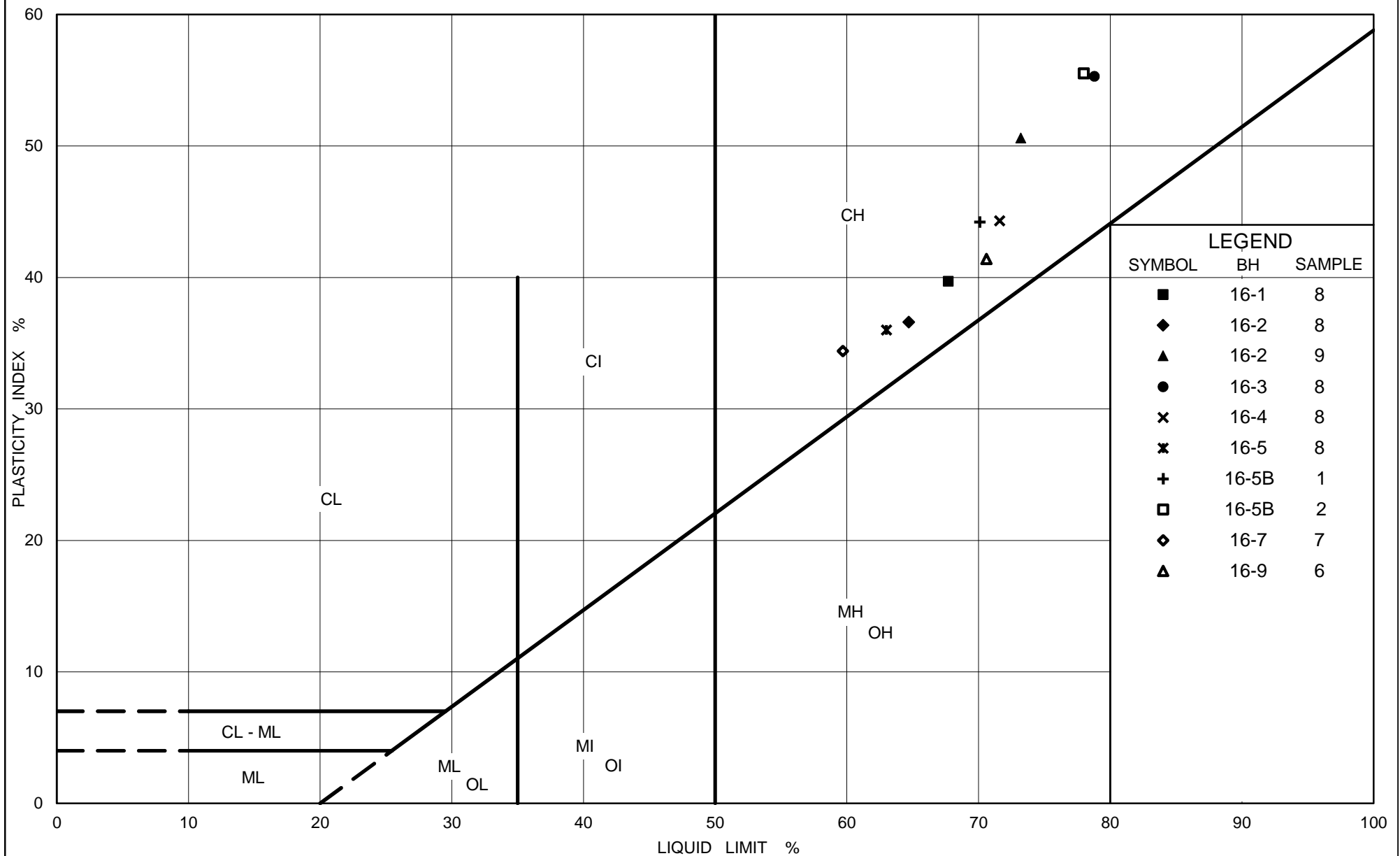
Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE B4



Borehole	Sample	Depth (m)
16-3	7	6.10-6.71



Ontario

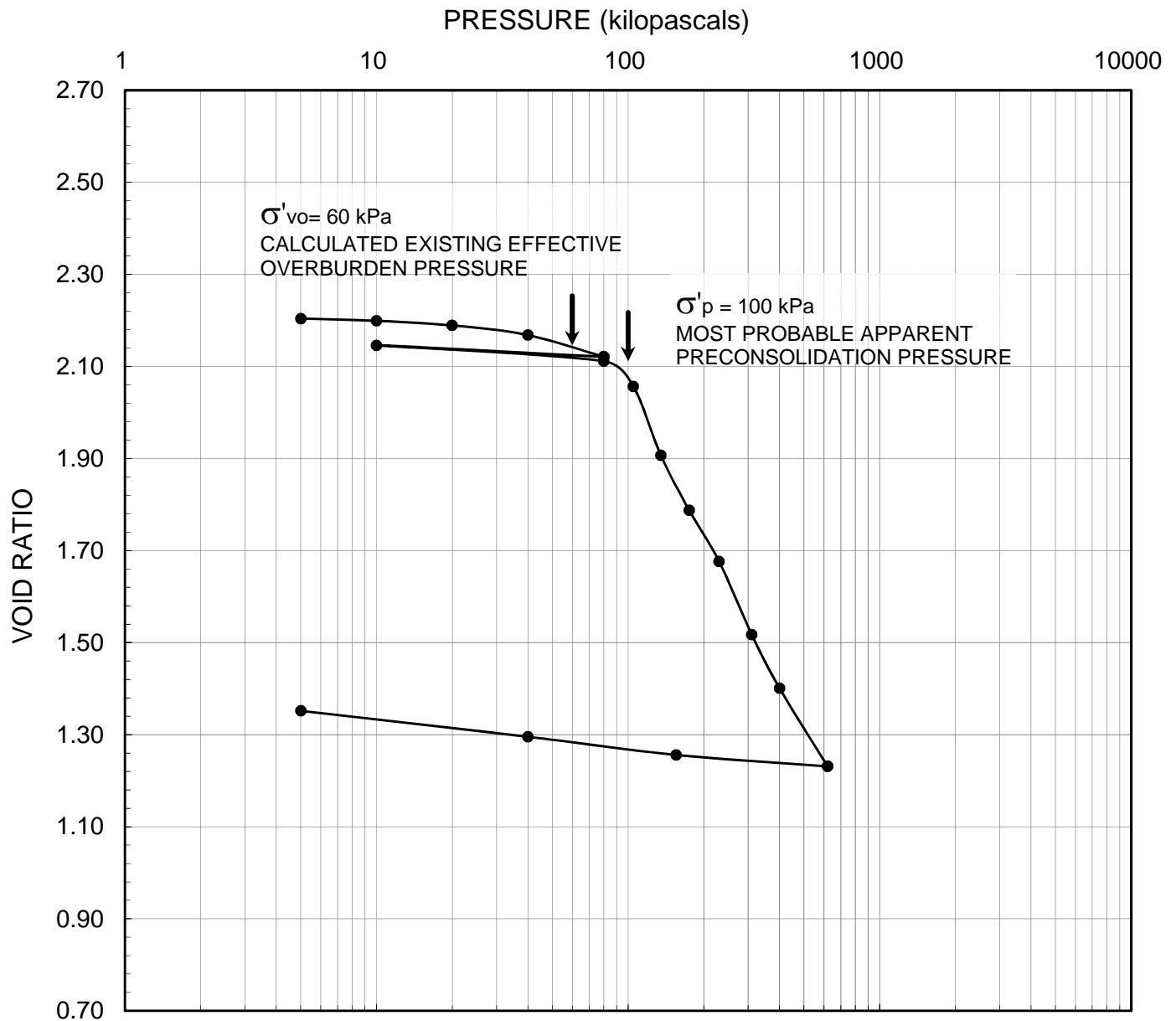
Ministry of Transportation

PLASTICITY CHART CLAY

FIG No. B5

Project No. 12-1121-0193 /1140

Compiled By : MI Checked By : CNM



LEGEND

Borehole: 16-5B	$w_i = 77\%$	$S_o = 98\%$	$\gamma = 15.2 \text{ kN/m}^3$
Sample: 1	$w_f = 49\%$	$e_o = 2.21$	$G_s = 2.80$
Depth (m): 5.7	$w_l = 70\%$	$C_c = 1.23$	
Elevation (m): 42.4	$w_p = 26\%$	$C_r = 0.039$	



SCALE	AS SHOWN
DATE	05/27/17
CADD	N/A
ENTERED	CW
CHECK	CNM
REVIEW	KSL

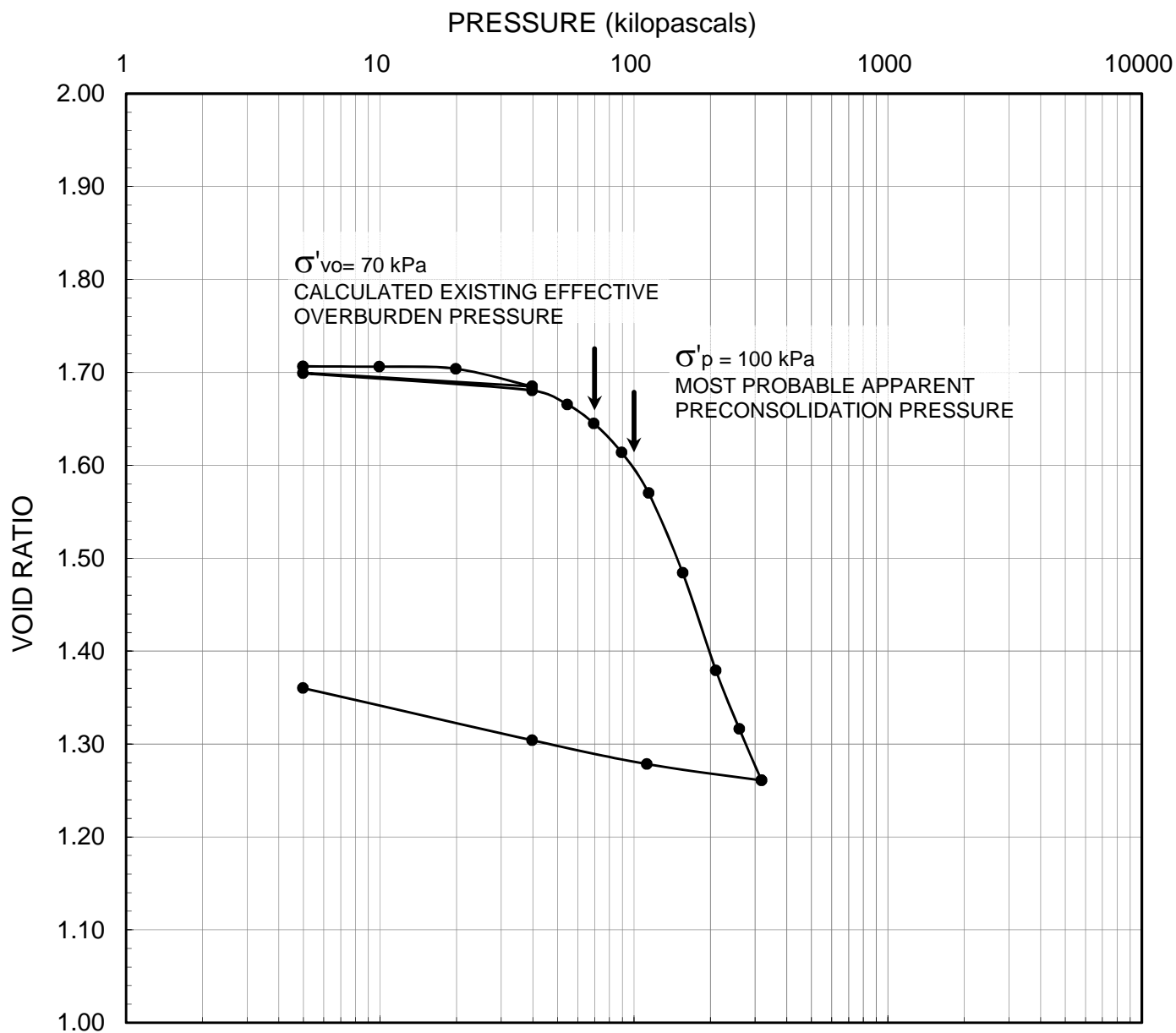
TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1140
REV.	1

FIGURE

B6



LEGEND

Borehole: 16-5B	$w_i = 60\%$	$S_o = 98\%$	$\gamma = 16.3 \text{ kN/m}^3$
Sample: 2	$w_f = 48\%$	$e_o = 1.71$	$G_s = 2.81$
Depth (m): 7.6	$w_l = 78\%$	$C_c = 0.81$	
Elevation (m): 40.5	$w_p = 23\%$	$C_r = 0.020$	



SCALE	AS SHOWN
DATE	05/27/17
CADD	N/A
ENTERED	CW
CHECK	CNM
REVIEW	KSL

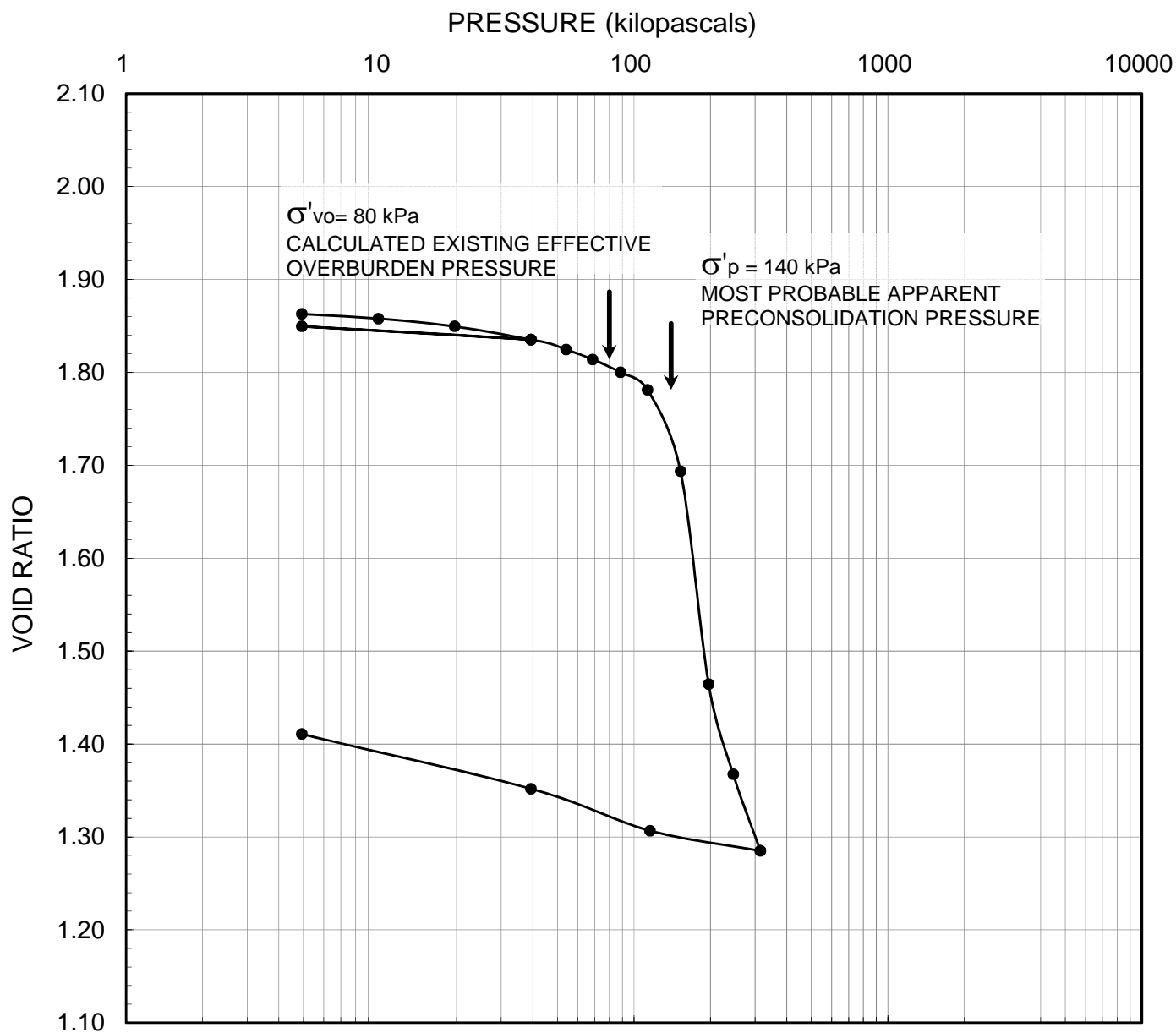
TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1140
REV.	1

FIGURE

B7



LEGEND

Borehole: 16-3	$w_i = 67\%$	$S_o = 100\%$	$\gamma = 15.9 \text{ kN/m}^3$
Sample: 8	$w_f = 52\%$	$e_o = 1.86$	$G_s = 2.78$
Depth (m): 8.9	$w_l = 79\%$	$C_c = 2.09$	
Elevation (m): 39.5	$w_p = 24\%$	$C_r = 0.017$	



SCALE	AS SHOWN
DATE	05/27/17
CADD	N/A
ENTERED	CW
CHECK	CNM
REVIEW	KSL

TITLE

CONSOLIDATION TEST RESULTS

FILE No.	Consolidation summary
PROJECT No.	12-1121-0193-1140
REV.	1

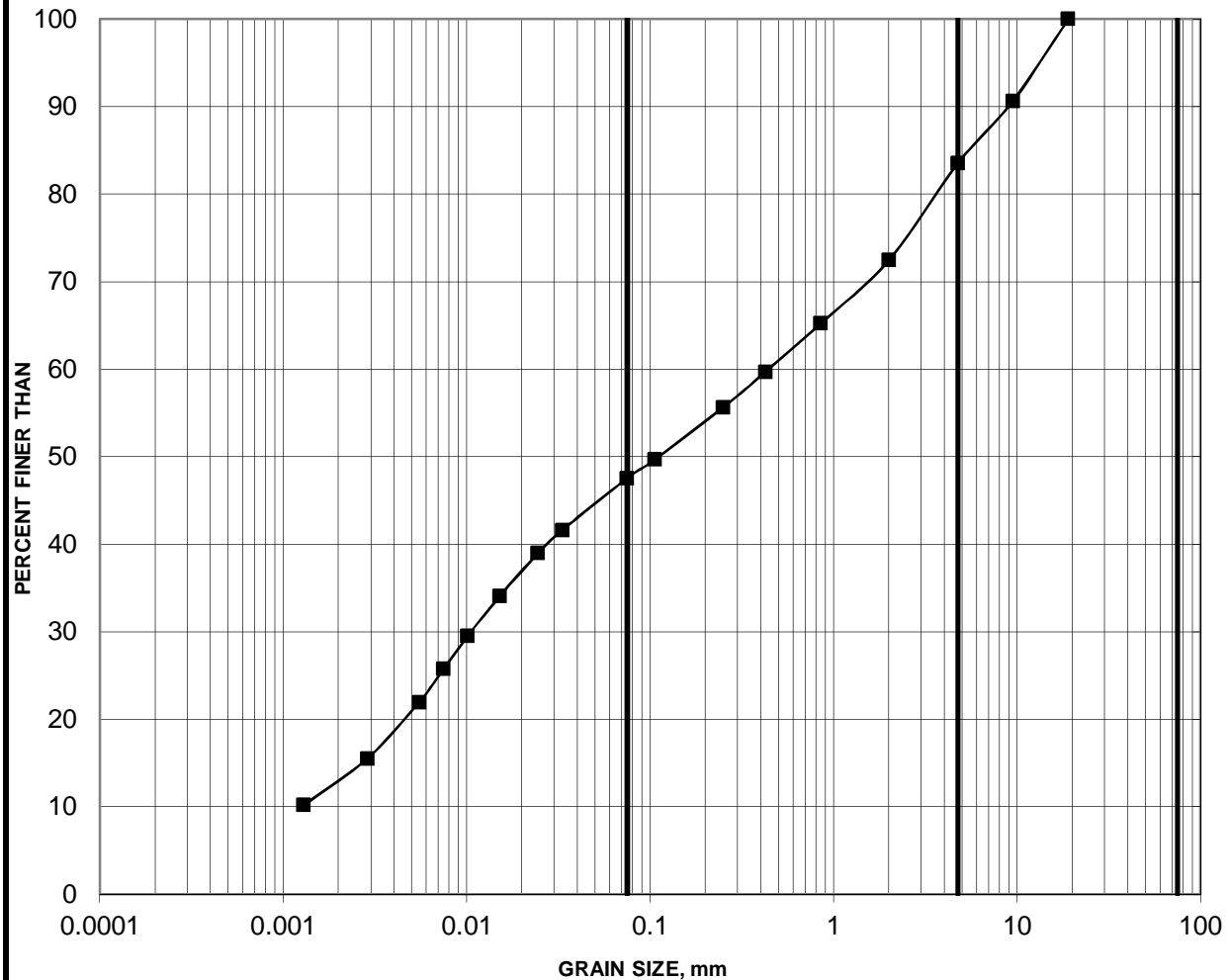
FIGURE

B8

GRAIN SIZE DISTRIBUTION

FIGURE B9

SILTY SAND (TILL)



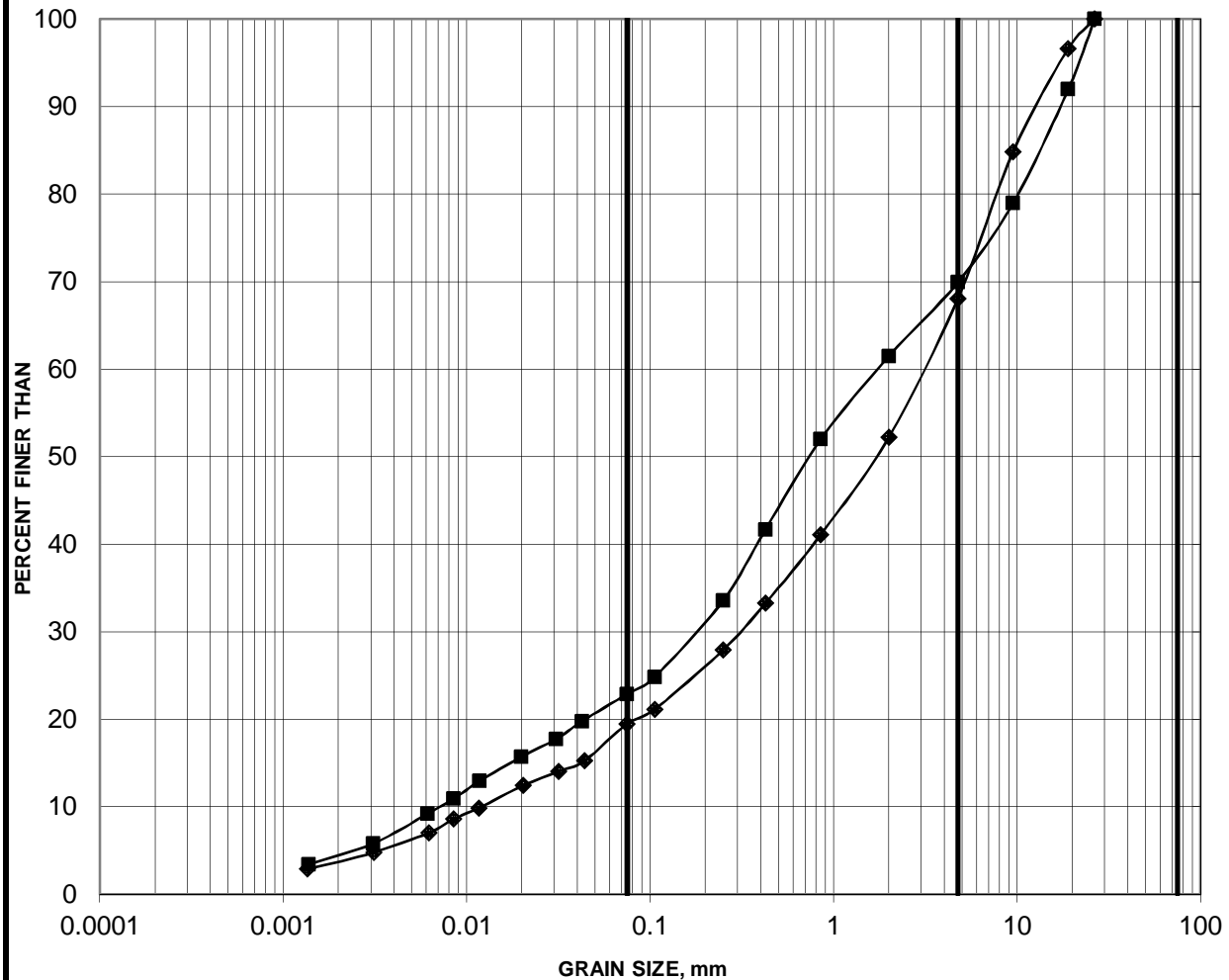
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-2	12	11.43-12.04

GRAIN SIZE DISTRIBUTION

FIGURE B10

GRAVELLY SAND



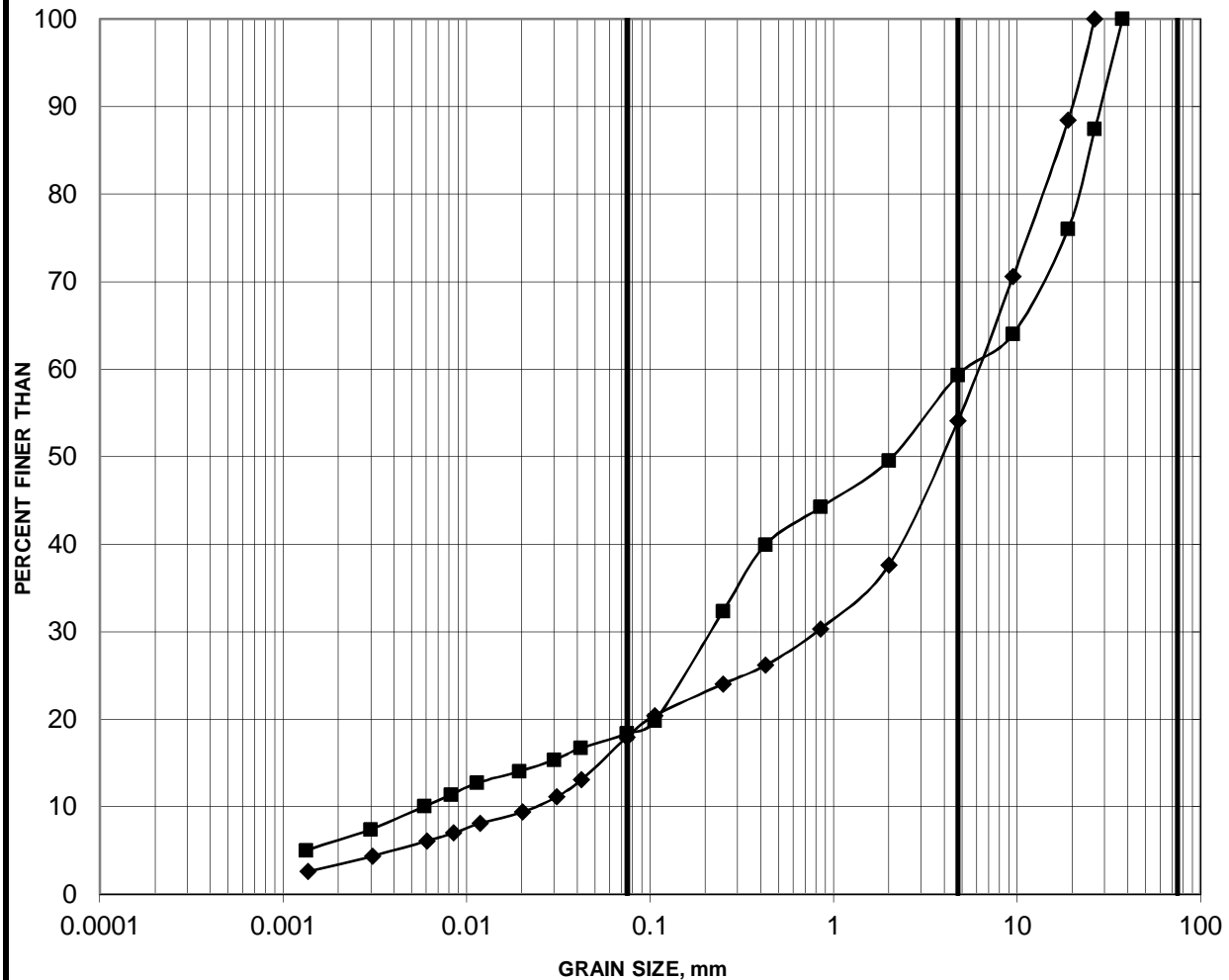
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-1	12	12.20-12.80
16-5	11	10.67-11.28

GRAIN SIZE DISTRIBUTION

FIGURE B11

SAND and GRAVEL

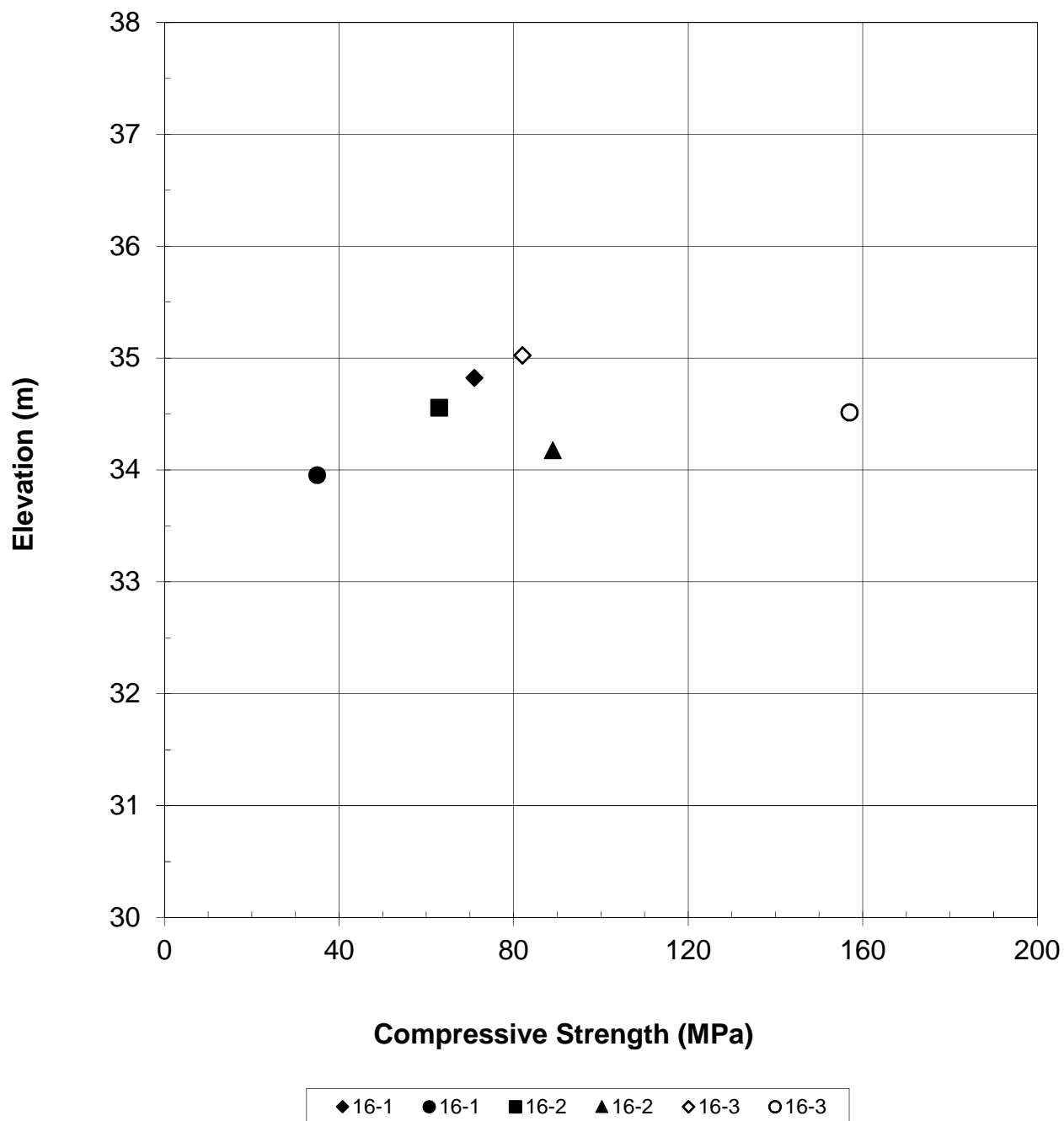


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-3	10	10.06-10.67
16-4	14	13.72-13.94

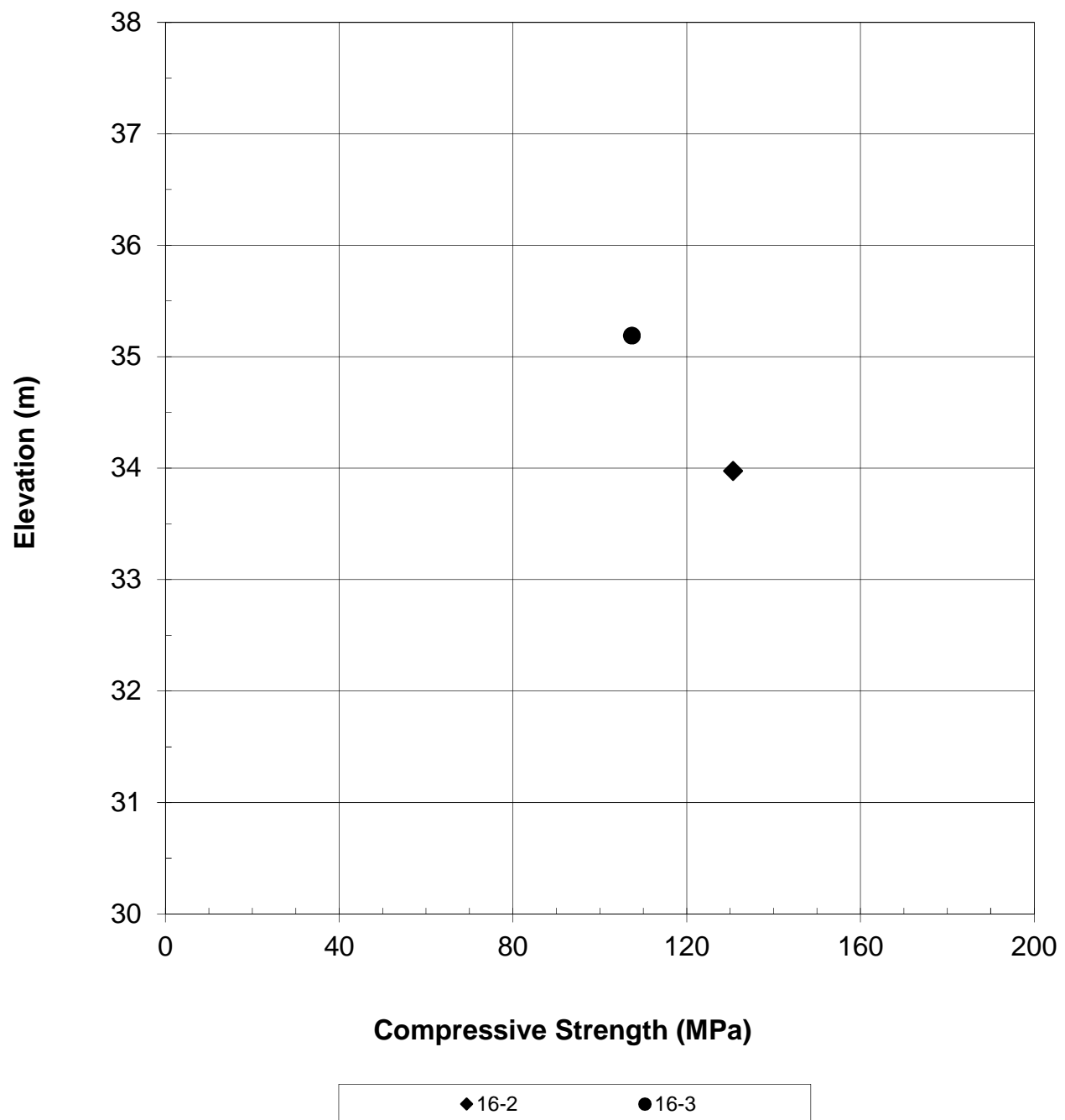
SUMMARY OF LABORATORY COMPRESSIVE STRENGTH POINT LOAD TESTING

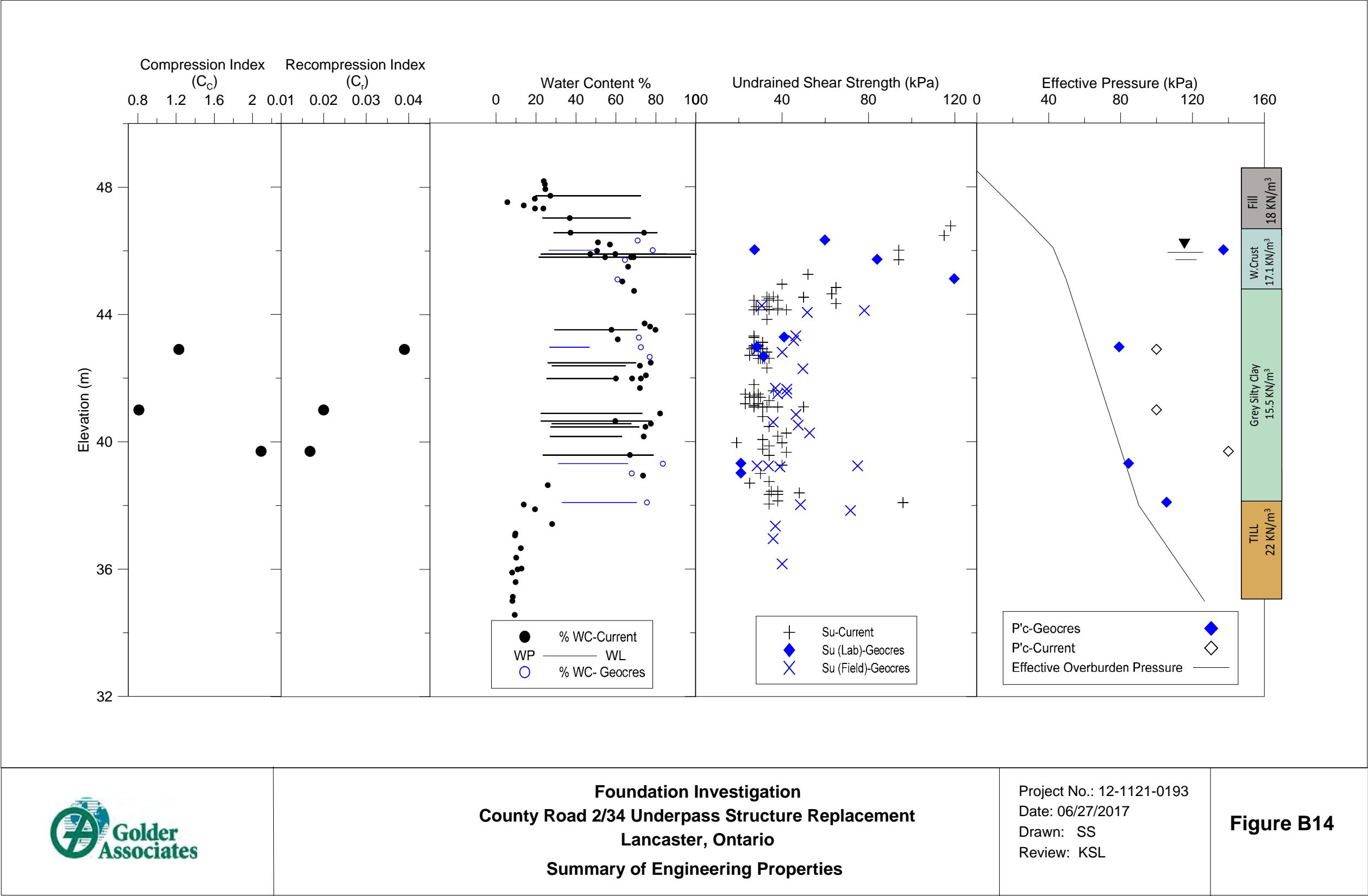
FIGURE B12



**SUMMARY OF LABORATORY COMPRESSIVE STRENGTH
UNCONFINED COMPRESSION TESTS**

FIGURE B13





Foundation Investigation
County Road 2/34 Underpass Structure Replacement
Lancaster, Ontario
Summary of Engineering Properties

Project No.: 12-1121-0193
Date: 06/27/2017
Drawn: SS
Review: KSL

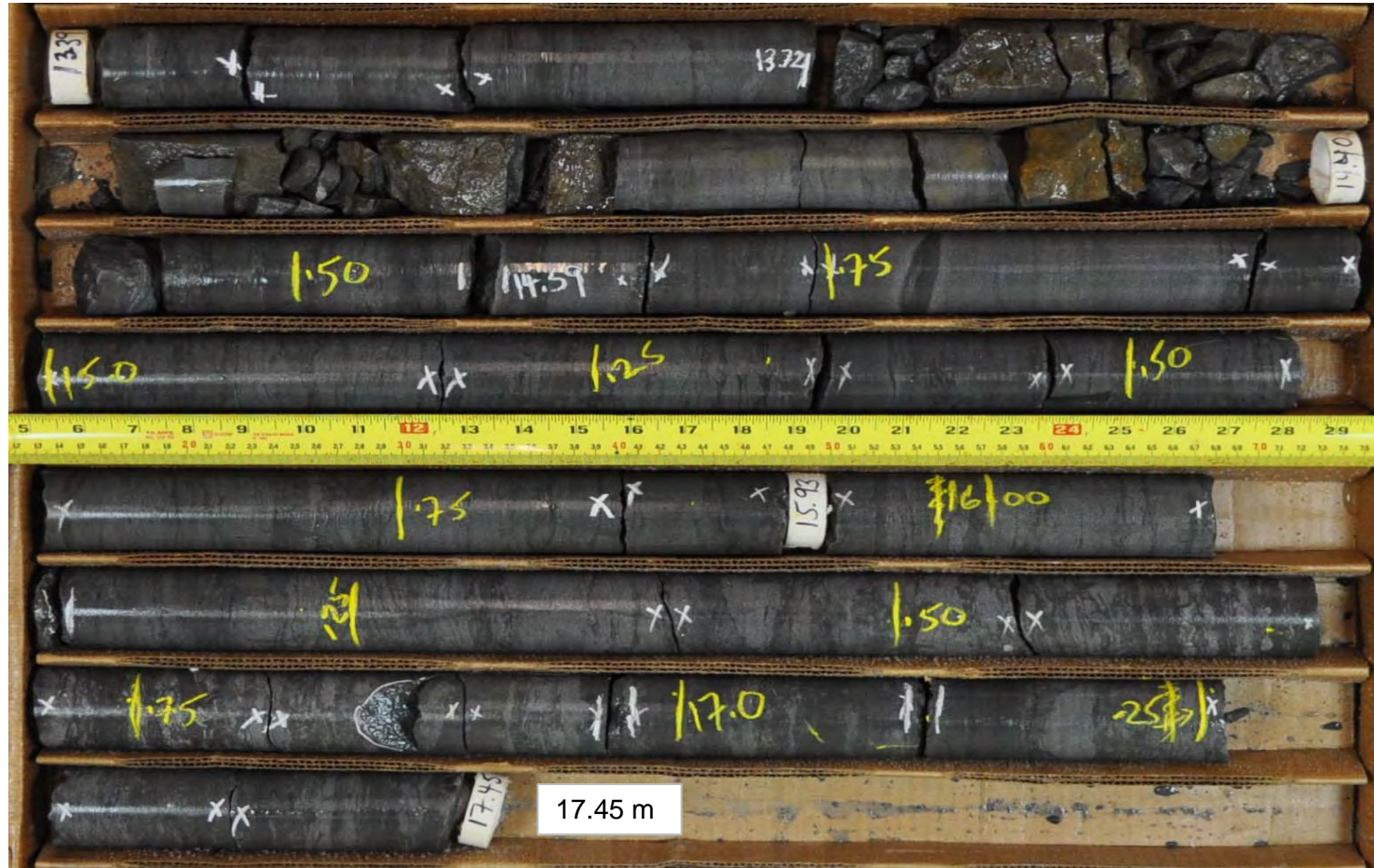
Figure B14

BH 16-1

Cored Length of 13.39 to 17.45 metres

13.39 m Depth –Top of Bedrock

Core Box 1 and 2



Foundation Investigation

County Road 2/34 Underpass Structure Replacement

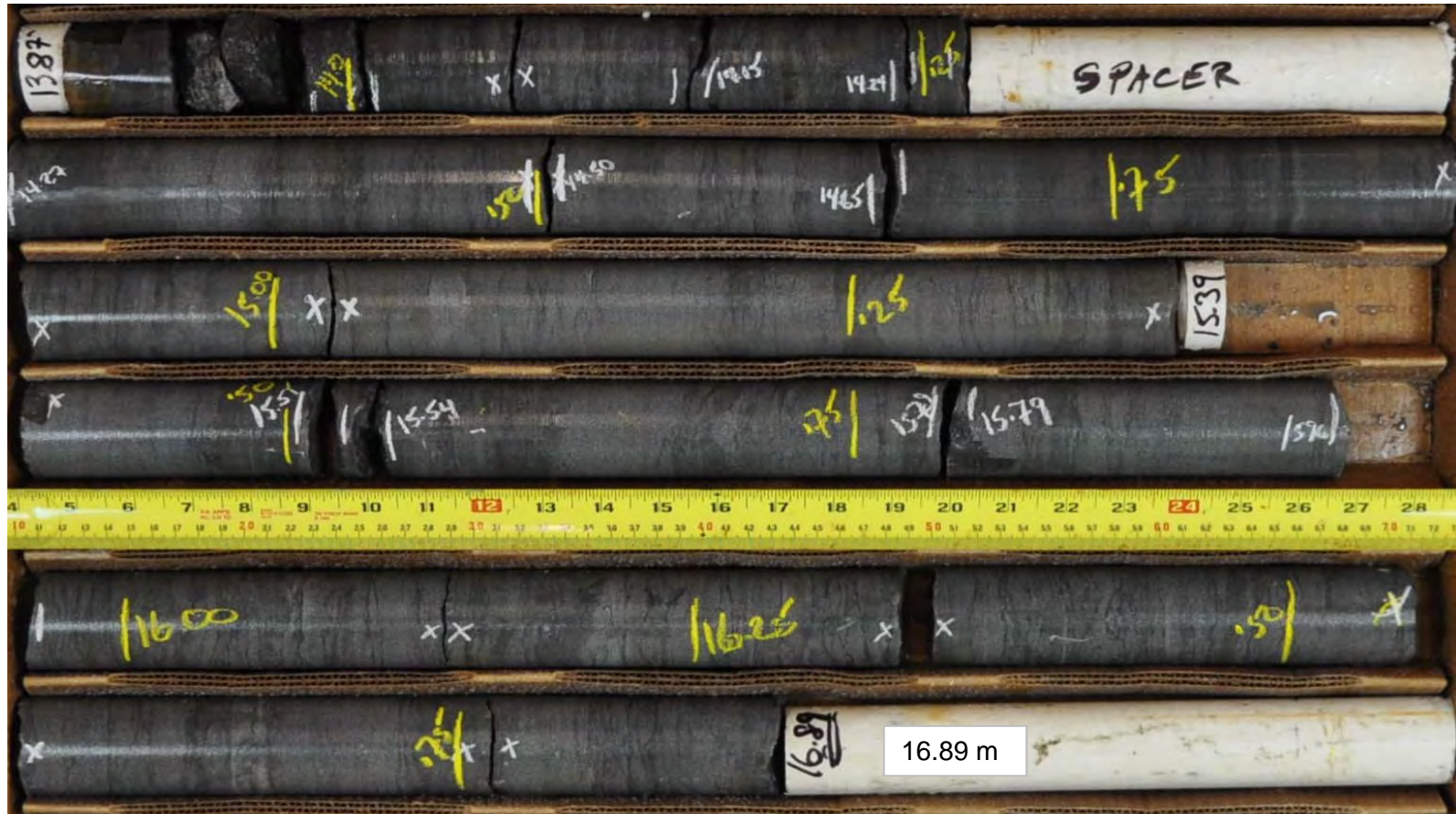
Lancaster, Ontario

Project No.	12-1121-0193
Drawn:	SS
Date:	06/27/2017
Checked:	KSL
Review:	

B15

BH 16-2
Cored Length of 13.87 to 16.89 metres
Core Box 1 and 2

13.87 m Depth – Top of Bedrock



Foundation Investigation
County Road 2/34 Underpass Structure Replacement
Lancaster, Ontario

Project No.	12-1121-0193
Drawn:	SS
Date:	06/27/2017
Checked:	KSL
Review:	

B16

BH 16-3
Cored Length of 12.55 to 15.60 metres
Core Box 1 and 2

12.55 m Depth –Top of Bedrock



Foundation Investigation
County Road 2/34 Underpass Structure Replacement
Lancaster, Ontario

Project No.	12-1121-0193
Drawn:	SS
Date:	06/27/2017
Checked:	KSL
Review:	

B17



APPENDIX C

Record of Boreholes, Previous Investigations

Records of Previous Boreholes 889-1 to 889-5 (Geocres No. 31G00-144)

Records of Previous Boreholes 1 to 4 (Geocres No. 31G00-145)

DRILLING REPORT

CLIENT Ontario Department of Highways JOB No. 889
 PROJECT W.P. 138-57 HOLE No. 889-1
 SITE Highway 401, Highway 2, Lancaster, Ontario SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 10:00 A.M. June 18 19 60
 FINISHED 1:30 P.M. June 21 19 60
 METHOD SOIL Modified Wash Boring CASING DIAM. BX
 OF
 DRILLING: ROCK Diamond Drill CORE DIAM. AXT

LOCATION: LATITUDE Ch. 507+73 ELEVATIONS: DATUM G.S.C.
 DEPARTURE 5 Feet Left DRILL PLATFORM
 BEARING GROUND SURFACE 155.8
 INITIAL DIP 90 Degrees ROCK SURFACE
 OTHER DIPS BOTTOM OF HOLE 114.3
 WATER TABLE 154.0

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE*	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows*
0.0	Clay	Light brown with darker brown spots, weathered, stiff						
3.0	Clay	Grey-green with brown spots stiff.	1	30	2	5.0 6.5	18	Pushed
8.0	Clay	Grey-blue medium stiff becoming softer with depth	Vane Test			8.5		
			2	30	2	11.0 12.5	18	Pushed
			Vane Test			14.3		
			3	30	2	15.0 16.7	17	Sank by own weight
			Vane Test			17.0		
			4	30	2	21.0 22.7	24	Sank by own weight
			Vane Test			24.3		
			5	30	2	27.0 28.8	24	Pushed

SAMPLING METHOD

* A - SPLIT TUBE
 R - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 Z - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR J. MacLeod
 LOGGED BY J. MacLeod

APPROVED *D.H. Macdonald*
 DATE July, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 889

PROJECT W.P. 138-57

HOLE No. 889-1

SITE Highway 401, Highway 2, Lancaster, Ontario

SHEET No. 2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY STRUCTURE WATER CONTENT PLASTICITY COMPACTION WATER LOSS OR GAIN ETC.	SAMPLE					PENETRATION TEST Blows*
			NO	TYPE	SIZE Inches	DEPTH Feet	RETD Inches	
				Vane Test		30.5		
33.0	Till	Grey sand, angular pebbles and stones, quite clayey	6	AQ	2	33.0		6
						33.5		3
						34.0		4
						34.5	5	
				A2	2	37.0		
						37.5		2-1/2
						38.0		4-1/2
						38.5	1/2	7
41.5		Possibly bedrock						
* - Penetration Test This is the number of blows of a 140-pound weight falling 30 inches required to advance the sampler to depth indicated.								

DRILLING REPORT

CLIENT Ontario Department of Highways JOB No. 889
 PROJECT W.P. 138-57 HOLE No. 889-2
 SITE Highway 401 and Highway 2, Lancaster, Ontario SHEET No. 1 OF 2
 CONTRACTOR: F.E. Johnston Drilling Company Limited STARTED 8:00 A.M. June 20 19 60
 FINISHED 10:30 A.M. June 22 19 60
 METHOD OF DRILLING: SOIL Modified Wash Boring CASING DIAM. BX and AX
 ROCK Diamond Drill CORE DIAM. AXT
 LOCATION: LATITUDE Ch. 567+72 ELEVATIONS: DATUM G.S.C.
 DEPARTURE 69 Feet Left DRILL PLATFORM
 BEARING GROUND SURFACE 156.0
 INITIAL DIP 30 Degrees ROCK SURFACE 115.0
 OTHER DIPS BOTTOM OF HOLE 107.0
 WATER TABLE 154.0

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST Blows
			NO.	TYPE*	SIZE Inches	DEPTH Feet	RET'D Inches	
0	Clay	Brown-grey, weathered						
5.0	Clay	Blue, homogeneous, tenacious	1	30	2	5.0		
						6.5	18	Pushed
				Vane Test		8.0		
			2	30	2	9.0		
						10.5	18	Pushed
				Vane Test		12.3		
			3	30	2	13.0		
						14.5	18	Pushed
		Some shells in wash		Vane Test		17.0		
			4	30	2	17.5		
						19.0	18	Pushed
				Vane Test		20.0		
				Vane Test		24.5		
			5	30	2	29.0		
						30.5	15	Pushed

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOFILM BAG
 Z — DISCARDED

INSPECTOR J. Bateson

LOGGED BY J. MacLeod

APPROVED

A. H. MacDonald

DATE

July, 1960

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
 NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Ontario Department of Highways
 PROJECT W.F. 138-57
 SITE Highway 401 and Highway 2, Lancaster, Ontario

JOB No. 889
 HOLE No. 889-2
 SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows
			Vane	Test		32.0		
				AZ	2	36.5		
						37.0		4
						37.5		7
						38.0	0	6
36.5	Till		6	FQ	2	38.0		
						38.5		
			7	AC	2	38.6		
						39.0		11
						39.6		13
						40.0	3	16
41.0	Bedrock	Calcareous Shale						
42.0		Hole Complete						
		BK casing to 41.0 feet						
		AX casing drilled to 42.3						
		feet because of water loss						
		at BX rock contact.						

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT	Ontario Department of Highways	JOB No.	889
PROJECT	W.P. 138-57	HOLE No.	889-3
SITE	Highway 401, Highway 2, Lancaster, Ontario	SHEET No.	1 OF 2
CONTRACTOR:	F.E. Johnston Drilling Company Limited	STARTED	3:00 P.M. June 21, 19 60
		FINISHED	5:30 P.M. June 22, 19 60
METHOD OF DRILLING:	SOIL Modified Wash Boring	CASING DIAM.	BX
	ROCK	CORE DIAM.	
LOCATION:	LATITUDE Ch. 567+97	ELEVATIONS: DATUM	G.S.C.
	DEPARTURE 200 Feet Left	DRILL PLATFORM	
	BEARING	GROUND SURFACE	155.9
	INITIAL DIP 90 Degrees	ROCK SURFACE	108.9
	OTHER DIPS	BOTTOM OF HOLE	108.9
		WATER TABLE	

DEPTH Feet	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST Blows
			NO	TYPE *	SIZE Inches	DEPTH Feet	RET'D Inches	
0.0	Clay	Gray-brown, weathered medium stiff	1	BO	2	4.0		
						5.5	18	
5.0	Clay	Blue-grey, medium stiff, homogeneous	2	BO	2	7.0		
						9.0	24	Pushed
13.0	Clay	Blue-grey, soft, homogeneous	Vane Test			11.5		
			3	BO	2	13.5		
						15.5	24	Pushed
			Vane Test			17.0		
			4	BO	2	21.0		
						23.0	24	Pushed
			Vane Test			24.5		
27.0	Clay	Blue-grey, stiff, homogeneous	5	BO	2	26.0		
						27.5	18	Pushed
			Vane Test			29.0		

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOTILM BAG
 Z — DISCARDED

INSPECTOR J. MacLeod
 LOGGED BY J. MacLeod

APPROVED

D. H. Macdonald

DATE

July, 1960

H. G. ACRES & COMPANY LIMITED — CONSULTING ENGINEERS
 NIAGARA FALLS, CANADA

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 589

PROJECT W.F. 138-57

HOLE No. 589-3

SITE Highway 401, Highway 2, Lancaster, Ontario.

SHEET No. 2 OF 2

DEPTH	SOIL TYPE	DESCRIPTION COLOUR CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTION, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO.	TYPE	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows
32.0	Clay	Blue-grey, soft, homogeneous	6	30	2	31.0		
						33.0	24	Pushed
			Vane Test			34.5		
37.0	Till	Blue-grey clay containing small angular pebbles	7	30	2	35.0		
						37.0	24	Pushed
			8	30	2	41.0		
						41.5		14
						42.0		13
						42.5	18	9
47.0	Bedrock	Dark grey calcareous shale						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 889

PROJECT W.P. 138-57

HOLE No. 889-4

SITE Highway 401, Highway 2, Lancaster, Ontario.

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 11:30 A.M. June 22, 1960
 FINISHED 5:30 P.M. June 24, 1960

METHOD SOIL Modified Wash Boring

CASING DIAM. BX

OF
 DRILLING: ROCK Diamond Drill

CORE DIAM. AXT

LOCATION: LATITUDE Ch. 567+79
 DEPARTURE 120 Feet Right
 BEARING
 INITIAL DIP 40 Degrees
 OTHER DIPS

ELEVATIONS: DATUM GSC
 DRILL PLATFORM
 GROUND SURFACE 156.0
 ROCK SURFACE 116.0
 BOTTOM OF HOLE 110.0
 WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION, COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	Blows
0	Clay	Grey-brown, weathered	1	30	2	5.0		
						6.5	15	Pushed
8.0	Clay	Blue, homogeneous, stiff, tenacious.	Vane Test			8.5		
			2	30	2	9.0		
10.0		Medium				10.5	18	Pushed
			Vane Test			12.8		
			3	30	2	13.0		
						14.5	18	Pushed
			Vane Test			16.3		
			4	30	2	17.0		
						18.5	18	Pushed
			Vane Test			20.3		
			5	30	2	21.0		
						22.5	18	Pushed
			Vane Test			24.3		
			Vane Test			28.5		

SAMPLING METHOD

* A — SPLIT TUBE
 B — THIN WALL TUBE
 C — PISTON SAMPLER
 D — CORE BARREL

E — AUGER
 F — WASH

SHIPPING CONTAINER

N — INSERT
 O — TUBE
 P — WATER CONTENT TIN
 Q — GLASS JAR

R — CLOTH BAG
 S — PLIOFILM BAG
 Z — DISCARDED

INSPECTOR J. Bateson

LOGGED BY J. MacLeod

APPROVED

D. H. MacDonell

DATE

July, 1960

DRILLING REPORT

CLIENT Ontario Department of Highways
 PROJECT W.F. 138-57
 SITE Highway 401, Highway 2, Lancaster, Ontario

JOB No. 869
 HOLE No. 889-4
 SHEET No. 2 OF 2

DEPTH Feet	SOIL TYPE	DESCRIPTION COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	S A M P L E					PENETRATION TEST
			NO	TYPE	SIZE Inches	DEPTH Feet	RET'D Inches	BLOWS
			6	30	2	31.0		125
						32.0	6	Pushed
32.0	Sand and Gravel		7	AC	2	33.0		125
33.0	Till					33.5		4
						34.0		7
						34.5	2	5
40.0	Bedrock	Calcareous shale						
46.0		End of hole						
		Drilled 40 feet						
		AX drilled 1 foot into bedrock to prevent water loss.						

DRILLING REPORT

CLIENT Ontario Department of Highways

JOB No. 889

PROJECT W.P. 138-57

HOLE No. 889-5

SITE Highway 401, Highway 2, Lancaster, Ontario

SHEET No. 1 OF 2

CONTRACTOR: F.E. Johnston Drilling
 Company Limited

STARTED 8:00 A.M. June 23, 19 60
 FINISHED 5:30 P.M. June 24, 19 60

METHOD SOIL Modified Wash Boring

CASING DIAM. BX

OF
 DRILLING: ROCK

CORE DIAM.

LOCATION: LATITUDE Ch. 567-65
 DEPARTURE 200 Feet Right
 BEARING
 INITIAL DIP 90 Degrees
 OTHER DIPS

ELEVATIONS: DATUM GSC
 DRILL PLATFORM
 GROUND SURFACE 156.5
 ROCK SURFACE 116.5
 BOTTOM OF HOLE 116.0
 WATER TABLE

DEPTH	SOIL TYPE	DESCRIPTION: COLOUR, CONSISTENCY, STRUCTURE, WATER CONTENT, PLASTICITY, COMPACTNESS, WATER LOSS OR GAIN, ETC.	SAMPLE					PENETRATION TEST
			NO	TYPE *	SIZE	DEPTH	RET'D	
Feet					Inches	Feet	Inches	
0.0	Clay	Gray-brown, weathered, medium stiff	1	BO	2	4.0		
						6.0	24	Pushed
			2	BO	2	8.0		
						10.0	24	Pushed
11.0	Clay	Blue-grey, homogeneous, soft	Vane Test			11.5		
			3	BO	2	14.0		
						16.0	24	Pushed
			Vane Test			17.5		
			4	BO	2	18.5		
						20.0	18	Pushed
20.0	Clay	Blue-grey, homogeneous, slightly stiffer than above material	Vane Test			21.5		
			5	BO	2	24.5		
						26.0	18	Pushed
			Vane Test			27.5		
30.0	Clay	Blue-grey, homogeneous, soft		BZ	2	31.0		
						32.5	0	Pushed

SAMPLING METHOD

* A - SPLIT TUBE
 B - THIN WALL TUBE
 C - PISTON SAMPLER
 D - CORE BARREL

E - AUGER
 F - WASH

SHIPPING CONTAINER

N - INSERT
 O - TUBE
 P - WATER CONTENT TIN
 Q - GLASS JAR

R - CLOTH BAG
 S - PLIOFILM BAG
 Z - DISCARDED

INSPECTOR J. MacLeod

APPROVED *D. H. MacDonald*

LOGGED BY J. MacLeod

DATE July, 1960

DRILLING REPORT

JOB No. 889
HOLE No. 889-5
SHEET No. 2 OF 2

Figure 1

<small>PL 125 54-90</small> DRILL RIG --- CORE DRILL --- CASING --- (STANDARD SAMPLERS TO FIT UNLESS NOTED) SAMPLER HAMMER WT --- 250 --- DROPP --- INCHES ---		MATERIALS LABORATORY DEPARTMENT OF HIGHWAYS - ONTARIO OFFICE REPORT ON SOIL EXPLORATION		JOB --- 55 F 15 LANCASTER --- BORING NO --- 3 --- DATUM Sta 367.64 P.A. ST. AL 22-47 --- DATE REPORT JUNE 25, 1955 --- COMPILED BY AH --- CHECKED BY W. H. H. BORING DATE JUNE 24, 1955 ---											
SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS											
 DISTURBED S.O.O. LOST	C.S. - CHUNK D.O. - DRIVE OPEN D.F. - DRIVE FOOT VALVE TO - THIN WALLED OPEN	V.S. - VASHEB SAMPLE R.C. - ROCK CORE	V - INSITU VANE SHEAR TEST M - MECHANICAL ANALYSIS U - UNCONFINED COMPRESSION Q _c - TRIAXIAL CONSOLIDATED QUICK Q - TRIAXIAL SLOW S - TRIAXIAL SLOW	γ - UNIT WEIGHT K - PERMEABILITY C - CONSOLIDATION CA - CASING W.L. - WATER LEVEL IN CASING W.T. - WATER TABLE IN SOIL											
SOIL PROFILE		SHEAR STRENGTH		WATER CONTENT		SAMPLES									
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	TONS/SQ. FT. OR Q _N /2			%		OTHER TESTS	CONDITION	TYPE	NO.	PENETRATION RESISTANCE	ELEV. RECOVER.	
				0.1	0.2	0.3	40	60							
				X CONE PENETRATION TEST			D.P.W.								
				RESISTANCE BLOWS PER FOOT			Δ LV								
				20 40 60			40 60								
				STD. ENERGY = 4200 LB. IN.											
125.6		GRAY CLAY (MARINE)		0									STD. ENER. 4200 LB. IN.	70	
	4														
	8														
	12														
	16														147.6
	20														100
	24														
	28														
	32														
	36														
122.6		STONES & CLAY (FLOATS)		36										125.6	
59.0				40											121.6
116.3		ROCK LEVEL		40											
59.3				44											
				48											

N.B. Sampler pushed 1.0' and then hit a string & refused to penetrate further.

[illegible]



APPENDIX D

Results of MASW Testing

DATE June 14, 2013**PROJECT No.** 12-1121-0193**TO** Kim Lesage
Golder Associates Ltd.**FROM** Patrick Finlay**EMAIL** pfinlay@golder.com**NBCC SEISMIC SITE CLASS TESTING RESULTS
MEGA 3: CONTRACT G, COUNTY ROAD 2/34 UNDERPASS AT HIGHWAY 401
SITE NO. 31-232, WP 4013-11-01**

This technical memorandum presents the processing and results of a Multichannel Analysis of Surface Waves (MASW) test performed for the purpose of National Building Code of Canada Seismic Site Classification of the overpass, Bridge 31-232, at the intersection of Highway 34 and Highway 401, near Lancaster, Ontario. The geophysical testing was performed by Golder Associates Ltd. (Golder) personnel on May 23, 2013.

Methodology

The MASW method measures variations in surface-wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface-waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium, surface-waves are dispersive, (i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that particular wavelength of surface-wave propagates through). The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledge hammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors, and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface-wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.



Field Work

The MASW field work was conducted on May 23, 2013, by personnel from the Golder Ottawa office. For the MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3-metre intervals, centered at 539601E, 4998124N (UTM NAD83, Zone 18T), as shown in Figure 1 (attached). A seismic weight-drop of 45 kg and a 9.9 kg sledge hammer were used as seismic sources for this investigation. Seismic records were collected with seismic sources located 5, 10, 15 and 20 metres from and collinear to the geophone array. An example of an active seismic record collected is shown in Figure 2 (below).

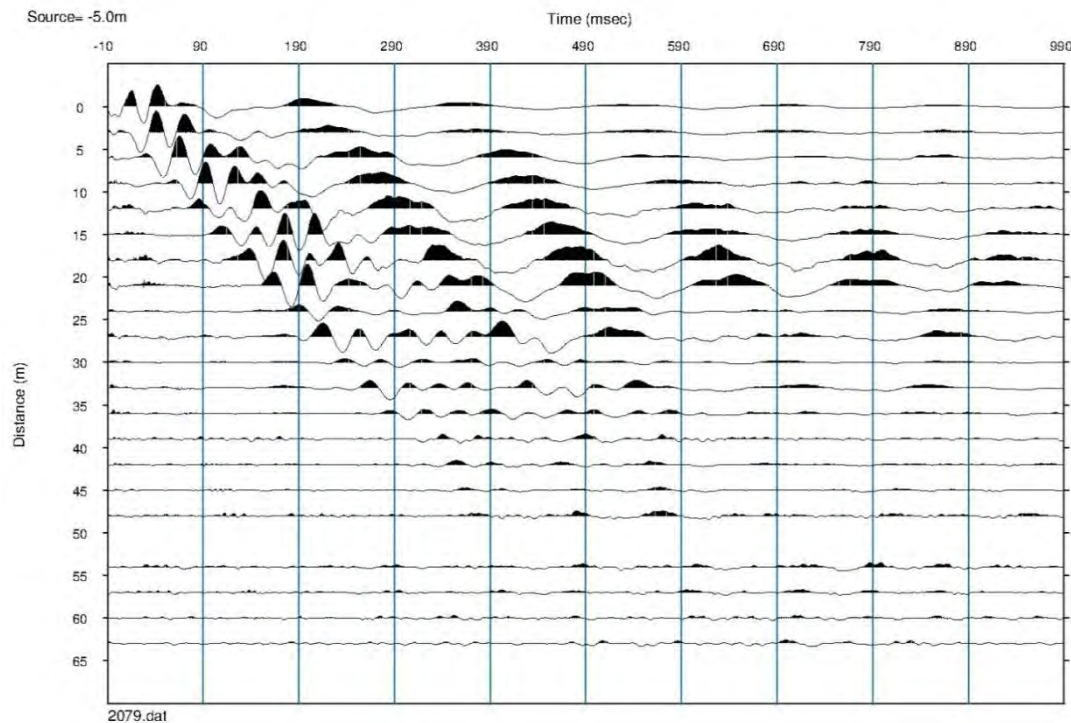


Figure 2: Typical seismic record collected at the site.

Data Processing

Processing of the MASW test results consisted of the following main steps:

- 1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
- 2) Calculation of the phase for each frequency component;
- 3) Linear regression to calculate phase velocity for each frequency component;
- 4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient (r^2) between the data and the linear regression best fit line used to calculate phase velocity;
- 5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and,
- 6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.

Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve was generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figure 3 (below). Shear-wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves. The active survey provided a dispersion curve in a relatively wide frequency range (5-18 Hz) providing information at both shallower and deeper depths. The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 5 Hz.

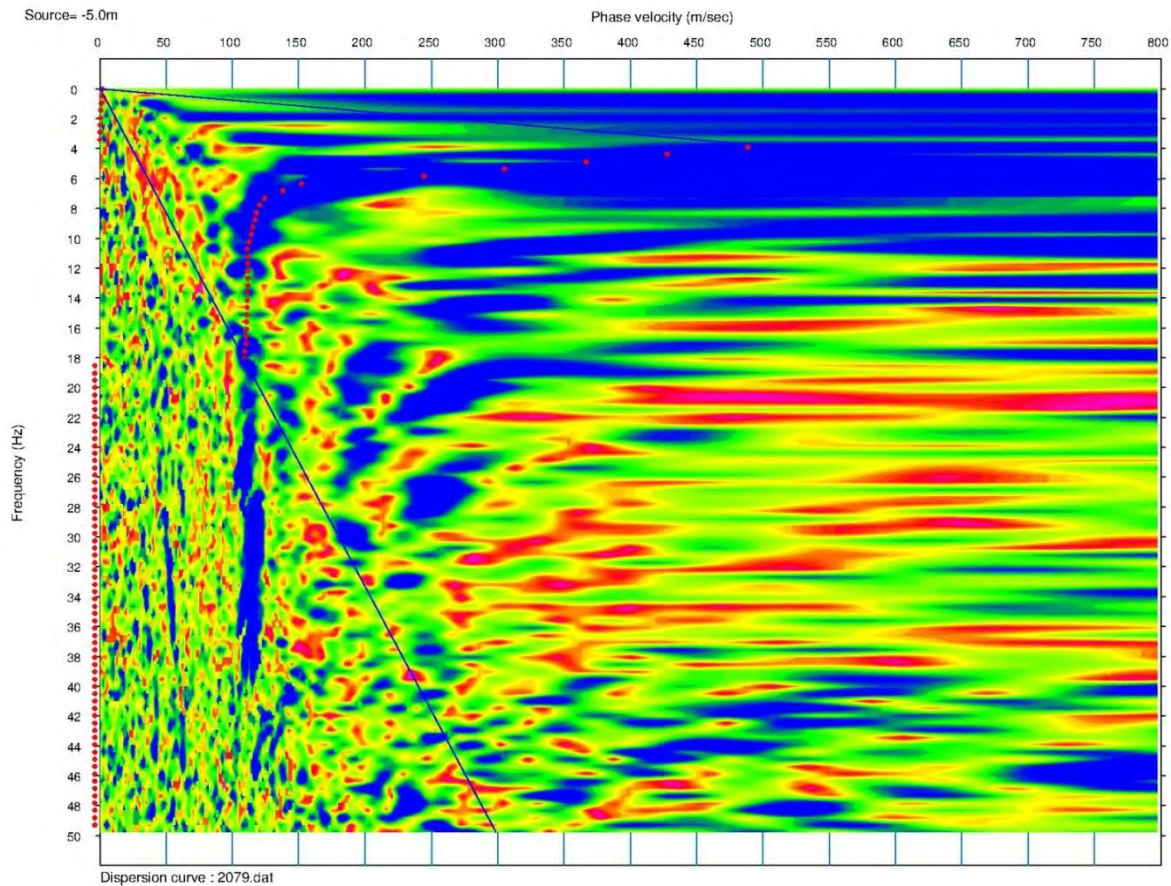


Figure 3: MASW Dispersion Curve Picks (red dots).

Results

The MASW test results are presented in Figure 4, which presents the calculated shear-wave velocity profiles measured from the field testing. The results have been inferred using a weight-drop located at 5 metres from the first geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figure 5. There is a good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 2%.

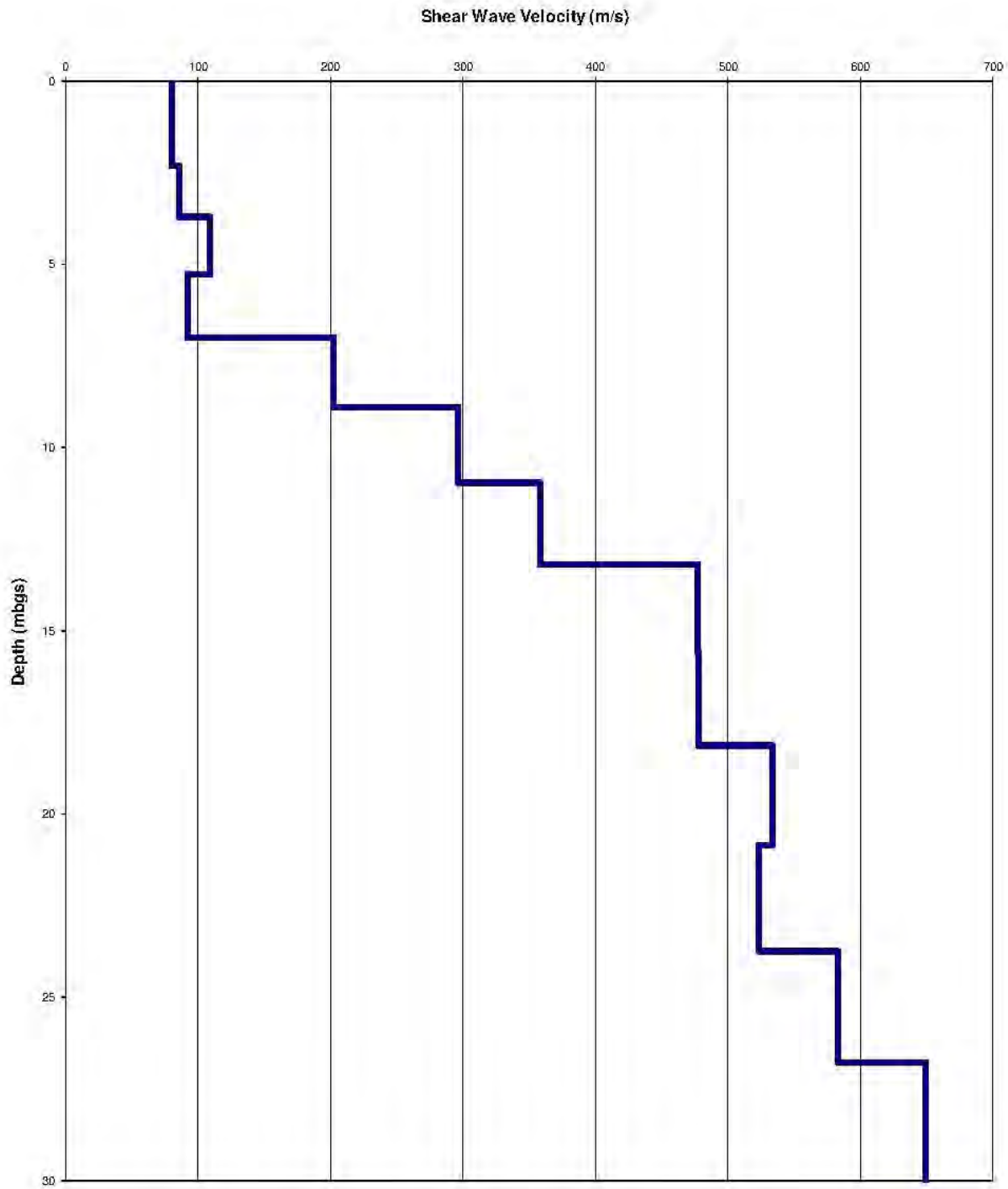


Figure 4: MASW Modelled Shear-Wave Velocity Depth profile.

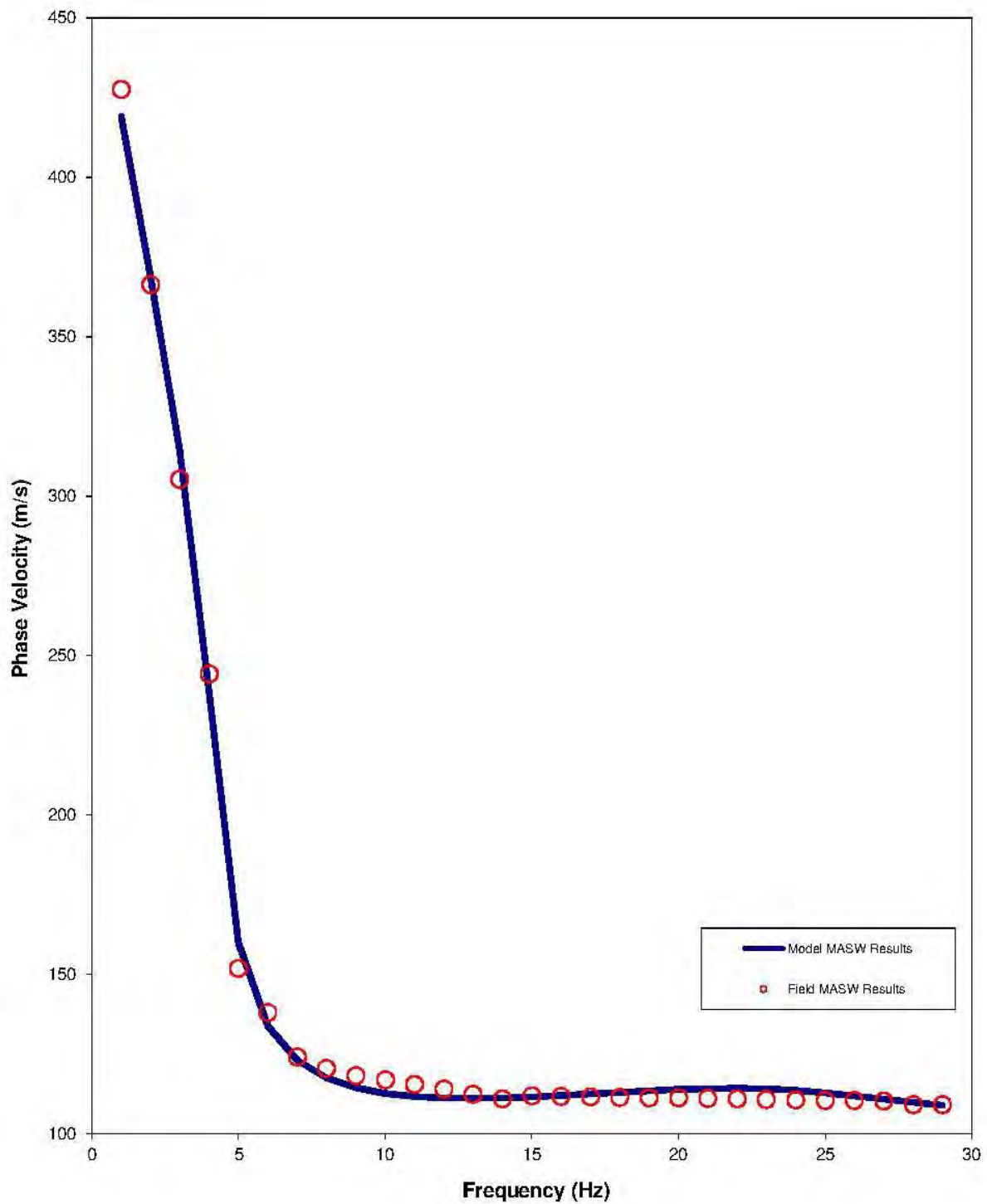


Figure 5: Comparison of Field (red dots) vs. Modelled Data (blue line).

To calculate the average shear-wave velocity as required by the National Building Code of Canada, 2010 (NBCC2010), the results were modelled to 30 metres below ground surface. The average shear-wave velocity was found to be 227 m/s (Table 1). Shear-wave velocities of this magnitude are classified according to the NBCC2005 as Site Class D (stiff soil), based solely on the average shear-wave velocity. The NBCC2010 requires special site specific evaluation if certain soil types are encountered on the site; therefore, the site classification stated here should be reviewed, and modified if necessary, according to borehole stratigraphy, standard penetration resistance results, and undrained shear-strength measurements, if available for this site.

Table 1: Shear-Wave Velocity Profile

Model Layer (mbgs)		Layer Thickness (m)	Shear-Wave Velocity (m/s)	Shear-Wave Travel Time Through Layer (s)
Top	Bottom			
0.0	1.1	1.07	80	0.013393
1.1	2.3	1.24	80	0.015434
2.3	3.7	1.40	86	0.016328
3.7	5.3	1.57	109	0.014360
5.3	7.0	1.73	92	0.018825
7.0	8.9	1.90	202	0.009369
8.9	11.0	2.06	296	0.006956
11.0	13.2	2.23	358	0.006212
13.2	15.6	2.39	477	0.005010
15.6	18.1	2.55	477	0.005352
18.1	20.9	2.72	533	0.005098
20.9	23.7	2.88	523	0.005513
23.7	26.8	3.05	583	0.005230
26.8	30.0	3.21	649	0.004952
Vs Average to 30 mbgs (m/s)				227

Closure

We trust that this memo meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience

GOLDER ASSOCIATES LTD.

Patrick Finlay, P.Geo.
Geophysics Group



Brian Byerley, M.Sc., P.Eng.
Senior Hydrogeologist/Principal

PIF/BTB/sg

n:\active\2012\1121 - geotechnical\12-1121-0193 dillon mega 3 eastern region\bridge surveys\31-232 hwy 2-34\geophysics\12-1121-0193 lancaster mto masw_14june2013.docx

Attachments: Figure 1 – MASW Testing Location

N:\Active\2012\1121 - Geotechnical\12-1121-0193 Dillon Mega 3 Eastern Region\Spatial IM\GIS\MXD\12-1121-0193-1140-01.mxd



LEGEND

— MASW SURVEY LINE (COORDINATES COLLECTED WITH HANDHELD GPS UNIT +/- 5 m)

NOTE

THIS FIGURE IS TO BE READ IN CONJUNCTION WITH THE ACCOMPANYING GOLDER ASSOCIATES LTD. REPORT No. 12-1121-0193/1140

REFERENCE

DATUM: NAD 83, COORDINATE SYSTEM: UTM ZONE 18

0 20 40 60 80 100 Metres

PROJECT
MEGA 3: CONTRACT G, COUNTY ROAD 2/34 UNDERPASS
AT HIGHWAY 401, SITE No. 31-232, WP 4013-11-01

TITLE

MASW TESTING LOCATION



PROJECT No. 12-1121-0193			SCALE AS SHOWN	REV. 0.0
DESIGN	PIF	May 2013	FIGURE 1	
GIS	JEM	May 2013		
CHECK	PIF	June 2013		
REVIEW	BTB	June 2013		



APPENDIX E

Non-Standard Special Provisions

Vibration Monitoring

CSP for Integral Abutments

Expanded Polystyrene Embankment

CSP FOR INTEGRAL ABUTMENTS - Item No.

Special Provision

Scope

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

References

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 906	Construction Specification for Structural Steel for Bridges
OPSS.PROV 909	Construction Specification for Prestressed Concrete - Precast Girders

Ontario Provincial Standard Specifications, Material:

OPSS 1605	Material Specification for Extruded Expanded Polystyrene Pavement Insulation
OPSS 1801	Material Specification for Corrugated Steel Pipe (CSP) Products

Canadian Standards Association Standards:

CSA G164-M Hot Dip Galvanizing of Irregularly Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

Definitions

For the purposes of this specification, the following definitions apply:

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

Submission and Design Requirements

Submissions

All submissions shall bear the seal and signature of the Design Engineer.

At least two weeks prior to commencement of installation of the abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS.PROV 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

Material

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

The sand fill for backfilling the CSP's shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

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

Expanded Extruded Polystyrene

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

Construction

General

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

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

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

The Contractor shall set the CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.

After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

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

After the sand fill has been placed to the top of each CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Expanded Extruded Polystyrene

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

Tolerances

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

<u>Criteria</u>	<u>Tolerance</u>
Maximum deviation of CSP from pile centroid.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified Elevation.	± 10 mm

Quality Assurance

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

Measurement for Payment

There will be no measurement for this item.

Basis of Payment

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

Non-Standard Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the expanded polystyrene embankment fill, including foundation preparation, excavation, leveling pad, polyethylene sheeting and associated works as shown on the contract drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

National Standards of Canada

CAN/CGSB - 51.20 M87 Thermal Insulation, Polystyrene, Boards and Pipe Covering

ASTM

ASTM D6817 Standard Specification for Rigid Cellular Polystyrene Geofoam

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Plastics

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS.PROV 212 Construction Specification for Earth Borrow

OPSS.PROV 501 Construction Specification for Compacting

OPSS.PROV 517 Construction Specification for Dewatering

OPSS 902 Construction Specification for Excavating and Backfilling - Structures

OPSS.PROV 1010 Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material

OPSS 1605 Material Specification for Extruded Expanded Polystyrene Pavement Insulation

OPSS 1860 Material Specification for Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the geotechnical investigation reports for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene

Molded rigid blocks produced by a process of pre-expansion, aging and forming of a petroleum based raw material.

Production Lot

The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and a method statement that provides full details of the materials and construction procedure.

6.2 Delivery, Storage, Handling and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirements.

6.3 Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of granular leveling pad.

- c) The method of placement of expanded polystyrene including temporary ballasting (if required) and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of protective concrete slab.
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

7. MATERIALS

7.1 Granular Leveling Pad

The leveling pad shall consist of a Granular 'A' or Granular 'B' Type II material with gradation and physical requirements as specified in OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Dimensional Stability
 6. Oxygen Index
 7. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Contract Administrator for review.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum design permanent stress level must not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 276 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

7.2.2.6 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.7 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant, limited or not resistant shall be submitted.

7.2.2.8 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.9 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' Type II material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Polystyrene

- 1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- 2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- 3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with a maximum joint opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- 4) Sloping end adjustments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- 5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- 6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- 7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.
- 8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- 9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- 10) The top surface and side surfaces of the expanded polystyrene shall be covered with 10 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

- 11) The side slope of the rigid expanded polystyrene embankment shall be covered with fill material as detailed elsewhere in this contract.
- 12) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted prior to the installation of the rigid expanded polystyrene embankments. The Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- 13) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer, a minimum of one week prior to the commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision, shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. *Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.*

10. EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11. QUALITY ASSURANCE

11.1 Quality Assurance

Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

11.2 Sampling and Testing

11.2.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 may be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

11.2.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, one (1) block shall be tested for the full suite of tests and three (3) blocks shall be tested for compressive strength.

11.2.3 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 Measurement for Payment

12.1 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 Payment

13.1 Basis of Payment

The granular leveling pad shall be included in the work and shall not be measured for separate payment.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials, and equipment to do the work as described above and no extra payments will be made.

VIBRATION MONITORING - Item No.

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the vibration monitoring during pile installation works and installation of driven protection systems.

2.0 REFERENCES - Not Used

3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Certificate of Conformance means a document issued by the Quality Verification Engineer confirming that the specified components of the Work are in General Conformance with the requirements of the Contract Documents.

Quality Verification Engineer means an Engineer retained by the Contractor qualified to provide the services specified in the Contract Documents. The Engineer shall have a minimum of five (5) years experience in the field of pile installation and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Submission Requirements

4.01.01 General

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- a) Qualifications of vibrations monitoring specialist.
- b) Proposed instrumentation.
- c) Proposed location of instruments.
- d) Proposed frequency of readings.
- e) Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

4.02.01.02 Monitoring Submissions

The measured results of the vibration monitoring shall be submitted to the Contract Administrator after each pile has been driven and prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

4.01.03 Certificate of Conformance

A completed Certificate of Conformance shall be submitted to the Contract Administrator upon completion of the pile installation work. The Certificate of Conformance shall be sealed and signed by a QVE and shall state that the pile installation work has been carried out in general conformance with the Contract Documents. The Certificate of Conformance shall also certify that the monitoring submissions have been completed as specified.

5.0 MATERIALS - Not Used

6.0 EQUIPMENT - Not Used

7.0 CONSTRUCTION

7.01 Monitoring

The Contractor shall take readings during driving of each pile. The readings shall be taken and recorded during the entire length of driving and during seating of the pile on the bedrock.

The pile(s) furthest from the monitored structure or utility should be driven first to assess the the vibration level at the existing structures. If necessary, the contractor must alter the pile driving procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven and prior to continuing with the subsequent piles. If the vibration monitoring results are acceptable, the Contractor may continue with the next pile. If the readings are not within the limits stated above, the Contractor shall alter the driving procedures until the vibrations are within acceptable levels.

8.0 QUALITY ASSURANCE - Not Used

9.0 MEASUREMENT FOR PAYMENT - Not Used

10.0 BASIS OF PAYMENT

10.01 Vibration Monitoring - Item

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



APPENDIX F

Results of Analysis

Figure F1

Figure F2

Figure F3

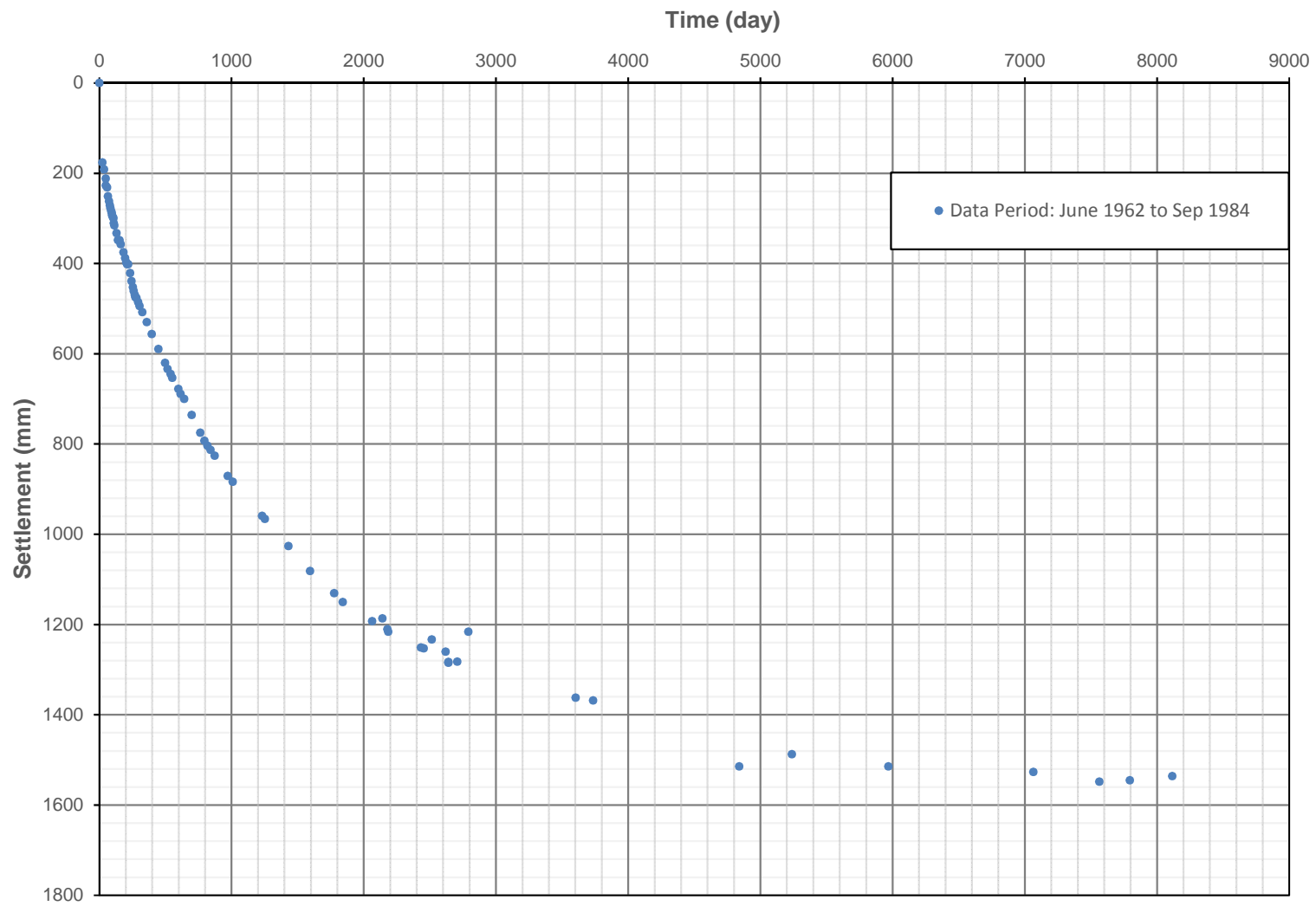
Figure F4/F5

Settlement History 1962 – 1984

Anticipated Settlement for Proposed
Granular Fill Embankment

Results of Slope Stability Analysis

P-Y Curves for H-Piles



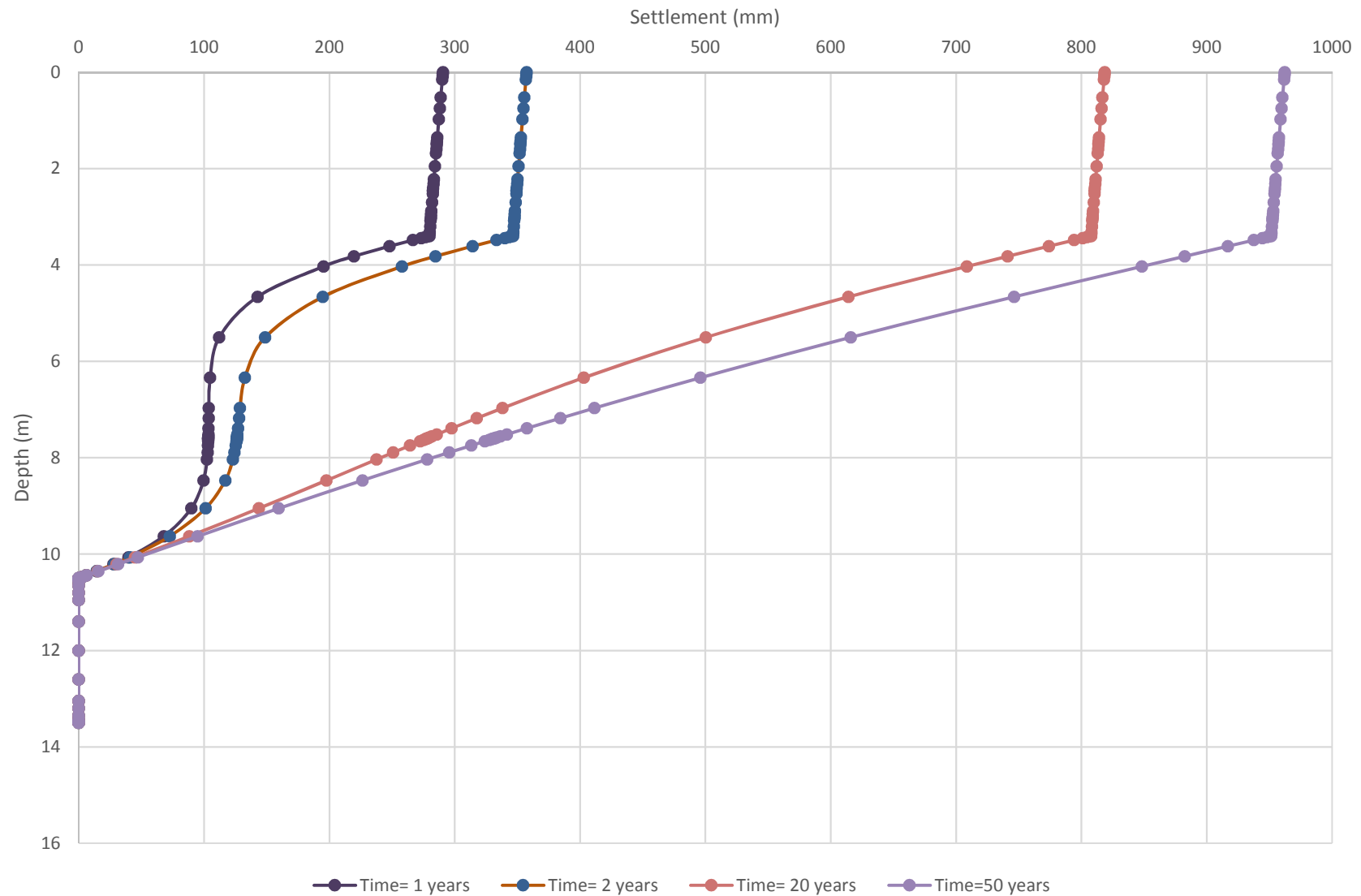
Settlement History 1962-1984

Highway 2/34 Underpass

Lancaster, Ontario

Project No.	12-1121-0193
Drawn:	WAM
Date:	17/08/2017
Checked:	KSL
Review:	FJH

Figure F1



Anticipated Settlement for Proposed Granular Fill Embankment

Highway 2/34 Underpass

Lancaster, Ontario

Project No.	12-1121-0193
Drawn:	WAM
Date:	17/08/2017
Checked:	KSL
Review:	FJH

Figure F2

Title: Slope Stability Short Term
 File Name: 12-1121-0193 Hwy 2-34 Embankment Static Undrained.gsz
 Method: Morgenstern-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0

Name: Existing Fill
 Model: Mohr-Coulomb
 Unit Weight: 20 kN/m³
 Cohesion: 0 kPa
 Phi: 28 °

Name: Embankment Fill
 Model: Mohr-Coulomb
 Unit Weight: 21.5 kN/m³
 Cohesion: 0 kPa
 Phi: 32 °

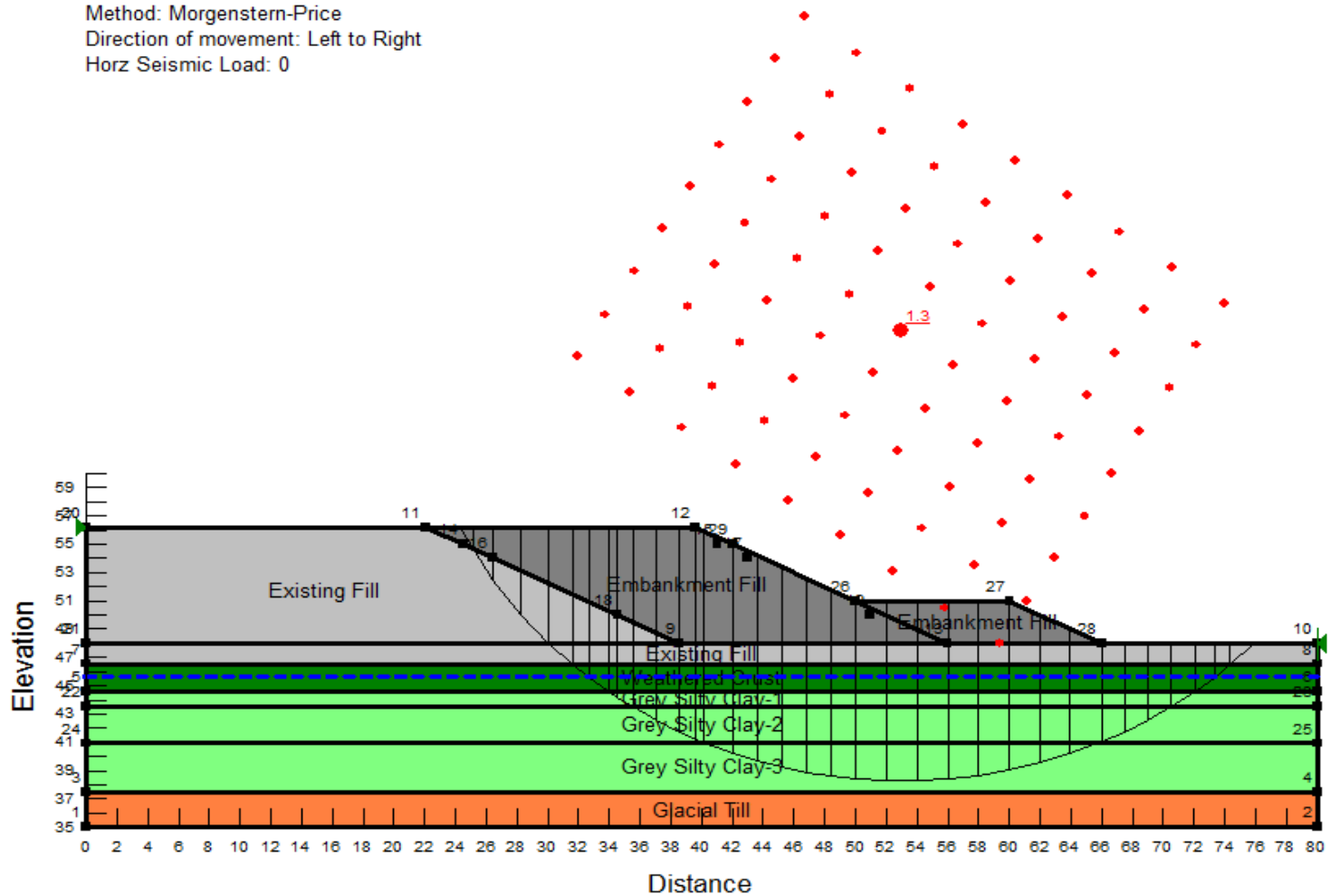
Name: Weathered Crust
 Model: Undrained (Phi=0)
 Unit Weight: 17.1 kN/m³
 Cohesion: 100 kPa

Name: Grey Silty Clay-1
 Model: S=f(depth)
 Unit Weight: 15.5 kN/m³
 C-Top of Layer: 32 kPa
 C-Rate of Change: -7.3 kPa/m
 Limiting C: 21 kPa

Name: Grey Silty Clay-2
 Model: Undrained (Phi=0)
 Unit Weight: 15.5 kN/m³
 Cohesion: 21 kPa

Name: Grey Silty Clay-3
 Model: S=f(depth)
 Unit Weight: 15.5 kN/m³
 C-Top of Layer: 21 kPa
 C-Rate of Change: 2.6 kPa/m
 Limiting C: 30 kPa

Name: Glacial Till
 Model: Bedrock (Impenetrable)



Results of Slope Stability
Highway 2/34 Underpass
Lancaster, Ontario

Project No.	12-1121-0193
Drawn:	WAM
Date:	17/08/2017
Checked:	KSL
Review:	FJH

Figure F3

SUMMARY OF P-y CURVES FOR A H-Pile 310x110 Pile - Abutments

Description Depth (z) * Elevation P-y Curves	Loose Sand (CSP)								Firm to Stiff Clay (Crust)				Soft to Firm Clay																					
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m	
	Elev. 49.5 m		Elev. 49.0 m		Elev. 48.5 m		Elev. 48.0 m		Elev. 47.5 m		Elev. 47.0 m		Elev. 46.5 m		Elev. 46.0 m		Elev. 45.5 m		Elev. 45.0 m		Elev. 44.5 m		Elev. 44.0 m		Elev. 43.0 m		Elev. 42.0 m		Elev. 41.0 m		Elev. 40.0 m		Elev. 39.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.004	4.613	0.003	7.368	0.004	14.820	0.005	24.784	0.006	37.259	0.007	51.921	0.000	9.300	0.000	9.300	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	
0.007	8.892	0.006	14.203	0.008	28.567	0.010	47.772	0.012	71.818	0.014	100.080	0.000	18.600	0.000	18.600	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	
0.011	12.593	0.009	20.114	0.012	40.456	0.014	67.654	0.017	101.708	0.020	141.733	0.000	27.900	0.000	27.900	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	
0.014	15.605	0.011	24.926	0.015	50.133	0.019	83.836	0.023	126.036	0.027	175.635	0.001	37.200	0.001	37.200	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	
0.018	17.938	0.014	28.651	0.019	57.627	0.024	96.368	0.029	144.876	0.034	201.889	0.002	46.500	0.002	46.500	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	
0.022	19.675	0.017	31.427	0.023	63.209	0.029	105.704	0.035	158.911	0.041	221.447	0.003	55.800	0.003	55.800	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	
0.025	20.933	0.020	33.436	0.027	67.250	0.034	112.461	0.041	169.068	0.047	235.602	0.005	65.100	0.005	65.100	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	
0.029	21.824	0.023	34.859	0.031	70.113	0.039	117.248	0.046	176.266	0.054	245.632	0.008	74.400	0.008	74.400	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	
0.032	22.446	0.026	35.853	0.035	72.111	0.043	120.590	0.052	181.291	0.061	252.634	0.011	83.700	0.011	83.700	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	
0.036	22.876	0.029	36.540	0.038	73.492	0.048	122.899	0.058	184.761	0.068	257.470	0.015	93.000	0.015	93.000	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	
0.040	23.171	0.032	37.010	0.042	74.439	0.053	124.482	0.064	187.142	0.074	260.787	0.020	102.300	0.020	102.300	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	
0.043	23.372	0.034	37.332	0.046	75.085	0.058	125.563	0.070	188.766	0.081	263.051	0.025	111.600	0.025	111.600	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	
0.047	23.509	0.037	37.550	0.050	75.524	0.063	126.298	0.075	189.871	0.088	264.591	0.032	120.900	0.032	120.900	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	
0.050	23.601	0.040	37.698	0.054	75.822	0.068	126.796	0.081	190.620	0.095	265.635	0.040	130.200	0.040	130.200	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	
0.054	23.664	0.043	37.799	0.058	76.024	0.072	127.134	0.087	191.128	0.101	266.343	0.050	139.500	0.050	139.500	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	
0.058	23.707	0.046	37.867	0.062	76.161	0.077	127.363	0.093	191.472	0.108	266.822	0.053	139.500	0.053	139.500	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	
Description Depth (z) * Elevation P-y Curves	Compact to Dense Sand to Sand and Gravel								z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m																			
	Elev. 38.0 m		Elev. 37.0 m		Elev. 36.0 m		Elev. 35.0 m																											
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)																										
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000																										
	0.001	238.749	0.001	258.856	0.001	279.167	0.001	299.274																										
	0.003	460.203	0.003	498.960	0.003	538.109	0.003	576.867																										
	0.004	651.737	0.004	706.626	0.004	762.069	0.004	816.957																										
	0.006	807.628	0.006	875.645	0.006	944.350	0.006	1012.367																										
	0.007	928.351	0.007	1006.535	0.007	1085.510	0.007	1163.694																										
	0.008	1018.288	0.008	1104.047	0.008	1190.672	0.008	1276.431																										
	0.010	1083.376	0.010	1174.616	0.010	1266.778	0.010	1358.019																										
	0.011	1129.497	0.011	1224.622	0.011	1320.708	0.011	1415.832																										
	0.013	1161.693	0.013	1259.530	0.013	1358.354	0.013	1456.191																										
	0.014	1183.935	0.014	1283.644	0.014	1384.361	0.014	1484.070																										
0.015	1199.188	0.015	1300.182	0.015	1402.196	0.015	1503.190																											
0.017	1209.597	0.017	1311.468	0.017	1414.367	0.017	1516.238																											
0.018	1216.676	0.018	1319.143	0.018	1422.645	0.018	1525.112																											
0.020	1221.479	0.020	1324.350	0.020	1428.261	0.020	1531.133																											
0.021	1224.733	0.021	1327.878	0.021	1432.066	0.021	1535.211																											
0.022	1226.935	0.022	1330.266	0.022	1434.641	0.022	1537.972																											

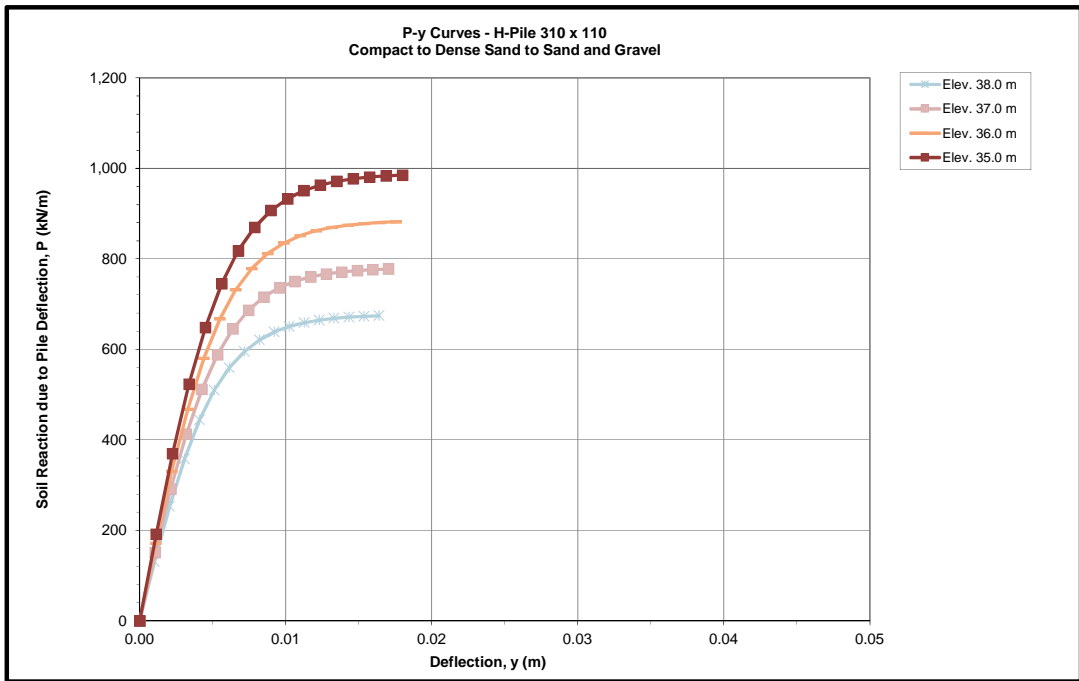
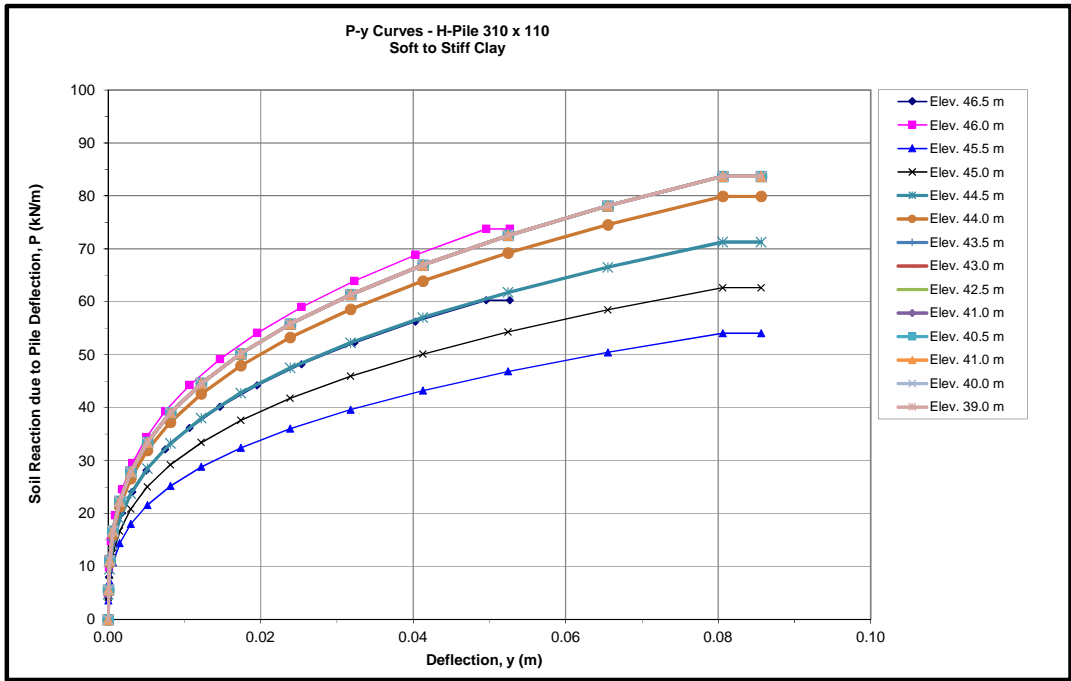
SUMMARY OF P-y CURVES FOR A H-Pile 310x110 Pile - Centre Pier

Description Depth (z) * Elevation P-y Curves	Firm to Stiff Clay (Crust)						Soft to Firm Clay																								Compact to Dense Sand to Sand and Gravel							
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m			
	Elev. 46.5 m		Elev. 46.0 m		Elev. 45.5 m		Elev. 45.0 m		Elev. 44.5 m		Elev. 44.0 m		Elev. 43.5 m		Elev. 43.0 m		Elev. 42.5 m		Elev. 41.0 m		Elev. 40.5 m		Elev. 41.0 m		Elev. 40.0 m		Elev. 39.0 m		Elev. 38.0 m		Elev. 37.0 m		Elev. 36.0 m		Elev. 35.0 m			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)		
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.000	4.018	0.000	4.918	0.000	3.604	0.000	4.178	0.000	4.753	0.000	5.327	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.000	5.580	0.001	131.224	0.001	151.331	0.001	171.642	0.001	191.749	
0.000	8.036	0.000	9.836	0.000	7.208	0.000	8.357	0.000	9.505	0.000	10.654	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.000	11.160	0.002	252.942	0.002	291.700	0.002	330.849	0.002	369.607			
0.000	12.054	0.000	14.753	0.001	10.812	0.001	12.535	0.001	14.258	0.001	15.981	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.001	16.740	0.003	358.216	0.003	413.104	0.003	468.547	0.003	523.436			
0.001	16.072	0.001	19.671	0.002	14.415	0.002	16.713	0.002	19.011	0.002	21.308	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.002	22.320	0.004	443.898	0.004	511.916	0.004	580.620	0.004	648.637			
0.002	20.090	0.002	24.589	0.003	18.019	0.003	20.891	0.003	23.763	0.003	26.635	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.003	27.900	0.005	510.251	0.005	588.436	0.005	667.410	0.006	745.595			
0.003	24.108	0.003	29.507	0.005	21.623	0.005	25.070	0.005	28.516	0.005	31.962	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.005	33.480	0.006	559.684	0.006	645.443	0.007	732.068	0.007	817.827			
0.005	28.126	0.005	34.424	0.008	25.227	0.008	29.248	0.008	33.269	0.008	37.289	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.008	39.060	0.007	595.458	0.007	686.699	0.008	778.861	0.008	870.101			
0.008	32.145	0.008	39.342	0.012	28.831	0.012	33.426	0.012	38.021	0.012	42.617	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.012	44.640	0.008	620.808	0.009	715.933	0.009	812.018	0.009	907.143			
0.011	36.163	0.011	44.260	0.017	32.435	0.017	37.604	0.017	42.774	0.017	47.944	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.017	50.220	0.009	638.504	0.010	736.340	0.010	835.165	0.010	933.001			
0.015	40.181	0.015	49.178	0.024	36.039	0.024	41.783	0.024	47.527	0.024	53.271	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.024	55.800	0.010	650.729	0.011	750.438	0.011	851.155	0.011	950.864			
0.020	44.199	0.020	54.095	0.032	39.643	0.032	45.961	0.032	52.279	0.032	58.598	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.032	61.380	0.011	659.112	0.012	760.106	0.012	862.121	0.012	963.115			
0.025	48.217	0.025	59.013	0.041	43.246	0.041	50.139	0.041	57.032	0.041	63.925	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.041	66.960	0.012	664.833	0.013	766.704	0.013	869.604	0.013	971.475			
0.032	52.235	0.032	63.931	0.052	46.850	0.052	54.317	0.052	61.785	0.052	69.252	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.052	72.540	0.012	668.724	0.014	771.191	0.014	874.693	0.015	977.160			
0.040	56.253	0.040	68.849	0.066	50.454	0.066	58.496	0.066	66.537	0.066	74.579	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.066	78.120	0.014	671.364	0.016	774.236	0.015	878.146	0.016	981.018			
0.050	60.271	0.050	73.767	0.081	54.058	0.081	62.674	0.081	71.290	0.081	79.906	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.081	83.700	0.015	673.153	0.016	776.298	0.016	880.486	0.017	983.631			
0.053	60.271	0.053	73.767	0.086	54.058	0.086	62.674	0.086	71.290	0.086	79.906	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.086	83.700	0.016	674.363	0.017	777.694	0.018	882.069	0.018	985.400			

NOTES: * Depth (z) is measured to be positive below the underside of the pile cap at centre pier (Elevation 47.0 m).

Please note the following assumptions:

1. P-y curves have been generated for vertical piles (i.e. no inclination)
2. Static loading condition is considered.
3. There are no pile group effects (i.e. analysis is based on a single pile).
4. P-y curves have been generated for strong axis of H-pile 310x110.



DRAFT



APPENDIX G

Technical Memorandum – Embankment Design Alternatives

DATE September 21, 2017**PROJECT No.** 12-1121-0193-1140**TO** Brad Craig, P.Eng.
Dillon Consulting Limited**FROM** Kim Lesage, P.Eng.
Fin Heffernan, P.Eng.**EMAIL** Kim_Lesage@golder.com
Fin_Heffernan@golder.com**EMBANKMENT DESIGN ALTERNATIVES
COUNTY ROAD 2/34 UNDERPASS
HIGHWAY 401, SITE 31-232
LANCASTER, ONTARIO
W.P. 4013-11-01**

This memo provides an assessment of the embankment design alternatives for the Country Road 2/34 Underpass replacement along Highway 401 in Lancaster, Ontario.

Background

The results of the foundation investigation for this project were provided in our draft report of August 2017 titled "*Foundation Investigation and Design, Country Road 2/34 Underpass Replacement, Highway 401, Site 31-232, Lancaster, Ontario, W.P. 4013-11-01*" (report number 12-1121-0193-1140).

The subsurface conditions at the site consist of up to 10 m of sensitive and compressible clay. The existing structure, built in 1963, is to be replaced with a two lane, two-span structure with a shift in the alignment to the west immediately adjacent to the existing alignment. A widening/new embankment will be required about 8 m in height and up to about 20 m in width at the crest to accommodate this shift. The west side slopes of the existing embankments will therefore form a small part of the new embankments.

The existing embankment loading over the clay deposit has led to very large settlements of the embankments since the original construction. Analysis results show that, if conventional earth fill or granular fill is used for the new 8 m high embankments (using conventional construction techniques), settlement of the approach embankments will be significant. It is important to limit these settlements not only to avoid roadway distortion, but also to avoid important differential settlements between the approach embankments and the bridge structure (which will be supported on deep foundations on bedrock, and therefore not subjected to these consolidation settlements) and to eliminate the corresponding downdrag loads on the piles at the abutments, which could in turn reduce the available capacity of the piles.

The following sections present the available options being considered for mitigating the anticipated settlements.



Embankment Design Alternatives

Given the magnitude of anticipated settlements, and their continuous/long-term nature, it is considered that periodic re-paving to correct for the settlement (as was done in the past) is not a feasible option for addressing/mitigating the settlement effects. Subexcavation of the clay would also not be feasible due to its thickness. The following options are therefore being considered for mitigating the anticipated settlements:

- 1) **Lightweight Fill**: Lightweight fill materials such as expanded polystyrene (EPS) could be used for the embankment construction, thereby reducing the stress increase on the compressible clay deposit to a level so that the settlements, within each specified distance from the abutment, would be within acceptable tolerances at paving time for the geotechnical criterion.
- 2) **Preloading with Wick Drains together with Lightweight Fill**: The new embankment areas could be preloaded, in part, and allowed to settle in advance of the roadway being paved or put into service over the new approach embankments. Wick-drains would be used to decrease the preload time and the magnitude of post-construction creep settlements. Due to the sensitive nature of the clay and consolidation characteristics, the preload height would have to be limited and the use of some EPS would still be required for this option to be feasible.
- 3) **Rigid Inclusions**: The installation of Rigid Inclusions (RI) is another alternative for mitigating settlements beneath embankments. RI's constructed of ready-mix concrete installed within the clay soil using specialty equipment would be suitable for this site. Rigid Inclusions could be installed in the clay deposit, up to original ground surface, to transfer the stress from the embankment loads down to the glacial till or bedrock. A Load Transfer Platform (LTP) created using granular material and geogrid, and/or concrete would be constructed above the Rigid Inclusions (i.e., beneath the embankment) to transfer the embankment loads to the columns.
- 4) **Deep Soil Mixing**: Deep soil mixing is another alternative for mitigating settlements beneath embankments. Deep soil mixing consists of in situ mechanical mixing of the native soil through a process that breaks down the soil without extraction while injecting a stabilizing agent in the mix at low pressure to create columns or panels of reinforced soil.

The advantages, disadvantages, relative costs, and risks associated with these feasible options are provided in the attached Table 1. In preparation of this table and memo, several specialty contractors were contacted and provided input to our summary, as follows:

- Plasti-Fab Ltd.;
- Menard Canada (Geopac);
- GeoSolv Design/Build Inc.; and,
- Keller Foundations Ltd. (GeoFoundations).

Further details for each alternative are presented in the following sub-sections.

Despite the selected option, the existing adjacent embankment makes for a complicated design and construction because the settlements will also have to be mitigated under the existing side slopes (otherwise there is the potential for significant post construction maintenance). This settlement could be reduced by installing a wedge of lightweight fill within the portion of the cross-section above the existing side slope. Some amount of shoring (i.e., roadway protection) would likely be required for all of the options listed above.

Due to the geometry of the new embankments with respect the existing embankments, the installation area and effectiveness of the wick drains, concrete columns or deep soil mixing would be limited to areas where the specialized equipment can install the various ground improvement components. The ground improvement methods would therefore likely be installed from at least two working platform elevations.

Lightweight Fill

To reduce the anticipated settlements, the embankment can be constructed of EPS lightweight fill.

Given the required thickness of material needed for the pavement structure (about 1.0 m), the protective concrete slab over the EPS (discussed below), and a granular working/levelling pad for placement of the EPS, it is considered that only minimal additional embankment fill could be placed to maintain the clay below its preconsolidation pressure. Therefore the entire thickness of the embankment (under the pavement structure) would need to consist of EPS for minimal settlements of the approach embankments to occur, as in the area adjacent to the pile supported abutments.

However, since the construction schedule allows for about a 1 year preload period, some thickness of granular embankment fill away from the abutment areas may be placed as a preload (i.e., to allow for some of the primary settlement to occur in advance of the roadway being paved or put into service) over the new approach embankments. This option would lessen the thickness of EPS required for embankment construction. Some of the existing embankment material would nonetheless need to be removed adjacent to the new embankments (i.e., steepening of the existing side slope adjacent to the new embankment) to substitute with lightweight fill.

It is considered that the post-construction settlements be restricted to the tolerable amounts accepted by MTO. These criteria would restrict the thickness of earth fill 'preload' used for the embankment construction, as follows.

Distance from Abutment (m)	Allowable Amount of Granular Fill¹ (m)
0 to 20	0.0
20 to 50	0.5
50 to 75	1.0
>75	1.5

Note 1: In addition to the pavement structure, expected to be about 1 m thick.

These limited thicknesses of granular embankment fill would likely bring the ultimate stress level in the underlying clay deposit to near or slightly over the preconsolidation pressure. A settlement monitoring program will need to be implemented to monitor the settlements prior to, during, and following the 'preload' placement. Settlement monitoring would provide an indication that the settlements are occurring as anticipated and to determine if the granular fill heights have to be altered in consideration of the 1 m pavement structure thickness to be placed above the EPS.

Prior to the placement of the EPS, the existing embankment should be cut down (i.e., unloaded) to form a temporary 1.5H:1V side slope prior to the new embankment construction. This unloading would allow for the placement of the new pavement structure without bringing the ultimate stress level in the underlying clay deposit to over it's current preconsolidation pressure and thus minimize settlements.

Further oedometer testing to provide additional information on the preconsolidation pressure should be considered for this option.

Preload with Wick Drains

Preloading can be used to mitigate the effects of settlements, with the use of wick drains to accelerate the settlements and mitigate the long term, post-construction embankment settlements.

Since the existing effective stress in the clay deposit is near its pre-consolidation pressure, it is recommended to limit the preload height at this site. Otherwise, the sensitive clay deposit has the potential to be remoulded, causing instability of the existing embankment and roadway. With this in mind, we would recommend a limited preload thickness, with wick drains, in addition to EPS lightweight fill. This option would therefore not eliminate the need for more expensive lightweight fill, but decrease the amount required for the embankment construction.

The following table presents the amount of preload recommended for the construction of the embankments.

Distance from Abutment (m)	Granular Fill Preload Amount¹ (m)
0 to 20	0.0
20 to 50	1.0
50 to 75	1.5
>75	2.5

Note 1: In addition to the pavement structure, expected to be about 1 m thick.

The wick drain design layout should consist of a triangular grid pattern at about a 1.25 m spacing. The wick drains should extend laterally to the design toe of the embankments as well as two rows beyond the toe. The wick drains should extend vertically from the top of the drainage layer (discussed below) to the bottom of the clay layer.

The wick drain materials and their installation should conform to OPSS.PROV 220 (Wick Drain Installation).

A 0.3 m thick granular drainage blanket composed of OPSS Granular B Type II should be placed on the ground surface following stripping and grading.

A monitoring program would need to be implemented which includes monitoring of the embankment settlements using settlement plates and monitoring of the dissipation of the excess pore water pressures in the clay deposit (which develop due to the load imposed by the embankment) using vibrating wire piezometers. The monitoring program would allow a review of the granular preload amount prior to placement of the EPS and pavement structure.

Rigid Inclusions

Rigid inclusions (RI) are used to transfer unacceptable embankment loads through compressible soils to stiffer soil or rock. This ground improvement method increases the load carrying capacity of the soil, reduces the compressibility/settlements and helps prevent slope instability.

Rigid inclusions can consist of cement-treated aggregate, grouted aggregate, grout mixed with soil, or concreted columns. Aggregate columns are not considered feasible for this site considering the underlying clay deposit. The clay would likely offer little resistance/confinement during the installation of the aggregates and therefore there would be a high risk of column bulging in addition to the potential for shearing failures and remoulding of the clay structure.

Concrete column rigid inclusions would be the most feasible RI system for the site, with a specifically designed Load Transfer Platform (LTP). The LTP, which transfers the load from the embankments to the rigid inclusions, is a key element of the design that will prevent arching between the columns. The system would also be designed to satisfy the MTO settlement and global stability criteria.

Due to the configuration of the proposed embankment with respect to the existing embankment, it would be recommended to install the RI's from two working elevations (i.e., two stages). The first stage would include installation of rigid inclusions from the existing ground level (i.e., about Elevation 48 m) beneath the footprint of the widened/new embankment. The second stage would include installation of RI's from a higher elevation, following granular embankment construction to a certain height (i.e., about Elevation 51 m), so that RI's can be placed beneath the future roadway and within the existing embankment side slopes. Roadway protection will be required for this second stage, adjacent to the existing lanes of traffic that will need to remain in service, so that RI's can be installed. Additional slope stability analysis would be required to ensure that the existing and future embankments remain stable during service.

A LTP would also be designed by the specialty ground improvement contractor to limit the load that is directly transmitted to the compressible clay soils. The load transfer system could consist of something as straightforward as several layers of geogrid or could be a concrete layer at the top of the RI's.

Wick drains may be required, in addition to RI's, depending on the specialty contractor that is retained and their proprietary design. Wick drains or other structural reinforcement can be used to mitigate against seismic instability.

At the abutment locations, the RI pattern is typically modified to allow for pile installation after some settlement has occurred. RI's may be reinforced to ensure they are not damaged during pile driving.

Some amount of EPS may still be required directly behind the abutments if the design cannot acceptably mitigate against downdrag forces or the resulting settlements.

Field trials would be recommended prior to or at the same time as design of the rigid inclusions to establish the range of strength that can be achieved from the ground improvement. Additional testing, such as pressuremeter testing or CPT testing may also be required by the specialty contractor.

Deep Soil Mixing

Improvement of weak and compressible soils by deep soil mixing (DSM) can be achieved by mixing the existing soils using either a slurry with binder (wet DSM) or a dry binder (dry DSM). Jetting of slurry can be also used to enhance mechanical mixing. Similar to Rigid Inclusions, deep soil mixing creates large columns of improved ground for embankment support.

Wet DSM would be suitable for this site. Approximately 1.8 m diameter columns would be created with the DSM procedures, placed in a specific pattern. A track-mounted drill rig would be used to directly inject the binder into the column areas, add the stabilizing agent, and mixing.

High plasticity clays with high shear strengths (i.e., weathered clay crust at the site) may require pre-treatment for successful performance. Furthermore, areas with stiff soils and/or obstructions, such as the existing embankments and side slopes, may require pre-drilling ahead of the soil mixing process.

Some amount of EPS may still be required directly behind the abutments if the design cannot acceptably mitigate against downdrag forces or the resulting settlements.

At least one pre-production test column would be required prior to construction.

Closure


Please feel free to contact us if you have any questions regarding this memo.

Yours truly,

GOLDER ASSOCIATES LTD.



Kim Lesage, P.Eng.
Geotechnical Engineer



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MTO Designate Foundation Contact



FJH/KSL/mvrd

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Attachments: Table 1 – Comparison of Embankment Settlement Mitigation Alternatives

TABLE 1
COMPARISON OF EMBANKMENT SETTLEMENT MITIGATION ALTERNATIVES

Embankment Option	Advantages	Disadvantages	Estimated Costs	Risks/Consequences
Option 1 Lightweight fill	<ul style="list-style-type: none"> Limits post-construction maintenance Eliminates downdrag forces on piles at the abutments Minimal impact on schedule 	<ul style="list-style-type: none"> Expensive 	<ul style="list-style-type: none"> \$4.5 to 5.0 Million 	<ul style="list-style-type: none"> Low risk option Abutment backfill will have to be carefully designed to resist seismic forces; a higher strength EPS may be needed behind the abutments
Option 2 Pre-loading with wick-drains	<ul style="list-style-type: none"> Reduces the amount of EPS required 	<ul style="list-style-type: none"> May delay paving May requires post-construction maintenance Mobilizing specialty subcontractor for limited amount of work EPS would still be required 	<ul style="list-style-type: none"> \$4.0 to 4.5 Million 	<ul style="list-style-type: none"> Some uncertainty about schedule, since cannot complete roadway construction until monitoring indicates sufficient settlement has occurred Would lead to unacceptable settlement if not used in conjunction with EPS Settlement monitoring recommended prior to final paving
Option 3 Rigid Inclusions - Concrete Columns	<ul style="list-style-type: none"> Relatively rapid installation Allows for greater bearing pressures and limited settlements 	<ul style="list-style-type: none"> Mobilizing specialty subcontractor may have impact on schedule 	<ul style="list-style-type: none"> \$2.5 to 3.5 Million (dependent on the design/contractor), plus earth fill embankment cost (up to \$0.5 Million) 	<ul style="list-style-type: none"> Some field testing ahead of production would be recommended Additional investigation, such as CPT testing may be required Settlement monitoring recommended prior to final paving
Option 4 Deep Soil Mixing	<ul style="list-style-type: none"> Relatively rapid installation Allows for greater bearing pressures and limited settlements 	<ul style="list-style-type: none"> Mobilizing specialty subcontractor may have impact on schedule 	<ul style="list-style-type: none"> \$3.0 Million, plus earth fill embankment cost (up to \$0.5 Million) 	<ul style="list-style-type: none"> Some field testing ahead of production would be recommended Slightly higher risk option for high plasticity clay May require predrilling where columns are required beneath the existing side slopes Settlement monitoring recommended prior to final paving

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