



Foundation Investigation and Design Report

High Fill Embankment Widening, Highway 401 Westbound Core and Collector Lanes, Neilson Road to Warden Avenue, City of Toronto, Ontario
MTO G.W.P. No. 2162-11-00

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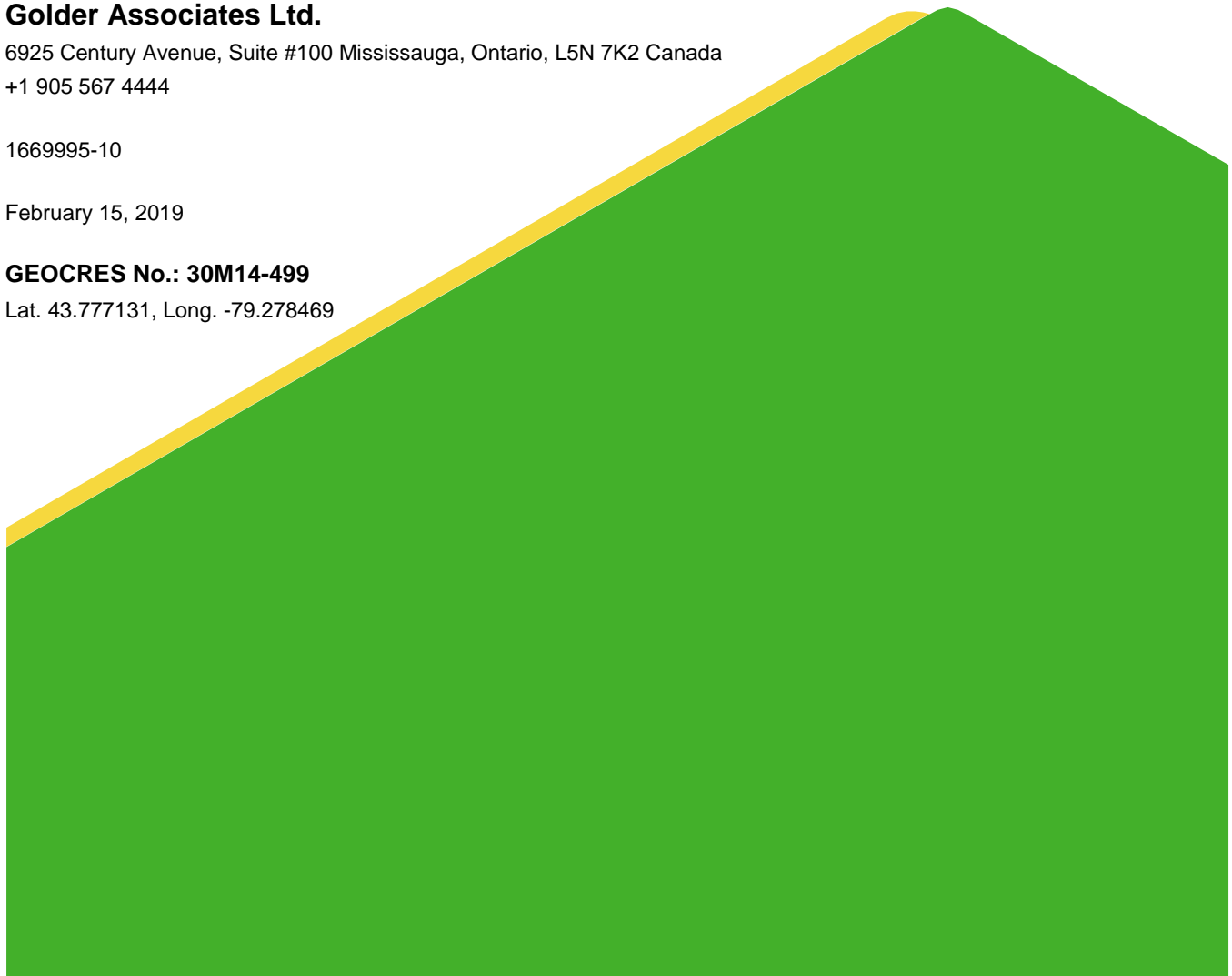
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Table of Contents

PART A – FOUNDATION INVESTIGATION REPORT

1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
3.0 INVESTIGATION PROCEDURES	1
3.1 1966 Investigation	1
3.2 2018 Investigation	2
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS.....	4
4.1 Regional Geology.....	4
4.2 Subsurface Conditions	4
4.2.1 Topsoil.....	4
4.2.2 Pavement Structure	5
4.2.3 Clayey Silt to Silt and Sand Fill	5
4.2.4 Surficial Silt to Clayey Silt	5
4.2.5 Interlayered Silt to Silty Sand with Clayey Silt Interlayers	6
4.2.6 Upper Clayey Silt Till.....	7
4.2.7 Gravelly Sand.....	7
4.2.8 Lower Clayey Silt	7
4.2.9 Lower Clayey Silt Till.....	8
4.3 Groundwater Conditions	8
5.0 CLOSURE	9

PART B – FOUNDATION DESIGN REPORT

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS	10
6.1 General.....	10
6.2 General Foundation Design Context.....	10
6.2.1 Consequence and Site Understanding Classification.....	10
6.2.2 Correlation of Automatic and Manual Hammer for SPT 'N'-Values.....	11

6.2.3	Seismic Design	11
6.2.3.1	Seismic Site Classification	11
6.2.3.2	Spectral Response Values and Seismic Performance Category	11
6.2.3.3	Soil Liquefaction.....	11
6.3	Global Stability of High Fill Embankment.....	12
6.3.1	Method of Analysis.....	12
6.3.2	Soil Shear Strength Parameters	12
6.3.3	Stability Analysis Results	13
6.4	High Fill Embankment Settlement.....	13
6.4.1	Settlement of Embankment Fill	13
6.4.2	Settlement of Foundation Soils	14
6.4.3	Discussion of Settlement Results and Recommendations	15
6.5	Construction Considerations	15
6.5.1	Subgrade Preparation Requirements - Removal of Organic Materials	16
6.5.2	Groundwater and Surface Water Control	16
6.5.3	Embankment Construction.....	16
7.0	CLOSURE	17

REFERENCES

DRAWINGS

Drawing 1	Borehole Locations
Drawing 2	Soil Strata

FIGURES

Figure 1	Static Global Stability Analysis – High Fill Area – Short-Term Conditions (Undrained)
Figure 2	Static Global Stability Analysis – High Fill Area – Long-Term Conditions (Drained)

APPENDICES

APPENDIX A - Borehole Records from 1966 Investigation (GEOCRE No. 30M14-67)

Records of Boreholes 67-3 and 67-4

APPENDIX B - Borehole Records from 2018 Investigation

Lists of Symbols and Abbreviations

Records of Boreholes CN-02, CN-03, HF-01, HF-02 and HF-03

APPENDIX C - Geotechnical Laboratory Test Results

Figure C-1	Grain Size Distribution – Clayey Silt Fill
Figure C-2	Plasticity Chart – Clayey Silt to Silt and Sand Fill
Figure C-3	Plasticity Chart – Surficial Silt to Clayey Silt
Figure C-4A/B	Grain Size Distribution – Silt to Silt and Sand to Silty Sand
Figure C-5	Plasticity Chart – Silt to Silty Sand
Figure C-6	Grain Size Distribution – Clayey Silt Interlayers
Figure C-7	Plasticity Chart – Clayey Silt Interlayers
Figure C-8	Grain Size Distribution – Upper Clayey Silt Till
Figure C-9	Plasticity Chart – Upper Clayey Silt Till
Figure C-10	Grain Size Distribution – Gravelly Sand
Figure C-11	Grain Size Distribution – Lower Clayey Silt
Figure C-12	Plasticity Chart – Lower Clayey Silt
Figure C-13	Grain Size Distribution – Lower Clayey Silt Till
Figure C-14	Plasticity Chart – Lower Clayey Silt Till

APPENDIX D - Operational Constraints

Delay of Paving Operations for High Fill Embankment Widening Area

PART A

FOUNDATION INVESTIGATION REPORT
HIGH FILL EMBANKMENT WIDENING
HIGHWAY 401 WESTBOUND CORE AND COLLECTOR LANES, NEILSON
ROAD TO WARDEN AVENUE, CITY OF TORONTO, ONTARIO
MTO G.W.P. NO. 2162-11-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the rehabilitation and operational improvements of the Highway 401 westbound (WB) core and collector lanes, from Neilson Road to Warden Avenue in the City of Toronto, Ontario (GWP 2162-11-00).

This report addresses the foundation investigation carried out for the proposed high fill embankment widening between West Highland Creek and the Canadian National (CN) Rail overhead structure associated with the northward widening of Highway 401. The high fill investigation area is shown in plan on Drawing 1. This report was developed based on information from the 2018 (current) investigation, supplemented with information from a 1966 (previous) foundation investigation completed by others at the culvert structure site near the east limit of the high fill area, reported as follows:

- **MTO GEOCRES No. 30M14-67:** Report titled “Foundation Investigation Report for Proposed Culvert Structure at Hwy. 401 and Highland Creek, 0.6 Mile East of Kennedy Road, Scarborough Twp., York County, District #6 (Toronto), W.J. 66-F-34 – W.P. 252-61-7”, prepared by Department of Highways Ontario, Foundation Section – Materials and Testing Division, dated July 26, 1966.

The results of the 1966 investigation are also summarized in the following report:

- **MTO GEOCRES No. 30M14-342:** “Preliminary Foundation Investigation and Design Report, Culvert Extensions, Highway 401 Rehabilitation from Warden Avenue to Brock Road, Toronto, Ontario, W.O. 07-20012,” by Golder Associates Ltd., dated April 2012.

The Terms of Reference and Scope of Work for the foundation engineering services are outlined in MTO’s Request for Proposal, dated November 21, 2016, which forms part of the Consultant Agreement (No. 2016-E-0009) for this project. The work has been carried out in accordance with Golder’s Supplementary Specialty Plan for foundation engineering services for this project, dated July 10, 2017.

2.0 SITE DESCRIPTION

The high fill area (shown on Drawing 1) is located on the north side of Highway 401 WBL from the CN Rail overhead structure to West Highland Creek (approximately Station 23+370 to Station 23+575) in the City of Toronto, Ontario. The natural ground surface in this area varies from about Elevation 164 m to 162.5 m, declining eastward, and the Highway 401 grade varies from approximately Elevation 173 m to 172 m, also declining eastward. The Highway 401 embankment is approximately 9 m to 10 m in height in this area. Based on visual observations, the existing north slope of the highway embankment in the high fill area has performed well.

3.0 INVESTIGATION PROCEDURES

3.1 1966 Investigation

A total of two boreholes were advanced as part of a 1966 investigation (GEOCRES No. 30M14-67) near the east limit of the high fill embankment area. The previous investigation boreholes used in this report have been renumbered to show the MTO GEOCRES reference number followed by the original borehole designation (i.e., 67-X, where X is the original borehole number).

The locations of the boreholes are summarized below and shown on Drawing 1. These borehole locations have been developed based on plotting the station and offset as shown on the 1966 borehole records and drawings, adjusted based on the site features shown on the drawings and converted to MTM NAD83 (Zone 10) coordinates. The borehole records from the 1966 investigation, including the summary results of the groundwater conditions and results of the geotechnical laboratory testing are presented in Appendix A and a summary of the borehole locations, ground surface elevation referenced to Geodetic datum and drilled depths are presented below. The summarized stratigraphy along the high fill area alignment is shown on the stratigraphic profile on Drawing 1.

Borehole No.	MTM NAD 83 (Zone 10)		Borehole Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
67-3	4,848,582.8	322,734.5	162.5 m	9.6 m
67-4	4,848,574.6	322,705.5	162.8 m	15.7 m

The Standard Penetration Test (SPT) “N”-values presented on the borehole records of the 1966 investigation were obtained using a manual hammer.

3.2 2018 Investigation

As part of the current investigation, three boreholes were advanced in the high fill embankment widening area (Boreholes HF-01 to HF-03) to supplement existing subsurface information. Additionally, two boreholes were advanced as part of the current investigation for the CN Rail overpass structure site (Boreholes CN-02 and CN-03), near the west limit of the high fill area, and these have been included in this report. Most boreholes for the current foundation investigation were drilled between November 1 and 5, 2018, with one borehole (Borehole CN-02) advanced earlier in the year between March 14 and 16, 2018. The borehole locations are shown on Drawing 1.

Most of the borehole investigation was carried out using CME-55 track-mounted and CME-75 truck-mounted drill rigs supplied and operated by Geo-Environmental Drilling Inc. of Acton, Ontario. Due to access constraints, Borehole HF-03 was advanced using portable drilling equipment (tripod) supplied and operated by OGS Inc. of Almonte, Ontario.

The boreholes were advanced through the overburden using 152 mm, 165 mm and 203 mm outside diameter hollow stem augers to depths generally ranging from 6.7 m to 11.3 m below existing ground surface; Borehole CN-02, which was drilled to provide coverage for the CN Rail overhead structure foundations, was drilled to a depth of 50.9 m. Borehole HF-03 was advanced through the overburden by wash boring methods using ‘NW’-sized casing to a depth of 7.7 m below existing ground surface.

In general, soil samples were obtained at 0.75 m, 1.5 m and 3 m intervals of depth using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer on the truck- and track-mounted drill rigs in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)¹. Borehole HF-03, advanced by portable equipment, employed a full-weight hammer lifted manually and dropped from the standard SPT height.

¹ ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. A standpipe piezometer was installed in Borehole CN-03 to permit monitoring of the water level. The installed piezometer consisted of a 50 mm diameter PVC pipe, with a 1.5 m slotted screen within a filter sand pack, with the piezometer sealed near the bottom of the borehole. The annulus between the piezometer pipe and the borehole wall above the filter sand pack was backfilled to the existing ground surface using bentonite. All other boreholes were backfilled with bentonite upon completion, in accordance with Ontario Regulation 903, Wells (as amended).

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, arranged for the clearance of underground utility services, observed the drilling, directed the sampling and in situ testing operations, logged the borehole and examined the soil samples. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual review. Geotechnical laboratory index and classification testing, consisting of natural moisture contents, Atterberg limits and grain size distributions, was conducted on selected samples in accordance with MTO and / or ASTM Standards as applicable. The results of geotechnical laboratory testing for the current investigation are included in Appendix C.

The borehole locations were laid out in the field by Golder personnel relative to existing road features and pre-selected coordinates using a hand-held global positioning system (GPS) unit with an accuracy of 1 m in the horizontal and vertical directions. The borehole locations were then measured relative to existing site features and the ground surface elevation at the borehole locations was established from the digital terrain model for the project. The location given on the borehole records and shown on Drawing 1 is positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations are referenced to Geodetic datum. The borehole locations, including both MTM NAD 83 and geographic coordinates, ground surface elevation and drilled depth are summarized below.

Borehole No.	MTM NAD83 (Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m) (Latitude °)	Easting (m) (Longitude °)		
HF-01	4848576.6 (43.776945)	322567.6 (-79.279282)	162.8	11.3
HF-02	4848597.4 (43.777131)	322633.0 (-79.278469)	162.6	11.2
HF-03	4848619.0 (43.777323)	322709.4 (-79.277519)	162.2	7.7
CN-02	4,848,512.6 (43.776370)	322562.4 (-79.279349)	173.8	50.9
CN-03	4,848,561.9 (43.776814)	322546.3 (-79.279548)	163.5	6.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the physiographic region known as the South Slope, according to *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)².

The South Slope region is comprised of calcareous clay till with lacustrine clay and silt reworked by glaciers, with numerous scattered drumlins and deep valley cuts caused by flowing streams towards Lake Ontario. The surface topography slopes gradually and uniformly southwards towards Lake Ontario. The overburden within the majority of the South Slope area is underlain by shale bedrock of the Queenston and Georgian Bay Formations, which contain limestone interlayers.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the 2018 investigation, including piezometer installation details and water level readings in the piezometers, and the results of the geotechnical laboratory tests carried out on selected soil samples are presented on the borehole records provided in Appendix B. The results of the in-situ field tests (i.e., SPT “N”-values) as presented on the borehole records and in Section 4.2 are uncorrected. The SPT “N”-values from the 1966 investigation are based on use of a manual hammer, while those in the 2018 investigation are based on use of an automatic hammer and the values are reported with no adjustment in this report, although it is recognized that SPT “N”-values obtained using a manual hammer are frequently higher than those obtained using an automatic hammer. Plots of the results of the geotechnical laboratory testing from the current investigation are presented in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 2 are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations; however, the factual data presented in the borehole records governs any interpretation of the site conditions. It is important to note that the current ground surface along the existing highway alignment is higher than the ground surface at the time of the 1966 borehole investigation as a result of subsequent highway construction.

In general, the subsurface conditions encountered in the boreholes advanced at the site for the current and previous investigations consist of the Highway 401 embankment fill (at the boreholes drilled from the highway platform) underlain by soil deposits that vary in composition from silt to silt and sand to silty sand interlayered by clayey silt. A surficial deposit of silt to clayey silt and localized thin existing fill is present in portions of the embankment widening area. These interlayered deposits are underlain in the deeper borehole (CN-02) by a sequence of deposits consisting of upper clayey silt till, gravelly sand, lower clayey silt, and lower clayey silt till. More detailed descriptions of the subsurface conditions are provided in the following sections of this report.

4.2.1 Topsoil

Approximately 70 mm to 150 mm of topsoil was encountered immediately below ground surface in Boreholes CN-03, HF-02 and HF-03 which were advanced north of the existing highway embankment. Approximately 300 mm of

² Chapman, L.J. and Putman, D.F., 1984, *The Physiography of Southern Ontario*, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.)

topsoil was encountered below the existing highway embankment fill in Borehole CN-02 which was advanced on the Highway 401 roadway.

4.2.2 Pavement Structure

Borehole CN-02 was advanced through the existing pavement structure on the westbound lane of Highway 401. The pavement is comprised of an approximately 203 mm thick layer of asphalt underlain by an approximately 1900 mm thick layer of gravelly sand (granular road base material). The measured Standard Penetration Test (SPT) “N”-values within the gravelly sand fill range between 18 blows and 86 blows per 0.3 m of penetration, indicating a compact to very dense level of compactness.

The natural water content measured on a selected sample of the gravelly sand was about 6 per cent.

4.2.3 Clayey Silt to Silt and Sand Fill

The pavement structure is underlain by an approximately 7.6 m thick layer of fill in Borehole CN-02. This fill is associated with the existing Highway 401 embankment and varies in composition from cohesive clayey silt with sand to non-cohesive silt and sand. The existing embankment fill extends to Elevation 164.3 m in Borehole CN-02.

An approximately 0.6 m thick layer of sandy silt fill was encountered underlying the topsoil in Borehole CN-03 which was advanced north of the existing highway, near the CN Rail line. This fill extends to Elevation 162.8 m.

The SPT “N”-values measured within the clayey silt to silt and sand fill in Borehole CN-02 range from 14 blows to 86 blows per 0.3 m of penetration, suggesting a stiff to hard consistency. One SPT “N”-value measured with the sandy silt fill in Borehole CN-03 was 5 blows per 0.3 m of penetration, suggesting a loose level of compactness.

The results of a grain size distribution test completed on one sample of the clayey silt fill from the existing highway embankment encountered in Borehole CN-02 is presented on Figure C-1 in Appendix C. Atterberg limits testing was carried out on two selected sample of the clayey silt to silt and sand fill from the existing highway embankment encountered in Borehole CN-02 and measured liquid limits ranging from about 15 to 17 per cent, plastic limits of about 12 per cent and plasticity indices ranging from about 3 to 5 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure C-2 in Appendix C, and indicate that the fill can be classified as clayey silt of low plasticity to silt with slight plasticity. The natural water content measured on selected samples of the fill ranges between about 9 and 10 per cent.

4.2.4 Surficial Silt to Clayey Silt

A 0.5 m to 0.8 m thick surficial deposit of silt to clayey silt with sand was encountered immediately below ground surface in Borehole HF-01, underlying the topsoil in Borehole HF-03, and underlying the fill in Borehole CN-03. The surface of the surficial silt to clayey silt layer was encountered between Elevations 162.8 m and 162.0 m, and the deposit extends to between Elevations 162.1 m and 161.5 m.

The SPT ‘N’-values measured within the surficial silt to clayey silt deposit are between 4 and 8 blows per 0.3 m of penetration, suggesting a firm to stiff consistency.

Atterberg limits testing was carried out on two selected samples of the surficial silt to clayey silt deposit encountered during the 2018 investigation and measured liquid limits of about 20 and 29 per cent, plastic limits of about 17 and 19 per cent, and plasticity indices of about 3 and 10 per cent. The results, which are plotted on a plasticity chart on Figure C-3 in Appendix C, indicate that the layer consists of silt with slight plasticity to clayey silt of low plasticity.

The natural water content measured on three selected samples of the surficial silt to clayey silt deposit range from about 20 to 27 per cent.

4.2.5 Interlayered Silt to Silty Sand with Clayey Silt Interlayers

Below the fill, surficial silt to clayey silt deposit or original ground surface, the boreholes encountered a non-cohesive soil deposit comprised of various layers / interlayers varying in composition from silt to sandy silt to silt and sand to silty sand, interlayered in places with clayey silt with sand. A gravelly sand layer was also encountered within this deposit at the base of Borehole HF-03.

The silt to silty sand deposit was fully penetrated in Borehole CN-02, where it is 9.3 m thick, with its base at Elevation 154.7 m; the remaining boreholes terminated within this deposit, penetrating it for about 5.2 m to 15.7 m. A clayey silt interlayer was encountered within these thicknesses in four of the boreholes; the elevations of the surface and base of the clayey silt interlayers, and interlayer thicknesses as encountered in the boreholes, are summarized below.

Borehole No.	Depth to Surface of Interlayer (m)	Interlayer Surface Elevation (m)	Interlayer Thickness (m)	Interlayer Base Elevation (m)
67-3	2.7	159.7	2.3	157.4
67-4	2.7	160.0	2.3	157.7
HF-01	2.5	160.3	4.7	155.6
HF-02	2.2	160.4	5.7	154.7

The measured SPT 'N'-values in the non-cohesive interlayers range from about 13 blows to 156 blows per 0.3 m of penetration, with numerous 'N'-values ranging from 102 blows to 125 blows per 0.3 m of penetration, and up to 100 blows for 0.13 m of penetration, indicating a compact to very dense level of compactness. The measured SPT 'N'-values in the cohesive clayey silt interlayers range from about 13 blows to 66 blows per 0.3 m of penetration, with one 'N'-value of 100 blows for 0.15 m of penetration, suggesting a stiff to hard and primarily very stiff to hard consistency.

Grain size distribution tests were carried out on twelve samples of the silt to silty sand deposit encountered during the 2018 investigation, and the results are shown on Figures C-4A and C-4B in Appendix C. Atterberg limits testing was carried out on three selected samples of the non-cohesive silt to silty sand portion of the deposit encountered during the 2018 investigation and measured liquid limits ranging from about 13 to 14 per cent, plastic limits ranging from about 10 to 11 per cent, and plasticity indices ranging from about 3 to 4 per cent. Atterberg limits testing was also carried out on two selected samples of the sandy silt to silty sand portion of the deposit from the 1966 investigation and measured liquid limits of about 13 and 15 per cent, plastic limits of about 11 and 12 per cent, and plasticity indices of about 2 and 3 per cent. These results, which are plotted on a plasticity chart on Figure C-5 or shown on the borehole records in Appendix A, confirm that the silt to silty sand deposit contains silt of slight plasticity.

Grain size distribution tests were carried on two samples of the clayey silt interlayers, and the results are shown on Figure C-6 in Appendix C. Atterberg limits testing was carried out on two selected samples of the clayey silt interlayers encountered during the 2018 investigation and measured liquid limits ranging from about 16 to 17 per cent, plastic limits ranging from about 10 to 11 per cent, and plasticity indices of about 6 per cent. The results,

which are plotted on a plasticity chart on Figure C-7 in Appendix C, indicate that the cohesive interlayers are comprised of clayey silt of low plasticity. Atterberg limits testing was also carried out on four selected samples of the clayey silt interlayer(s) from the 1966 investigation and measured liquid limits ranging from about 15 to 17 per cent, plastic limits ranging from about 9 to 11 per cent, and plasticity indices ranging from about 4 to 6 per cent.

The natural water content measured on selected samples of the interlayered deposit range between about 7 and 20 per cent.

4.2.6 Upper Clayey Silt Till

A 7.1 m thick upper till deposit was encountered at about Elevation 154.7 m, underlying the silty sand to silt deposit in Borehole CN-02. The till deposit is comprised of clayey silt to clayey silt with sand, which shows a relatively broader grain size distribution than the interlayered non-cohesive / cohesive deposit.

The SPT 'N'-values measured within the upper till deposit range from 35 blows to 140 blows per 0.3 m of penetration, suggesting a hard consistency.

Grain size distribution tests were carried out on one selected sample of the upper clayey silt till deposit encountered during the 2018 investigation, and the results are shown on Figure C-8 in Appendix C. Atterberg limits testing was carried out on one selected sample of the upper till deposit encountered during the 2018 investigation and measured a liquid limit of 19 per cent, a plastic limit of 12 per cent, and a corresponding plasticity index of 7 per cent. The result, which is plotted on a plasticity chart on Figure C-9 in Appendix C, indicates that the upper till deposit is classified as clayey silt of low plasticity. The natural water content of two samples of the upper clayey silt till are about 10 per cent and 13 per cent.

4.2.7 Gravelly Sand

A deposit or layers of gravelly sand was encountered underlying the upper clayey silt till in Borehole CN-02. The layer is 3.1 m thick, with its base Elevation at 144.5 m.

The SPT 'N' value measured in the gravelly sand layer is 49 blows per 0.3 m of penetration, indicating a dense compactness condition. A grain size distribution test was carried out on one sample from the gravelly sand layers encountered during the 2018 investigation, and the result is shown on Figure C-10 in Appendix C. The natural water content measured on one selected sample of the gravelly sand layers is about 11 per cent.

4.2.8 Lower Clayey Silt

A 12.2 m thick lower deposit of clayey silt was encountered underlying the gravelly sand in Borehole CN-02. The surface of this deposit was encountered at Elevation 144.5 m in the borehole, and the deposit extends to Elevation 132.3 m.

The SPT 'N' values measured within the lower clayey silt deposit range from 28 blows to 119 blows per 0.3 m of penetration, with one 'N'-value of 100 blows for 0.15 m of penetration, suggesting a very stiff to hard consistency.

Grain size distribution tests were carried out on one sample of the clayey silt deposit encountered during the 2018 investigation, and the results are shown on Figure C-11 in Appendix C. Atterberg limits testing was carried out on two selected samples of the clayey silt deposit encountered during the 2018 investigation and measured liquid limits of 21 and 35 per cent, plastic limits of 15 and 17 per cent, and plasticity indices of 6 and 18 per cent. The results, which are plotted on a plasticity chart on Figure C-12 in Appendix C, indicate that the deposit consists of clayey silt

of low plasticity. The natural water content measured on selected samples of the clayey silt deposit ranges between about 14 and 23 per cent.

4.2.9 Lower Clayey Silt Till

A deposit of clayey silt till was encountered underlying the lower clayey silt deposit in Borehole CN-02, with the surface of the lower till deposit at Elevation 132.3 m. The borehole terminated within this deposit, penetrating it for a thickness of 9.4 m.

The SPT 'N' values measured within the lower clayey silt till deposit range from 22 to 30 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

A grain size distribution test was carried out on one sample of the lower clayey silt till deposit encountered during the 2018 investigation, and the results are shown on Figure C-13 in Appendix C.

Atterberg limits testing was carried out on one selected sample of the lower clayey silt till deposit encountered during the 2018 investigation and measured a liquid limit of 17 per cent, a plastic limit of 11 per cent, and a corresponding plasticity index of 6 per cent. The result, which is plotted on a plasticity chart on Figure C-14 in Appendix C, indicates that the lower till deposit consists of clayey silt of low plasticity. The natural water content measured on selected samples of the lower clayey silt till ranges between about 12 and 17 per cent.

4.3 Groundwater Conditions

The groundwater levels in the open boreholes were measured upon completion of drilling operations during the 2018 and 1966 investigations, and in the piezometer installed in Borehole CN-03, as summarized below.

Borehole No.	Ground Surface Elevation (m)	Depth to Groundwater (m)	Groundwater Elevation (m)	Date	Comments
CN-02	173.8	-*	-	Mar. 16, 2018	Open borehole
CN-03	163.5	Dry	-	Mar. 16, 2018	Open borehole
		0.3	163.2	Nov. 1, 2018	Standpipe piezometer
HF-01	162.8	6.6	156.2	Nov. 1, 2018	Open borehole
HF-02	162.6	6.8	155.8	Nov. 2, 2018	Open borehole
HF-03	162.2	0.7**	161.5	Nov. 5, 2018	Open borehole
67-3	162.5	0.0	162.5	Apr. 18, 1966	Open borehole
67-4	162.8	0.0	162.8	Apr. 19, 1966	Open borehole

Notes:

* A water level reading was not taken on completion of drilling the borehole as water / drilling mud was added during the drilling operations.

** Depth to groundwater estimated from observation of a wet split spoon sample during drilling. A water level reading was not taken on completion of drilling.

As some of these water levels were measured immediately after completion of drilling, they may not represent the stabilized groundwater level at the site, nor the current level in the case of the 1966 data. The groundwater level in

the area will be subject to seasonal fluctuations and should be expected to be higher during the spring season or during and following periods of heavy precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared Matt Soderman, P.Eng, a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent technical and quality control review of the report.



Matt Soderman, P.Eng.
Geotechnical Engineer



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Principal, MTO Foundations Designated Contact

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PART B

FOUNDATION DESIGN REPORT
HIGH FILL EMBANKMENT WIDENING
HIGHWAY 401 WESTBOUND CORE AND COLLECTOR LANES, NEILSON
ROAD TO WARDEN AVENUE, CITY OF TORONTO, ONTARIO
MTO G.W.P. NO. 2162-11-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the proposed high fill embankment widening between West Highland Creek and the Canadian National (CN) Rail overhead, associated with the northward widening of Highway 401 as part of the rehabilitation and operational improvements of the Highway 401 westbound core and collector lanes, from Neilson Road to Warden Avenue in the City of Toronto, Ontario.

These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the 2018 subsurface investigation at this site, supplemented with data from the 1966 investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the requirements for stability and settlement of the widened high fill embankment. The Foundation Investigation Report, discussion and recommendations are intended for the use of the MTO and their designers, and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. Contractors must make their own interpretation based on the factual data in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

Based on the cross-sections of the proposed widened Highway 401 westbound (WB) lanes (electronic design drawing files provided to Golder by WSP on June 11, 2018), it is understood that the existing high fill embankment will be widened laterally northward by up to 12 m (relative to the existing embankment crest). This will require placement of a vertical thickness of up to approximately 5 m of additional fill atop the existing embankment side slopes. The overall height of the embankment fill within the high fill widening areas will range from 10 m to 12 m. The existing high fill embankment in the area appears to have performed well (no indications of excessive settlements or instability) based on field observations of the embankment slopes, crest and existing pavement made during Golder's 2018 subsurface investigation.

6.2 General Foundation Design Context

6.2.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code (CHBDC, 2014)* and its *Commentary*, the proposed northward widening of the Highway 401 WB high fill embankment is expected to carry high traffic volumes and its performance may have potential impacts on other transportation corridors; hence, the proposed high fill embankment works have been assessed as having a “typical consequence level” associated with exceeding limits states design.

Based on the level of foundation investigation carried out for the high fill area in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding 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.

For a “typical degree of understanding”, *CHBDC (2014)* requires a minimum Factor of Safety of 1.33 for the short-term/temporary condition and 1.54 for the long-term/permanent condition for global slope stability of embankments.

6.2.2 Correlation of Automatic and Manual Hammer for SPT 'N'-Values

It is recognized that some differences in Standard Penetration Test (SPT) 'N'-values will occur due to the use of an automated hammer with higher efficiency in the 2018 investigation as compared to a manually operated hammer (i.e., rope cathead) that was used in the 1966 investigation. The 2018 SPT "N"-values correlate reasonably well with the 1966 data when corrected to a 60% efficiency of hammer energy transfer. The foundation options and recommendations presented below are based on the correlated "N₆₀"-values, where applicable.

6.2.3 Seismic Design

6.2.3.1 Seismic Site Classification

The subsurface conditions for seismic site characterization were assessed based on the results of the field investigation and laboratory testing. The SPT "N"-values measured in the soil layers and the interpreted shear wave velocity of soils up to 30 m below ground surface were used to define the seismic site classification in accordance with Table 4.1 of the CHBDC (2014). Based on this methodology, it is considered that a Site Class C would be applicable for this high fill embankment area.

6.2.3.2 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.4 of the CHBDC (2014), the peak ground acceleration (PGA) values and design spectral acceleration (Sa) values for Site Class C are presented below.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.040	0.071	0.134
PGV (m/s)	0.031	0.051	0.090
Sa (0.2) (g)	0.068	0.115	0.209
Sa (0.5) (g)	0.043	0.067	0.112
Sa (1.0) (g)	0.024	0.036	0.059
Sa (2.0) (g)	0.011	0.018	0.028
Sa (5.0) (g)	0.0024	0.0040	0.0069
Sa (10.0) (g)	0.0011	0.0017	0.0029

6.2.3.3 Soil Liquefaction

Given the generally very stiff to hard consistency / dense to very dense compactness condition of the soils present at the site and the low seismic hazard classification for the site, the risk of potential soil liquefaction due to a seismic event is very low.

6.3 Global Stability of High Fill Embankment

The following sub-sections outline the method and parameters used to evaluate static global stability of the proposed northward widening of the high fill embankment, together with the results of the stability analyses.

6.3.1 Method of Analysis

Two-dimensional limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern Price method of analysis. Morgenstern Price is a general method of slices which is based on equilibrium of forces and moments acting on each slice of soil mass above the potential failure surface. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. For the purpose of the stability analysis, the Factor of Safety is equal to the inverse of the product of the consequence factor, Ψ , and the geotechnical resistance factor, ϕ_{gu} . (i.e., $FoS = 1/(\Psi \cdot \phi_{gu})$). Accordingly, minimum Factors of Safety of 1.33 and 1.54 have been used for the design of the embankment slopes for the short term/temporary and long term/permanent static conditions, as per Table 6.2 of CHBDC (2014).

The stability analyses have been completed for a single representative typical critical section of the general widening area based on the highest embankment height and the “poorest” (i.e. “weakest”) soil conditions encountered within the high fill embankment area. The widened embankment geometry is based on the typical cross sections provided by WSP with an assumed maximum embankment height of 12 m. The widened embankment side slopes are designed at grade of 2 horizontal to 1 vertical and include a 2 m wide mid-slope bench. The piezometric groundwater level used in the analyses was interpreted to be at approximately 0.3 m below ground surface, as observed in the standpipe piezometer installed in Borehole CN-03. It is assumed for the analysis that topsoil and other unsuitable material will be removed from the footprint of the high fill embankment widening during foundation preparation.

6.3.2 Soil Shear Strength Parameters

For the non-cohesive soils present at the site, the effective stress parameters employed in the analyses were estimated from empirical correlations based on the results of the in-situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For the upper surficial clayey silt cohesive deposit, total stress parameters were employed in the analyses of the short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were estimated from correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. Effective stress parameters were also assigned to the upper surficial cohesive deposit to evaluate the stability based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle (ϕ') for the cohesive deposits were estimated from empirical correlations based on the plasticity index. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted using engineering judgment based on precedent experience in similar soil conditions.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the proposed high fill embankment widening area.

Soil Deposit	Bulk Unit Weight, γ (kN/m ³)	Effective Friction Angle, ϕ' (°)	Cohesion, c' (kPa)	Undrained Shear Strength, s_u (kPa)
New embankment fill	21	32	-	-
Compact to very dense / stiff to hard existing embankment fill or existing surficial fill	19	30	-	-
Firm surficial clayey silt	19	32	-	50
Compact to very dense silt to silty sand	20	33	-	-
Very stiff to hard clayey silt interlayers	20	32	-	150
Hard upper clayey silt till	21	34	-	300

6.3.3 Stability Analysis Results

The stability analyses indicate that the proposed widened high fill embankment with a maximum height of 12 m constructed with side slopes inclined at a minimum of 2 horizontal to 1 vertical (with 2 m wide mid-slope benches (discussed further in Section 6.5.3)) will have a FoS of greater than 1.33 and 1.54 against global instability for the short-term (undrained) and long-term (drained) conditions, respectively. An example of the static global stability analyses results for the short-term and long-term conditions are provided on Figures 1 and 2, respectively.

6.4 High Fill Embankment Settlement

The existing Highway 401 WBL high fill embankment will be widened by up to about 12 m, which would require placement of a vertical thickness of up to approximately 5 m of additional fill atop the existing embankment side slope.

Settlement of the high fill embankment widening areas will occur as a result of compression of the new embankment fill used in the widening itself, as well as short-term compression of the cohesionless soils and generally very stiff to hard cohesive soils that underlie the embankment widening area. A limited surficial deposit of firm to stiff clayey silt (based on SPT 'N' values of 4 to 8 blows per 0.3 m of penetration) was encountered in the high fill embankment widening area in Boreholes HF-01, HF-03 and CN-03. It is recommended that all surficial topsoil, organic matter and any other unsuitable materials near surface are removed prior to fill placement.

6.4.1 Settlement of Embankment Fill

Settlement of new fill that is properly placed and compacted for construction of the embankment widening will occur during construction. Provided that the embankment widening material consists of clean earth fill or granular fill, the settlement of the embankment fill itself is expected to be less than approximately 25 mm for the proposed widened embankment height. Although not required, use of granular fill is preferred for the new embankment construction because this would reduce the magnitude of settlement, since the majority of settlement of granular fills will occur during, or immediately following construction.

6.4.2 Settlement of Foundation Soils

To estimate the magnitude of the expected settlements of the foundation materials below the high fill embankment widening area, settlement analyses were carried out for a typical critical section using both hand calculations and the commercially available software Settle-3D from Rocscience. Similar to the slope stability analysis discussed in Section 6.3.1, the typical critical section corresponds to the greatest fill height on a simplified representative “worst-case” foundation stratigraphy developed based on the conditions encountered in boreholes advanced in the high fill widening area.

Settlement analyses were carried out for the foundation materials using the estimated elastic deformation moduli and consolidation parameters as given below, based on correlations with the SPT “N” values, laboratory test results and correlations published in literature (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974), and engineering judgement from experience with similar soils in this region of Ontario. The surficial, firm clayey silt deposits encountered in Boreholes HF-01, HF-03 and CN-03 were assumed to be normally to slightly over-consolidated for the analyses. The coefficient of consolidation, c_v (cm^2/s), required in the time rate analysis was established using correlation with plasticity indices. A bulk unit weight of 21 kN/m^3 was employed for the proposed embankment widening fill in calculating the loading of the new fill on the subgrade subsoils.

Foundation Soil	Bulk Unit Weight (kN/m^3)	Approximate Thickness in Representative Critical Section (m)	Estimated Deformation Properties
Loose existing non-cohesive fill	20	0.6	$E' = 15 \text{ MPa}$
Firm surficial clayey silt	19	0.7	$e_0 = 0.65$, $C_c = 0.15$
Compact to dense silt to silty sand	20	2.0	$E' = 55 \text{ MPa}$
Dense to very dense silt to silty sand	20	5.0	$E' = 100 \text{ MPa}$
Very stiff to hard clayey silt interlayer	20	4.5	$E' = 75 \text{ MPa}$
Hard upper clayey silt till	21	6.0	$E' = 150 \text{ MPa}$

The presence and thickness of the compressible soils and the thickness/height of the new fill will vary along the proposed widened high fill embankment alignment, and as such the settlements will similarly vary. Given that the analysis was carried out at a critical fill section (i.e. at the thickest fill and/or less stiff founding soil locations) of the widened embankment, the settlement estimated will generally represent the maximum values.

The maximum total settlements are comprised of immediate settlement (i.e. settlement during or shortly after construction) due mainly to compression of the loose non-cohesive existing embankment fill, compact to very dense silt to silty sand, very stiff to hard clayey silt interlayers, and clayey silt till, and primary consolidation settlement (i.e. time-dependent settlement) of the firm surficial clayey silt layer. The maximum settlements of the soils under the high fill embankment widening are estimated to range from 75 mm to 90 mm, with the maximum at the crest of the widened embankment, and lesser values at the existing crest and new embankment toe, as discussed further below.

The initial (immediate) compression settlement of the foundation materials under the loading from the new embankment widening fill is expected to range from 5 mm to 10 mm in the areas between the existing embankment crest and newly widened crest/shoulder and from the approximately the mid slope height of the widened embankment to the toe of the widened embankment slope; and up to a maximum of 15 mm to 30 mm between the new widened crest/shoulder to approximately the mid slope height of the widened embankment where the increase to the effective stresses in the foundation relative to the initial stress state is the greatest. The immediate settlements are expected to occur immediately during construction in response to the placement of the new fill.

The time-dependent settlement of the surficial firm clayey silt under the new fill loading is estimated to be between 50 mm and 60 mm. Similar to the location of the maximum compression settlement, the maximum consolidation settlement would occur between the crest of the newly widened crest/shoulder to approximately the mid slope height of the widened embankment. Consolidation settlements would reduce to an estimated range 25 mm between the existing embankment crest and newly widened crest/shoulder and from the mid slope height of the widened embankment to the toe of the widened embankment slope, where the increase to the effective stress in the foundation relative to the initial stress is less due to a relatively high initial effective stress under the maximum existing embankment fill level (crest of slope) and/or a lower vertical thickness of new fill placement. Secondary consolidation of the surficial clayey silt layer is anticipated to be negligible.

Based on an estimated co-efficient of consolidation (c_v) of $5 \times 10^{-3} \text{ cm}^2/\text{s}$ for the firm cohesive soils (and the imposed loading conditions, and assuming two-way drainage of the thin cohesive deposit), the majority (up to 90%) of the primary consolidation settlement within the surficial clayey silt subsoils below the new fill would occur within 15 days after completion of construction of embankment widening.

6.4.3 Discussion of Settlement Results and Recommendations

The settlement performance criterion for design of embankment widenings within the high fill area is outlined in MTO's Guideline titled, "Embankment Settlement Criteria for Design", dated July 2010. The allowable total post-construction settlement and differential settlement limits for freeway embankment widenings is 50 mm and 200:1, respectively, over a 20-year period following completion of construction for a King's highway.

Given that the immediate settlement is expected to occur during or immediately after construction, and considering that the majority of the consolidation settlement, although it exceeds the allowable total post-construction settlement limit of 50 mm, is estimated to occur within a 15-day period given the relatively thin and low plastic nature of the surficial cohesive clayey silt deposit, settlement mitigation measures (such as pre-loading) are not required. The magnitude of any remaining consolidation settlement occurring after a 15-day period is expected to be less than 25 mm, satisfying the post-construction settlement criterion of 50 mm over a 20-year period. In this regard, it is recommended that the final grading and paving of high fill embankment widening be carried out a minimum of 15 days after completion of bulk fill placement to allow for any additional fill placement that may be required to accommodate for any post-construction settlement. It is anticipated that construction of the pavement structure is unlikely to occur within 15 days of completion of the embankment filling; however, an operational constraint has been developed (see Appendix D) to address this timing requirement, for inclusion in the Contract Documents.

6.5 Construction Considerations

The following sections discuss general aspects of subgrade preparation and embankment construction for the high fill embankment widening.

6.5.1 Subgrade Preparation Requirements - Removal of Organic Materials

Based on the information from the boreholes advanced during the field investigation, the thickness of organic deposits (mainly topsoil) generally ranges between about 100 mm and 200 mm. After clearing and grubbing of the high fill widening areas, particularly in the treed area within 100 m of West Highland Creek, and prior to the placement of any fill for new construction, it is recommended that all surficial and near surface layers of topsoil, organic soils, and any deposits containing deleterious materials within the high fill widening area be stripped from the plan limits of the proposed works regardless of height in accordance with OPSS.PROV 206 (*Grading*).

The exposed subgrade soils in the stripped area should be proof-rolled prior to fill placement in accordance with the applicable specifications. Any unsuitable subgrade materials encountered during proof-roll operations should be sub-excavated and backfilled with approved material and compacted.

6.5.2 Groundwater and Surface Water Control

Excavations within the plan limits of the proposed works will be required to remove topsoil and/or any organic soils/deposits containing deleterious materials prior to embankment widening fill placement, which will generally be maintained above the groundwater table. However, if construction operations are carried out during the wet season or periods of heavy or sustained precipitation, some groundwater flow into deeper sub-excavations (if required) may occur due to the presence of a high groundwater table and relatively permeable granular (silty sand) deposits encountered within the project limits. Unwatering is not required for the sub-excavation and backfilling along the high fill widening area; however, surface water should be directed away from any sub-excavations at all times.

6.5.3 Embankment Construction

Non-cohesive fill is recommended for the construction of the embankment widening, as settlement within non-cohesive embankment fill will essentially occur during placement and compaction, whereas some nominal post-construction settlement of cohesive fill could occur.

Placement of Select Subgrade Material (SSM) as per OPSS.PROV 1010 (*Aggregates*), earth fill meeting the requirements of OPSS.PROV 212 (*Borrow*), or granular fill (satisfying OPSS.PROV 1010 Granular 'B' Type I or Type II requirements) above the water table for construction of the high fill embankment widening (including backfilling operations) should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*). Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials for the widening, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

The SSM, earth fill or granular fill should be compacted in accordance with OPSS.PROV 501 (*Compacting*). Inspection and field testing should be carried out by qualified personnel during construction to confirm that appropriate materials are being utilized and that adequate levels of compaction are being achieved. Side slopes for the embankment should be no steeper than 2 horizontal to 1 vertical (2H:1V). The embankment side slopes should also include a minimum 2 m wide bench at mid-height for all fill heights greater than 8 m as suggested in OPSD 202.010 (*Slope Flattening*).

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.PROV 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.PROV 1004 (Aggregates – Miscellaneous) will be required to reduce the potential for erosion

and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Matt Soderman, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., an MTO Foundations Designated Contact and Principal of Golder, conducted an independent technical and quality control review of the report.

Golder Associates Ltd.



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Geotechnical Engineer



Lisa Coyne, P.Eng.
Principal, MTO Foundations Designated Contact

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ASTM International:

- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils |
|------------|---|

Commercial Software:

- Slide (Version 6.0) by Rocscience Inc.
- Settle^{3D} (Version 4.0) by Rocscience Inc.

Ontario Provisional Standard Drawing:

- | | |
|--------------|--------------------------|
| OPSD 202.010 | Slope Flattening |
| OPSD 208.010 | Benching of Earth Slopes |

Ontario Provincial Standard Specification:

- | | |
|---------------|--|
| OPSS.PROV 206 | Construction Specification for Grading |
| OPSS.PROV 212 | Construction Specification for Earth Borrow |
| OPSS.PROV 501 | Construction Specification for Compacting |
| OPSS.PROV 511 | Construction Specification for Rip Rap, Rock Protection and Granular Sheetting |
| OPSS 802 | Construction Specification for Topsoil |

OPSS.PROV 804 Construction Specification for Seed and Cover

OPSS.PROV 1004 Material Specification for Aggregates – Miscellaneous

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

OPSS.PROV 1860 Material Specification for Aggregates - Geotextiles

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

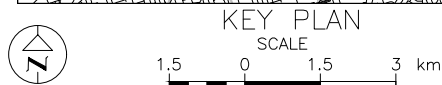
Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects (as amended)





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

HIGH FILL EMBANKMENT
HIGHWAY 401 WESTBOUND CORE AND COLLECTORS
BOREHOLE LOCATIONS



LEGEND

-  Borehole — 2018 Investigation
 Borehole — 1966 Investigation
 (GEOCREs No. 30M14-67)

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
67-3	162.5	4848582.8	322734.5
67-4	162.8	4848574.6	322705.5
CN-02	173.8	4848512.6	322562.4
CN-03	163.5	4848561.9	322546.3
HF-01	162.8	4848576.6	322567.6
HF-02	162.6	4848597.4	322633.0
HF-03	162.2	4848619.0	322709.4

NOTES

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

REFERENCE

Base plan provided in digital format by WSP, drawings files no. H17M-01449-00_X01.dwg, No.H17M-01449-00_X01.dwg and H17M-01449-00_XN01.dwg, received October 26, 2018.

Design Layout provided in digital format by WSP, drawing files no. H17M-01449-00_XN01.dwg, received November 28, 2017 and New Construction.dwg , received June 11, 2018.

General Arrangement provided in digital format by WSP, drawings files no. 17M-01449-00-302-001GA.dwg, received June 5, 2018.

Existing ground provided in digital format by WSP, drawing file no. Contours Sept. 12, 2019.dwg, received September 12, 2018.

-	-	-	-
NO.	DATE	BY	REVISION
Geocres No. 30M14-499			
HWY. 401		PROJECT NO. 1669995	DIST. .
SUBM'D. MS	CHKD. MS	DATE: 01/25/2019	SITE:
DRAWN: DD	CHKD. LCC	APPD. LCC	DWG. 1

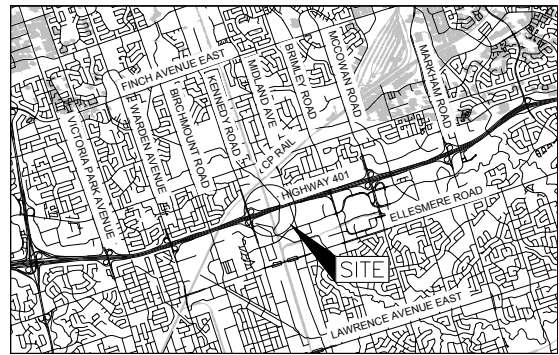
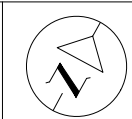


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. _____
WP No. 2219-14-00

HIGH FILL EMBANKMENT
HIGHWAY 401 WESTBOUND CORE AND COLLECTORS

SOIL STRATA



KEY PLAN
SCALE
1.5 0 1.5 3 km

LEGEND

- Borehole - 2018 Investigation
- Borehole - 1966 Investigation (GEOCRETS No. 30M14-67)
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ▽ WL in piezometer
- ▽ WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
67-3	162.5	4848582.8	322734.5
67-4	162.8	4848574.6	322705.5
CN-02	173.8	4848512.6	322562.4
CN-03	163.5	4848561.9	322546.3
HF-01	162.8	4848576.6	322567.6
HF-02	162.6	4848597.4	322633.0
HF-03	162.2	4848619.0	322709.4

NOTES

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

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

REFERENCE

Base plan provided in digital format by WSP, drawings files no. H17M-01449-00_XA01.dwg, No.H17M-01449-00_XB01.dwg and H17M-01449-00_XY01.dwg, received October 26, 2017.
Design Layout provided in digital format by WSP, drawing files no. H17M-01449-00_XN01.dwg, received November 28, 2017 and New Construction.dwg, received June 11, 2018.
Existing ground provided in digital format by WSP, drawing file no. Contours Sept. 12, 2019.dwg, received September 12, 2018.

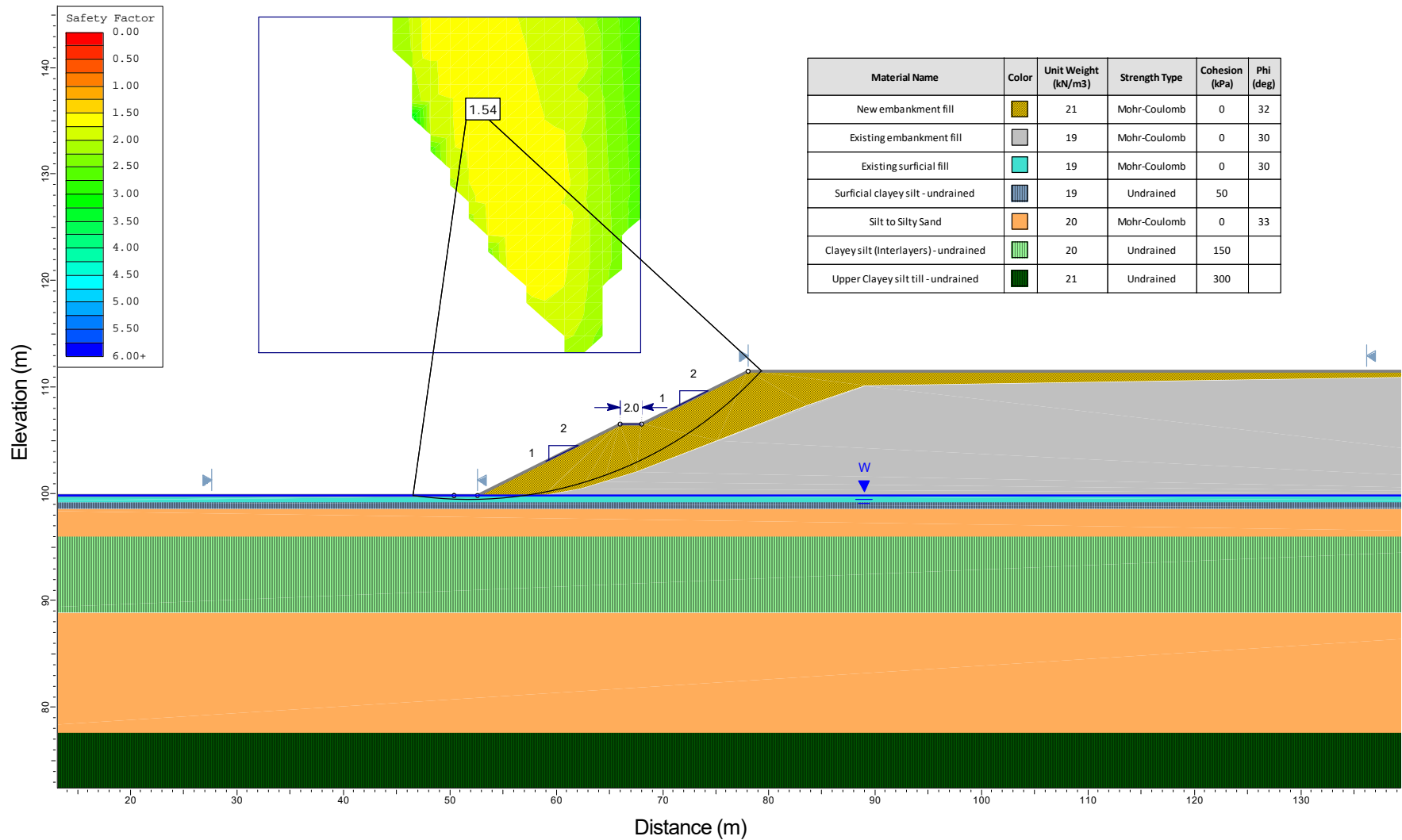
NO.	DATE	BY	REVISION
Geocres No. 30M14-499			
HWY. 401	PROJECT No. 1669995		DIST. .
SUBM'D. MS	CHKD. MS	DATE: 01/25/2019	SITE: .
DRAWN: DD	CHKD. LCC	APPD. LCC	DWG. 2



A-A PROFILE
1

VERTICAL SCALE
5 0 5 10 m

HORIZONTAL SCALE
10 0 10 20 m



CLIENT
Ministry of Transportation Ontario (MTO)

CONSULTANT

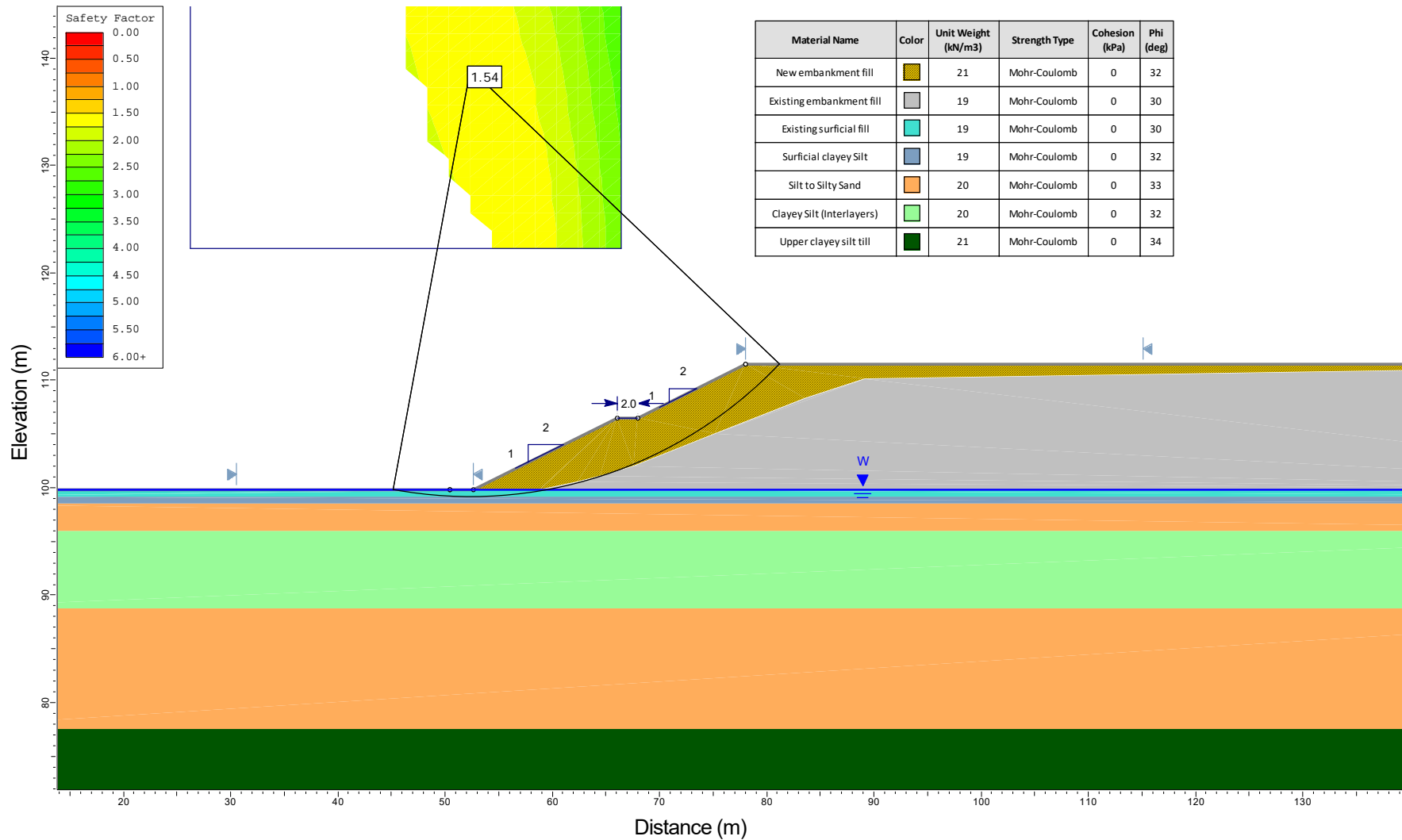


YYYY-MM-DD 2019-02-15
PREPARED MAS
DESIGN MAS
REVIEW LCC
APPROVED LCC

PROJECT
HIGHWAY 401 WESTBOUND CORE AND COLLECTOR LANES
NEILSON ROAD TO WARDEN AVENUE, CITY OF TORONTO
G.W.P. NO. 2162-11-00

TITLE
HIGH FILL AREA
STATIC GLOBAL STABILITY ANALYSIS
SHORT-TERM CONDITIONS (UNDRAINED)

PROJECT No.
1669995



CLIENT
Ministry of Transportation Ontario (MTO)

CONSULTANT



YYYY-MM-DD 2019-02-15
PREPARED MAS
DESIGN MAS
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APPROVED LCC

PROJECT
HIGHWAY 401 WESTBOUND CORE AND COLLECTOR LANES
NEILSON ROAD TO WARDEN AVENUE, CITY OF TORONTO
G.W.P. NO. 2162-11-00

TITLE
HIGH FILL AREA
STATIC GLOBAL STABILITY ANALYSIS
LONG-TERM CONDITIONS (DRAINED)

PROJECT No.
1669995

APPENDIX A

**Borehole Records from
1966 Investigation
(GEOCRES No. 30M14-67)**

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

BH 67-3

FOUNDATION SECTION

JOB 66-P-34

LOCATION Hwy. 401 & Highland Cr. Sta. 364/32, 116' Lt.

ORIGINATED BY V.K.

W.P. _____


BORING DATE April 18, 1966

COMPILED BY V.K.

DATUM Geodetic

BOREHOLE TYPE Drive PX Casing and Wash

CHECKED BY HL

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT % 15 30 45	
(w)																		
162.5	533.0	W.L. Ground Level																
159.7	524.0		1	SS	20	530												
2.7	9.0		2	SS	100/6"													
			3	SS	100/6"													
157.4	516.5		4	SS	27	520												
5.0	16.5		5	SS	25													
			6	SS	75/6"	510												
			7	SS	44													
152.9	501.5		8	SS	110													
9.6	31.5	End of Borehole				500												

Gr-4 Sa-35
Si-43 Cl-18

Gr-3 Sa-43
Si-44 Cl-10

Gr-1 Sa-15
Si-81 Cl-3

BH 67-4

FOUNDATION SECTION

ORIGINATED BY V.K.

COMPILED BY V.K.

CHECKED BY AK

SOIL PROFILE				SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W			BULK DENSITY P.C.F.	REMARKS
(W)	ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT 20 40 60 80 100					WATER CONTENT % 15 30 45				
								SHEAR STRENGTH P.S.F.					wp w WL				
162.8	534.0	W.L. Ground Level															
		Compact to very Dense Silty Sand to Sandy Silt with traces of Gravel and Clay		1	SS	22	530										Gr-6 Sa-41 Si-50 Cl-3
160.0	525.0			2	SS	114											
2.7	9.0	Clayey Silt. Sand and Gravel (V. Stiff to Hard)		3	SS	57	520										Gr-3 Sa-35 Si-51 Cl-11
				4	SS	28											
157.7	517.5			5	SS	22	510										Gr-8 Sa-35 Si-45 Cl-12
5.0	16.5			6	SS	44											
154.8	508.0						500										Gr-31 Sa-30 Si-25 Cl-8
7.9	26.0	Clayey Silt. Sand, Gravel		7	SS	46											
		Silty Sand with traces of gravel (Compact to V. Dense)		8	SS	125	490										Gr-0 Sa-87 Si-Cl-13
				9	SS	60											
				10	SS	102											
				11	SS	114											
147.1	482.5			12	SS	106	480										
15.7	51.5	End of Borehole															

APPENDIX B

**Borehole Records from
2018 Investigation**

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

Compactness	N
Condition	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

PROJECT		1669995		RECORD OF BOREHOLE No CN-02		SHEET 1 OF 4		METRIC								
G.W.P.		2162-11-00		LOCATION		N 4848512.6; E 322562.4 MTM NAD 83 ZONE 10 (LAT. 43.776370; LONG. -79.279349)		ORIGINATED BY								
DIST		Central HWY 401		BOREHOLE TYPE		CME 75 Truck-Mounted Drill Rig, 165 mm O.D. Hollow Stem Augers		COMPILED BY								
DATUM		Geodetic		DATE		March 14 to 16, 2018		CHECKED BY								
								NK								
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								WATER CONTENT (%)
173.8	GROUND SURFACE							20	40	60	80	100				
0.0	ASPHALT (203 mm)							20	40	60	80	100				
0.2	Gravelly sand, some silt, trace clay (FILL) Compact to very dense Brown Moist		1	SS	18		173									
			2	SS	36											
171.9			3A	SS	86		172									
1.9	Silt and sand, some clay to clayey silt with sand, trace gravel (FILL) Stiff to hard Grey-brown Moist		3B													
			4	SS	16		171									
			5	SS	25											
			6	SS	20		170									
			7	SS	16		169									
			8	SS	26		168									
	- Oxidation stains between depths of approximately 6.1 m and 8.2 m		9	SS	39											
							167									
			10	SS	14		166									
							165									
164.3	TOPSOIL		11	SS	28		164									
9.8	Silty SAND, trace clay, trace gravel Very dense Grey-brown Wet						163									
			12	SS	56											
							162									
			13	SS	22		161									
160.5							160									
13.3	Sandy SILT to SILT and SAND, trace to some gravel, trace to some clay Compact to very dense Grey Moist		14	SS	47											
							159									

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

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PROJECT 1669995		RECORD OF BOREHOLE No CN-02		SHEET 2 OF 4		METRIC												
G.W.P. 2162-11-00		LOCATION N 4848512.6; E 322562.4 MTM NAD 83 ZONE 10 (LAT. 43.776370; LONG. -79.279349)		ORIGINATED BY AB														
DIST Central HWY 401		BOREHOLE TYPE CME 75 Truck-Mounted Drill Rig, 165 mm O.D. Hollow Stem Augers		COMPILED BY KAW														
DATUM Geodetic		DATE March 14 to 16, 2018		CHECKED BY NK														
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)					
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 10 20 30			GR SA SI CL		
--- CONTINUED FROM PREVIOUS PAGE ---																		
	Sandy SILT to SILT and SAND, trace to some gravel, trace to some clay Compact to very dense Grey Moist		15	SS	22		158											
	- Grinding on inferred cobble between depths of approximately 16.2 m and 16.5 m						157											
							156											
	- Sampler bouncing on inferred cobble at depth of approximately 18.3 m, no sample recovery		16	SS	100/ 0.08		155											
							154											
			17	SS	140		153											
152.9	CLAYEY SILT, some sand to with SAND, trace to some gravel (TILL) Hard Grey Moist		18	SS	86		152											
20.9							151											
							150											
			19	SS	35		149											
							148											
147.6	Gravelly SAND, some silt, trace clay Dense Grey Wet		20	SS	49		147											
26.2							146											
	- Slight organic odour at depth of approximately 28.0 m						145											
144.5	CLAYEY SILT, trace gravel, trace to some sand						144											
29.3																		

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

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

PROJECT <u>1669995</u>		RECORD OF BOREHOLE No CN-02				SHEET 4 OF 4		METRIC	
G.W.P. <u>2162-11-00</u>		LOCATION <u>N 4848512.6; E 322562.4 MTM NAD 83 ZONE 10 (LAT. 43.776370; LONG. -79.279349)</u>				ORIGINATED BY <u>AB</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>CME 75 Truck-Mounted Drill Rig, 165 mm O.D. Hollow Stem Augers</u>				COMPILED BY <u>KAW</u>			
DATUM <u>Geodetic</u>		DATE <u>March 14 to 16, 2018</u>				CHECKED BY <u>NK</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p — W — W _L 10 20 30					
	--- CONTINUED FROM PREVIOUS PAGE ---															
	CLAYEY SILT with SAND, trace gravel (TILL) Very stiff to hard Grey Moist		27	SS	24											
			28	SS	24											
			29	SS	22											
122.9 50.9	END OF BOREHOLE															
	NOTES: 1. No water level reading taken upon completion of drilling due to the addition of water/drill mud.															

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PROJECT		1669995		RECORD OF BOREHOLE No CN-03				SHEET 1 OF 1		METRIC							
G.W.P.		2162-11-00		LOCATION		N 4848561.9; E 322546.3 MTM NAD 83 ZONE 10 (LAT. 43.776814; LONG. -79.279548)		ORIGINATED BY		KN							
DIST		Central HWY 401		BOREHOLE TYPE		203 mm O.D. Hollow Stem Augers; CME 55 Track-Mounted Drill Rig		COMPILED BY		EN							
DATUM		Geodetic		DATE		November 1, 2018		CHECKED BY		NK							
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									
163.5	GROUND SURFACE																
0.7	TOPSOIL (70 mm)																
162.8	Sandy silt, some rootlets, trace gravel, trace clay (FILL)		1	SS	5												
0.7	Loose Brown Moist		2	SS	5												
162.1	SILT, some sand, trace rootlets, trace gravel																
1.5	Loose Grey mottled red and brown Moist		3	SS	28												
	SILT and SAND, trace to some gravel, trace clay, sand pockets																
	Compact to dense Brown to grey at 3.0 m		4	SS	46												6 43 46 5
	Moist to wet below 5.5 m		5	SS	45												
			6	SS	17												4 45 41 10
			7	SS	13												
			8	SS	18												1 47 41 11
156.8	END OF BOREHOLE																
6.7	NOTES:																
	1. Open borehole dry on completion of drilling.																
	2. Water level measured in piezometer as follows:																
	Date Depth (m) Elev. (m)																
	11/27/18 0.3 163.2																

PROJECT		1669995		RECORD OF BOREHOLE No HF-01				SHEET 1 OF 1		METRIC			
G.W.P.		2219-14-00		LOCATION		N 4848576.6; E 322567.6 MTM NAD 83 ZONE 10 (LAT. 43.776945; LONG. -79.279282)		ORIGINATED BY		KN			
DIST		Central HWY 401		BOREHOLE TYPE		CME 55 Truck-Mounted Drill Rig, 152 mm O.D. Hollow Stem Augers		COMPILED BY		MAS			
DATUM		Geodetic		DATE		November 1, 2018		CHECKED BY		LCC			
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)			
162.8	GROUND SURFACE												
0.0	CLAYEY SILT with SAND, trace to some gravel, containing trace to some rootlets		1	SS	4								
162.1	Firm												
0.7	Brown Wet		2	SS	25								5 43 46 6
	SILT and SAND, trace gravel, trace to some clay												
	Compact to dense		3	SS	44								
	Brown Moist												
160.3			4A	SS	41								
2.5	CLAYEY SILT with SAND, trace gravel		4B	SS									
	Very stiff to hard												
	Grey		5	SS	66								
	Moist becoming wet below 6.1 m		6	SS	47								
			7	SS	17								4 40 42 14
			8	SS	39								
			9	SS	42								
155.6	SILT and SAND, trace gravel and clay												
7.2	Dense to very dense												
	Grey		10	SS	54								3 57 36 4
	Wet												
			11	SS	60								
151.5													
11.3	END OF BOREHOLE												
	NOTES:												
	1. Water level in open borehole at a depth of 6.6 m on completion of drilling.												

PROJECT		1669995		RECORD OF BOREHOLE No HF-02		SHEET 1 OF 1		METRIC								
G.W.P.		2219-14-00		LOCATION		N 4848597.4; E 322633.0 MTM NAD 83 ZONE 10 (LAT. 43.777131; LONG. -79.278469)		ORIGINATED BY								
DIST		Central HWY 401		BOREHOLE TYPE		CME 55 Truck-Mounted Drill Rig, 152 mm O.D. Hollow Stem Augers		COMPILED BY								
DATUM		Geodetic		DATE		November 2, 2018		CHECKED BY								
								LCC								
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								
162.6	GROUND SURFACE															
0.0	TOPSOIL															
0.2	SILT and SAND, trace to some gravel and clay Compact to dense Brown Moist		1	SS	20											
			2	SS	31											
			3	SS	42											
160.4	CLAYEY SILT with SAND, trace gravel Hard to very stiff Grey Moist		4	SS	32											
2.2			5	SS	30											
			6	SS	38											
			7	SS	24											
			8	SS	13											
154.7	Silty SAND, trace clay Compact Grey Wet		9	SS	22											
7.9			10	SS	14											
152.4	SILT, trace to some sand, trace gravel and clay Very dense Grey Wet		11	SS	68											
10.2																
151.4	END OF BOREHOLE															
11.2	NOTES: 1. Water level in open borehole at 6.8 m below ground surface on completion of drilling.															

PROJECT 1669995		RECORD OF BOREHOLE No HF-03				SHEET 1 OF 1		METRIC							
G.W.P. 2219-14-00		LOCATION N 4848619.0; E 322709.4 MTM NAD 83 ZONE 10 (LAT. 43.777323; LONG. -79.277519)				ORIGINATED BY LK									
DIST Central HWY 401		BOREHOLE TYPE CME 55 Truck-Mounted Drill Rig, 152 mm O.D. Hollow Stem Augers				COMPILED BY MAS									
DATUM Geodetic		DATE November 5, 2018				CHECKED BY LCC									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED							
162.2	GROUND SURFACE					▽	162								
0.0	TOPSOIL		1A	SS	8										
0.2	CLAYEY SILT with SAND, trace gravel		1B												
161.5	Stiff		2A	SS	51										
0.7	Mottled brown Moist		2B												
	SILT, some sand, to SILT and SAND, trace to some gravel and clay		3	SS	100/0.14			161							0 14 80 6
	Very dense		4	SS	62/0.08										
	Grey		5	SS	65/0.10			160							2 44 46 8
	Moist		6	SS	77/0.15			159							
			7	SS	60/0.08										
			8	SS	156		158							10 35 45 10	
			9	SS	60/0.08		156								
155.2	GRAVELLY SAND, trace to some silt		10	SS	100/0.14		155								
7.0	Very dense														
154.5	Grey Wet														
7.7	END OF BOREHOLE														
	NOTE: 1. Water level estimated to be 0.7 m below ground surface based on observation of wet split spoon sampler.														

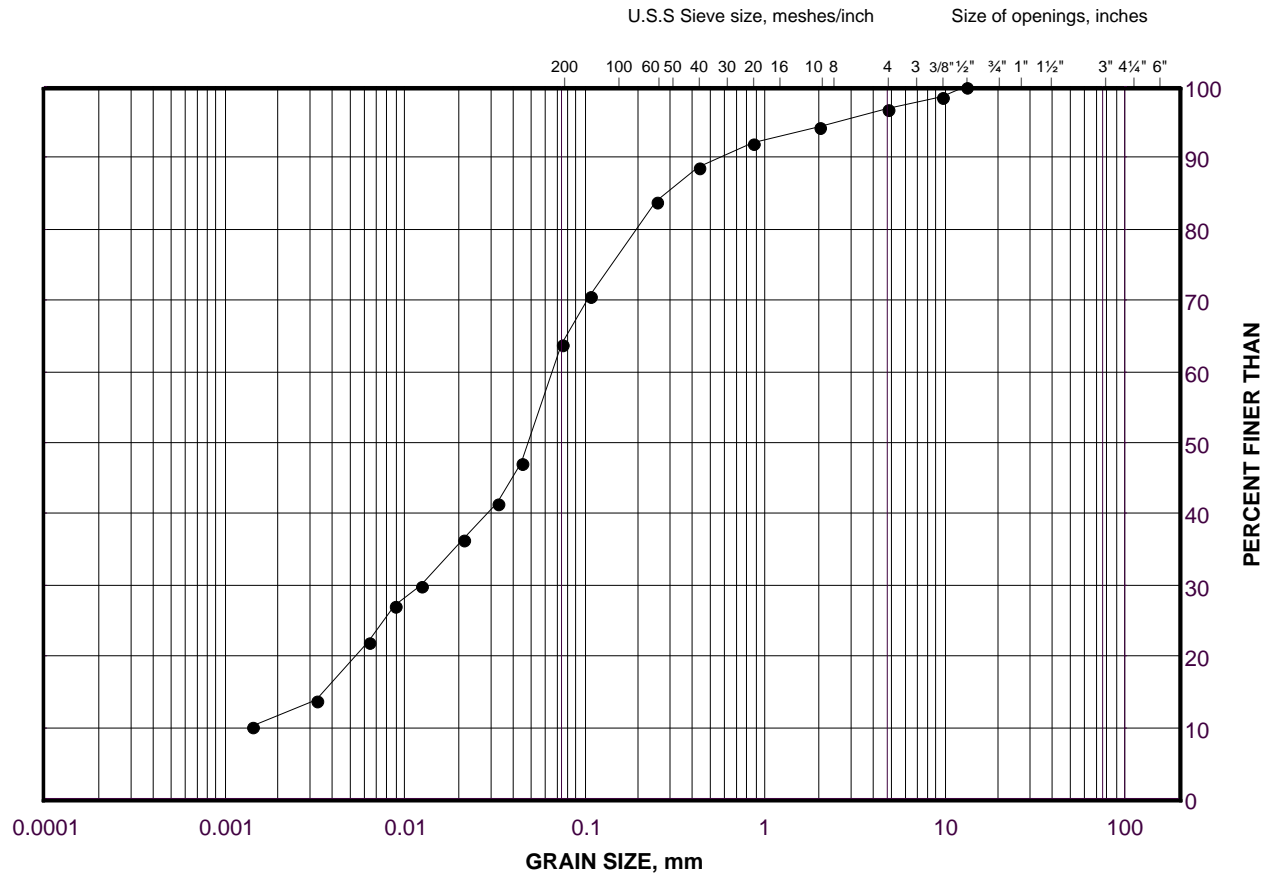
APPENDIX C

**Geotechnical Laboratory
Test Results**

GRAIN SIZE DISTRIBUTION

Clayey Silt Fill

FIGURE C-1



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

LEGEND

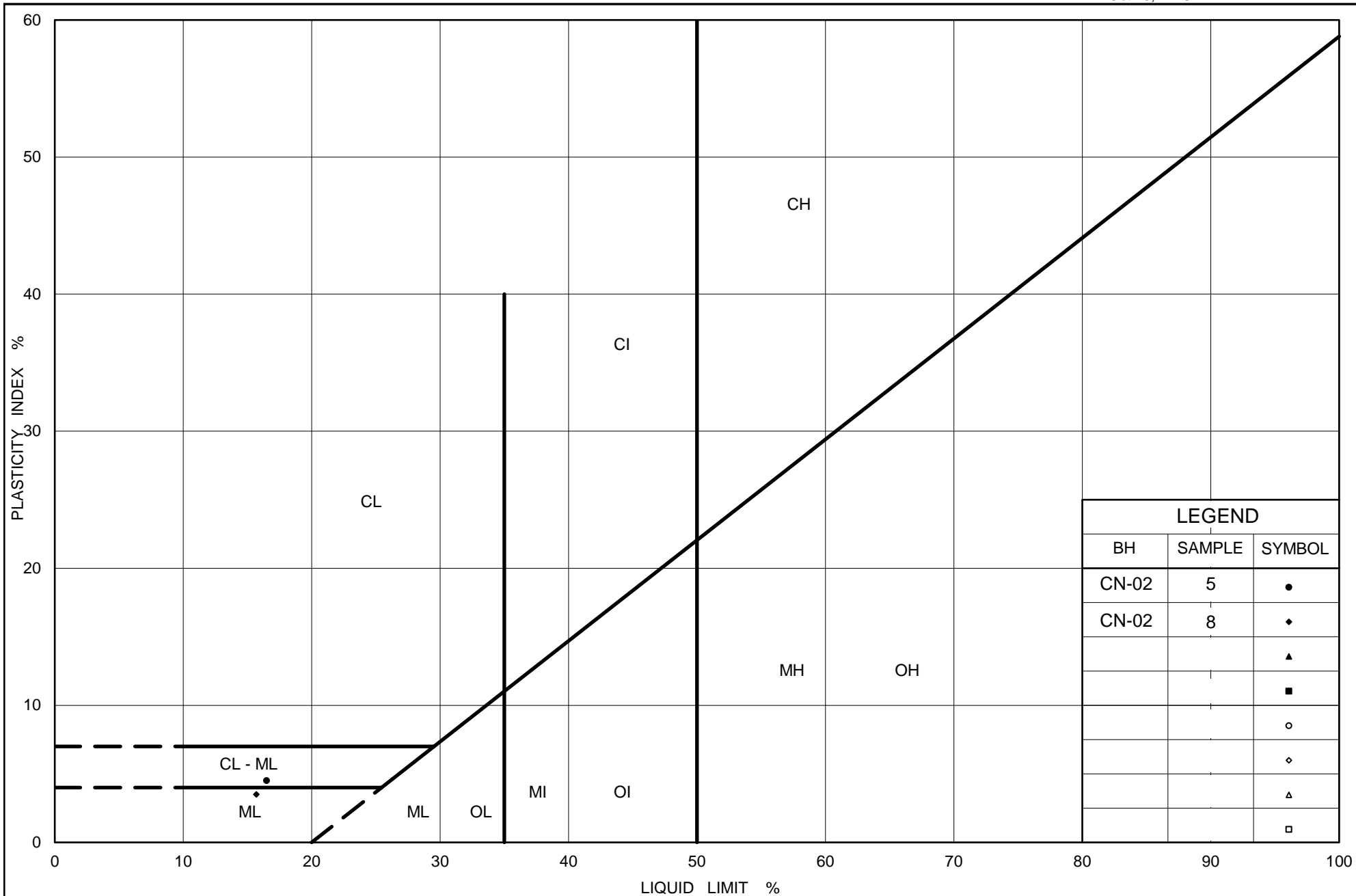
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	CN-02	5	170.4

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Golder Associates

Date: 10-Jan-19



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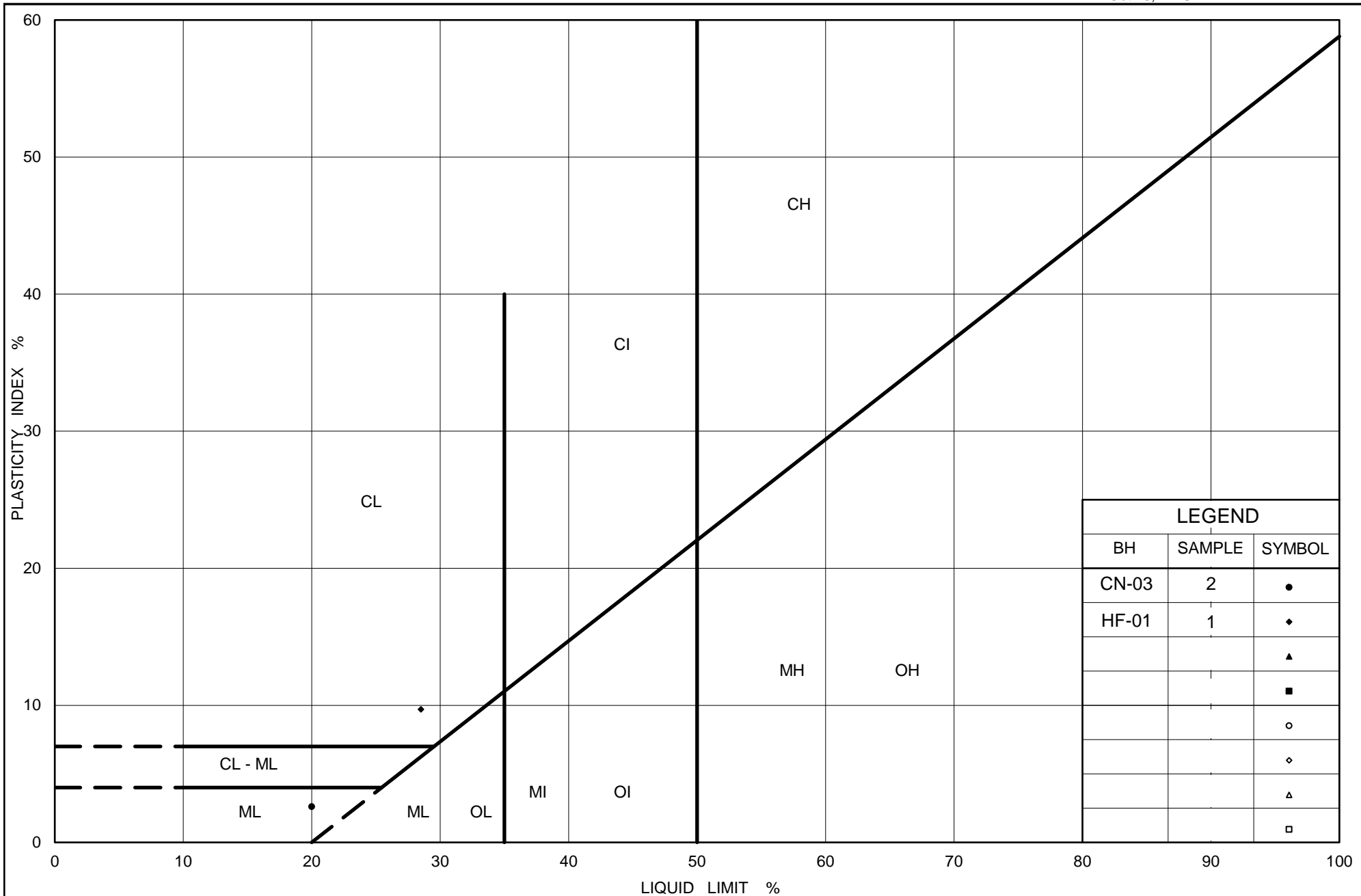
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PLASTICITY CHART Clayey Silt to Silt and Sand Fill

Figure No. C-2

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PLASTICITY CHART Surficial Silt to Clayey Silt

Figure No. C-3

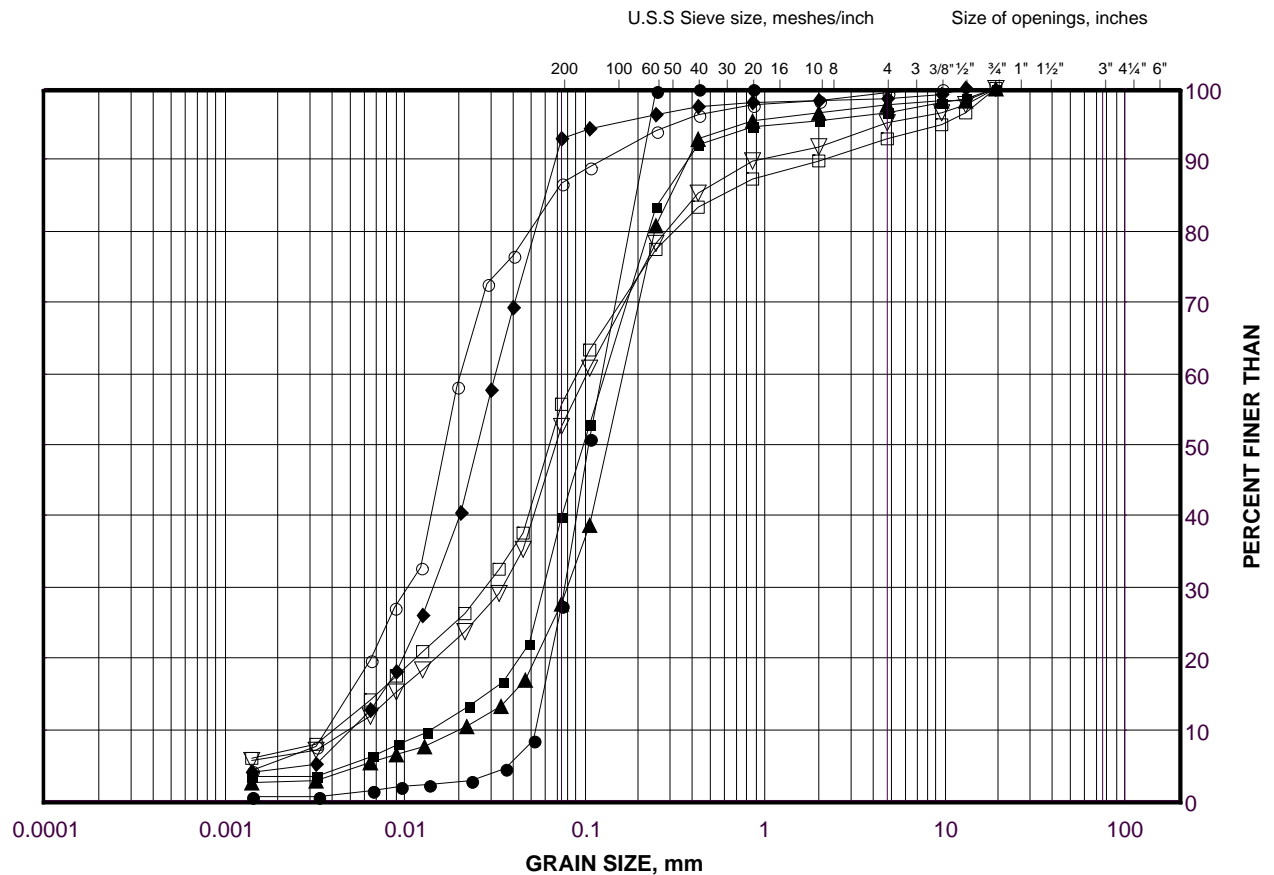
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GRAIN SIZE DISTRIBUTION

Silt to Silt and Sand to Silty Sand

FIGURE C-4A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-02	10	153.2
■	HF-01	10	153.4
◆	HF-02	11	151.6
▲	CN-02	12	162.8
▽	HF-01	2	161.7
○	HF-03	2B	161.1
□	HF-02	3	160.8

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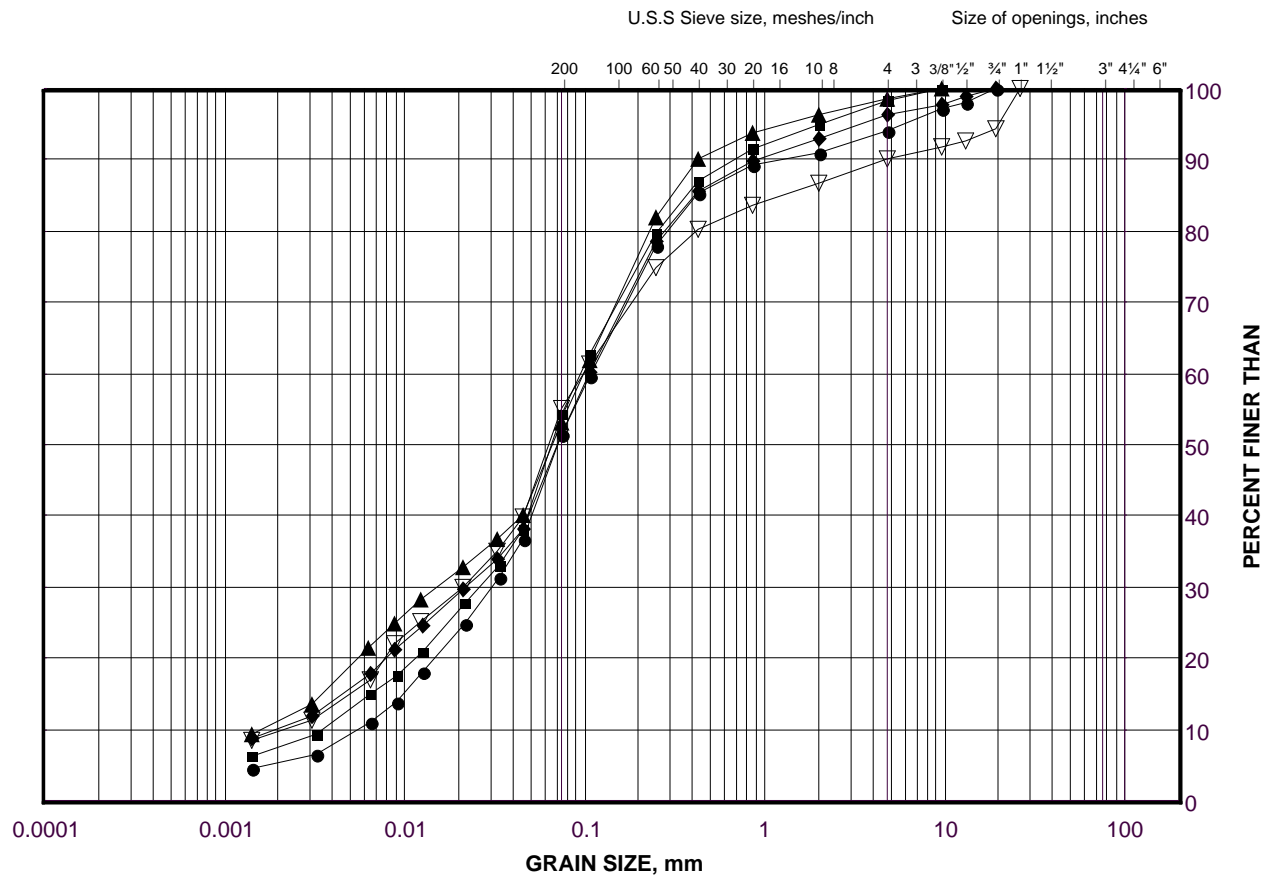
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Silt to Silt and Sand to Silty Sand

FIGURE C-4B



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

LEGEND

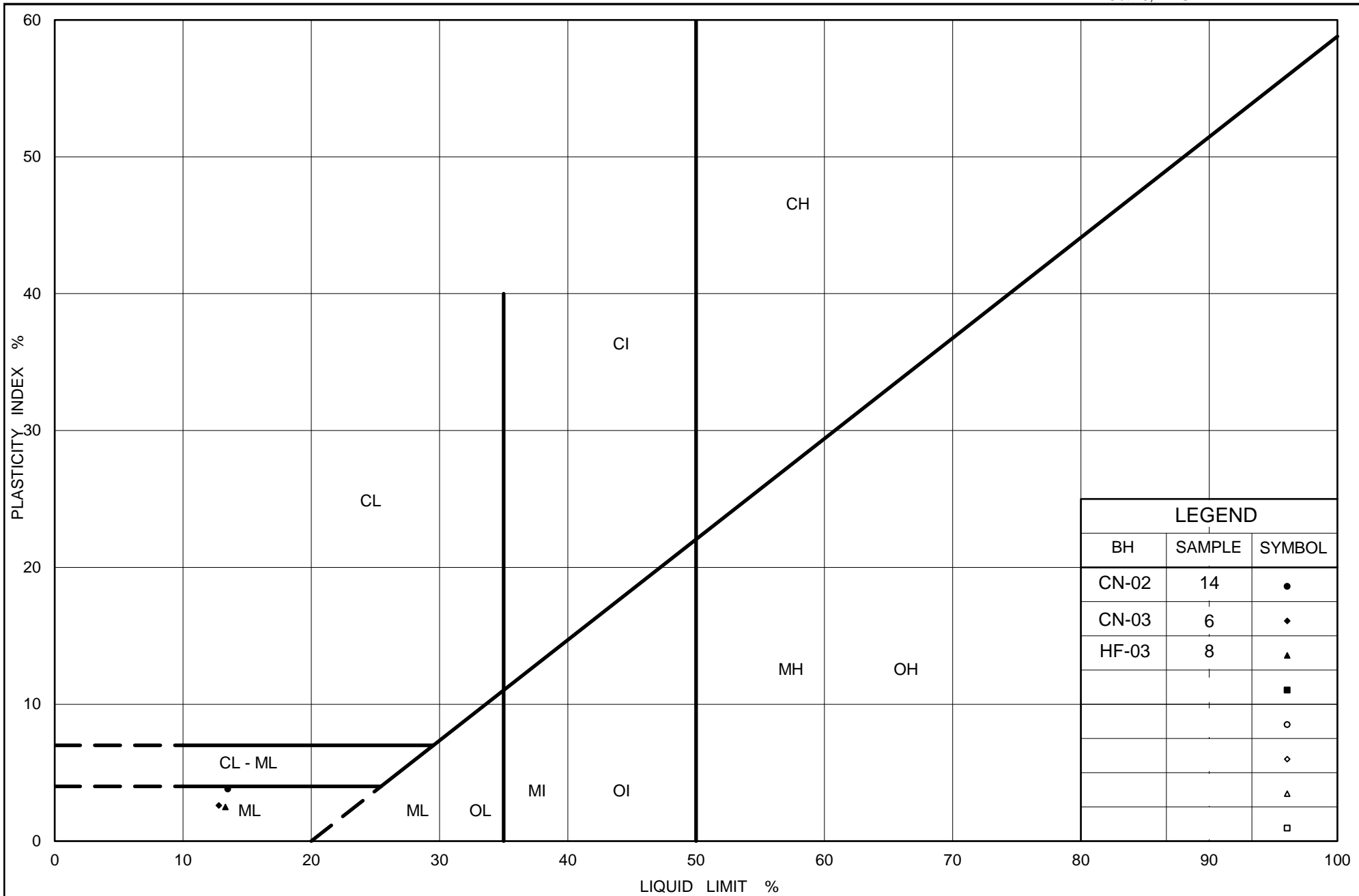
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CN-03	4	160.9
■	HF-03	4	160.3
◆	CN-03	6	159.4
▲	CN-03	8	157.1
▽	HF-03	8	157.6

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PLASTICITY CHART Silt to Silty Sand

Figure No. C-5

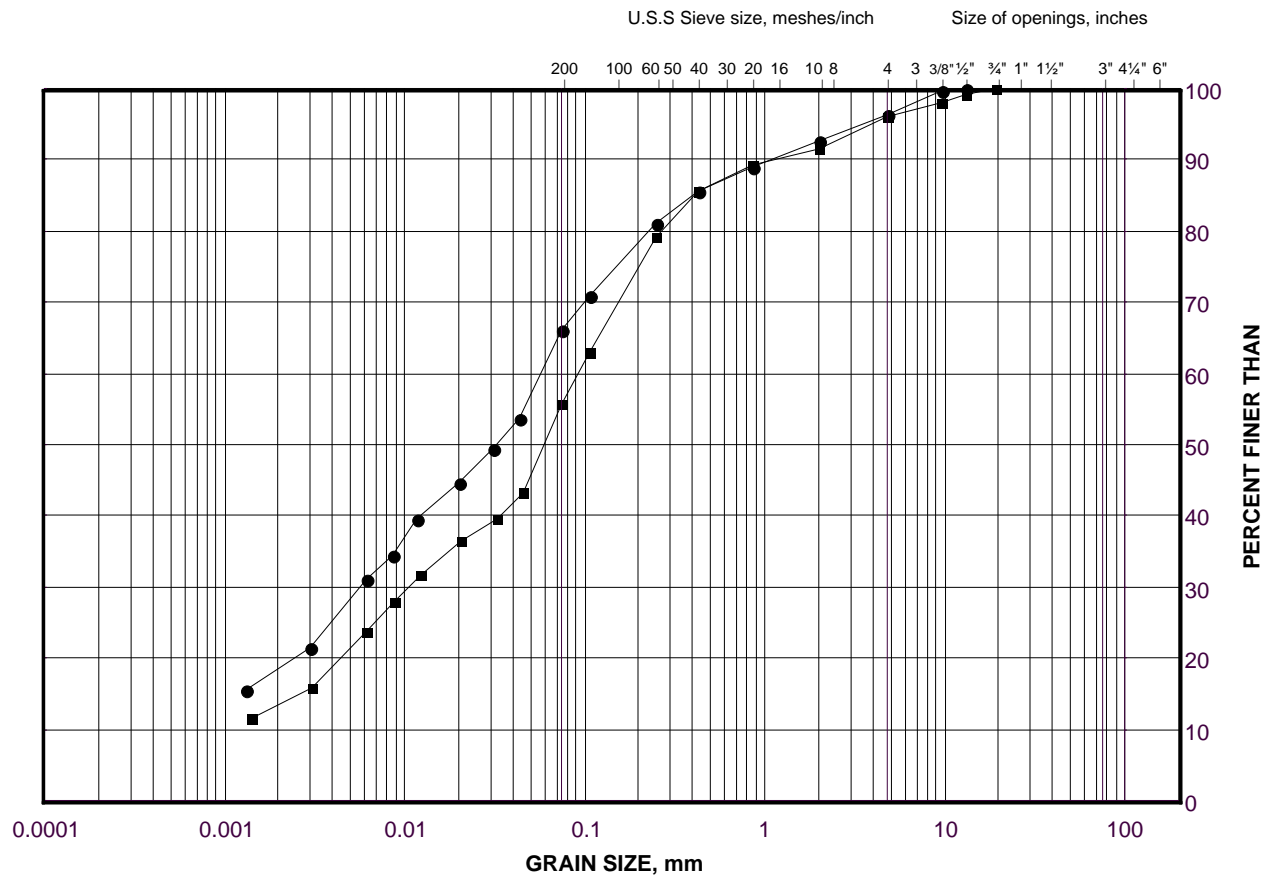
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GRAIN SIZE DISTRIBUTION

Clayey Silt Interlayers

FIGURE C-6



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

LEGEND

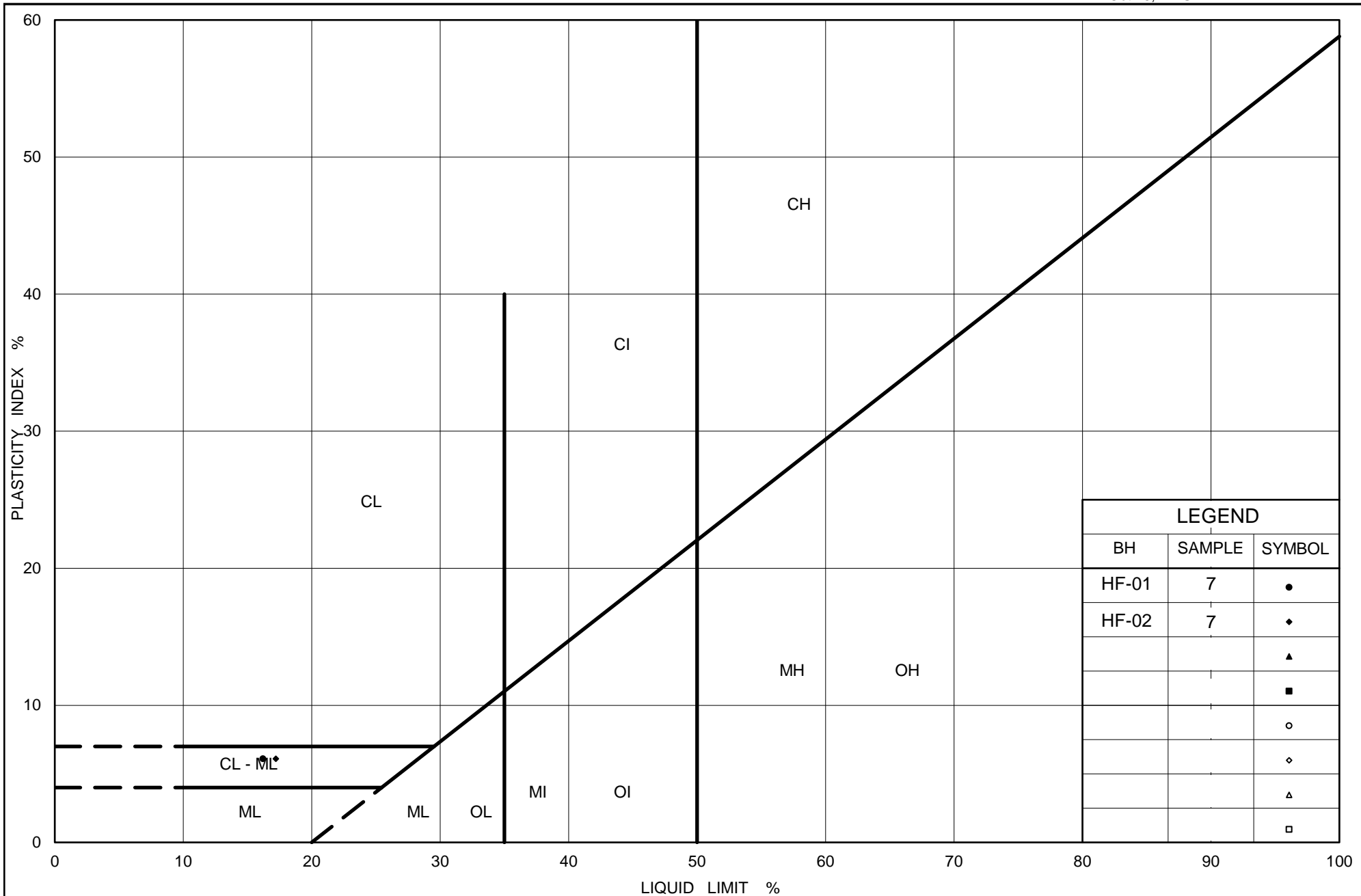
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	HF-02	7	157.7
■	HF-01	7	157.9

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PLASTICITY CHART

Clayey Silt Interlayers

Figure No. C-7

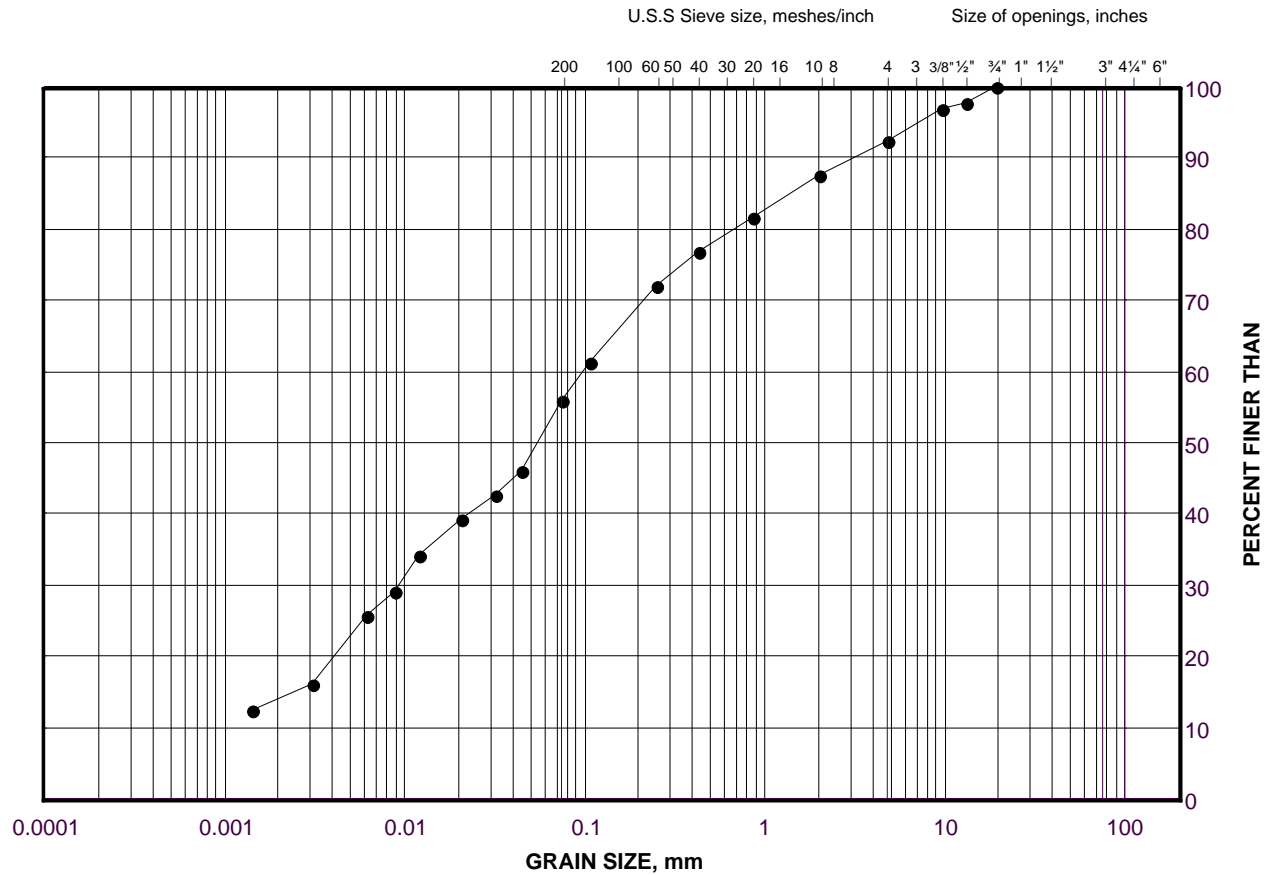
Project No. 1669995

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GRAIN SIZE DISTRIBUTION

Upper Clayey Silt Till

FIGURE C-8



LEGEND

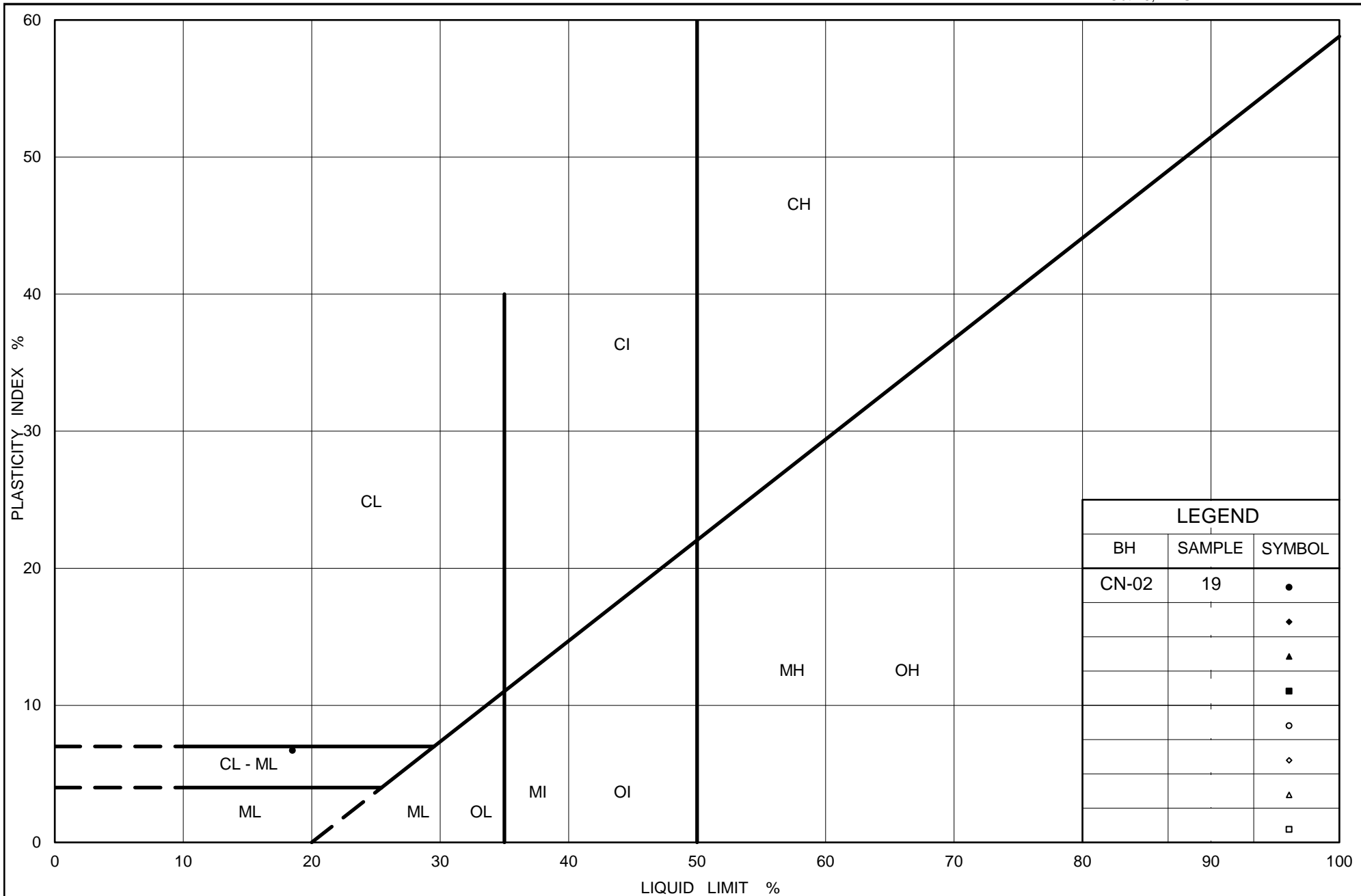
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	CN-02	19	149.1

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PLASTICITY CHART Upper Clayey Silt Till

Figure No. C-9

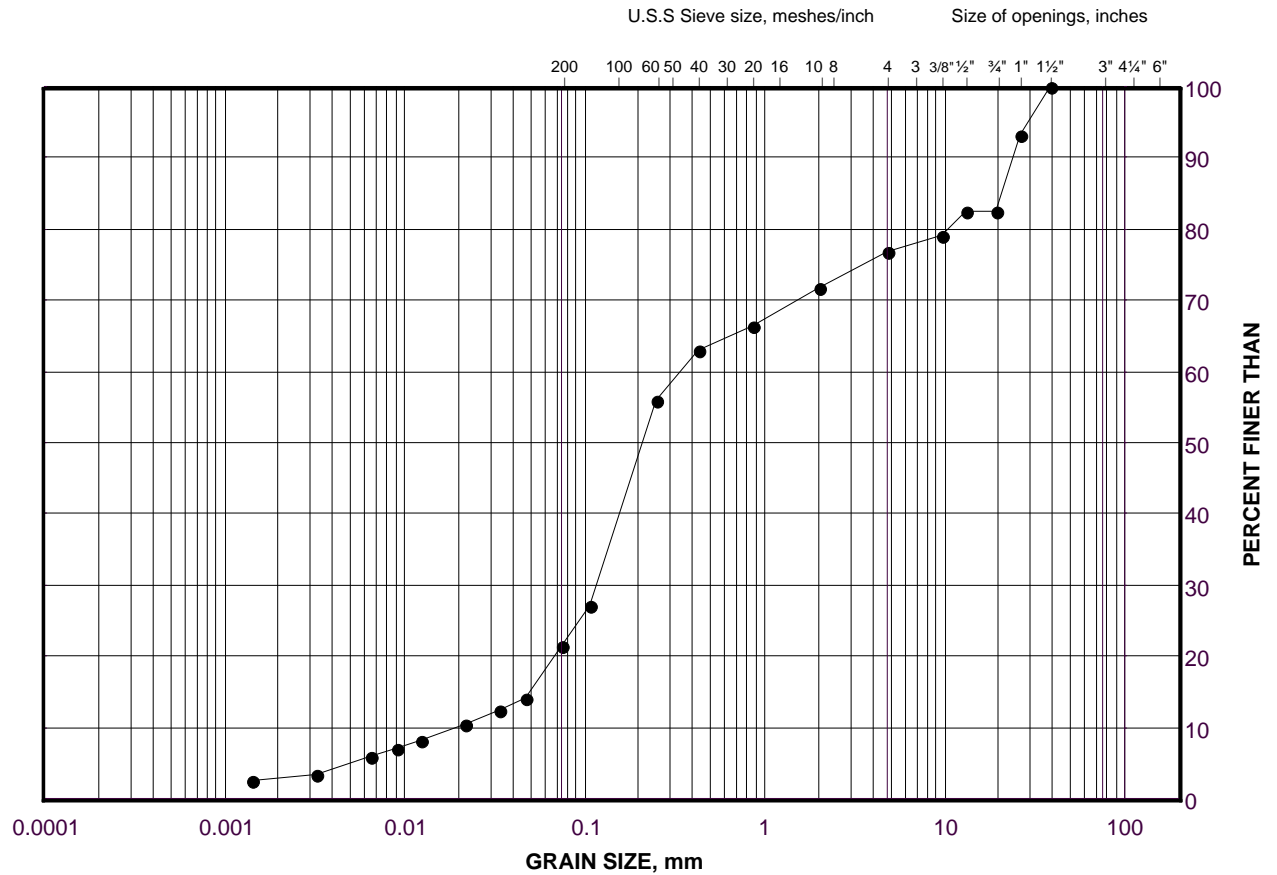
Project No. 1669995

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GRAIN SIZE DISTRIBUTION

Gravelly Sand

FIGURE C-10



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	CN-02	20	146.1

Project Number: 1669995

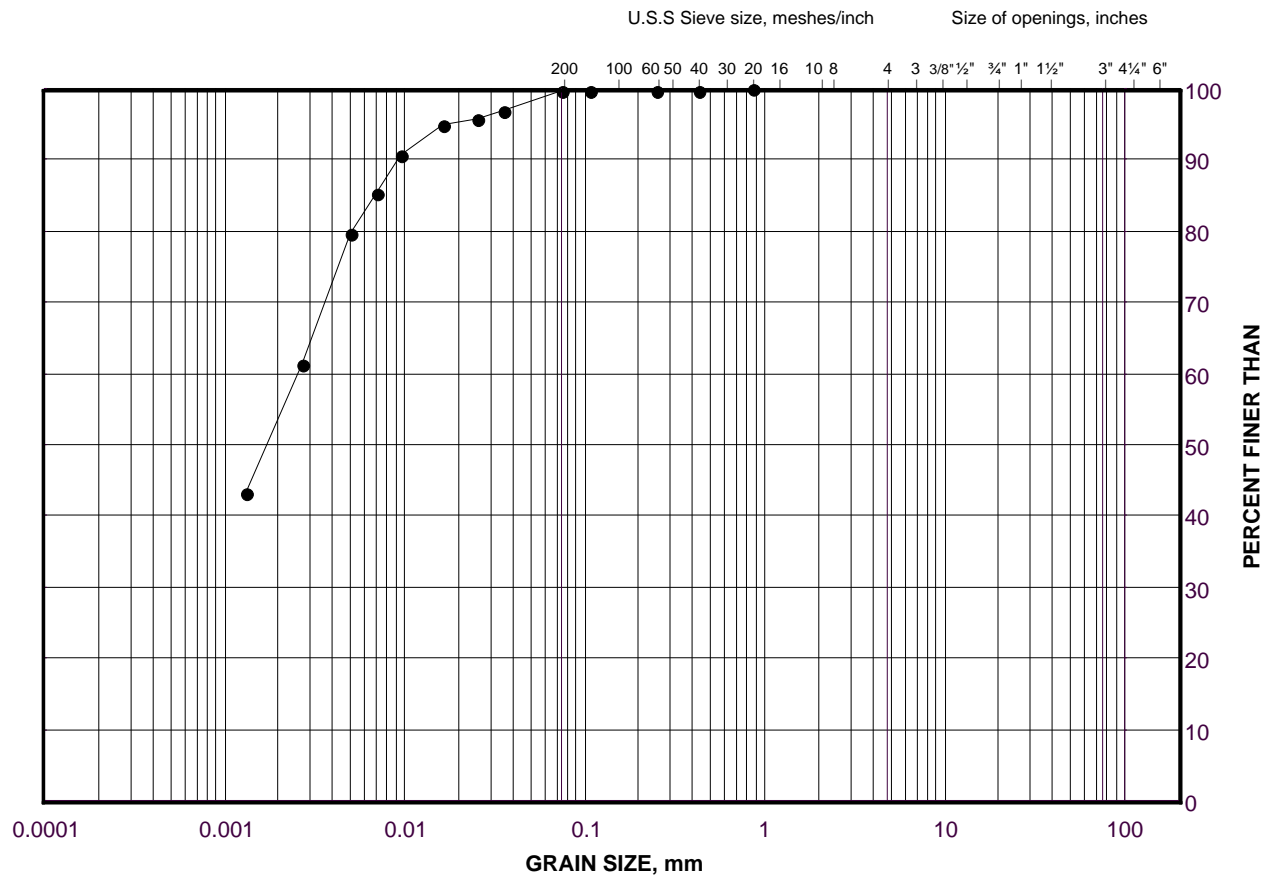
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Lower Clayey Silt

FIGURE C-11



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

LEGEND

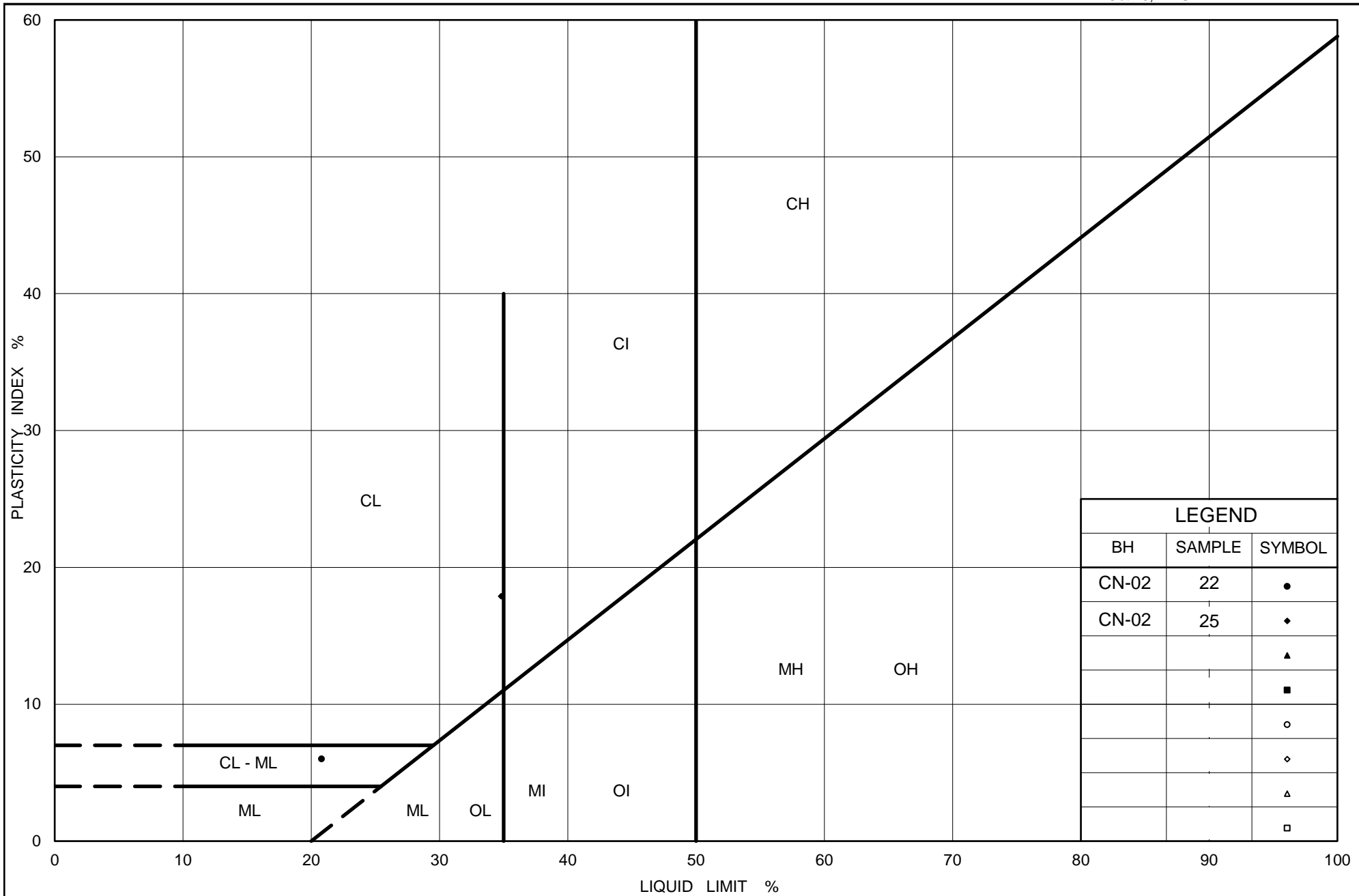
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	CN-02	25	133.9

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PLASTICITY CHART

Lower Clayey Silt

Figure No. C-12

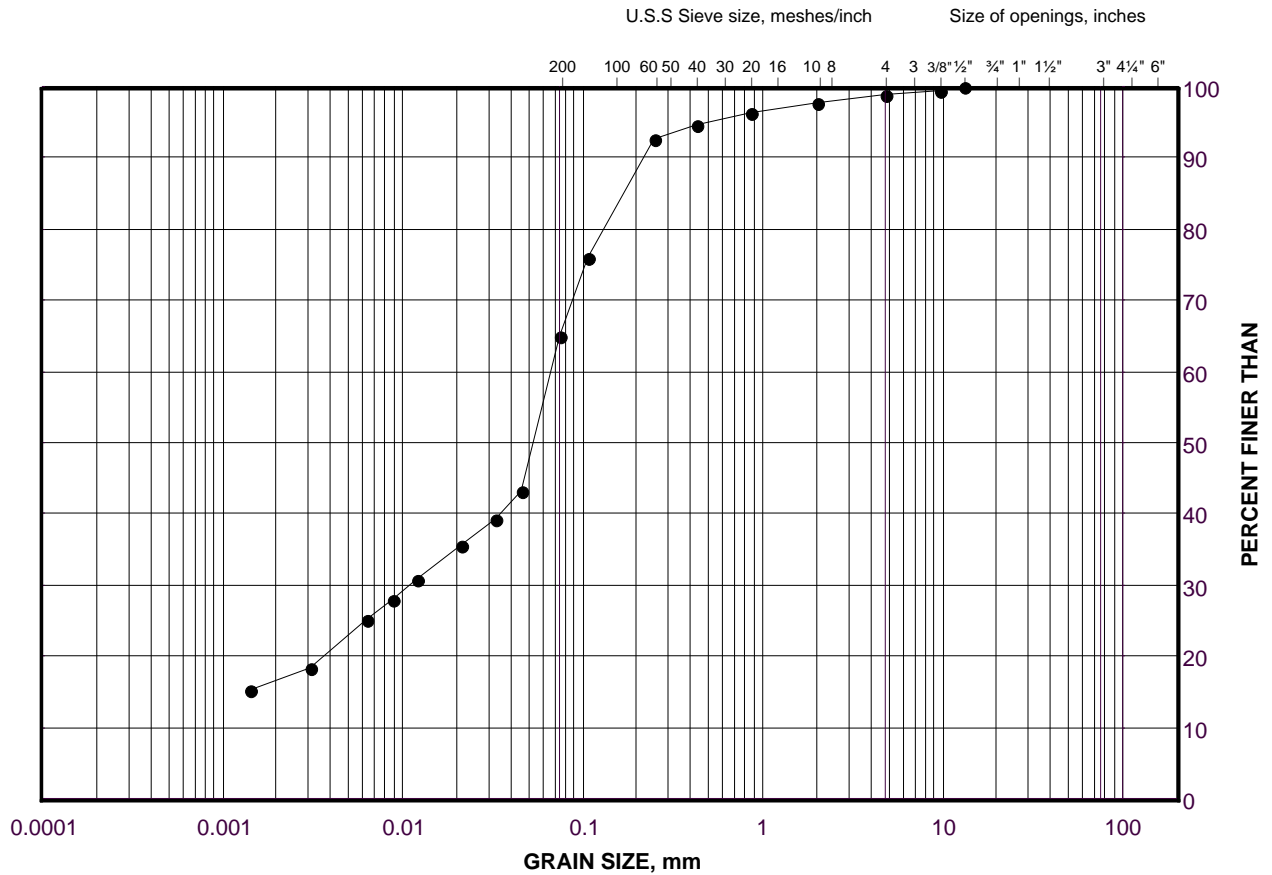
Project No. 1669995

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GRAIN SIZE DISTRIBUTION

Lower Clayey Silt Till

FIGURE C-13



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

LEGEND

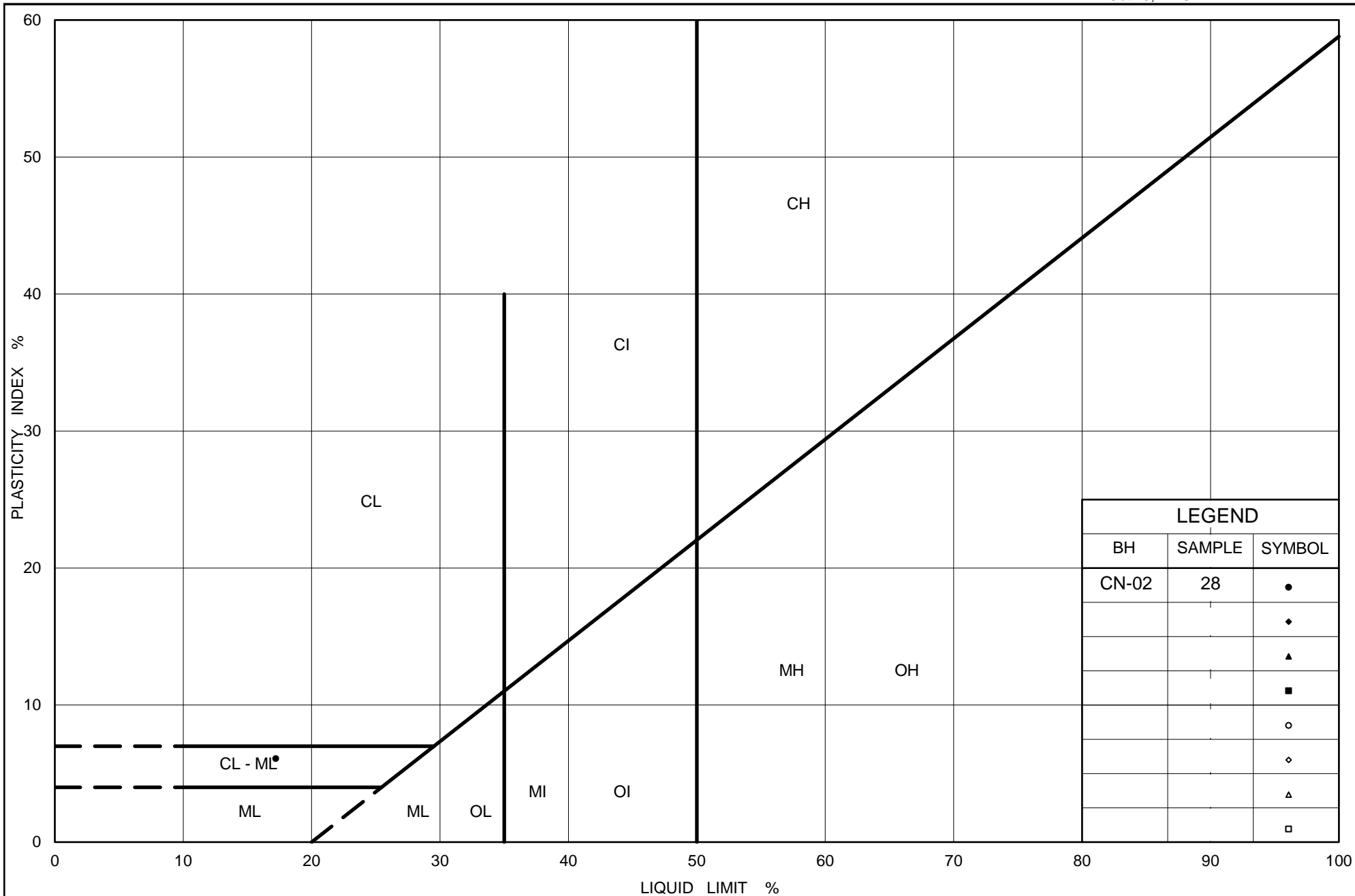
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	CN-02	28	124.7

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PLASTICITY CHART Lower Clayey Silt Till

Figure No. C-14

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APPENDIX D

Operational Constraints

OPERATIONAL CONSTRAINT – Delay of Paving Operations for High Fill Embankment Widening Area

Special Provision

Following completion of placement of the engineered fill for the high fill embankment widening, up to the top of the granular sub-base for the pavement structure, the Contractor is required to delay installation of the pavement structure granular base material and surface course/paving for a minimum period of 15 days. Prior to placement of the pavement structure granular base material and paving, the Contractor shall conduct a survey to determine the elevation of the top of the granular sub-base material, and shall place additional granular sub-base material as and where required to achieve the pavement design sub-base elevation.

The Contractor shall not proceed with final granular base placement and paving until approval has been given by the Contract Administrator.



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