FOUNDATION INVESTIGATION AND DESIGN REPORT

Highway 406 S - Geneva Street N/S Ramp, Structure Site 18-168, Highway 406 Structural Rehabilitation from Fourth Avenue to Westchester Avenue, St. Catharines, Ontario

Ministry of Transportation, Ontario G.W.P. 2453-13-00

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PART A

FOUNDATION INVESTIGATION REPORT

HIGHWAY 406 S – GENEVA STREET N/S RAMP (STRUCTURE SITE 18-168)
HIGHWAY 406 STRUCTURAL REHABILITATION FROM FOURTH AVENUE TO WESTCHESTER AVENUE, ST. CATHARINES, ONTARIO
MINISTRY OF TRANPORTATION, ONTARIO
G.W.P. 2453-13-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the replacement of the Highway 406 S – Geneva Street north/south (N/S) off-ramp structure associated with the Highway 406 rehabilitation project from Fourth Avenue to Westchester Avenue in the City of St. Catharines, Regional Municipality of Niagara, Ontario.

The purpose of this investigation is to establish the subsurface soil and groundwater conditions at the existing ramp structure by borehole drilling and geotechnical/analytical laboratory testing on selected soil samples.

The Terms of Reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal, dated September 2015, which forms part of the Consultants Agreement for Assignment No. 2014-E-0075 for this project. The scope of work for the Geneva Street N/S Off-Ramp structure site is outlined in Golder's Revised Change Request, dated May 25, 2017. The work has been carried out in accordance with Golder's Supplementary Specialty Plan for this project, dated June 2016.

2.0 SITE DESCRIPTION

The Highway 406 S - Geneva Street S N/S Off-Ramp structure is located north of the Highway 406 underpass structure at Westchester Avenue and connects Highway 406 northbound vehicle traffic to Geneva Street, near downtown St. Catharines at the location shown on the Key Plan on Drawing 1. The structure spans the broad gully that was part of the old second Welland Canal (now called the Canal Valley adjacent to Twelve Mile Creek), of which the crest-to-crest width is about 210 m. The General Plan of the site available in GEOCRES 30M3-43 (Drawing D 5147-2, dated May 1963) shows topographic contours indicating that the crest of the valley bank was about 8.3 m above the ground surface of the base of the valley. The General Plan drawing shows the location of the original Old Welland Canal and indicates that a culvert was constructed to the east of the Old Welland Canal. It is understood that Old Welland Canal discharged to Twelve Mile Creek approximately 680 m to the west of the Ramp Bridge through a three-cell buried structural culvert that was constructed between Piers 1 and 2 of the Ramp Bridge. The General Plan further indicates that Old Welland Canal was filled to Elevation 87.5 m, and that in the vicinity of Pier 4 and between Pier 1 and the east abutment the "gravely clay fill, miscellaneous fill, ash and rubble fill" was to be subexcavated to "firm strata" and the subexcavation replaced with "select earth fill".

The existing Ramp is a five-span bridge that was constructed in about 1964, and has a total length of approximately 114 m. The current grade of the Ramp is at about Elevation 97 m near the east abutment and rises to about Elevation 102 m at the west abutment.

Drawing No. D 5147-3 titled "Foundation Layout" indicates that the abutments and piers are reportedly supported on pile caps founded on 14BP73 steel H-piles (equivalent to HP360x108), driven into a till stratum underlying the silty clay deposit, to practical refusal as determined by the Hiley Formula (D.H.O. Std. BD 16-3,4).

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigation

The results of a previous geotechnical investigation carried out at the site of the existing Highway 406 south Geneva Street N/S Off-Ramp between May 3 and October 4, 1962 are obtained from the MTO GEOCRES library, and are summarized in the report prepared by the Materials and Research Division (Foundation Section) titled, "Highway #58 and Geneva Street, Access Ramp at Old Welland Canal, City of St. Catharines, Dist. #4" dated July 12, 1962, GEOCRES No. 30M03-043. During the 1962 investigation, a total of nineteen boreholes (Boreholes 2 to 20) were advanced in the general vicinity of the existing ramp structure. The location of the boreholes advanced during the previous investigation are shown on Figure A1 in Appendix A. The relevant records for eighteen borehole (Boreholes 3 to 20) advanced in the immediate vicinity of the structure during the 1962 investigation are presented in Appendix A.

The GEOCRES foundation investigation report indicates that soil samples were obtained at 0.75 m to 3 m depth intervals using 50 mm outside diameter split-spoon samplers driven by manual hammers, in accordance with the Standard Penetration Test (SPT) procedure. In the soft to stiff cohesive deposit, thin-walled Shelby tube samples were also taken and in situ field vane testing was conducted to measure the undrained shear strength of the deposit. Dynamic Cone Penetration Testing (DCPT) was conducted from the ground surface in the immediate vicinity of Boreholes 3, 4 and 6 to 11.

Observations of the water levels in the boreholes were recorded on some boreholes logs; however, piezometers were not installed in any of the boreholes.

Selected samples obtained from the boreholes were subjected to classification testing and the results are resented the Record of Borehole sheets in Appendix A.

The boreholes locations as provided on the Record of Borehole sheets in Station and Off-set were plotted on the General Arrangement Drawing No. R2-1, dated Nov. 2016, provided by MTO on January 31, 2017, and the borehole coordinates were interpreted from the coordinate system superimposed on the plan. The borehole locations in MTM NAD 83 Zone 10 Coordinates, geographic coordinates (latitude / longitude) the ground surface elevations in Geodetic Datum and the drilled depths as presented on or derived from the 1962 borehole records are summarized below.

Borehole No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude,°)		
3	4779872.8 (43.158411)	326101.6 (-79.238065)	88.1	14.2
4	4779919.8 (43.158835)	326064.9 (-79.238514)	90.5	15.4
5	4779944.1 (43.159055)	326041.6 (-79.238800)	96.3	20.3
6	4779894.3 (43.158605)	326083.7 (-79.238285)	87.3	20.3
7	4779859.3 (43.158289)	326118.1 (-79.237863)	88.8	15.7
8	4779866.7 (43.158355)	326140.7 (-79.237585)	89.0	15.2
9	4779844.5 (43.158155)	326163.9 (-79.237300)	89.7	15.5
10	4779855.2 (43.158250)	326189.6 (-79.236984)	90.2	15.8

Borehole No.	Location (MTM NAD 83 Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude,°)		
11	4779908.1 (43.158729)	326068.2 (-79.238106)	90.8	18.4
12	4779860.7 (43.158301)	326155.0 (-79.237409)	89.5	15.7
13	4779850.7 (43.158211)	326141.3 (-79.237578)	89.3	4.6
14	4779879.9 (43.158474)	326139.3 (-79.237601)	88.7	3.5
15	4779882.6 (43.158498)	326153.6 (-79.237426)	89.0	3.5
16	4779875.5 (43.158433)	326175.7 (-79.237154)	89.6	4.6
17	4779933.1 (43.158955)	326063.1 (-79.238536)	91.1	3.5
18	4779899.4 (43.158652)	326050.5 (-79.238693)	90.5	5.5
19	4779905.9 (43.158711)	326032.6 (-79.238913)	90.2	14.8
20	4779922.9 (43.158864)	326033.8 (-79.238897)	91.4	2.3

3.2 Current Investigation

The field work for the current foundation investigation was carried out between October 30 and November 1, 2017 and between April 9 and May 1, 2018, during this time, a total of ten boreholes, (designated as Boreholes 17-1, 17-2, 17-2A, 17-3, 17-3A, 17-4, 17-5, 17-6, 17-7 and 17-8) were advanced near the footprint of the foundation elements, at the locations shown on Drawing 1 as follows:

Foundation Element	Nearest Relevant Boreholes
West Abutment	17-1
Pier 4	17-2, 17-2A
Pier 3	17-3, 17-3A
Pier 2	17-4
Pier 1	17-5

Foundation Element	Nearest Relevant Boreholes
East Abutment	17-6
Proposed Crane Pad	17-7, 17-8

The field borehole investigation was completed using a track-mounted CME 850 drill rig, supplied and operated by Aardvark Drilling Inc., of Guelph, Ontario, and a track-mounted CME 55 drill rig, supplied and operated by Davis Drilling, of Milton, Ontario. The boreholes were advanced through the overburden using 150 mm outer diameter solid stem augers or 203 mm outer diameter hollow stem augers. All boreholes, with the exception of Borehole 17-2 and 17-3, also used an 86 mm diameter tricone with wash boring techniques and used drilling mud to balance hydrostatic heads and to maintain the boreholes open. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler operated by an automatic hammer on the drill rigs, performed in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586[1]). In situ field vane shear testing, using MTO standard “N”-sized vanes, was carried out to measure the undrained shear strength of cohesive soils (ASTM D2573[2]). Dynamic cone penetration tests (DCPT) were advanced immediately adjacent to Boreholes 17-2A, 17-3A, 17-4 and 17-5 from depths ranging from 12.2 m to 21.3 m below ground surface.

The groundwater conditions and water levels in the open boreholes were observed during and immediately following drilling operations. A standpipe piezometer was installed in Borehole 17-4 to permit monitoring of the groundwater level over time. The standpipe piezometer consists of a 50 mm diameter PVC pipe with a slotted screen sealed at a selected depth within the borehole. The borehole and annulus surrounding the piezometer pipe above the screen sand pack was backfilled to the ground surface with bentonite pellets. The remaining boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903: Wells (as amended).

The field work was observed by members of Golder’s engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder’s Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples.

Three selected soil samples were submitted to Maxxam Analytics, a Standards Council of Canada (SCC) accredited laboratory of Mississauga, Ontario, for chemical analysis. The soil samples were analysed for a suite of corrosivity parameters, including conductivity, resistivity, soluble chloride, soluble sulphate, and pH.

The borehole locations and ground surface elevations were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and 0.1 m in the horizontal directions. The borehole locations, given on the borehole records and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) northing and easting coordinates and the ground surface elevations referenced to Geodetic Datum. The borehole locations including geographic coordinates (latitude / longitude), ground surface elevations and borehole depths are summarized below.

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[1] ASTM D1586-11 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils, ASTM International, West Conshohocken, PA, 2011

[2] ASTM D2573-15 Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils, ASTM International, West Conshohocken, PA, 2015

Borehole No.	Location (MTM NAD 83, Zone 10)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
17-1	4,779,913.3 (43.158778)	326,043.0 (-79.238784)	98.0	32.6
17-2	4,779,907.6 (43.158725)	326,069.7 (-79.238456)	90.3	8.2
17-2A	4,779,907.6 (43.158725)	326,070.1 (-79.238451)	90.5	21.2
17-3	4,779,896.7 (43.158627)	326,090.3 (-79.238203)	87.4	8.8
17-3A	4,779,896.0 (43.158620)	326,089.7 (-79.238211)	87.7	21.3
17-4	4,779,889.0 (43.158557)	326,105.4 (-79.238018)	88.1	23.5
17-5	4,779,880.2 (43.158477)	326,126.3 (-79.237761)	88.3	20.4
17-6	4,779,867.0 (43.158358)	326,151.9 (-79.237447)	96.8	28.0
17-7	4,779,898.7 (43.158644)	326,114.1 (-79.237911)	87.2	11.3
17-8	4,779,910.2 (43.158748)	326,105.8 (-79.238012)	86.9	11.3

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 406 is located within the Iroquois Plains physiographic region, as delineated in the *Physiography of Southern Ontario* (Chapman and Putnam, 1984)[3]. The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession. This site is bound to the north by shoreline beach deposits from Glacial Lake Iroquois such as the Homer Bar on which downtown St Catharines is located, and the Niagara Escarpment located some 3 km to the south.

Surficial soil in this area of the Iroquois Plain is typically comprised of silty and clayey till of the Halton Till sheet according to the *Quaternary Geology of the Niagara-Welland Area* (Ontario Geological Survey Map 2496; Feenstra,

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[3] 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.

1984)[4]. The Halton Till sheet is underlain by an older red sandy and silty till, possibly the Wentworth Till sheet (OGS Preliminary Map 764, Feenstra, 1972)[5]. Shallow depressions on the surface of the clay plain upslope of the Homer Bar are infilled with bog sediments while fill materials comprised of earth and rock fill associated with the canal construction occur in the vicinity of the former Welland Canal (OGS Preliminary Map 764, Feenstra 1972)[5].

4.2 Subsurface Conditions

Detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the current investigation, including details of the standpipe piezometer installation and water level reading, and the results of the laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets provided in Appendix B. List of Abbreviations and symbols are also provided in Appendix B to assist in the interpretation of the borehole records. The results of the in situ field tests (i.e. SPT "N"-values, field vane and dynamic cone penetration test (DCPT)) as presented on the Record of Borehole sheets and in sub-sections of Section 4.2 are uncorrected. The geotechnical laboratory testing plots are contained in Appendix C. The results of the analytical testing of these soil samples are presented in Appendix D.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the stratigraphic profile on Drawing 1 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 govern any interpretation of the site conditions. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

In general, the subsurface conditions consist of pavement structure (borehole advanced at the east abutment) or topsoil (all the remaining of boreholes) underlain by fill associated with the construction of the existing off-ramp structure in turn, underlain by a cohesive clayey silt to clay deposit, underlain by a cohesive till deposit. The till deposit is underlain by a layered granular deposit consisting of silt to silt and sand to sand, as well as cohesive interlayers of clayey silt, at some borehole locations.

A more detailed description of the subsurface conditions encountered in the boreholes from the current investigation is provided in the following sections.

4.2.1 Pavement Structure

Borehole 17-6 was advanced from the off-ramp pavement surface and encountered an approximately 203 mm thick layer of asphalt and a 366 mm thick layer of concrete (including a reinforcing steel bars).

4.2.2 Topsoil

A 30 mm to 152 mm thick layer of topsoil was encountered at the ground surface in Boreholes 17-1, 17-5, 17-7 and 17-8.

4.2.3 Cohesive Fill

Cohesive fill comprised of sandy clayey silt to clayey silt to silty clay was encountered in Boreholes 17-1 below the surficial top soil layer and 17-6 below the sand to sand and gravel fill underlying the pavement, advanced at the west and east abutments, respectively. The fill extends to depths of 7.2 m and 11.2 m below ground surface

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[4] Feenstra, B.H. 1984. Quaternary Geology of the Niagara-Welland Area. Ontario Geological Survey, Map 2496, Quaternary Geology Series. Scale 1:50,000

[5] Feenstra, B.H. 1972. Quaternary Geology of the Niagara Area, Southern Ontario. Ontario Division of Mines, Preliminary Map P.764, Geological Survey. Scale 1:50,000

(Elevations 90.8 m and 85.6 m), respectively. In Boreholes 17-2 to 17-5 advanced at the bottom of the valley, the cohesive fill was encountered at ground surface and extended to depths of between about 1.3 m and 3.0 m below ground surface (between Elevations 87.3 m and 85.3 m). In Boreholes 17-7 and 17-8 the fill layer was underlying the topsoil and extends to depth of 0.7 m and 0.9 m below ground surface (Elevations 86.5 m and 86.0 m), respectively.

The SPT “N”-values measured within the cohesive fill deposit generally range from 3 blows to 14 blows per 0.3 m of penetration, suggesting a soft to stiff consistency. The SPT “N”-values recorded at Borehole 17-3 are 100 blows per 0.05 m of penetration and 100 blows per 0.08 m of penetration, inferred due to the presence of concrete fragments in the fill material and these values are not considered representative of the overall fill composition.

A grain size distribution test was completed on one sample of the cohesive fill material and the result is shown on Figure C-1 in Appendix C. The cohesive fill deposit consists of trace to some gravel, trace rootlets and deleterious material including brick and asphalt fragments. An organic odour was noted in Borehole 17-2, at a depth of 2.3 m below ground surface. An Atterberg limits test carried out on one sample of the cohesive fill material measured a liquid limit of about 44 per cent, a plastic limit of about 21 per cent, and a plastic index of about 23 per cent. The result, which is plotted on a plasticity chart on Figure C-2 in Appendix C, indicates that the fill material consists of silty clay of medium plasticity.

The water content measured on select samples of the fill deposit ranges from about 10 per cent to 29 per cent.

4.2.4 Non-Cohesive Fill

Non-cohesive fill consisting of silty sand to sand to sand and gravel to gravel was encountered in Boreholes 17-2, 17-3, 17-5, 17-6 and 17-8, underlying the asphalt and concrete pavement (in Borehole 17-6) or topsoil (in boreholes advanced at the base of the valley). A layer of fill consisting of black sand exhibiting a hydrocarbon odour was also encountered underlying the cohesive fill in Borehole 17-6 at a depth of 11.2 m (Elevation 85.6 m) and is about 1.5 m thick. The surface of the non-cohesive fill was encountered at depths between about 0.6 m and 3.0 m, (between Elevations 96.2 m and 85.3 m) below ground surface, respectively and the thickness of the non-cohesive fill ranges from 0.7 m to 4.0 m. Hydrocarbon odours were noted at Borehole 17-5 at depths between 3.0 m and 5.2 m below ground surface (Elevation 85.3 m and 83.0 m, respectively.

The SPT “N”-values measured within the non-cohesive fill range from 2 blows to 27 blows per 0.3 m of penetration, indicating that the fill layer has a very loose to compact compactness condition.

A grain size distribution test was carried out on one sample of the non-cohesive fill material and the result is shown on Figure C-3 in Appendix C. The non-cohesive fill contains of trace to some silt, trace clay, clayey silt pockets, inferred cobbles and glass fragments.

The water content measured on select samples of the fill deposit ranges from about 17 per cent to 61 per cent.

4.2.5 Clayey Silt with Sand to Clay

Underlying the fill deposit in all boreholes advanced for the current investigation, a cohesive deposit consisting of clayey silt with sand to sandy clayey silt to clayey silt to silty clay to clay was encountered at depths between about 0.7 m and 12.7 m. At Boreholes 17-1 and 17-6 advanced at the west and east abutment, respectively the cohesive deposit extends to depths of about 22.4 m and 22.6 m (Elevations 75.6 and 74.2 m), below ground surface, respectively. At Boreholes 17-1 and 17-6 the overall thickness of the cohesive deposit is about 15.2 m and 9.9 m, respectively. At Boreholes 17-2 to 17-5, advanced at the bottom of the valley the cohesive deposit extends to depths of between about 13.3 m and 15.9 m (between Elevation 75.0 m and 70.3 m), below ground surface and the thickness of the deposit ranges from about 7.7 m to 14.9 m. Boreholes 17-7 and 17-8 were terminated within this deposit at a depth of 11.3 m (Elevation 75.9 m and 75.6 m), below ground surface. In Borehole 17-7, a 1.5 thick layer of sand was encountered within this cohesive deposit at a depth of 3.1 m (Elevation 84.1 m) below ground surface.

The SPT “N”-values recorded within the cohesive deposit range from 1 blow to 25 blows per 0.3 m of penetration. In situ field vane tests carried out within the cohesive stratum measured undrained shear strengths ranging from about 34 kPa to greater than 96 kPa, with sensitivities ranging from about 1 to 4. The field vane test results together with the SPT “N”-values indicate that the cohesive deposit has a generally firm to very stiff consistency. The SPT “N”-value recorded within the gravel layer is 31 blows per 0.3 m of penetration, indicating a dense compactness condition.

The results of grain size distribution tests carried out on nine samples of the deposit are shown on Figures C-4A, C-4B and C-4C, in Appendix C. Hydrocarbon odours were encountered within the cohesive deposit at Borehole 17-2 from depths between 3.8 m and 4.4 m (Elevation 86.5 m and 85.9 m) below ground surface, respectively and in Borehole 17-3 from depths between 2.3 m and 2.9 m (Elevation 85.1 m and 84.5 m) below ground surface, respectively.

The results of grain size distribution testing carried out on one sample of the gravel inter layer encountered in Borehole 17-7 is shown on Figure C-5, in Appendix C.

Atterberg limits tests were carried out on eleven samples of the cohesive deposit and measured liquid limits ranging between about 28 per cent and 52 per cent, plastic limits ranging between about 14 per cent and 24 per cent, and plastic indices ranging between about 12 per cent and 31 per cent. These results, which are plotted on a plasticity chart on Figures C-6A and C-6B in Appendix C, indicate that the deposit consists of clayey silt of low plasticity to clay of high plasticity.

The natural water content measured on samples of the cohesive deposit ranges from about 18 per cent to 53 per cent. The natural water content measured on a sample of the gravel interlayer encountered at Borehole 17-7 is about 10 per cent.

4.2.6 Sandy Silt to Sandy Clayey Silt to Clayey Silt (Till)

A till deposit was encountered underlying the cohesive deposit in Boreholes 17-1, 17-2A, 17-3A, 17-4, 17-5 and 17-6. In Boreholes 17-1 and 17-6 advanced at the west and east abutments the surface of the till deposit was encountered at depths of 22.4 m and 22.6 m below ground surface (Elevations 75.6 m and 74.2 m), and the thickness of the deposit is 1.5 m and 1.7 m, respectively. In Boreholes 17-2A, 17-3A, 17-4 and 17-5, advanced at the base of the valley, the surface of the till deposit was encountered at depths between about 13.3 m and 17.1 m (between Elevations 75.0 m and 70.3 m) below ground surface and the thickness of the deposit ranges between 1.2 m and 3.0 m. A 0.2 m thick layer of gravelly sand was encountered within the till deposit at Borehole 17-4 at a depth of 15.3 m (Elevation 72.8 m) below ground surface.

SPT “N”-values measured within the till deposit range from 28 blows to 76 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency. As SPT “N”-value of 8 blows per 0.3 m of penetration was measured across the interface of the till and overlying clayey silt deposit in Borehole 17-3A, indicating a loose compactness condition.

The results of grain size distribution tests carried out on six samples from the till deposit are shown on Figure C-7 in Appendix C. The till is composed of primarily of clayey silt with sand to sandy clayey silt to clayey silt with a zone of sandy silt trace to some clay in Borehole 17-3A. Atterberg limits testing carried out on seven samples of this deposit measured liquid limits ranging from about 17 per cent to 22 per cent, plastic limits ranging from about 13 per cent to 14 per cent, and plastic indices ranging from about 3 per cent to 7 per cent. These results, which are plotted on a plasticity chart on Figure C-8 in Appendix C, indicate that the deposit consists of clayey silt of low plasticity and a zone of sandy silt of slight plasticity.

The natural water content measured on samples of the till deposit ranges from about 9 per cent to 15 per cent.

4.2.7 Sand to Silt and Sand to Sandy Silt to Silt

A granular deposit consisting of interlayered silt to sandy silt to silt and sand to sand was encountered underlying the till in Boreholes 17-1, 17-2A, 17-3A, 17-4 and 17-6, and underlying a clayey silt interlayer in Borehole 17-5 (see Section 4.2.8). The surface of this granular deposit was encountered at depths between about 23.0 m and 24.3 m (Elevations 74.1 m and 72.5 m) below ground surface in Boreholes 17-1 and 17-6 advanced at the west and east abutments, respectively and in Boreholes 17-1, 17-2A, 17-3A, 17-4 and 17-5, advanced at the bottom of the valley the granular deposit was encountered at depths of between 16.3 m and 19.7 m (between Elevations 73.2 m and 67.7 m) below ground surface. All boreholes advanced during the current investigation, with the exception of Borehole 17-3A, 17-7 and 17-8, terminated within this granular deposit at depths between 20.4 m and 23.5 m below the bottom of the valley and at depths of 32.6 m (Elevation 65.4 m) and 28.0 m (Elevation 68.8 m), below ground surface at the west and east abutment, respectively.

SPT “N”-values measured within the various layers of the granular deposit generally range between 18 blows per 0.3 m of penetration with one “N”-value of 102 blows per 0.26 m of penetration, indicating a compact to very dense compactness condition.

The results of grain size distribution tests carried out on ten samples of the granular deposit are shown on Figures C-9A and C-9B in Appendix C. Atterberg limits testing was carried out on one sample of the silt and sand layer of this deposit from Borehole 17-5 and the test indicates that the silt and sand portion of the deposit is non-plastic.

The natural water content measured on samples of this granular deposit ranges from about 14 per cent to 21 per cent.

4.2.8 Clayey Silt

A cohesive layer of clayey silt was encountered underlying the glacial till deposit in Borehole 17-5 at Elevation 72.0 m and underlying the silt deposit in Borehole 17-3A at Elevation 66.5 m. The thickness of the clayey silt layer encountered in Borehole 17-5 is about 1.5 m and Borehole 17-3A was terminated within the clayey silt layer after penetrating into it for a depth of 0.6 m.

The SPT “N”-values recorded within this deposit are 39 blows and 41 blows per 0.3 m of penetration, suggesting a hard consistency.

Atterberg limits testing was carried out on two samples of this deposit and measured liquid limits at about 20 per cent and 23 per cent, plastic limits at about 13 per cent and 16 per cent, and plastic indices at about 7 per cent. The results, which are plotted on a plasticity chart on Figure C-10 in Appendix C, indicates that the material comprising these cohesive layers is a clayey silt of low plasticity.

The natural water content measured two samples of this cohesive deposit ranges from about 11 per cent to about 21 per cent.

4.2.9 Groundwater

The overburden samples obtained from the borehole investigation were generally moist to wet. The groundwater levels in the open borehole or inside the drill casing were measured upon completion of drilling operations whenever possible; however, water drilling mud was used to advance all borehole with the exception of Boreholes 17-2 and 17-3. Upon advancement of Borehole 17-2A at a depth of about 4.6 m below ground surface (Elevation 85.7 m), artesian conditions were recorded with the water level rising to about 0.4 m above ground surface.

A standpipe piezometer was installed in Borehole 17-4 to permit the monitoring of groundwater level at this site. The piezometer at Borehole 17-4, screened within the granular deposit underlying the sandy clayey silt till deposit. Details of the piezometer installation and measured groundwater levels are shown on the borehole records in Appendix B. The groundwater levels recorded are summarized below.

Borehole / Test Pit No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date	Comments
17-2A	90.3	2.5	87.8	Oct 13, 2017	Open Borehole
17-3A	87.4	3.4	84.0	Nov 1, 2017	
17-4	88.1	0.0	88.1	Nov 1, 2017	Measured in Standpipe Piezometer
		1.2	86.9	April 4, 2018	
		1.2	86.9	May 1, 2018	

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year.

4.2.10 Analytical Testing Results

Analytical testing was carried out on selected soil samples recovered from Borehole 17-4. The soil samples were submitted to Maxxam Analytics of Mississauga, Ontario for corrosivity testing. Detailed analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix D, and summarized below.

Borehole No.	Sample ID	Depth (m)	Parameters				
			Resistivity (ohm-cm)	Electrical Conductivity (mS/cm)	Soluble Sulphate (So_4) Content (µg-g)	Chlorides (CL) Content (µg-g)	pH (pH)
17-4	SS7	6.1 – 6.7	3,200	317	62	81	7.7
17-4	SS12	13.7 – 14.3	2,200	460	170	130	8.4
17-4	SS16	19.8 – 20.4	2,200	460	140	180	8.6

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Katelyn Nero, and was reviewed by Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a MTO Foundations Designated Contact for Golder and Senior Consultant conducted a technical and quality control review of the report.

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PART B

FOUNDATION DESIGN REPORT
HIGHWAY 406 S – GENEVA STREET N/S RAMP (STRUCTURE SITE 18-168)
HIGHWAY 406 STRUCTURAL REHABILITATION FROM FOURTH AVENUE TO WESTCHESTER AVENUE, ST. CATHARINES, ONTARIO
MINISTRY OF TRANPORTATION, ONTARIO
G.W.P. 2453-13-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering recommendations for design of the proposed replacement of the Geneva Street N-S Off-Ramp bridge from Highway 406 South in the City of St. Catharines, Regional Municipality of Niagara, Ontario. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the recent subsurface investigation by Golder and previous investigations carried out by the Ministry of Transportation, Ontario (MTO). The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasibility of reuse of the existing foundation and carry out the detail design of the bridge foundations. The Foundation Investigation Report, discussion and recommendations are intended for the use of the MTO and its 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. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project 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.

6.1 General

Based on the General Arrangement (GA) drawing R3-1, provided by the MTO on February 17, 2017, it is understood that the MTO plans to remove the existing superstructure and construct a new structure using the existing driven pile foundations to support the structure. At the pier locations the existing pile cap will be left in place and a new cap overlay will be constructed upon the existing pile cap. The GA drawing also indicates that new abutments and wingwalls will be constructed upon the existing pile cap. It is understood that the surface grade of the bridge will not be raised at the piers or abutments.

6.2 Summary of Existing Foundations

Based on the General Plan (Drawing D 5147-2, revision dated July 9, 1969 - “as-constructed”) obtained from GEOCRES, the existing bridge is a five-span structure with a total length of approximately 114 m. The abutments and piers are founded on pile caps supported on 14BP73 steel H - Piles (equivalent of HP 360 x 108 steel H-piles). The piles are shown to be battered at inclination between 8 vertical to 1 horizontal (8V:1H), 6V:1H and 3V:1H. The Foundation Layout drawing (D 5147-3, revision dated July 9, 1969 - “as constructed”), which is available in the GEOCRES documents, indicates that the piles were to be driven to practical refusal as determined by the Hiley Formula (D.H.O. Std. BD 16-3,4). The pile length below the “cut-off elevation” is noted on the Foundation Layout drawing. The following summarizes the cut-off elevation for the piles, the pile length, and the pile tip elevation:

Foundation Unit	Cut-off Elevation of Pile at Pile Cap (m)	Pile Length below Cut-off Elevation (m)	Estimated Pile Tip Elevation (m)
West Abutment	97.7	23.8	73.9
Pier 4	88.7	16.5	72.2
Pier 3	85.0	17.1	68.0
Pier 2	85.0	13.1	71.9
Pier 1	87.3	14.0	73.3

Foundation Unit	Cut-off Elevation of Pile at Pile Cap (m)	Pile Length below Cut-off Elevation (m)	Estimated Pile Tip Elevation (m)
East Abutment	92.7	20.4	72.2

The Foundation Layout drawing recommends a "design load" of about 489 kN (55 tons) for each pile at Piers 2 and 3 and a "design load" of about 445 kN (50 tons) for the abutments and Piers 1 and 4. The October 19, 1962 Foundation Investigation Report (GEOCRES 30M3-43) suggests as an alternate a "safe load" of 530 kN (60 tons) per pile for 12 ¾ inch (0.324 m) diameter steel tube piles driven to practical refusal and recommends that they be driven some 1.8 m into the till stratum, although, as noted above, the as-constructed drawings indicate that 14BP73 H-piles were ultimately chosen / used as the deep foundation elements at this site.

It is understood from the MTO Bridge office that the existing structure has performed adequately and there is no observable evidence of settlement or lateral movement of the structure at the ground surface. In addition, the existing approach embankment side slopes appear to have performed adequately and do not show any signs of instability.

6.3 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 Canadian Highway Bridge Design Code and its Commentary (CHBDC 2014), the proposed bridge and its foundation system is considered to be classified as having a "typical consequence level" associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the 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 ULS and SLS consequence factor, ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the CHBDC (2014) have been used for design (i.e. assessment of the existing and new, if required foundation elements).

6.4 Seismic Design

6.4.1 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 founding level 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 D would be applicable for the design of the Geneva Street N-S Off-Ramp replacement bridge structure.

6.4.2 Spectral Response Values and Seismic Performance Category

Based on the location of the Highway 406 S - Geneva Street N-S Off-Ramp replacement bridge (Latitude: 43.158620° ; Longitude: -79.238211°), the reference Site Class C spectral acceleration values were obtained based on the 5th generation seismic hazard maps published by the Geological Survey of Canada (GSC).

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 D 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)	0.074	0.138	0.226
PGV (m/s)	0.053	0.093	0.157
Sa (0.2) (g)	0.113	0.207	0.347
Sa (0.5) (g)	0.072	0.122	0.202
Sa (1.0) (g)	0.039	0.064	0.099
Sa (2.0) (g)	0.019	0.030	0.046
Sa (5.0) (g)	0.0040	0.0066	0.0112
Sa (10.0) (g)	0.0016	0.0025	0.0042

6.5 Existing Driven Steel H-piles

6.5.1 Geotechnical Axial Resistance

Based on the as-constructed drawing referenced in Section 6.2, the existing HP360x108 steel H-piles were reportedly driven to practical refusal to the elevations calculated based on the provided cut-off elevation and the pile lengths for each foundation unit. Based on our interpretation of the borehole information obtained from the current investigation and the available information in the GEOCRES reports, and applying the applicable resistance factors from Tables 6.1 and 6.2 of the CHBDC (2014) for a "typical" consequence level and degree of site understanding, the factored ultimate geotechnical resistance at Ultimate Limit States (ULS) and the factored serviceability geotechnical resistance at Serviceability Limit States (SLS for 25 mm of settlement) for the abutment and piers founded on a pile cap supported on the existing steel H-piles driven to the estimated tip elevations given in Section 6.2 are provided below.

Foundation Unit	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance at SLS (kN) (for 25 mm of Settlement)
West Abutment	1,600 kN	--[1]
Piers 3 and 4	1,000 kN	--[1]
Pier 2	850 kN	--[1]
Pier 1	950 kN	--[1]
East Abutment	1,350 kN	--[1]

Note 1. The factored serviceability geotechnical resistance at SLS (for 25 mm of settlement) is greater that the factored ultimate geotechnical resistance at ULS, therefore the ULS condition will govern.

The above resistances are based on the current ground surface elevation relative to the existing pile tip elevation(s) at the foundation elements. If the ground surface is lowered this will result in a lower factored ultimate geotechnical resistance due to reduced overburden pressure and confining stress along the length of the piles. Section 6.5.2 provides a discussion on the estimated drag loads at the abutments, at Piers 1 and 4 (due to the approach embankment construction), and at Piers 2 and 3 (due to the backfilling of the Old Welland Canal).

6.5.2 Downdrag and Drag Loads

Based on a review of the original MTO design and construction records and discussion with MTO staff active during the time period of construction of the existing ramp bridge, there is no evidence or recollection as to when the 14BP73 steel H- piles were driven relative to the construction of the 8 m to 10 m high approach embankments and relative to the the placement of 2.3 m thicnkess of fill between Piers 2 and 3 to fill in the Old Welland Canal. If the existing piles were driven prior to placement of the fill and approach embankments or if the piles were driven prior to completion of the majority of the consolidation settlement of the underlying clayey silt with sand to clay deposits, then drag loads would have developed on the piles at the abutments and piers, as a result of long-term consolidation settlement of the underlying cohesive deposit, resulting in the development of negative skin friction along the length of the piles. If the piles were driven after the majority of the consolidation settlement from the embankment loading and the fill placed between Piers 2 and 3 had occurred, then it is unlikely that downdrag on the piles would have occurred. However, as noted above the actual staging of construction and when the piles were driven relative to the construction of the embankments and the fill placed between Piers 2 and 3 is unknown.

Based on case studies of long-term monitoring of drag loads presented in literature (Fellenius, 2006) instrumented piles were found to still have drag loads acting on the pile shaft over a period of 10 years to 15 years after installation. We were unable to find any published research information that measured the drag loads over a period of 50 years (i.e. about the time period that the existing piles at the Geneva Street Ramp Bridge site have been in place). Although the porewater pressures that would have built up in the cohesive deposit due to the embankment / fill loading have likely dissipated, it is noted that the piles tips are founded in hard / dense strata and in our opinion, the hard / dense strata that the piles were driven in to would likely restrict or even not permit settlement of the piles. Therefore, in our opinion the drag loads that would have developed on the piles during the consolidation period of the cohesive deposit are likely still acting on the steel H-piles. As such, we consider that it is prudent that the drag loads should be included in the assessment of the structural capacity of the existing steel H-piles, along with the factored dead loads of the structure.

Analyses to estimate the magnitude of the drag load(s) on the pile foundations at the abutments and piers were carried out in accordance with CHBDC (2014) and its Commentary. It is noted that the method used to assess the drag load is dependent on a number of factors including the pile length, foundation conditions at the pile tip, the unfactored dead load on the pile and the anticipated post construction settlement profile of the foundation soils. If any of these factors are different from those assumed in the analysis, and / or if embankment settlement mitigation options were undertaken at the time of the original construction of the bridge, the estimated drag loads would have to be reassessed.

The unfactored drag loads (based on the neutral plane conservatively estimated to be located at about the bottom of the cohesive deposit, corresponding to about Elevation 75 m) acting on the piles at the abutments and piers is summarized below.

Foundation Unit	Unfactored Drag Load
West Abutment	1,000 kN
Pier 4	725 kN
Pier 3	650 kN
Pier 2	475 kN
Pier 1	600 kN
East Abutment	900 kN

The unfactored drag load(s) noted above, in combination with the design dead load, should be considered by the structural engineer in the assessment of the structural capacity of the existing HP 360 x 108 steel H-piles. Should the magnitude of the combined and approximately factored drag loads and dead loads exceed the stuctural capacity of the piles, the location of the neutral plane could be estimated more accurately (using the method proposed by Briand and Tucker (1994)) and the magnitude of the drag loads recalculated and potentially revised for this condition.

6.5.3 Resistance to Lateral Loads

The resistance to lateral loading on deep foundations will be derived solely from the soil in front of the existing piles. Where ground conditions are generally competent and the lateral loads on piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory (as outlined below). However, it should be noted that the response of a pile to lateral loads is highly nonlinear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than 1 percent of the pile diameter, where the loading is static (no cycling) and where the pile material is linear (CFEM, 2006). Where these conditions are not met, the non-linear lateral behaviour of the soil should be considered by the use of P-y curves.

The factored serviceability geotechnical response of the soil in front of the piles under lateral loading at this site may be calculated using subgrade reaction theory suggested in CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h, (kPa/m) is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (CFEM 1992).

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction (kPa/m), as given below;

Z is the depth (m) below the in-ground drilled shaft cap; and,

B is the drilled shaft diameter/width (m)

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and,

B is the drilled shaft diameter/width (m)

The following values of n_h and s_u (Terzaghi, (1955) CFCM (1992) and American Petroleum Institute (API), 2002) may be incorporated into the calculations of horizontal subgrade reaction (k_h) for structural analyses for a single vertical pile. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that k_h is a function of deflection).

Foundation Element	Soil Unit	Elevation Interval (From Underside of Existing Pile Cap (m)	n_h (kPa/m)	s_u (kPa)
West Abutment	Firm to very stiff clayey silt to silty clay (Fill)	97.2 – 90.8	--	80
	Firm to very stiff clayey silt to silty clay	90.8 - 75.6	--	60
	Hard sandy clayey silt (Till)	75.6 – 74.1	--	250
Pier 4	Soft to stiff sandy clayey silt (Fill)	88.2 – 86.6	--	30
	Firm to very stiff clayey silt to silty clay to clay	86.6 – 74.4	--	80
	Hard sandy clayey silt (Till)	74.4 – 72.2	--	200
Pier 3	Firm to stiff clayey silt with sand	84.6 – 70.3	--	65
	Compact to dense sandy silt (Till)	70.3 – 68.0	8,000	--
Pier 2	Firm to stiff clayey silt	84.6 – 74.8	--	70
	Hard clayey silt (Till)	74.8 – 71.9	--	250
Pier 1	Soft to firm silty clay (Fill)	86.9 – 85.3	--	30
	Compact sand and gravel (Fill)	85.3 – 82.7	6,000	--
	Firm to stiff clayey silt	82.7 – 75.0	--	60
	Hard clayey silt (Till)	75.0 – 73.3	--	250
East Abutment	Stiff to very stiff clayey silt (Fill)	92.2 – 85.6	--	80
	Compact sand	85.6 – 84.1	3,500	--
	Stiff to very stiff clayey silt	84.1 – 74.2	--	75
	Hard sandy clayey silt (Till)	74.2 – 72.5	--	200
	Very dense silt and sand	72.5 – 72.2	11,000	--

Based on the above, both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ULS. At SLS, the horizontal resistance of the piles will be controlled by deflections and the horizontal resistance of the piles should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should be taken as that corresponding to a horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (see Section C6.11.2.2.2 of the Commentary to the CHBDC, 2014).

The upper zone of the soil down to a depth below the underside of the pile cap equal to about 1.5 times B (after Broms, 1964, where B is the pile diameter) should be neglected in the calculation of lateral resistance of the caisson to account for disturbance effects during installation.

Group action for lateral loading should also be considered when the pile spacing in the direction of loading is less than eight (8) pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R (U.S. Navy, 1986), as follows:

Pile Spacing in Direction of Loading (d = caisson diameter)	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above summary. Reduction for group effects is negligible when the centre-to-centre caisson spacing exceeds three caisson diameters measured in the direction perpendicular to loading.

6.5.4 Frost Protection

Pile caps, should be constructed not less than 1.2 m below the surrounding finished grade for protection from frost penetration, as interpreted from OPSD 3090.101 (Frost Penetration Depths for Southern Ontario).

6.6 Lateral Earth Pressure for Design of Abutments and Wing Walls

The lateral earth pressures acting on the abutment walls and any associated wing 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.

The following recommendations are made regarding the design of the abutment/wing walls:

- 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 subdrains 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 OPSD 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 (2014) 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 immediately behind the walls as per OPSS.PROV 501 (*Compacting*). 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 1.2 m (equivalent to the depth of frost penetration) behind the back of the wall on Figure C6.20(a) of the Commentary to the CHBDC (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line flatter than at 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the footing or pile cap on Figure C6.20(b) of the Commentary to the CHBDC (2014).

6.6.1 Lateral Earth Pressures for Static Design

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

- For a restrained wall, the pressures are based on the proposed embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill:

Material	**Earth Fill (Granular)**
Soil Unit Weight:	20 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a At rest, K_o	 0.31 0.47

- For an unrestrained wall, 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 At rest, K_o	 0.27 0.43	 0.27 0.43

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary to the CHBDC*, 2014.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 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must also be taken into account in the design of retaining / wing walls in accordance with Section 4.6.5 of the CHBDC (2014). 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 abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC (2014) and its Commentary, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA. For structures that do not allow lateral yielding, k_h is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient k_v is taken as zero.
- The seismic active pressure coefficients (K_{AE}) presented below may be used in design. These coefficients reflect the maximum K_{AE} obtained for each of the earthquake design periods and backfill conditions. 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 level. 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.

	Design Earthquake	Site PGA	Seismic Active Pressure Coefficients, K_{AE}		
			Granular A	**Granular B Type II**	**Earth Fill**
Yielding Wall	**475-Yr**	0.074g	0.27	0.27	0.30
	975-Yr	0.138g	0.29	0.29	0.32
	2,475 Yr	0.226g	0.31	0.31	0.35
Non-Yielding Wall	**475-Yr**	0.074g	0.29	0.29	0.32
	975-Yr	0.138g	0.33	0.33	0.37
	2,475 Yr	0.226g	0.40	0.40	0.45

- The K_{AE} value for a yielding wall is applicable provided that the wall can move up to $250k_h$ mm, where k_h is the site specific PGA as given in the table above. This corresponds to displacements of 18 mm, 35 mm and 56 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.

- 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 per Section C4.6.5 of the *Commentary to CHBDC* (2014).

6.7 Cyclic Mobility Potential of Cohesive Deposit

Cyclic mobility is a liquefaction phenomenon, triggered by seismically-induced cyclic loading, which can occur in soil deposits with static shear stresses lower than the soil strength. Deformations due to cyclic mobility develop incrementally because of static and dynamic stresses that develop during an earthquake and the associated strength loss of the soil due to prolonged shaking.

The loss of strength of the cohesive deposit can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading”, or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. As discussed in Section C4.6.4 of the Commentary to the CHBDC (2014), cyclic mobility can also cause settlement of the cohesive deposit which in turn results in the downdrag on the pile.

An assessment of the susceptibility of the cohesive deposit to cyclic mobility at this site was carried out using the methodology outlined by Bray and Sancio (2006). Using the results of the Atterberg limits tests carried out on samples of the cohesive deposit from boreholes advanced at the site, the plasticity index was plotted versus the ratio of the water content to the liquid limit (PI vs. w_n/LL) and of the fourteen samples tested / analysed twelve results plot as in the range classified as “not susceptible” to cyclic mobility and two results plot within the range classified as “moderately susceptible”. As such, the overall clayey silt to silty clay deposit is considered to be not susceptible to cyclic mobility during the 2,475-year design earthquake.

6.8 Approach Embankment Design and Construction

Based on the General Arrangement (GA) drawing provided by the MTO on February 17, 2017, the grade will not be raised at the abutments and approach embankments of the new structure. The highway grade at the west and east abutments is at about Elevation 102 m and 96.8 m, respectively. The ground surface at the base of the north side slope is generally between about Elevation 88 m and 90 m at the west and east abutments, respectively, therefore resulting in approach embankments that are about 10 m and 8 m high at the west and east approaches, respectively.

Borehole 17-6 was advanced immediately east of the east abutment from the highway grade and encountered sand and gravel fill to a depth of 4.6 m, underlain by clayey silt fill to a depth of 11.2 m, which is further underlain by a fill consisting of sand to a depth of 12.7 m below the highway grade. At the west abutment Borehole 17-1 was advanced near the toe of the approach embankment slope south of the structure and encountered sandy clayey silt fill material from ground surface to a depth of about 7.2 m below ground surface.

6.8.1 Subgrade Preparation and Embankment Construction

Following to the removal of the existing approach embankments, for the construction of the new abutments and wingwalls it is recommended that the excavation be inspected for the presence of any loosened/softened fill and topsoil/organic soils, and if present, it is recommended that these material be removed from the footprint of the approach embankments adjacent to the new structure. All construction operation adjacent to and at the abutments, including reconstruction works, should be carried out in accordance with OPSS.PROV.902 (Excavating and Backfilling Structure).

Fill for construction of the reconstructed approach embankments beyond the zone of Granular A or B Type II material against the abutment stem walls could consist of Granular ‘B’ Type I meeting the specifications of OPSS.PROV 1010 (Aggregates). The embankment fill should be placed and compacted in accordance with OPSS.PROV 501 (Compacting) and placed in accordance with OPSS.PROV 206 (Grading). Embankment side slopes should be re-constructed no steeper than 2 Horizontal to 1 Vertical (2H:1V) in granular fill.

All granular fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted to at least 95 per cent of the Standard Proctor Maximum Dry Density of the material. Inspection and field density testing should be carried out by qualified personnel during fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved. The excavated ends of the approach embankments should be benched to integrate the new reconstruction fill into the existing fill along the excavation faces, in accordance with OPSD 200.010 (Benching of Earth Slopes).

In accordance with MTO’s standard practice, a minimum 2 m wide bench should be provided where embankment slopes are greater than 8 m in high, such that the uninterrupted slope height does not exceed 8 m, consistent with 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.

6.8.2 Approach Embankment Slope Stability

Limit equilibrium slope stability analyses were performed on the east and west approach embankments side slopes for the area of reconstruction adjacent to the abutments using the commercially available program Slide (Version 6.0) produced by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of embankment slopes under static conditions at the end of construction as per the CHBDC (2014). This FoS is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to assess if the target minimum FoS was achieved for the design embankment height and geometries. In general, circular slip surfaces were analysed in the design.

For the non-cohesive soils present at the site, the effective stress parameters employed in the analysis were estimated from empirical correlations based on the results of the in-situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1967) 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 cohesive deposits, 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, in situ field vane shear strength test and other laboratory test data (i.e., natural water content), where appropriate. Effective stress parameters were also assigned to the cohesive deposits 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.

Summarized below are the simplified stratigraphy and the associated strengths and unit weights employed for the different soil types in the approach embankment areas.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Undrained Shear Strength (kPa)
Granular A or Granular B Type I and II Fill	22	35°	--
Compact sand Fill	22	33°	--
Firm to stiff sandy clayey silt Fill	20	32°	60
Firm to stiff clayey silt	20	26°	75
Hard clayey silt till	20	34°	250

The results of the analyses indicate that the factored FoS against global instability for the short-term (undrained) and long-term (drained) cases is greater than 1.5, as shown in Figures 1 and 2 for the east and west approach embankments, respectively.

6.8.3 Settlement

It is understood that the grade of the new N-S off-ramp bridge and approach embankments will be constructed to match the grade of the existing Geneva Street N-S off-ramp structure and the off-ramp approach embankments east and west of the bridge, therefore since there will not be any grade raise no additional settlement of the subgrade under the east and west approach embankment areas is expected

6.9 Analytical Testing for Construction Materials

The results of an analytical tests on three soil samples (clayey silt, clayey silt till and silt and sand) are presented in Section 4.2.10 and the laboratory test report is presented in Appendix D. The potential for sulphate attack and corrosion are discussed in the following paragraphs; however, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1-14 Section 4.1.1 "Durability Requirements" are followed when designing concrete elements.

6.9.1 Potential for Sulphate Attack

The analytical test results were compared to CSA A23.1 Table 3 *("Additional requirements for concrete subjected to sulphate attack")* to assess for potential sulphate attack on concrete. The sulphate concentrations measured in the tested samples are below the exposure class of S-3 (Moderate). Therefore, based on the three samples of soil tested, when the designer is selecting the exposure class for the structure, the effects of sulphates may not need to be considered.

6.9.2 Potential for Corrosion

The analytical test results of the soil samples were also compared to Table 2 of the U.S. Criteria for Assessing Ground Corrosion Potential (as derived from Federal Highways Administration (FHWA) 2003) for the potential attack on buried steel. The sulphate and chloride concentrations and the resistivity measured in the soil samples indicate “Mild to no corrosion potential” in the soil samples tested.

Based on the results of the samples tested, and given that the structure is located adjacent to the roadway and will be exposed to de-icing salt, consideration should be given by the designer to designing for a “C” type exposure class as defined by CSA A23.1 Table 1, for any new construction.

It is ultimately up to the structural designer to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 “Durability Requirements” are followed.

6.10 Construction Considerations

6.10.1 Excavation and Control of Groundwater and Surface Water

Excavations for the replacement of the abutments wall and wingwalls will extend about 3.8 m below ground surface into the compact sand and gravel fill material. Excavations at the piers for the addition of the concrete overlay above the existing pile caps will extend to depths of between 0.4 m and 2.5 m below current ground surface. Open-cut excavations must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act and Regulation for Construction Activities (OHSA, O. Reg. 213). The existing fill material is classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

The groundwater level at the site is anticipated to be at about Elevation 88 m, (as measured in the piezometer in Borehole 17-4 and similar to that in the open Borehole 17-2 and 17-3) and may be higher during wetter periods of the year. Given that the underside of the pile caps at the piers is between Elevations 88.2 m and 84.6 m, excavations at Piers 1 to 3 to expose the top of the pile caps may extend to about the elevation of the groundwater table. It is expected that water seepage will be relatively minor and can be handled by pumping from well filtered sumps located outside the foundation footprint. In this case, the dewatering should be carried out in accordance with OPSS 517 as amended by SP FOUND003. The SP FOUN0003 is included in Appendix E.

Excavations for the abutments for the replacement of the abutment and wingwalls and at Piers 1 and 4 will be carried out above the water table; however, perched groundwater may be encountered in the fill material overlying the cohesive deposit. Excavations for Piers 2 and 3 will extend to just below the groundwater level; however, it is anticipated that water inflow from these layers can be handled by pumping from filtered sump pumps placed at the base of the excavation. Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.10.2 Crane Pad and Falsework

It is understood that foundations for falsework and for a crane pad which will be constructed in the base of the valley will be required in support of the construction of the new superstructure. Based on discussions with MTO Structural Engineers the footings for the falsework would be on the order of 600 mm wide and designed on the basis that a factored ultimate geotechnical resistance of about 100 kPa is available. Based on the measured undrained shear strength and SPT “N”-values in the upper portion of the cohesive deposit it is estimated that the subgrade can provide a factored ultimate geotechnical resistance of about 100 kPa. The contract should specify that depending on the location of the crane pad and falsework, the contractor may need to advance additional boreholes to satisfy himself that there is adequate information to design the crane pad and falsework support. In addition, the contractor should also be required to retain foundation engineers to complete the design and design check; a Special Provision to this effect is included in Appendix E.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a senior geotechnical engineer and Associate with Golder. Mr. Jorge Costa, P.Eng., a MTO Foundations Designated Contact for Golder and Senior Consultant conducted a technical and quality control review of the report.

Golder Associates Ltd.

LICENSED PROFESSIONAL ENGINEER
S.M. McGAGHRAN
October 26, 2018
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LICENSED PROFESSIONAL ENGINEER
J. M. A. COSTA
October 26, 2018
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Sandra McGaghran, M.Eng., P.Eng.	Jorge M. A. Costa, P.Eng.
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KN/SMM/JPD/JMAC/rb

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https://golderassociates.sharepoint.com/sites/19544g/2000 foundation/5 - reports/4 - geneva street ramp investigation/final/1541610 fidr geneva street ns ramp structure 2018oct26.docx

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ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573-15	Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils

Commercial Software:

Slide (Version 6) by Rocscience Inc.

Ontario Provisional Standard Drawing:

OPSD 202.010	Slope Flattening using Surplus Excavated Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specification:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 802	Construction Specification for Topsoil
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 904	Construction Specifications for Concrete Structures
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

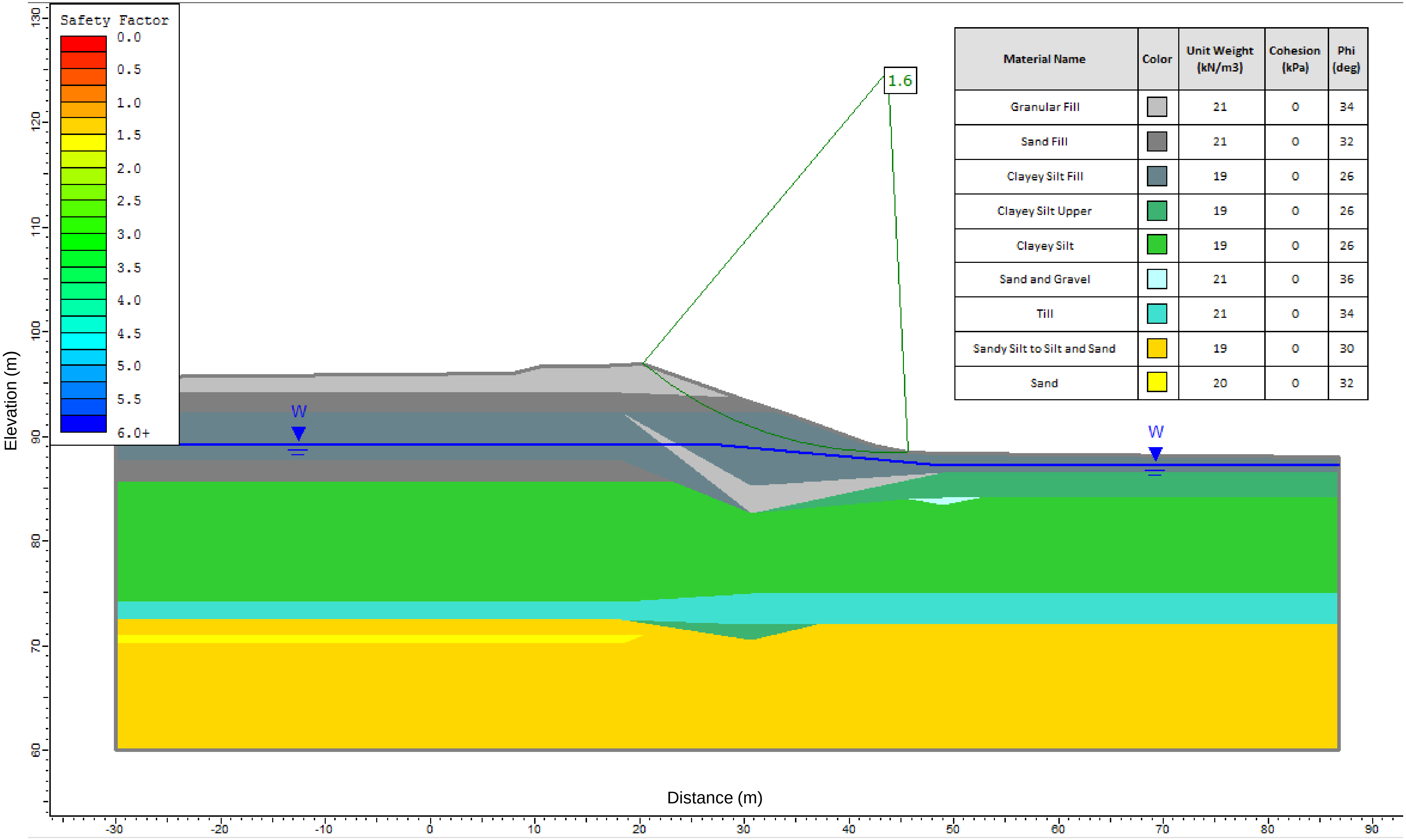
Ontario Regulation 903	Wells (as amended)

Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91	Construction Projects (as amended)

Highway 406 S – Geneva Street N/S Off-Ramp Bridge, East Approach Embankment Static Global Stability Analysis (Drained Case)

Figure 1



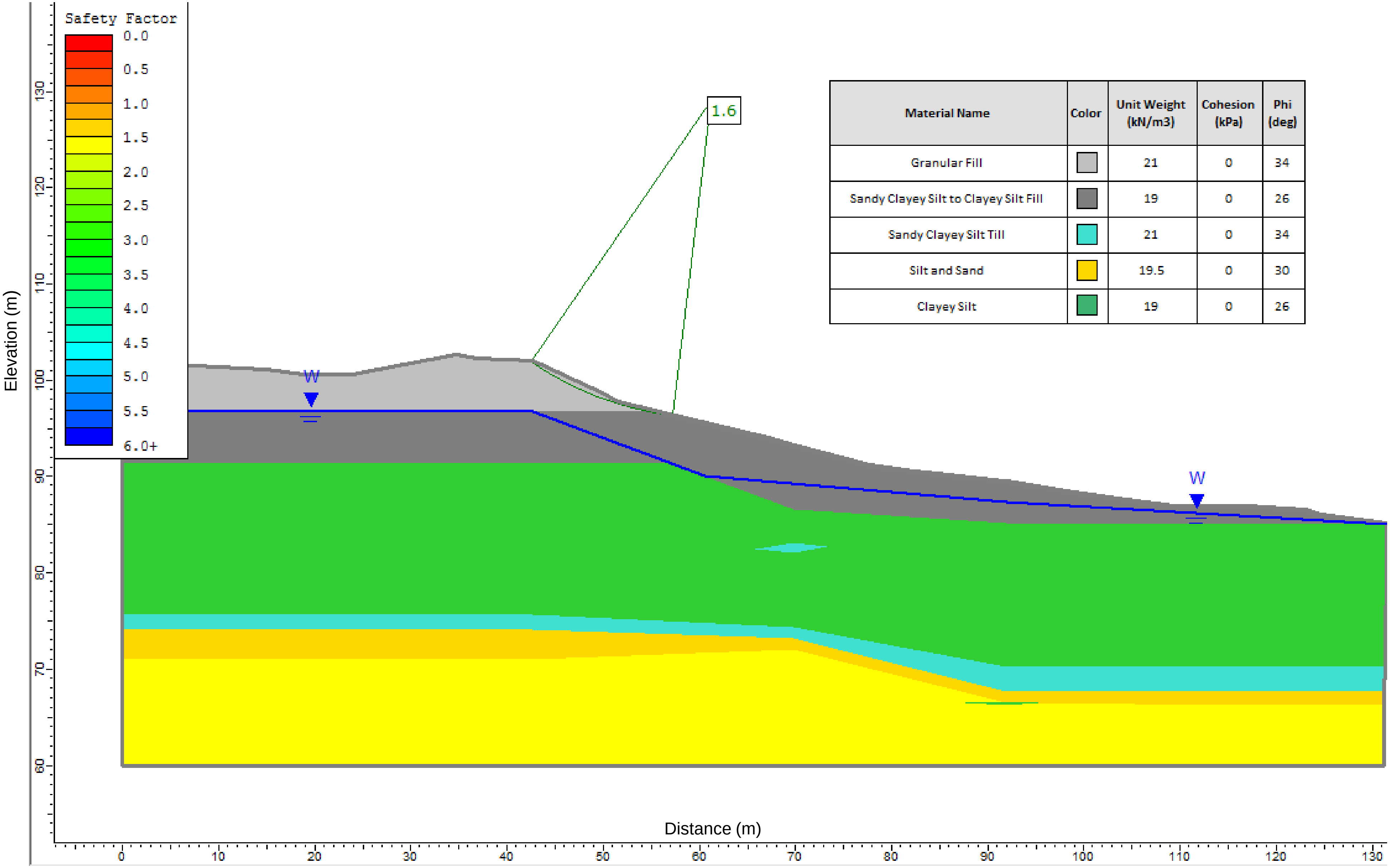
Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Granular Fill	⬜	21	0	34
Sand Fill	⬛	21	0	32
Clayey Silt Fill	⬛	19	0	26
Clayey Silt Upper	🟩	19	0	26
Clayey Silt	🟩	19	0	26
Sand and Gravel	⬜	21	0	36
Till	🟦	21	0	34
Sandy Silt to Silt and Sand	🟨	19	0	30
Sand	🟨	20	0	32

Date: July 31, 2018
Project No: 1541610

Analysis By: MCK **Reviewed By:** SMM

Highway 406 S – Geneva Street N/S Off-Ramp Bridge, West Approach Embankment Static Global Stability Analysis (Drained Case)

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Granular Fill	⬜	21	0	34
Sandy Clayey Silt to Clayey Silt Fill	⬛	19	0	26
Sandy Clayey Silt Till	🟦	21	0	34
Silt and Sand	🟨	19.5	0	30
Clayey Silt	🟩	19	0	26

Date: July 31, 2018

Project No: 1541610

Analysis By: MCK **Reviewed By:** SMM

CONT No.
G.W.P. No. 2453–13–00

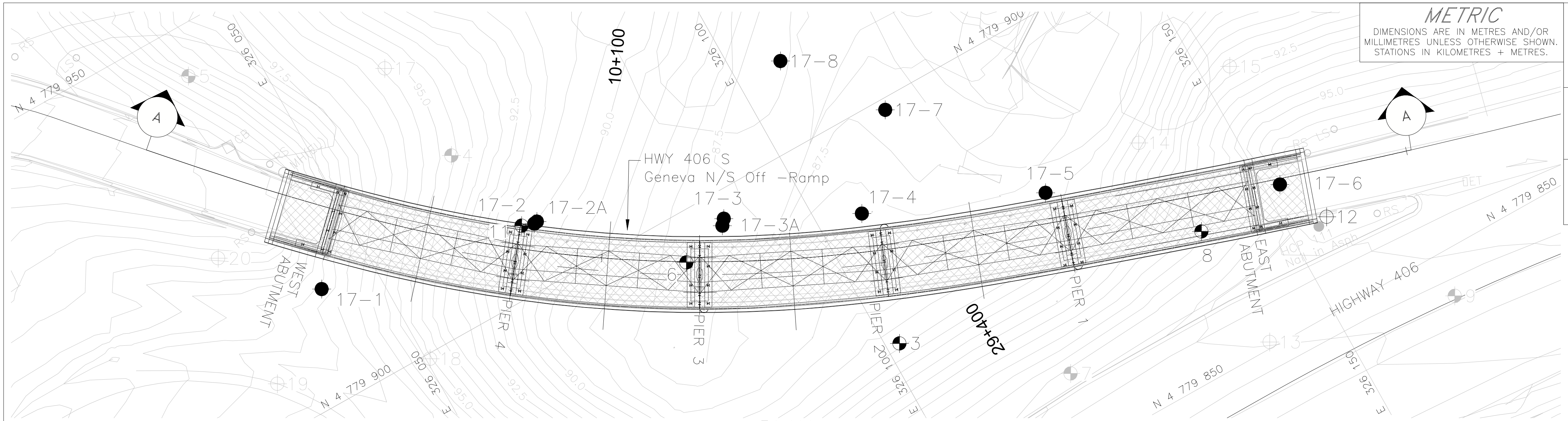
SHEET

HIGHWAY 406 SOUTH –
GENEVA STREET N/S RAMP BRIDGE REPLACEMENT

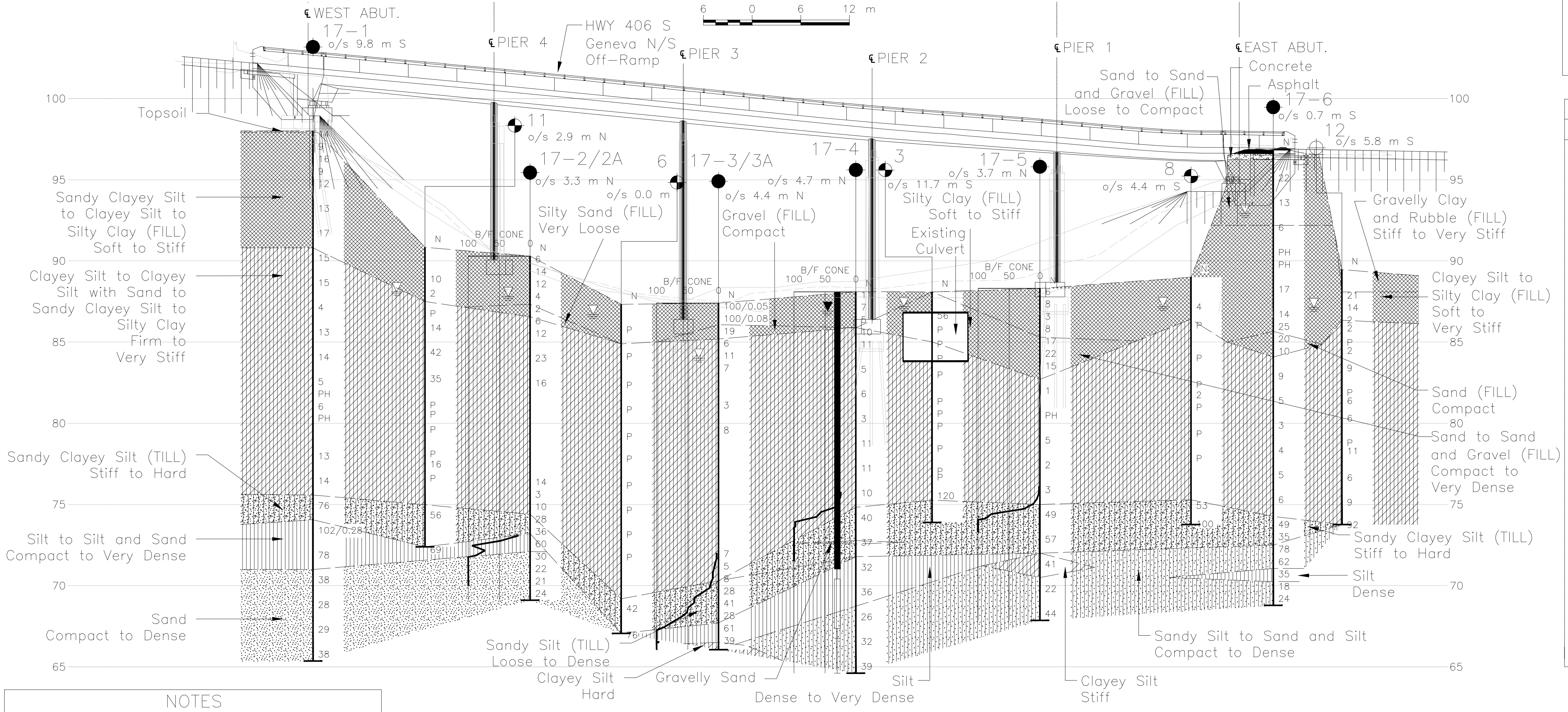
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER

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



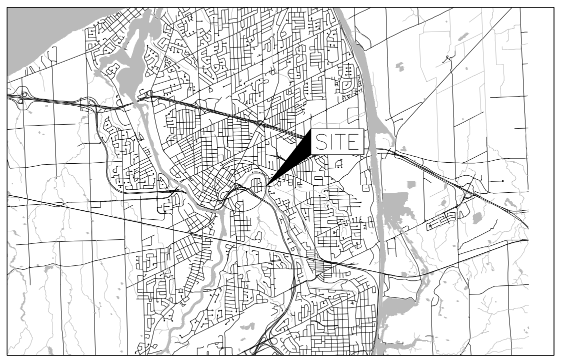
PLAN
SCALE
6 0 6 12 m



CROSS SECTION A – A'

SCALE HORIZONTAL
6 0 6 12 m

SCALE VERTICAL
3 0 3 6 m



KEY PLAN
SCALE
2 0 2 4 km

BOREHOLE CO–ORDINATES NAD83 ZONE 10			
No.	ELEVATION	NORTHING	EASTING
17–1	98.0	4779913.3	326043.0
17–2	90.3	4779907.6	326069.7
17–2A	90.5	4779907.6	326070.1
17–3	87.4	4779896.7	326090.3
17–3A	87.7	4779896.0	326089.7
17–4	88.1	4779889.0	326105.4
17–5	88.3	4779880.2	326126.3
17–6	96.8	4779867.0	326151.9
17–7	87.2	4779898.7	326114.1
17–8	86.9	4779910.2	326105.8
3	88.1	4779872.8	326101.6
4	90.5	4779919.8	326064.9
5	96.3	4779944.1	326041.6
6	87.3	4779894.3	326083.7
7	88.8	4779859.3	326118.1
8	89.0	4779866.7	326140.7
9	89.7	4779844.5	326163.9
11	90.8	4779908.1	326068.2
12	89.5	4779860.7	326155.0
13	89.3	4779850.7	326141.3
14	88.7	4779879.9	326139.3
15	89.0	4779882.6	326153.6
17	91.1	4779933.1	326063.1
18	90.5	4779899.4	326050.5
19	0.0	4779905.9	326032.6
20	91.4	4779922.9	326033.8

NOTES

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. Hwy406–9str_bgd.dwg and Hwy406–9str_plan.dwg, received March 13, 2018. Existing groun provided in digital format by Aecom, file no. ACAD–OG–mod_20180312.dwg, received March 14, 2018. General Arrangement plan provided in digital format by AECOM, drawing file nos. S1 GeneralArr.dwg, received July 27, 2018 and 18–168_R3_GA_pt.dwg, received July 31, 2018.

J.M.A.COSTA
Oct. 16, 2018

S. McGaghran
Oct. 16, 2018

LEGEND

●	Borehole – Current Investigation
⊕	Borehole – GEOCRES 30M03–043 (October 1962)
⊕	Borehole and Dynamic Cone Penetration Test – GEOCRES 30M03–043 (May/June 1962)
▮	Seal
▯	Piezometer
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
▼	WL in piezometer May 1, 2018
▽	WL upon completion of drilling

Dynamic Cone Penetration Test
B/F CONE
100 50 0

NO.	DATE	BY	REVISION

Geocres No. 30M3–306

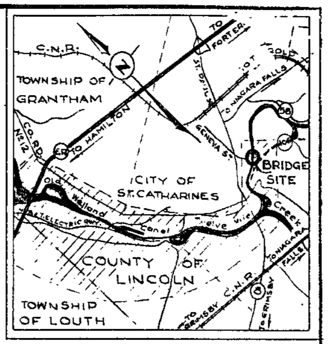
HWY. 406	PROJECT NO. 1541610		DIST. CENTRAL
SUBM'D. KN	CHKD. KN	DATE: 10/16/2018	SITE: 18–168
DRAWN: DD	CHKD. SMM	APPD. JMAC	DWG. 1

PLOT DATE: October 16, 2018
FILENAME: S:\Clients\MTO\Hwy_406\99_PROJ\1541610_Aecom_Geotech_Invest\40_PROD\0002_NS_Ramp_Bridge_Design\1541610-0002-BG-0001.dwg

APPENDIX A

Appendix A – Previous Investigation – MTO GEOCRES No. 30M03-43

CONT 79-21
SHEET 192



KEY PLAN
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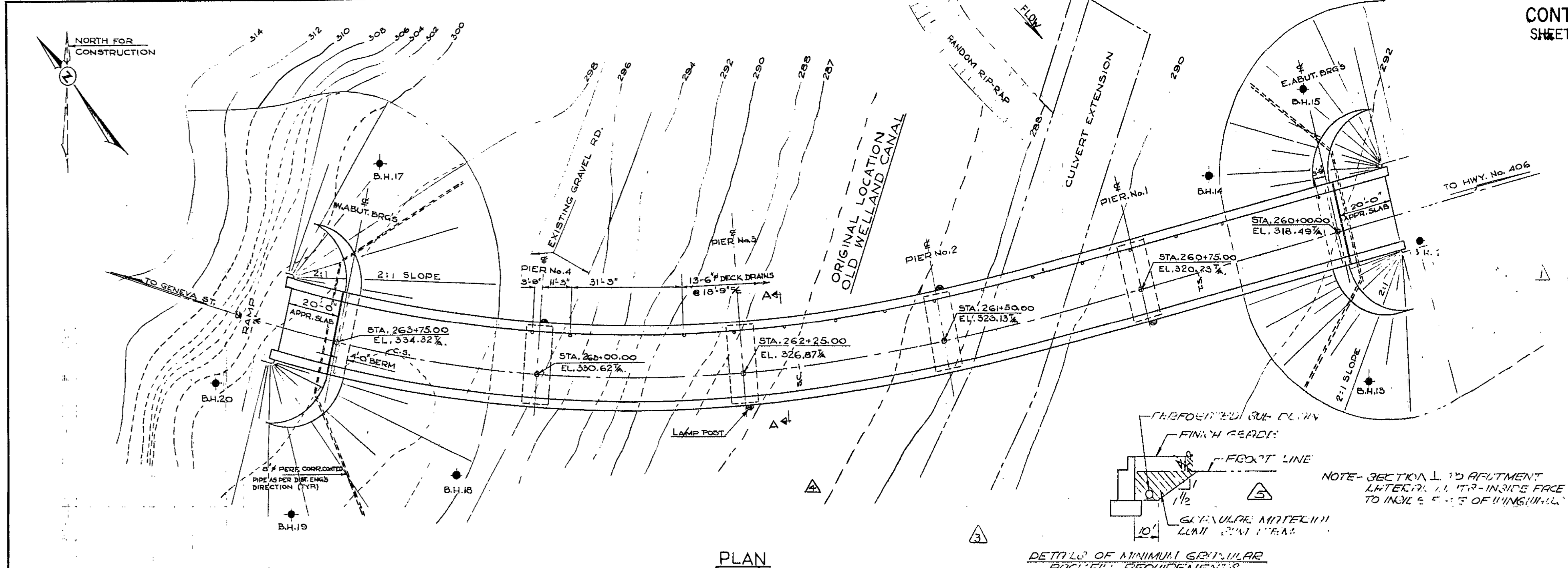
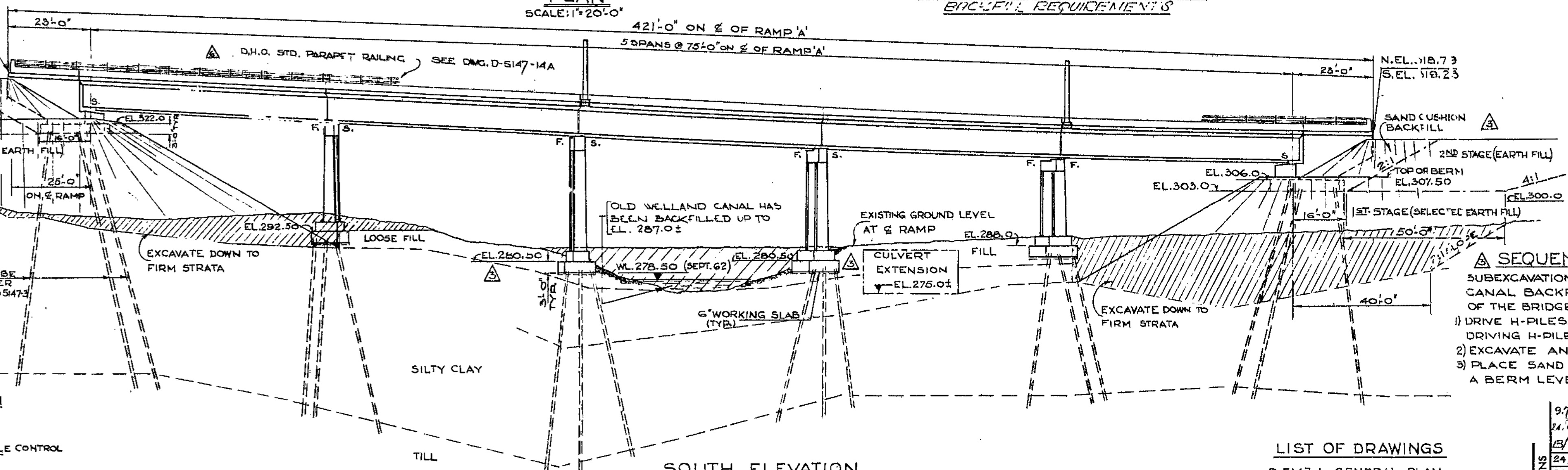
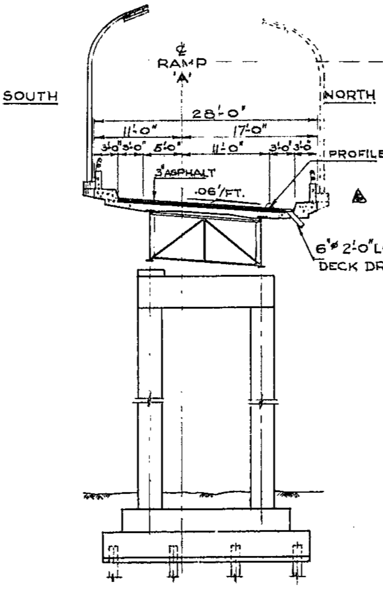


PLAN
SCALE: 1" = 20'-0"

DETAILS OF MINIMUM GRANULAR BACKFILL REQUIREMENTS



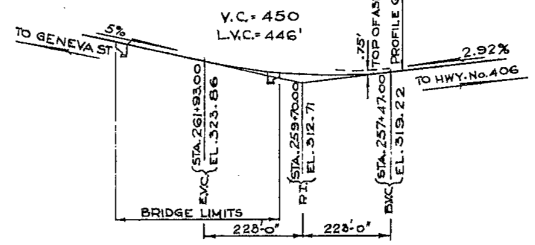
SOUTH ELEVATION
SCALE: 1" = 20'-0"



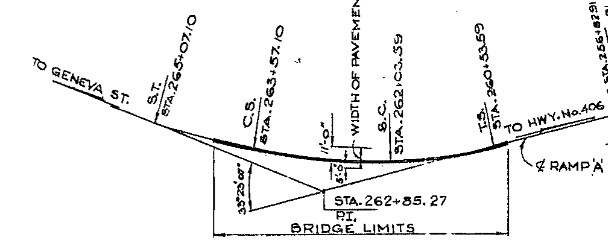
SECTION A-A
SCALE: 1" = 10'-0"

CURVE DATA

Δ = 33°23'07"		Ls = 150.00'
Δc = 16°53'07"		ϕs = 8°15'00"
D = 11°00' RT.		SUPERELEVATION DATA
R = 520.87'		S = .06 FT/FT. AT S.C. & C.S.
TS = 231.68'		S = .02 FT/FT. AT T.S. & S.T.
Lc = 153.51'		PROFILE CONTROL
Es = 24.79'		INSIDE EDGE OF PAVEMENT



PROFILE DIAGRAM (AT PROFILE CONTROL)
N.T.S.



ALIGNMENT DIAGRAM (AT ℄ RAMP 'A')
N.T.S.

NOTES

TO ENGINEER

CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DISTRICT ENGINEER.

TO CONTRACTOR.

STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM No. 9 AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER.

CONCRETE MIX

	MINIMUM STRENGTH AT 28 DAYS	MAXIMUM SIZE OF AGGREGATE
DECK	4000 P.S.I.	¾"
FOOTINGS, PIERS & ABUTMENTS	3000 P.S.I.	¾"

APPROVED ADMIXTURES SUPPLIED BY THE CONTRACTOR WILL BE ADDED TO ALL CONCRETE AS SPECIFIED BY THE EGINEER.

BORING DATA

THE COMPLETE SOIL INVESTIGATION REPORT FOR THIS STRUCTURE MAY BE EXAMINED AT THE BRIDGE OFFICE OR FOUNDATION OFFICE, DOWNSVIEW OR AT THE REGIONAL OFFICE AND AT THE HAMILTON DISTRICT OFFICE.

CLEAR COVER ON REINFORCING STEEL.

FOOTINGS-3", ABUTMENTS-3", DECK-1"

CONSTRUCTION NOTES.

ALL EXPOSED EDGES TO BE CHAMFERED 1"x1" EXCEPT AS [illegible]ED. ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE BRIDGE ENGINEER. THE GENERAL CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BRIDGE SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF PLUS OR MINUS ⅛ INCH. IF THEY ARE C[illegible] TOO HIGH THEY SHALL BE GROUND DOWN BY THE GENERAL CONTRACTOR. IF THEY ARE CAST TOO LOW THE GENERAL CONTRACTOR SHALL PROVIDE FULL BEARING SHIMS TO BRING THEM UP TO THE CORRECT ELEVATIONS. THE US[illegible] GROUT IS PROHIBITED.

THE GENERAL CONTRACTOR SHALL BE RESPONSIBLE FOR ENSURING THAT THE FINAL DECK ELEVATIONS CO[illegible] WITH THE ELEVATIONS SHOWN.

NO CONCRETE SHALL BE PLACED ABOVE THE BRID[illegible] UNTIL THE STRUCTURAL STEEL HAS BEEN PLACED.

SEQUENCE OF CONSTRUCTION

SUBEXCAVATION, 1ST STAGE FILL, CULVERT EXTENSION AND CANAL BACKFILL SHALL BE COMPLETED PRIOR TO COMMENCEMENT OF THE BRIDGE PROJECT.

1) DRIVE H-PILES. ALL H-PILES FOR PIERS TO BE PLACED PRIOR TO DRIVING H-PILES FOR ABUTMENTS.
2) EXCAVATE AND POUR FOOTINGS.
3) PLACE SAND CUSHION BACKFILL AT ABUTMENTS UP TO A BERM LEVEL PRIOR TO ERECTION OF STRUCTURAL STEEL.

DATE	BY	DESCRIPTION
9.7.69	R.T.	AS-CONSTR. NOT SUPPLIED
24.11.64	VV	STD. H RAILS CHANGED TO PARAPET RAILING.
13/7/64	WM	5 GRANULAR BACKFILL DETAIL ADDED
24/10/63	J.G.G.	4 GRAVEL ROAD REMOVED, PLAN REV.
29/8/63	A.U.	3 NOTES & SOUTH ELEVATION-PLAN-REV.
~~16/8/63~~	~~J.G.G.~~	~~2 NOTES~~
6/8/63	J.G.G.	1 NOTES ~~& ELEVATION REV~~

REVISIONS

LIST OF DRAWINGS

D 5147-1	GENERAL PLAN
D 5147-2	BORE HOLE DETAILS
D 5147-3	FOUNDATION LAYOUT
D 5147-4	SETTING DATA & ELEVATIONS
D 5147-5	ABUTMENTS
D 5147-6	PIERS
D 5147-7	STRUCTURAL STEEL & BEARING LAYOUT
D 5147-8	GIRDERS DETAILS
D 5147-9	BRACING & CONNECTION DETAILS
D 5147-10	BEARING DETAILS
D 5147-11	DECK DETAILS
D 5147-12	APPROACH SLABS & END POST DETAILS
D 5147-13	HANDRAIL & LIGHTING LAYOUT.
D 5147-14A	DETAILS OF PARAPET RAILING
D 5147-15A	BRIDGE LIGHTING DETAIL
D 5147-16	STEEL SCHEDULE (DECK & FOOTINGS)
D 5147-17	—— do —— (ABUT & PIERS)

REFERENCE DRAWINGS

SITE PLAN
PROFILE - CONTR. No. 62-316 SHEET ㊳
PLAN ——— ㉛
SOILS REPORT BA 1463 & 1463 A

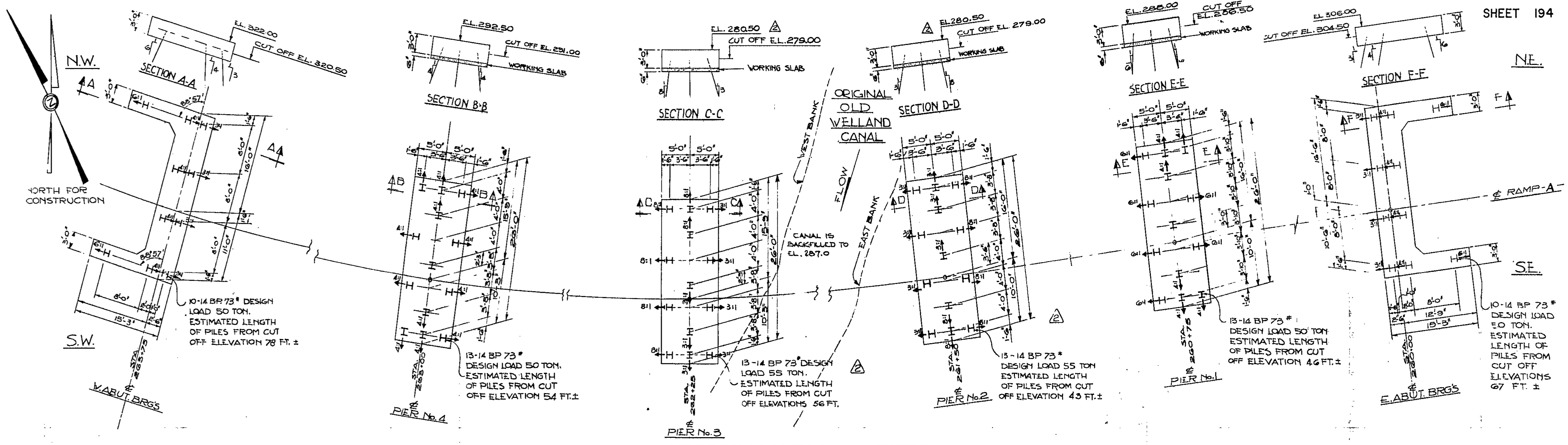
DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

BRIDGE OVER OLD WELLAND CANAL
CONNECTION BETWEEN HWY. No. 406 AND GENEVA ST.
IN ST. CATHARINES
KING'S HIGHWAY No. —— DIST. No. 4
CO. LINCOLN
~~THE~~ CITY OF ST. CATHARINES LOT —— CON. ——

GENERAL PLAN

APPROVED	BRIDGE ENGINEER	SITE No.	19-168	W.P. No.	274-62
DESIGN	A.U.	CHECK	G.S.	CONTRACT Nos.	61-05 / 64-295
DRAWING	H.M.	CHECK	G.S.		
DATE	MAY 63	LOADING	H 20 S 16	DRAWING No.	D-5147-1

SHEET 194



FOUNDATION LAYOUT SHOWING H-PILES

SCALE: 1/8" = 1'-0"

NOTES:

SETTING:

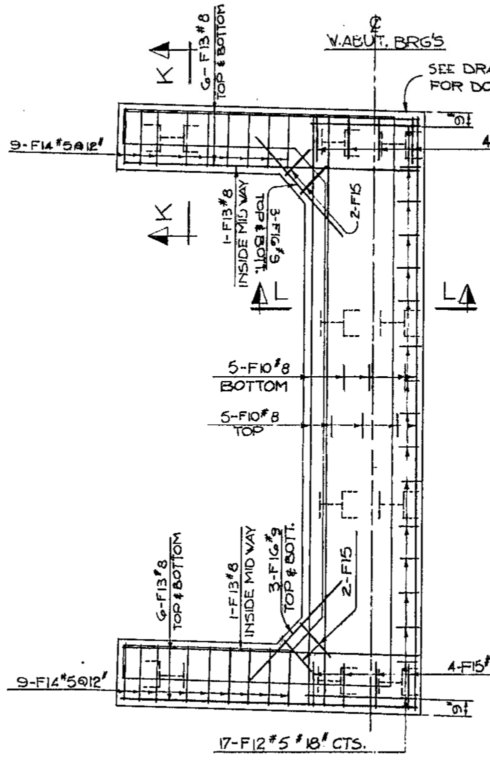
1. PIERS & ABUTMENTS TO BE SET IN FIELD USING DATA GIVEN ON DRAWING D-5147-4 TABLE 2. THESE POINTS SHALL NOT BE SET BY USE OF STATIONS ON ℄ OF RAMP A. STATIONS GIVEN FOR FUTURE REFERENCE ONLY.
2. ALL ℄ PIERS AND ℄ ABUTMENTS BRG'S ARE RADIAL.
3. LOCATION OF PIERS #2 & #3 SHALL BE CHECKED TO BE AS SHOWN ON DRAWINGS IN RELATION TO THE OLD WELLAND CANAL.

SOIL DATA AND PILING

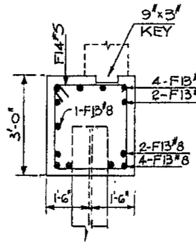
⚠ 4. FOR SOILS INFORMATION CONTRACTOR SHALL SEE THE SOILS REPORT. THE FOLLOWING POINTS ARE MENTIONED IN THE REPORT
- PRESENCE OF ⚠ BOULDERS AT PIERS #2 & #3 (REMOVE BEFORE PILING)
- NEED FOR TEMPORARY SHEET PILING FOR DEWATERING PURPOSES

5. ALL "H" PILES TO BE DRIVEN TO A PRACTICAL REFUSAL AS DETERMINED BY HILEY FORMULA (SEE D.H.O. STD. BD 16-3,4) ALL LOADS ARE VERTICAL.
SYMBOL: $H \xrightarrow{8:1}$ BATTER IN DIRECTION OF ARROW.

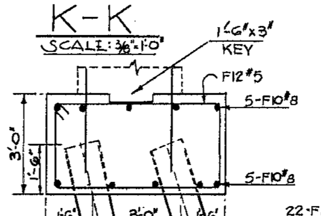
6. BACKFILLING OF EXCAVATION IN VICINITY OF PIER FOOTINGS TO BE IN 6" COMPACTED LAYERS UP TO TOP OF FOOTING ELEVATIONS.



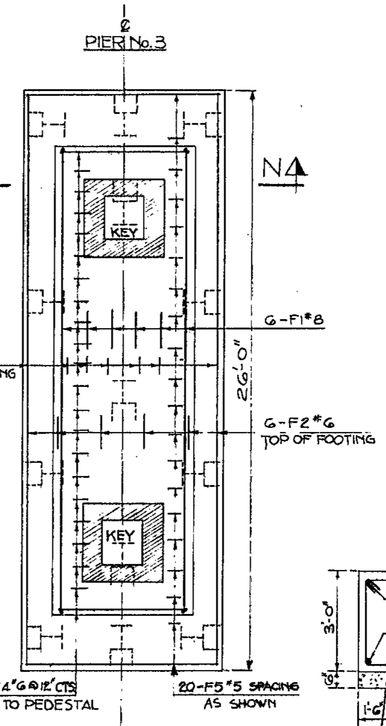
PLAN OF W. ABUTMENT
SHOWING FOOTING REINFORCEMENT
(REINFORCEMENT OF E. ABUT. SIMILAR)
SCALE: 1/4" = 1'-0"



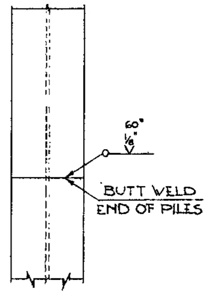
K-K
SCALE: 3/8" = 1'-0"



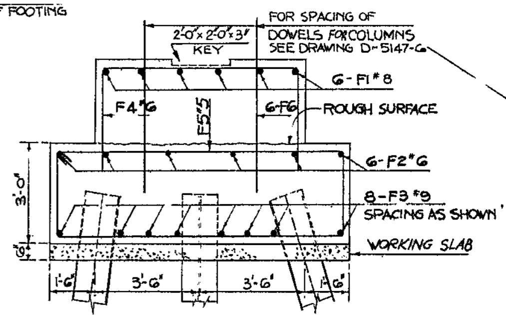
L — L
SCALE: 3/8" = 1'-0"



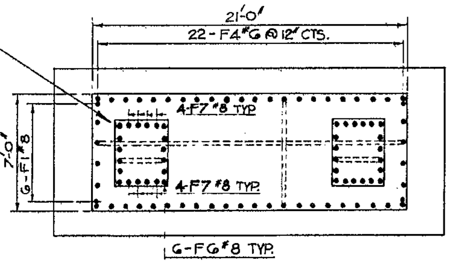
PLAN OF PIER #3
SHOWING FOOTING REINFORCEMENT
SCALE: 1/4" = 1'-0"



PILE SPLICE
SCALE: 3/4" = 1'-0"



N — N
SCALE: 3/8" = 1'-0"



DOWELS TO FOOTINGS
SCALE: 3/16" = 1'-0"

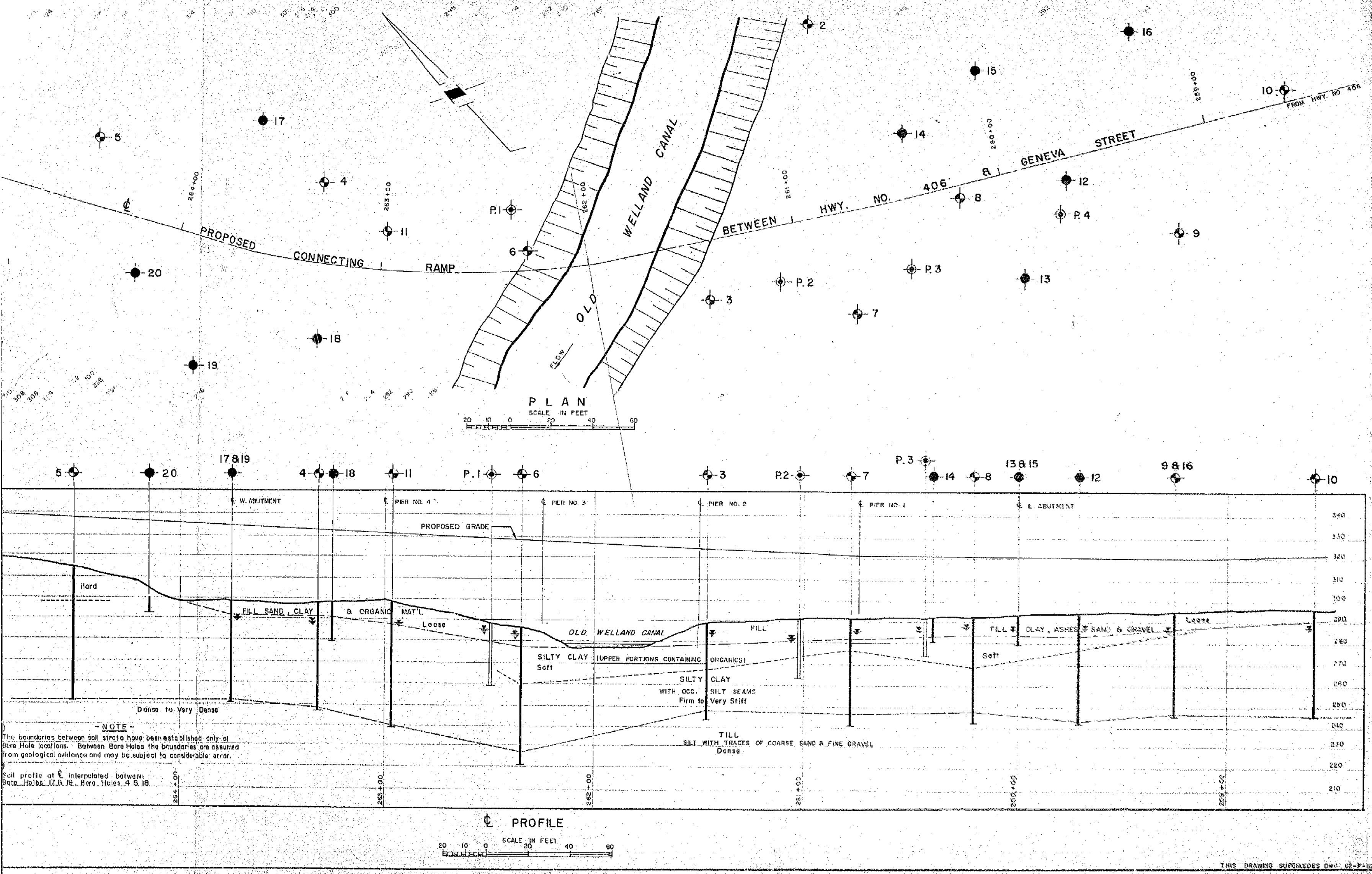
REVISIONS DATE	BY	DESCRIPTION
9.7.69	R.T.	REVISED AS-CONSTR.
24 10 63	JGG	⚠ WORD 'SURFACE' REMOVED
29/8/63	A.U.	⚠ REV ELEVATION NOTES - PIER 3 - PIER 2
6/8/63	JGG	⚠ NOTES REV.

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

BRIDGE OVER OLD WELLAND CANAL
CONNECTION BETWEEN HWY. No. 406 AND GENEVA ST. IN ST. CATHARINES
KING'S HIGHWAY No. ______ DIST. No. 4
CO. LINCOLN
~~TWP.~~ CITY OF ST. CATHARINES LOT ______ CON. ______

FOUNDATION LAYOUT

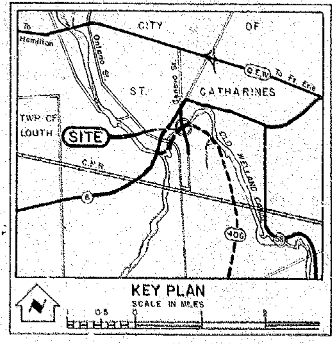
APPROVED	A.W. Lay BRIDGE ENGINEER			SITE No. 19-168	W.P. No. 274-62	
DESIGN	A U	CHECK	G.S.	CONTRACT Nos.	ST. STEEL 64-05	GENERAL 64-295
DRAWING	N T	CHECK	G.S.			
DATE	MAY. 63	LOADING	H-20 S-16	DRAWING No.	D-~~5147-3~~	



- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

Soil profile at ℄ interpolated between Bore Holes 17 & 19, Bore Holes 4 & 18.



KEY PLAN
SCALE IN MILES

LEGEND

●	Bore Hole
○	Cone Penetration Hole
◐	Bore & Cone Penetration Hole
◉	Piezometer Hole
▽	Water Levels established at time of field investigation June 1962

NO.	ELEVATION	STATION	OFFSET
2	289.0	260+78	60'RT.
3	289.0	261+46	28'LT.
4	307.0	263+52	38'RT.
5	316.0	264+49	31'RT.
6	288.8	262+35	9'RT.
7	291.3	260+70	48'LT.
8	292.0	260+21	6'LT.
9	294.3	259+28	49'LT.
10	295.8	258+59	5'RT.
11	298.0	262+97	18'RT.
12	293.5	259+70	10'LT.
13	293.0	260+00	50'LT.
14	291.0	260+40	30'RT.
15	292.0	260+00	80'RT.
16	294.0	259+25	50'RT.
17	299.0	263+75	60'RT.
18	297.0	263+25	35'LT.
19	296.0	263+75	60'LT.
20	300.0	264+15	25'LT.

NO.'S 1 & 21 NOT SHOWN ON PLAN

THIS DRAWING SUPERSEDES DWG. 62-F-62A

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

OLD WELLAND CANAL
AND
HWY. NO. 406 - GENEVA ST. CONNECTION
ST. CATHARINES

ORIGINATED G. CHERRINGTON	DISTRICT NO. 4	DATE OCT. 12, 1962
DRAWN F. CLARK	W.P. NO. 128-58-1	JOB NO. 62-F-62
CHECKED	CONT. NO.	DRAWING NO. 62-F-62 E
APPROVED		

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS : -

CONSISTENCY	'N' BLOWS / FT.	c LB. / SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUPE SAMPLE		

P.H.	SAMPLE ADVANCED HYDRAULICALLY
P.M.	SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \dfrac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
$_s$	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \dfrac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \dfrac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \dfrac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \dfrac{-\Delta e}{(1+e)\Delta\sigma'}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \dfrac{\Delta e}{\Delta \log_{10} \sigma'}$
T_v	TIME FACTOR $= \dfrac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
	IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
	IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	= 3·1416
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

FORM OB - ML - 126 REV. 1961
61-4391

OFFICE REPORT ON SOIL EXPLORATION

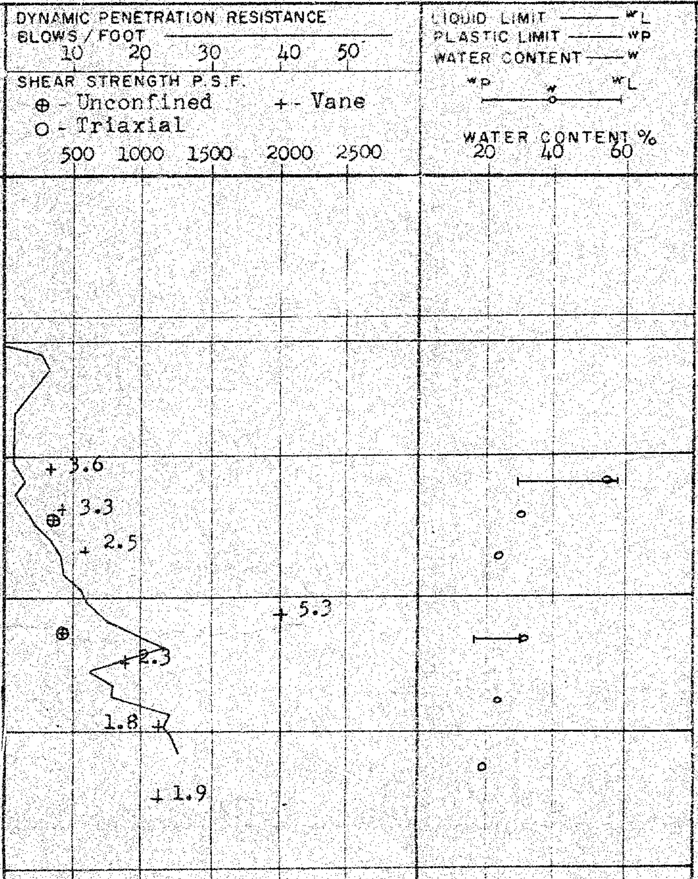
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 62-F-62
W.P. 126-58(-1)
DATUM 288.3'
LOCATION Sta. 260+75 (90' Rt.)
BORING DATE May 7, 1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY B.K.
COMPILED BY B.K.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
						290
289.0	Groundlevel. Topsoil					
	Fill-of sand silt and clay pockets grey and brown some org. material.		1	TW	P	
279.8	Very loose.		2	SS	2	280
8.5	Grey-black silty clay with org. matter.		3	TW	P	
273.3	Soft to med. stiff.		4	TW	P	
15.0	Grey to grey-brown silty clay with seams of silty sand.		5	TW	P	270
	Soft to stiff.		6	TW	P	
			7	TW	P	260
255.3			8	TW	P	
33.0	End of borehole.					250



BULK DENSITY γ P.C.F.
121.0

REMARKS
wl 282.0

FORM OB-ML-126 REV. 1961
61-4391

OFFICE REPORT ON SOIL EXPLORATION

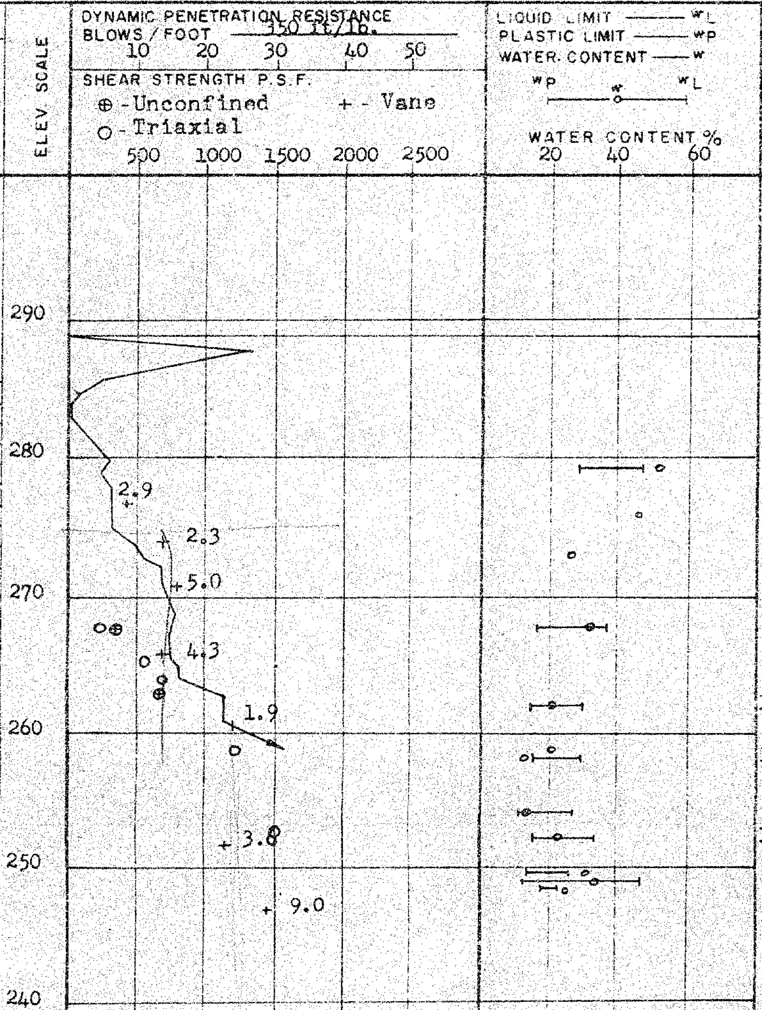
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 62-F-62
LOCATION Sta. 261+46 (28' Lt.)
ORIGINATED BY B.K.
W.P. 126-58(-1)
BORING DATE May 8, 1962.
COMPILED BY B.K.
DATUM 288.9ft.
BOREHOLE TYPE Washboring.
CHECKED BY M.D.

SOIL PROFILE		SAMPLES				
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
289.0	Groundlevel Topsoil					
	Fill of sand, silt and clay pockets, some org. material and gravel, grey and brown	1	SS	56		wl 286.0
		2	TW	P		
278.9 10.0	Grey-black silty clay with org. material Soft to med. stiff.	3&3A	TW	P		
		4	TW	P		
272.9 16.0	Silty clay with seams of silty sand. Soft to stiff. Grey to grey brown.	5	TW	P		
		6	TW	P	117.0	
		6A	TW	P		
		7&7A	TW	P	129.0	
		8&8A	TW	P	142.0	
		9	TW	P		
		10	TW	P	131.0	
		11	SS	120		
246.9 42.0	Red glacial till V. dense.					
242.4 46.5	End of borehole.					



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 4

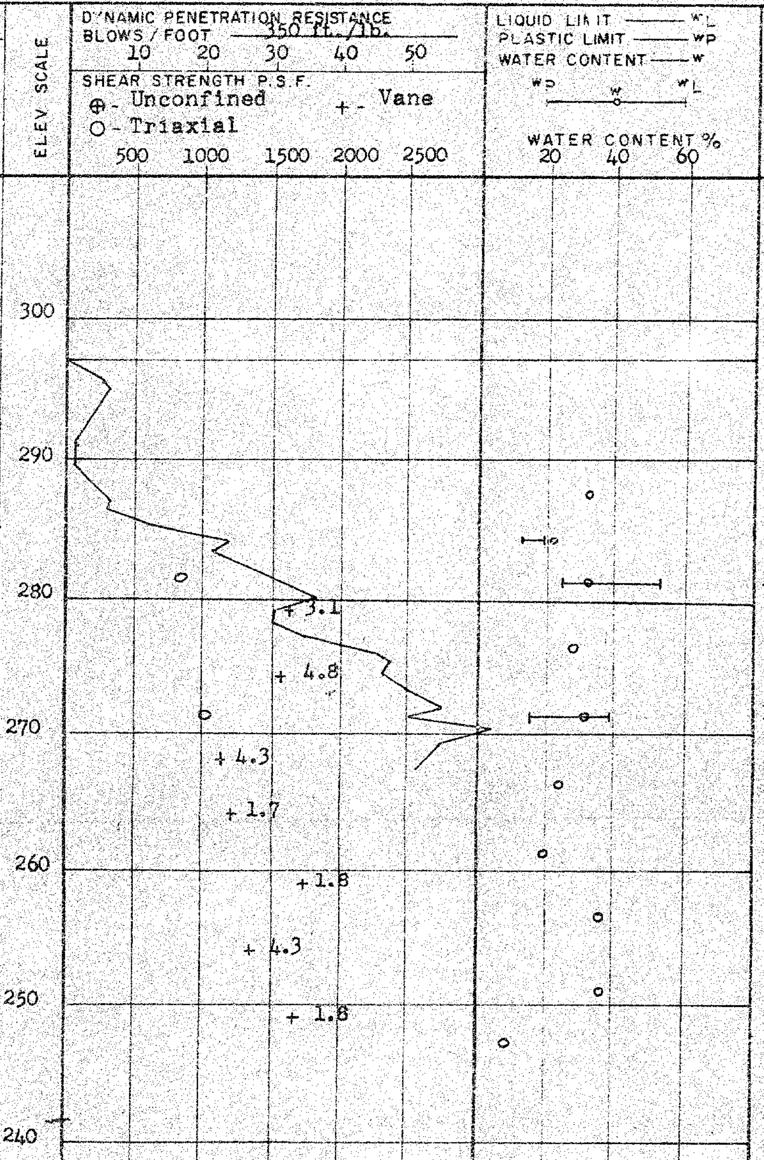
FOUNDATION SECTION

JOB 62-F-62
LOCATION Sta. 263+32 (38' Rt. ¢)
ORIGINATED BY B.K.
W.P. 126-58-1
BORING DATE May 9, 1962.
COMPILED BY B.K.
DATUM 297.0'
BOREHOLE TYPE Washboring.
CHECKED BY M.D.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION
297.0	Groundlevel Topsoil
	Fill of sand, silt & clay pockets, some org. mat'l. Gravel & cinders; brown to red brown.
287.0	Loose to very loose.
10.0	Grey to grey-brown silty clay with varying amounts of silt and sand, hard to stiff. Layer of red-brown gravel, sandy firm clay between elev 285.0 and 281.0'
248.0	
246.5	Red till-Very dense.
50.5	End of borehole.

SAMPLES NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.
1	SS	10	
2	SS	3	
3	SS	5	
4	TW	32	
5	TW	P	115
6	TW	P	
7	TW	P	123
8	TW	P	
9	TW	P	
10	TW	P	
11	TW	P	
12	SS	115	

REMARKS
wl 289.0



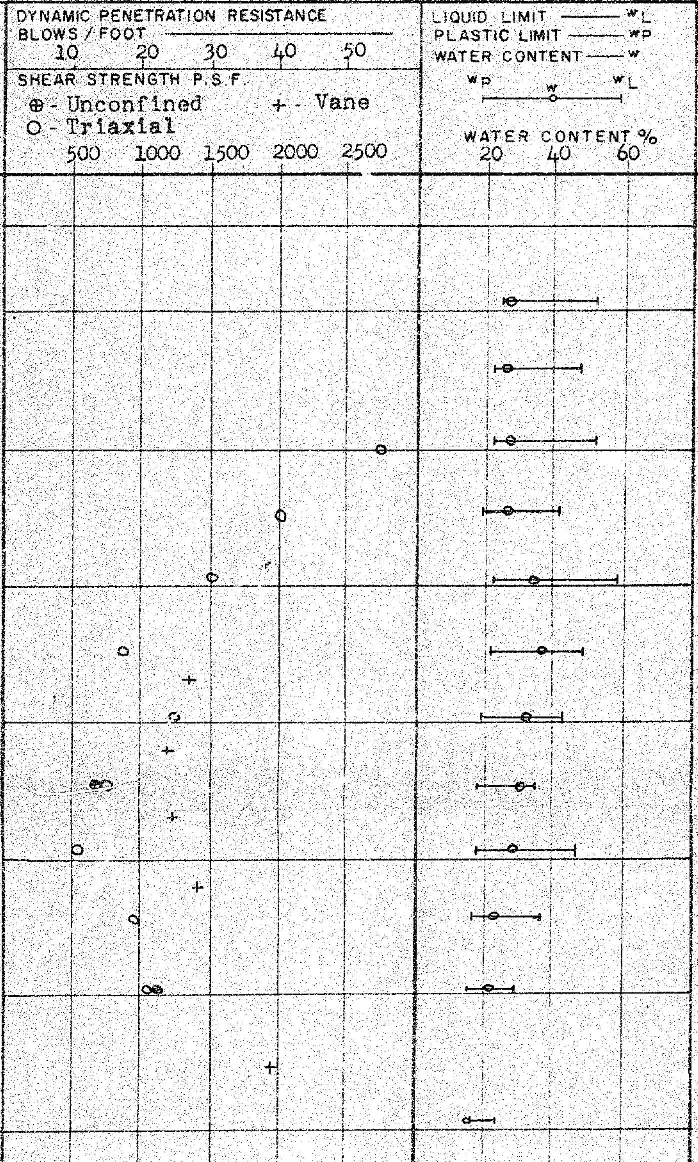
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 62-F-62
W.P. 126-58-1
DATUM Geodetic
LOCATION Sta. 264+49 (31' Rt.)
BORING DATE June 20,1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY G.C.
COMPILED BY G.C.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES					
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS
316.0	Groundlevel							
0.0	Brown clay to silty clay. Hard to stiff.		1	SS	38	310		
			2	SS	29			
			3	SS	24	300		
			4	SS	17			
291.0 25.0	Gray clay to silty clay. Med. stiff to stiff.		5	SS	10	290		
			6	TW	P			
			7	TW	P	280		
			8	TW	P			
			9	TW	P	270	109	
			10	TW	P		125	
			11	TW	P	260	134	
251.0 249.5	Till (Dense)		12	SS	34	250	134	
66.5	End of borehole.							



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OFFICE REPORT OF SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

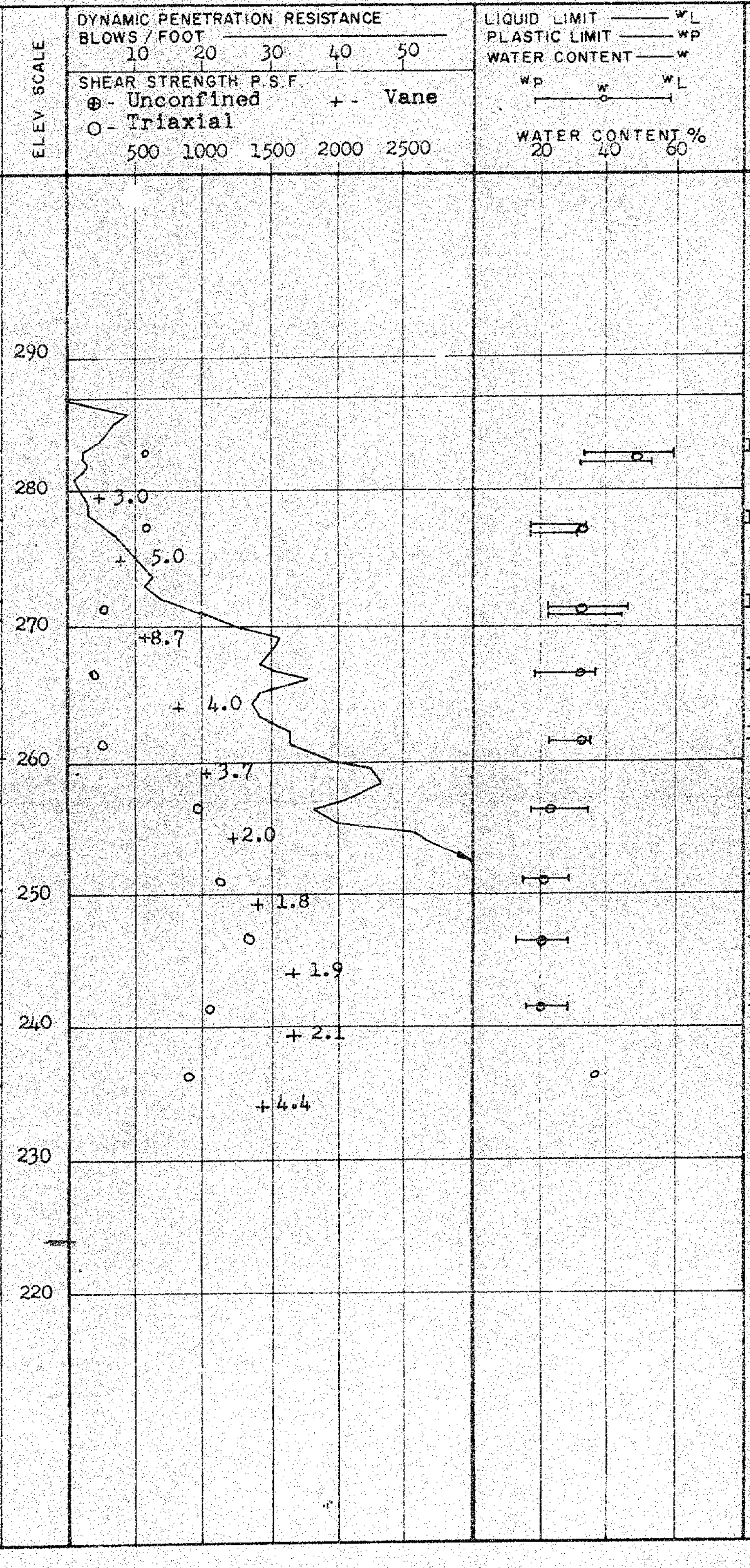
JOB 62-F-62
W.P. 126-58-1
DATUM 286.5'
LOCATION Sta. 262+35 (7' Rt.)
BORING DATE June 19, 1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY B.K.
COMPILED BY B.K.
CHECKED BY M.D.

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
286.5 0.0	Groundlevel Fill
▽282.0 4.5	Ashes, sand clay & gravel.
278.5 8.0	Gray silty clay mixed with organics. Soft to med. stiff.
260.0 26.5	Brown silty clay (stiff) changing to gray clayey silt. Med. stiff to stiff.
227.0 59.5	Till Very dense.
220.0 66.5	End of borehole.

SAMPLES

NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
1	TW	P	100	106% organics
2	TW	P	126	2.3% organics
3	TW	P	116.5	0.7% organics
4	TW	P	116	
5	TW	P	122	
6	TW	P	126	
7	TW	P	131	
8	TW	P	132	
9	TW	P	132	
10	TW	P		
11	SS	42		
12	SS	76		



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MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 7

FOUNDATION SECTION

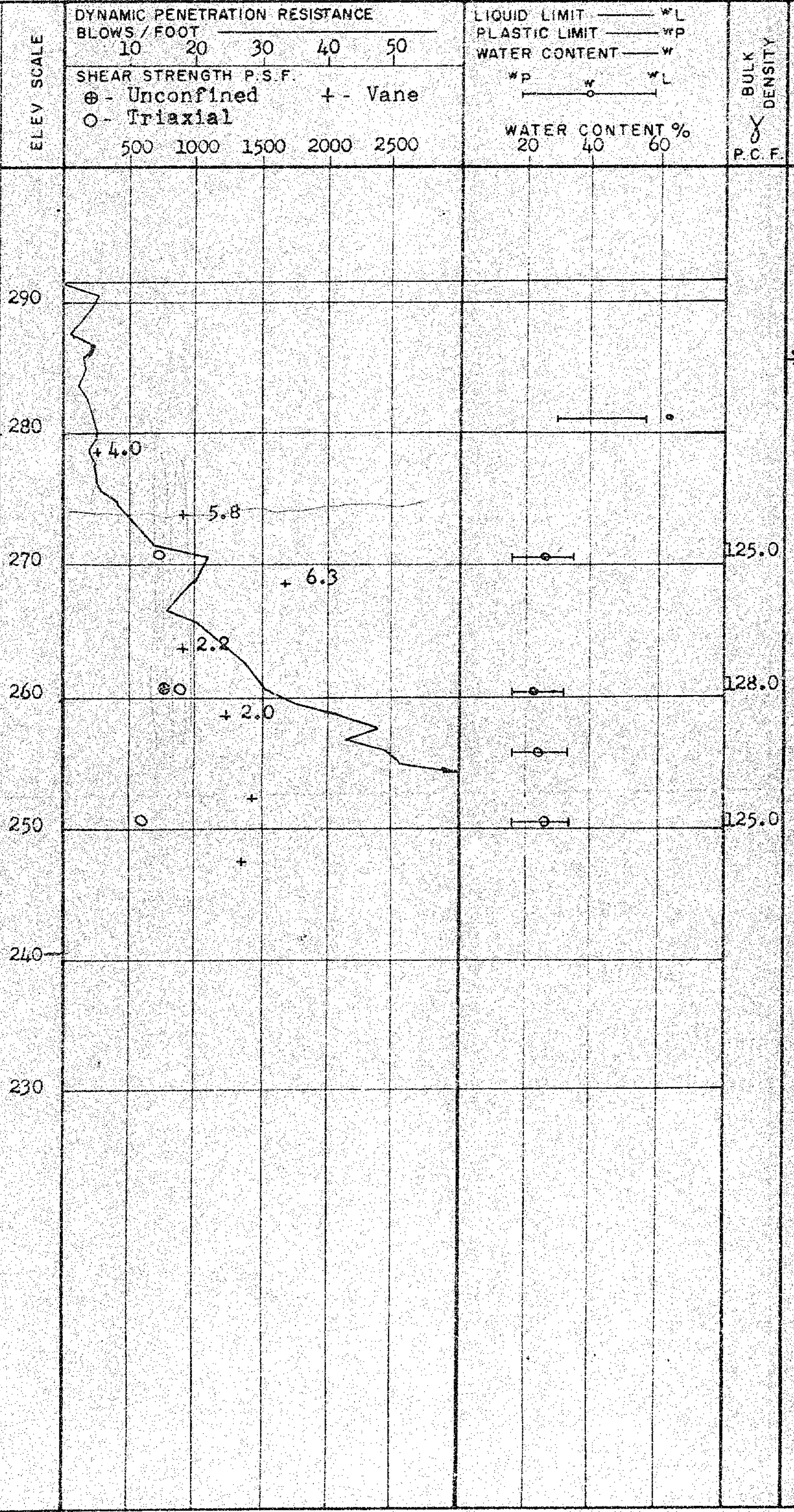
JOB 62-F-62
W. P. 126-58-1
DATUM 291.3'
LOCATION Sta. 260+79 (48' Lt.)
BORING DATE June 21, 1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY B.K.
COMPILED BY B.K.
CHECKED BY M.D.

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
291.3	Groundlevel
0.0	Fill Ashes, sand and gravel Very loose.
281.8 9.5	Clay with organics. Soft.
278.3 13.0	Silty clay. Soft to med. stiff.
268.3 23.0	Clayey silt. Med. stiff to stiff. (Occasional silt seams).
245.8 45.5	Till Med. dense to very dense.
239.6 51.5	End of borehole.

SAMPLES

NUMBER	TYPE	BLOWS/FOOT
1	SS	2
2	TW	P
3	TW	P
4	TW	P
5	TW	P
6	TW	P
7	TW	P
8	TW	P
9	TW	P
10	SS	22
11	SS	120



REMARKS

wl 285.3

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MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 8

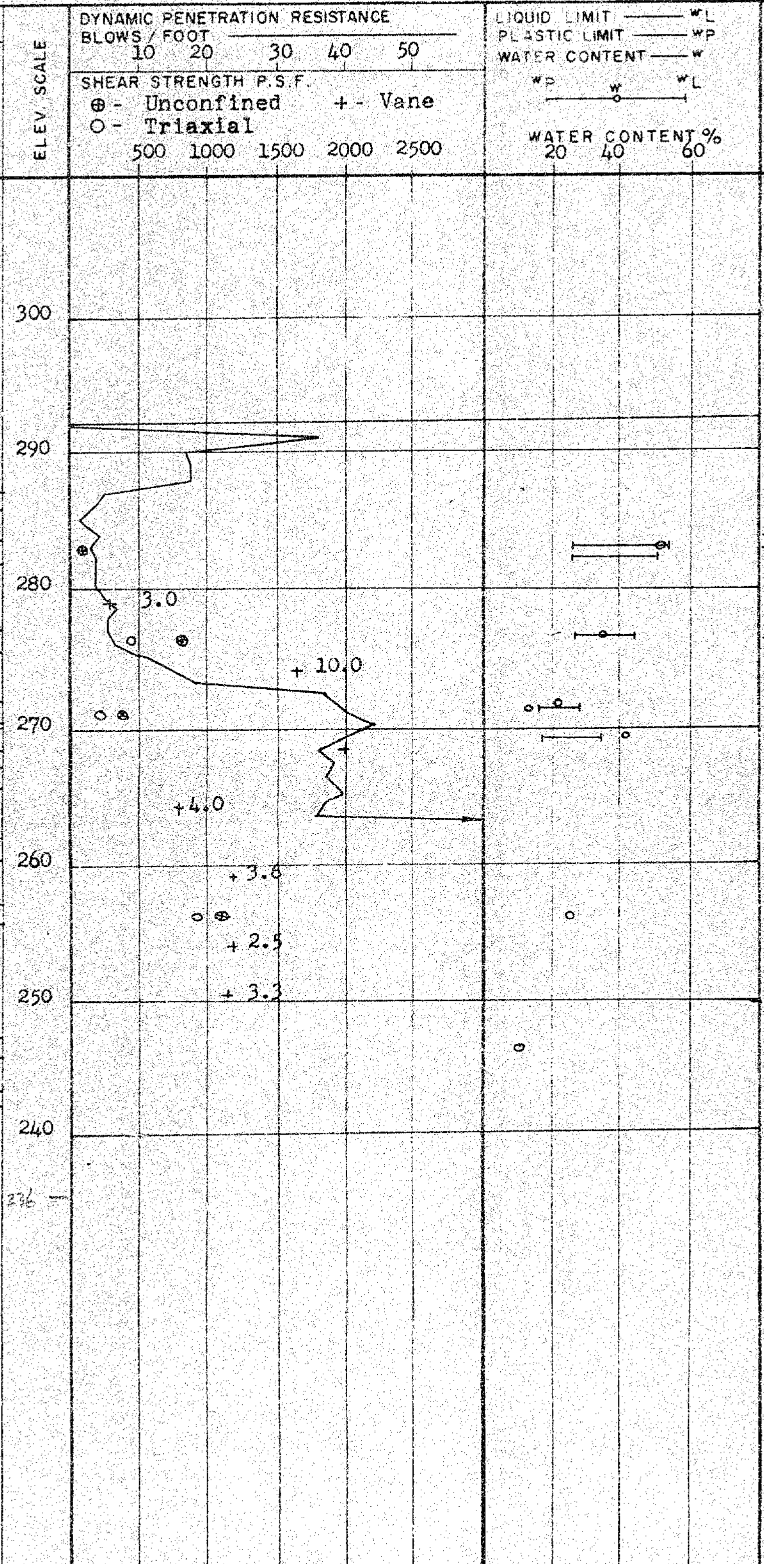
FOUNDATION SECTION

JOB 62-F-62
W.P. 126-58-1
DATUM 292.0'

LOCATION Sta. 260+21 (6' Lt.)
BORING DATE June 22, 1962.
BOREHOLE TYPE Washboring

ORIGINATED BY B.K.
COMPILED BY B.K.
CHECKED BY M.D.

SOIL PROFILE ELEV. DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT
292.0	Groundlevel			
0.0	Fill Clay, ashes, sand Loose.	1	SS	4
283.5 8.5	Clay with organics. Very soft.	2	TW	P
277.0 15.0	Gray clayey silt with silt seams. Soft to very stiff.	3	TW	P
		4	TW	P
		5	SS	2
		6	TW	P
		7	TW	P
		8	TW	P
247.0 45.0	Till Very dense.	9	SS	53
		10	SS	100
242.0 50.0	End of borehole.			



BULK DENSITY γ P.C.F.	REMARKS
	wl 286.0
102.0	Air Dry Oven dry.
106.0	
125	Brown.
128	

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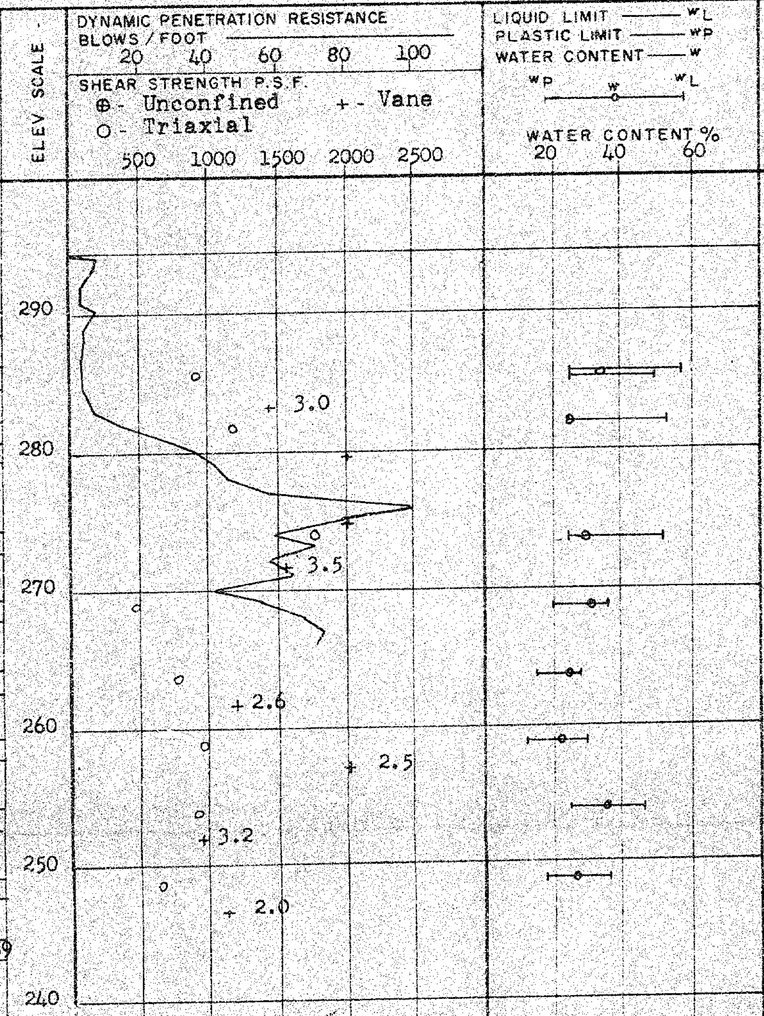
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 9

FOUNDATION SECTION

JOB 62-F-62
W.P. 128-58-1
DATUM Geodetic
LOCATION Sta. 259+25 (49' Lt.)
BORING DATE June 18&19, 1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY G.C.
COMPILED BY G.C.
CHECKED BY M.D

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT
294.3	Groundlevel			
0.0	Fill Ashes, sand, gravel and clay, Very loose.	1	SS	2
288.3 6.0	Brown to grey-brown silty clay with occasional silt seams. Very stiff to med. stiff.	2	TW	P
		3	TW	P
		4	SS	26
		5	TW	P
		6	TW	P
		7	TW	P
		8	TW	P
		9	TW	P
		10	TW	P
245.3 49.0	Till Very dense.	11	SS	159
243.3 51.0	End of borehole.			



BULK DENSITY γ P.C.F.	REMARKS
113.0	▽ wl 286.3 3.8% organic content
125.0	0.4% org. content.
122.0	
131.0	
129.0	
118.0	

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DEPARTMENT OF HIGHWAYS - ONTARIO
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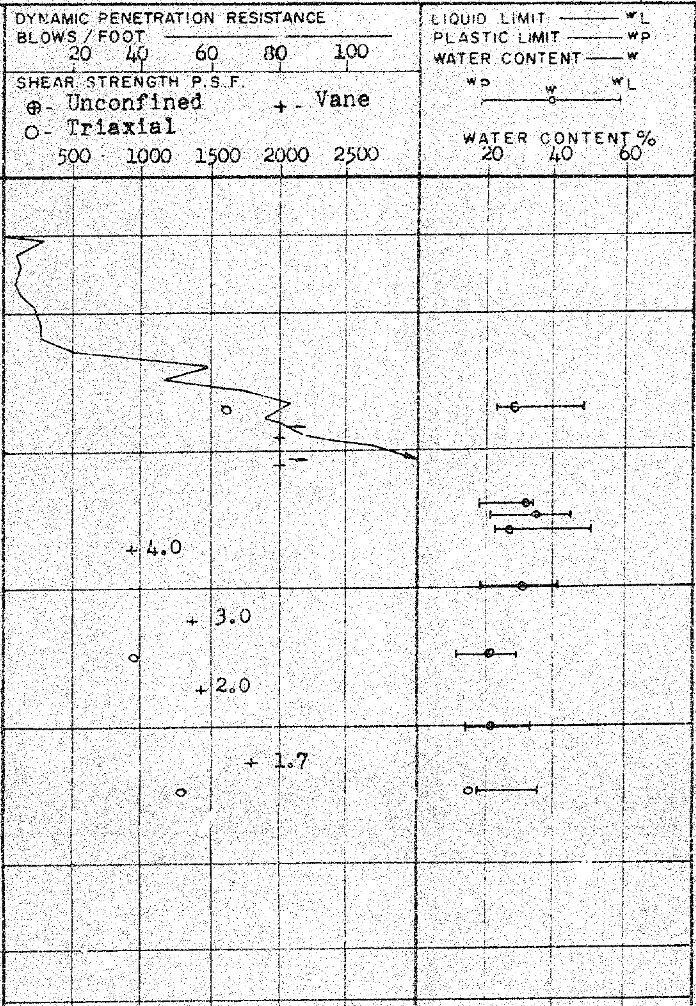
RECORD OF BOREHOLE NO. 10

FOUNDATION SECTION

JOB 62-F-62
W.P. 126-58-1
DATUM Geodet.
LOCATION Sta. 258+59 (5' Rt.)
BORING DATE June 21, 1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY G.C.
COMPILED BY G.C.
CHECKED BY M.D

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS
295.8	Groundlevel							
0.0	Fill							
292.3	Sand, gravel & clay.							
3.5	Brown silty clay. Very stiff to med. stiff.		1	SS	22			
			2	SS	21			⊻ wl 284.3
			3	TW	P		122	
277.8								
18.0	Gray brown silty clay. Very stiff to med. stiff.							Top Middle Bottom
			4	TW	P			
			5	TW	P		115	
			6	TW	P		130.0	
			8	TW	P	260	129.0	
			9	TW	P		127.0	
						250		
244.8								
51.0	Till							
52.0						240		



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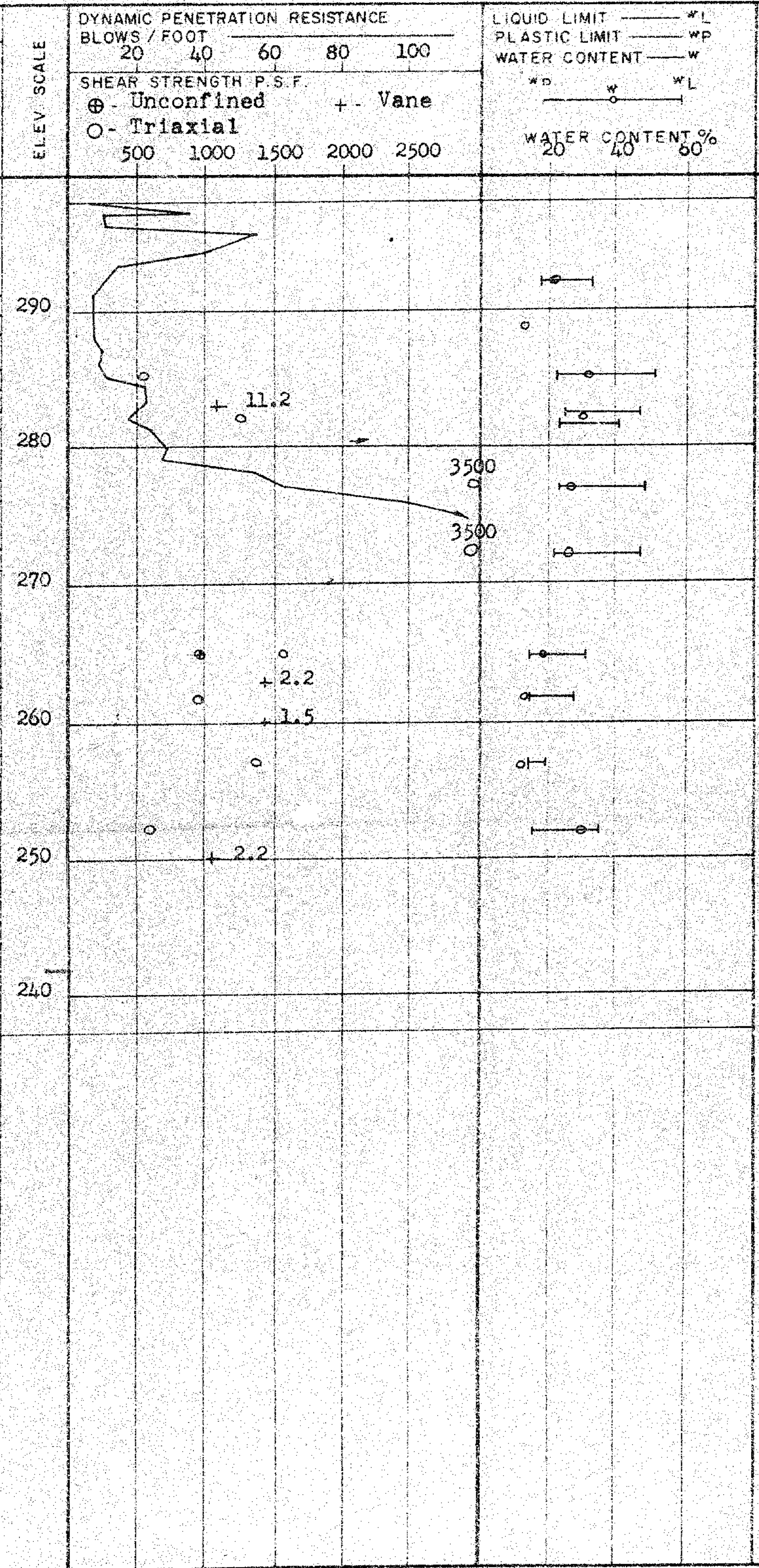
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 11

FOUNDATION SECTION

JOB 62-F-62
W. P. 126-58-1
DATUM Geodetic
LOCATION Sta. 262+97 (18' Rt.)
BORING DATE June 21, 1962.
BOREHOLE TYPE Washboring.
ORIGINATED BY G.C.
COMPILED BY G.C.
CHECKED BY M.D.

SOIL PROFILE		SAMPLES			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT		
298.0	Groundlevel					
0.0	Fill. Sand, clay, gravel & organics. Loose to very loose.	1	SS	10		
		2	SS	2		wl 289.0
287.0 11.0	Gray clay mixed with organics. Med. stiff to stiff.	3	TW	P	118.0	
		4	TW	14	119.0	Air Dry Oven Dry
280.0 18.0	Brown clay. Very stiff.	5	SS	42	130.0	
		6	SS	35		
270.0 28.0	Gray clayey silt with occasional silt seams. Stiff to med. stiff.					Lost
		7	TW	P		
		8	TW	P	133.0	
		9	TW	P	129.0	
		10	TW	P	142.0	
		10A	SS	16		
		11	TW	P	107.0	
246.0 52.0	Till Very dense.	12	SS	56		Lost
		13	SS	69		
237.5 60.5	End of borehole.					



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DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO 12

FOUNDATION SECTION

JOB 62-F-62
W. P.
DATUM
LOCATION Sta. 259 + 70 (10' Lt.)
BORING DATE 27th Sept. and 1st Oct. 1962
BOREHOLE TYPE Augerhole
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION
293.5	Ground level
0.0	Grey-brown gravelly clay fill
289.0 / 4.5	Rubble fill (ash, clay, gravel sand, organics)
283.2 / 10.3	Soft black organic clay
281.5 / 12.0	grey
23.0	grey with traces of red
266.5 / 27.0	silty Clay
40.0	grey with red seams (varved)
43.0	very silty, many silt seams
242.5 / 51.0	Reddish brown (silt till)

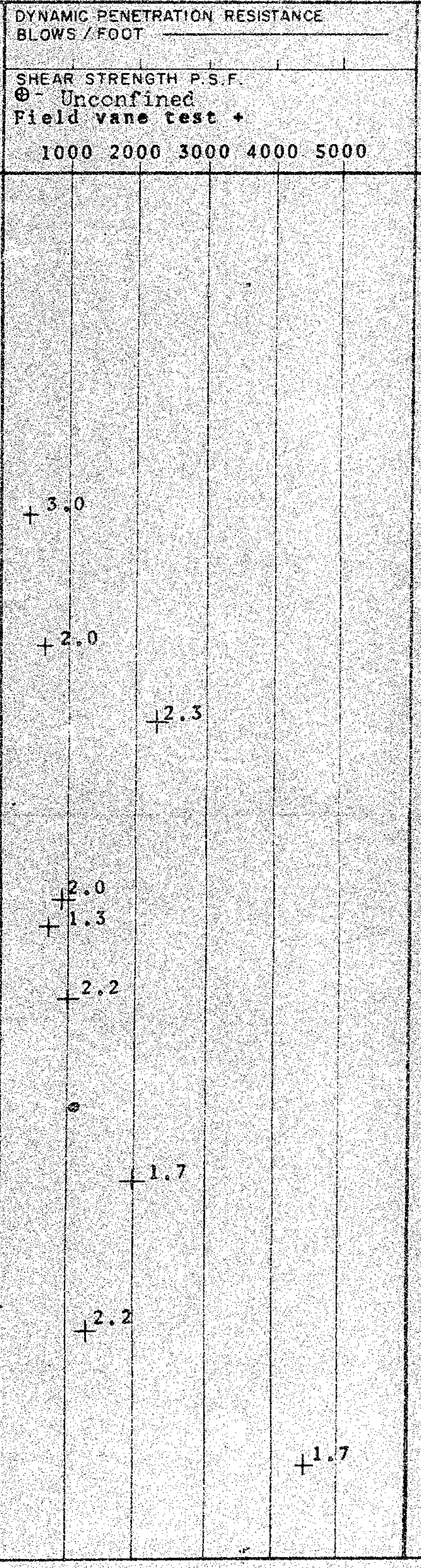
51.5 End of borehole

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	21
2	SS	14
3	SS	2
4	SS	2
5	TW	p
6	SS	2
7	SS	9
8	TW	p
9	SS	6
10	SS	6
11	TW	p
12	SS	11
13	SS	6
14	SS	9
15a		
15b	SS	92

ELEV. SCALE: 290, 285, 280, 275, 270, 265, 260, 255, 250, 245, 242.0

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F.
⊕ - Unconfined
Field vane test +



LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
w_P w w_L
WATER CONTENT %

BULK DENSITY γ P.C.F.

REMARKS
285.8
7.7
1/10/62

* Compiled by Dominion Soil Investigation Limited. Ref: 2-9-L6

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OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 13

FOUNDATION SECTION

JOB 62-F-62
W.P.
DATUM
LOCATION Sta. 260 + 00 (50' Lt.)
BORING DATE Oct. 1, 1962
BOREHOLE TYPE Augerhole
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F.

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT %

BULK DENSITY γ P.C.F.

SOIL PROFILE			SAMPLES				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
293.0	Ground level						
0.0	Gravelly clay fill					290	
289.0 4.0	Rubble fill (ash, clay, gravel, sand, organics)		1	SS	6	285	285.4 7.6 1/10/62
283.5 9.5	Organics		2	SS	2		
281.7 11.3	Soft black organic clay					280	
280.5 12.5	Soft grey clay						
278.0 15.0	End of borehole						

* Compiled by Dominion Soil Investigation Limited. Ref: 2-9-L6

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OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 14

FOUNDATION SECTION

JOB 62-F-62
W.P.
DATUM
LOCATION Sta. 260 + 40 (30' Rt.)
BORING DATE 2nd Oct. 1962
BOREHOLE TYPE Augerhole
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE				
291.0	Ground level									
0.0	Gravelly clay fill, trace of organics					290				
285.5 5.5	Rubble fill (ash, clay, gravel, sand, organics)		1	SS	5	285				285.2 5.8 5/10/62
283.0 8.0	Soft black organic clay									
280.5 10.5	Soft grey clay		2	SS	0	280				
279.5 11.5	End of borehole									

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MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 15

FOUNDATION SECTION

JOB 62-F-62
W.P.
DATUM
LOCATION Sta. 260 + 00 (50' Rt.)
BORING DATE 1st October 1962
BOREHOLE TYPE Augerhole
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

SOIL PROFILE			SAMPLES							
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F.	LIQUID LIMIT —— w_L PLASTIC LIMIT —— w_P WATER CONTENT —— w w_P —— w —— w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
292.0	Ground level									
0.0	Miscellaneous fill (clay, gravel, sand, ash, organics)					290		507 282 21		
			1	SS	4					285.7 6.3 1/10/62
	Loose					285				
283.5 8.5	Soft black organic clay									
281.2 10.8	Soft grey clay		2	SS	0					
280.5 11.5	End of borehole					280				

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DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 16

FOUNDATION SECTION

JOB 62-F-62
W.P.
DATUM
LOCATION Sta. 259 + 25 (50' Rt.)
BORING DATE 1st Oct. 1962
BOREHOLE TYPE Augerhole
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
294.0	Ground level									
0.0	Gravelly clay fill, some organics					290				
288.5 5.5	Rubble fill, (clay, gravel, sand, ash, organics) Loose		1	SS	13	285				286.7 7.3 1/10/62
			2	SS	4					
282.7 11.3	Soft black organic clay									
281.0 13.0	Soft grey clay					280				
279.0 15.0	End of borehole									

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OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 17

FOUNDATION SECTION

JOB 62-F-62
LOCATION Sta. 263 + 75 (60' Rt.)
ORIGINATED BY * JP
W.P.
BORING DATE 3rd Oct, 1962
COMPILED BY MC
DATUM
BOREHOLE TYPE Augerhole
CHECKED BY JP

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — wL PLASTIC LIMIT — wP WATER CONTENT — w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
299.0	Ground level									
0.0	Clay, gravel and ash fill									Open and dry 5/10/62
297.0 2.0	Stiff brown clay									
						295				
			1	SS	10					
						290				
			2	SS	10					
287.5 11.5	End of borehole									

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DEPARTMENT OF HIGHWAYS - ONTARIO
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RECORD OF BOREHOLE NO. 18

FOUNDATION SECTION

JOB 62-F-62
W.P.
DATUM
LOCATION Sta. 263 + 25 (35' Lt.)
BORING DATE 3rd Oct. 1962
BOREHOLE TYPE See remarks
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

SOIL PROFILE			SAMPLES							
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w w_P w w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
297.0	Ground level									
0.0	Fill gravelly					295				0 - 5'0" - wash bore
2.0	Stiff Silty Clay with traces of organics - Brown to Grey-Brown.		1	SS	11	290				5'0" - 18'0" - auger 297.0 3.0 5/10/62 (apparently rain water) Open and dry 3/10/62
			2	SS	20	285				
			3	SS	33	280				
279.0 18.0	End of borehole		4	SS	19					

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RECORD OF BOREHOLE NO. 19

FOUNDATION SECTION

JOB 62-F-62
W. P.
DATUM
LOCATION Sta. 263 + 75 (60' Lt.)
BORING DATE 2nd and 3rd Oct. 1962
BOREHOLE TYPE See remarks
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

SOIL PROFILE

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT
296.0	Ground level	
0.0 – 291.5 / 4.5	Sandy gravelly clay fill	≋
291.5 / 4.5 – 288.5 / 7.5	Ash	≋
288.5 / 7.5 – 281.5 / 14.5	Layers of brown and black organic clay Soft	▨
281.5 / 14.5 – 251.5 / 44.5	Reddish-brown clay, stiff to very stiff	▨
251.5 / 44.5 – 247.5 / 48.5	Reddish-brown sandy silt till	▥
247.5 / 48.5	End of borehole	

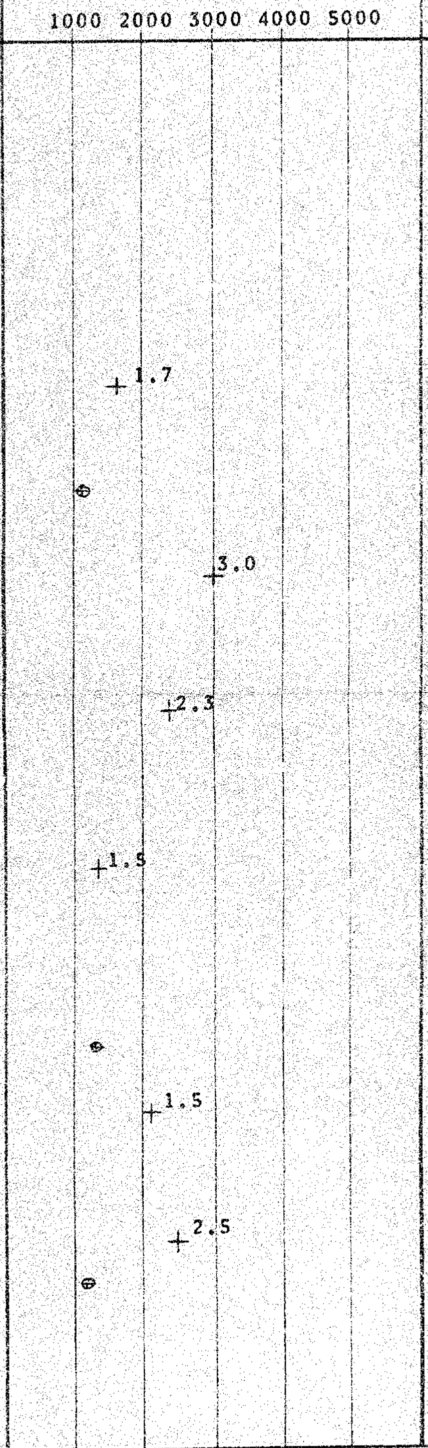
SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1	SS	1
2	SS	0
3	TW	p
4	SS	10
5	SS	11
6	TW	p
7	SS	11
8	TW	p
9	TW	p
10	SS	16
11	SS	9
12	TW	p
13	SS	23
14	SS	101

ELEV. SCALE: 295, 290, 285, 280, 275, 270, 265, 260, 255, 250

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
⊕ - Unconfined
Field vane test +



LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT %

BULK DENSITY γ P.C.F.

REMARKS

0 -35'0" auger

35'0" - 48'6" washbore

▽ 290.0
6.0
5/10/62

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OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
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RECORD OF BOREHOLE NO. 20

FOUNDATION SECTION

JOB 62-F-62
W.P.
DATUM
LOCATION Sta. 264 + 15 (25' Lt.)
BORING DATE 4 October 1962
BOREHOLE TYPE Augerhole
ORIGINATED BY * JP
COMPILED BY MC
CHECKED BY JP

SOIL PROFILE			SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
300.0	Ground level									
0.0	Organic clayey topsoil									Open and dry 5/10/62
299.0 1.0	Stiff brown clay									
						295				
292.5			1	SS	20					
7.5	End of borehole									

* Compiled by Dominion Soil Investigation Limited. Ref: 2-9-L6

APPENDIX B

Appendix B – Record of Boreholes - Current 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
υ	Poisson’s ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_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

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

(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 δ
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

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (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, 60º 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 cm^2 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 Condition	N 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	c_u, s_u 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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

RECORD OF BOREHOLE No 17-1

SHEET 1 OF 3

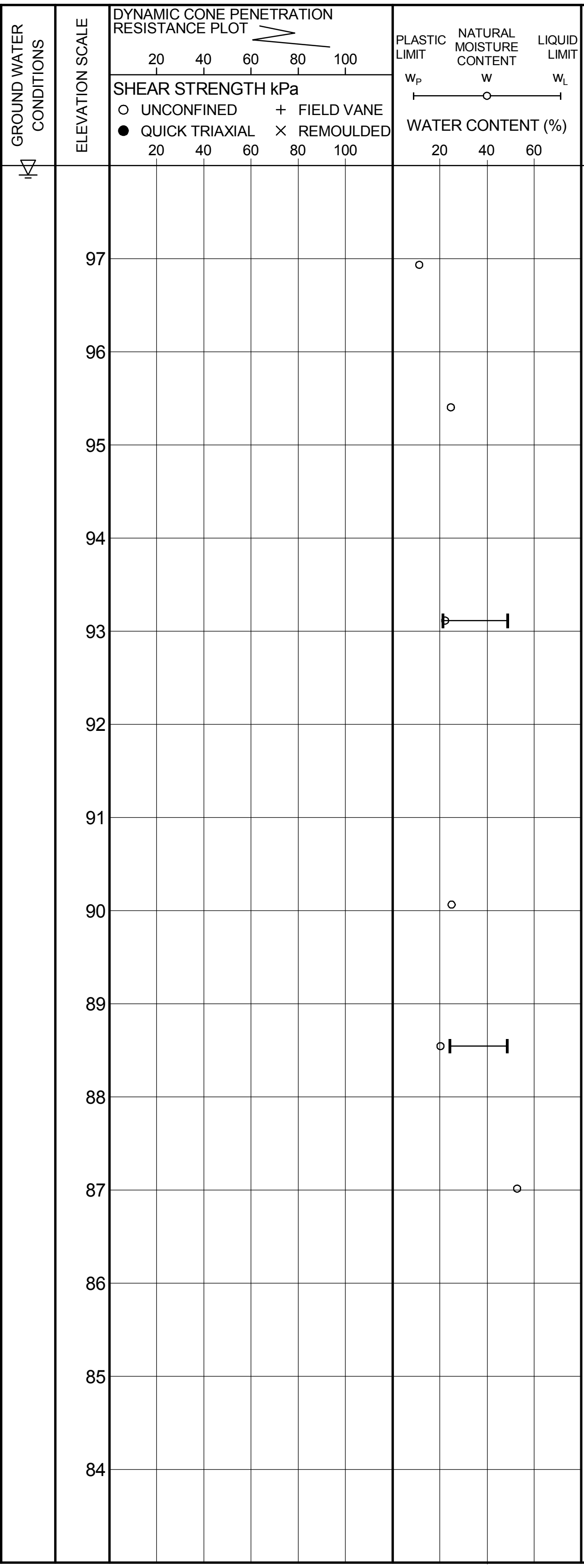
METRIC

PROJECT 1541610
G.W.P. 2453-13-00
DIST Central HWY 406
DATUM Geodetic
LOCATION N 4779913.3; E 326043.0 MTM NAD 83 ZONE 10 (LAT. 43.158778; LONG. -79.238784)
BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE October 30-31, 2017
ORIGINATED BY KN
COMPILED BY DH
CHECKED BY SMM

ELEV / DEPTH	DESCRIPTION
98.0	GROUND SURFACE
0.0 0.1	TOPSOIL (76 mm)
	Sandy clayey silt, trace gravel, trace rootlets, trace asphalt and brick fragments (FILL) Stiff Brown Moist
96.7 1.3	Clayey silt to silty clay, some sand, trace to some gravel, trace organics (FILL) Firm to very stiff Brown to grey-brown Moist to moist-wet
90.8 7.2	Sandy CLAYEY SILT, trace gravel Hard Reddish-brown Moist - Organic odour from depths between 9.2 m and 11.3 m - Grey to black-grey from depths between 9.2 m and 11.3 m - Trace organics and shell fragments from depths between 10.7 m and 11.3 m - Wood fragments / organics from depths between 12.2 m and 12.8 m

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	14				
2	SS	9				
3	SS	16				
4	SS	9				
5	SS	12				
6	SS	13				
7	SS	17				
8	SS	15				
9	SS	15	3	19	45	33
10	SS	4				
11	SS	13				
12	SS	14				

GROUND WATER CONDITIONS: ∇



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

RECORD OF BOREHOLE No 17-1

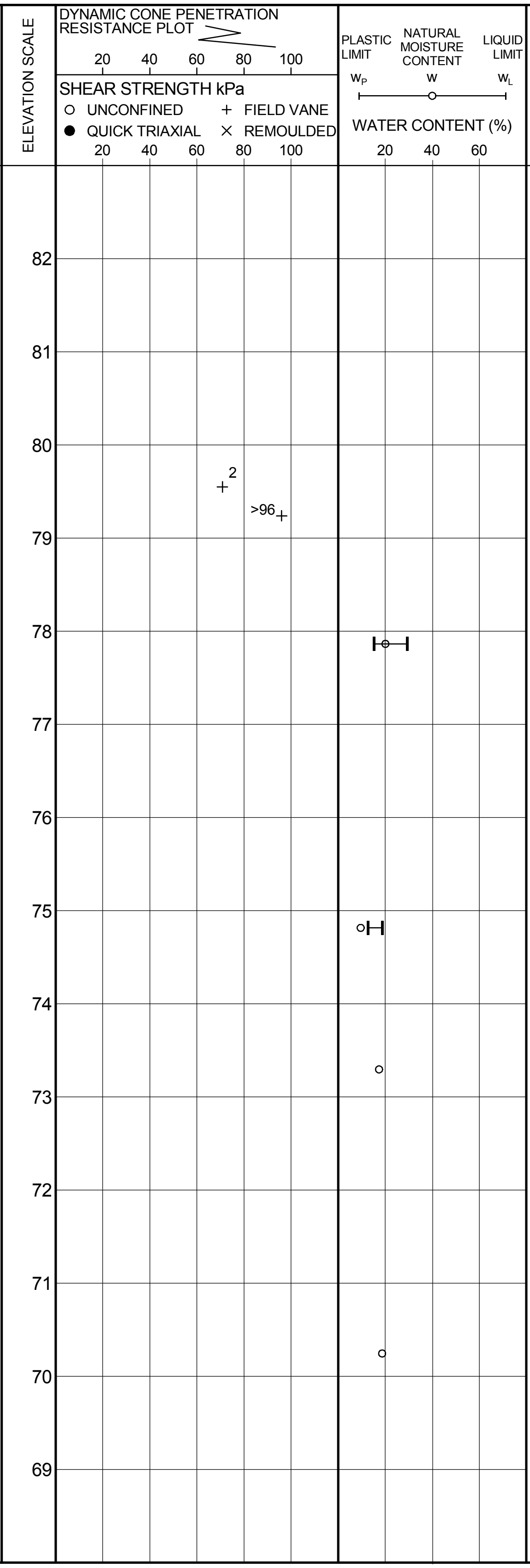
SHEET 2 OF 3

METRIC

PROJECT 1541610			
G.W.P. 2453-13-00	LOCATION N 4779913.3; E 326043.0 MTM NAD 83 ZONE 10 (LAT. 43.158778; LONG. -79.238784)	ORIGINATED BY KN	
DIST Central HWY 406	BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud	COMPILED BY DH	
DATUM Geodetic	DATE October 30-31, 2017	CHECKED BY SMM	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	Sandy CLAYEY SILT, trace gravel Hard Reddish-brown Moist	13	SS	5				
	- No recovery in Shelby tube 1	1	TO	PH				
		14	SS	6				
	- No recovery in Shelby tube 2	2	TO	PH				
		15	SS	13	3	15	45	37
	- Sand lens at 21.7 m	16	SS	14				
75.6 22.4	Sandy CLAYEY SILT, trace gravel (TILL) Hard Redish-brown Moist	17	SS	76	4	24	58	14
74.1 23.9	SILT and SAND, trace clay Very dense Grey to reddish-grey Moist to wet	18	SS	102/0.28	0	53	44	3
		19	SS	78				
71.0 27.0	SAND, some silt, trace clay, trace to some gravel Compact to dense Grey to brown-grey Wet	20	SS	38	0	82	15	3
		21	SS	28				



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

PROJECT 1541610

RECORD OF BOREHOLE No 17-1

SHEET 3 OF 3

METRIC

G.W.P. 2453-13-00

LOCATION N 4779913.3; E 326043.0 MTM NAD 83 ZONE 10 (LAT. 43.158778; LONG. -79.238784)

ORIGINATED BY KN

DIST Central HWY 406

BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud

COMPILED BY DH

DATUM Geodetic

DATE October 30-31, 2017

CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- *CONTINUED FROM PREVIOUS PAGE* ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	SAND, some silt, trace clay, trace to some gravel Compact to dense Grey to brown-grey Wet										
			22	SS	29						
							67				
							66				
65.4			23	SS	38						
32.6	END OF BOREHOLE										

NOTES:

1. Borehole dry prior to beginning of wash boring operations at a depth of 4.6 m (Elev. 93.4 m) below ground surface.

2. Water level measurement in the casing at the beginning of each work shift

Date	Depth (m)	Elev. (m)
31/10/17	0.1	97.9

The water level measurement are not considered representative of the groundwater level due to introduction of water / drilling mud during wash boring operations.

$+^3, \times^3$: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-2/2A

SHEET 1 OF 2

METRIC

PROJECT 1541610

G.W.P. 2453-13-00

LOCATION N 4779907.6; E 326069.7 MTM NAD 83 ZONE 10 (LAT. 43.158725; LONG. -79.238456)

ORIGINATED BY KN

DIST Central HWY 406

BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud

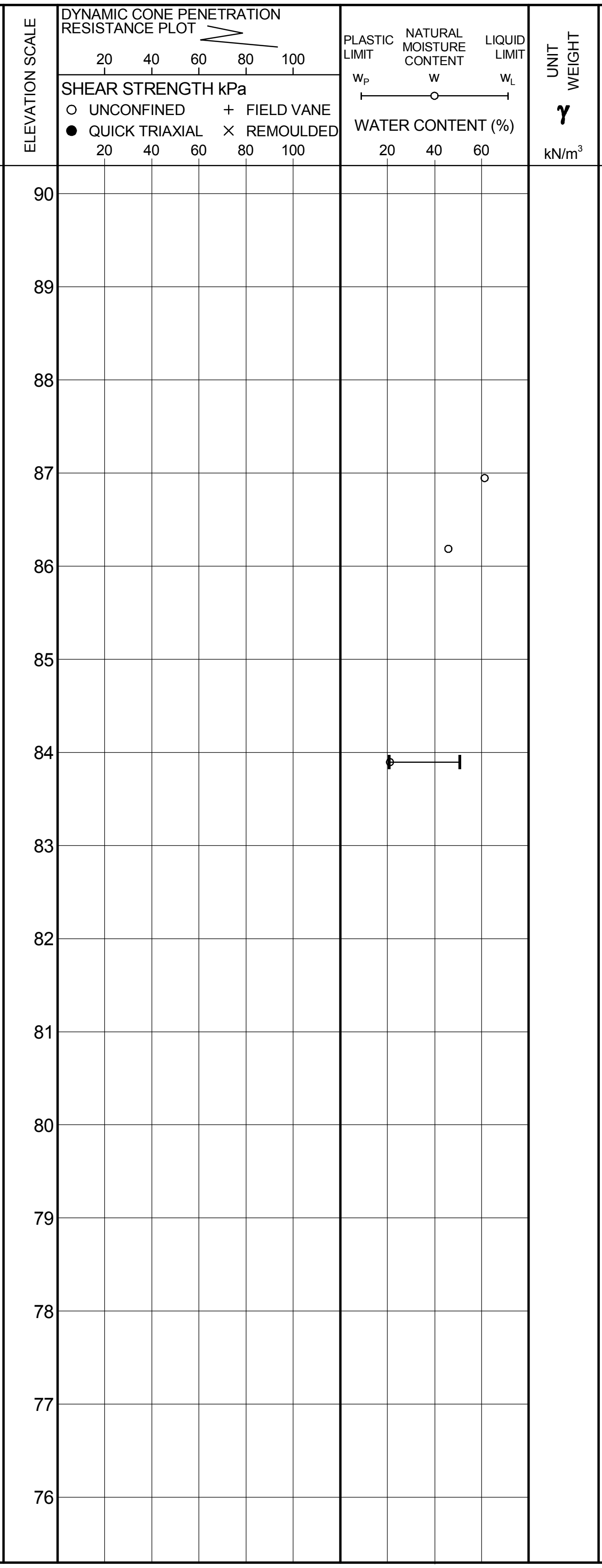
COMPILED BY DH/SK

DATUM Geodetic

DATE October 31, 2017 and April 12 and 17, 2018

CHECKED BY SMM

SOIL PROFILE		SAMPLES				REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
90.3	GROUND SURFACE								
0.0	Sandy clayey silt to clayey silt, trace rootlets, trace to some gravel, trace sand, trace brick fragments (FILL) Stiff to soft Brown to black Moist to moist-wet	1	SS	6					
		2	SS	14					
		3	SS	12					
	- Organic odour at a depth below 2.3 m	4	SS	4	∇				
87.3 3.0	Silty sand, trace clay, trace glass fragments, black clayey silt pockets (FILL) Very loose Grey Wet	5	SS	2					
86.6 3.7	SILTY CLAY to CLAY, trace wood / organics, trace sand, trace gravel Firm to very stiff Grey to brown Wet to moist - Hydrocarbon odour from depths between 3.8 m and 4.4 m	6	SS	6					
		7	SS	12					
		8	SS	23		0	4	38	58
83.1 7.2	Sandy CLAYEY SILT, trace gravel Very stiff Brown Moist	9	SS	16					
82.1 8.2	END OF BOREHOLE 17-2 Advanced Borehole 17-2A 0.4 m east of Borehole 17-2.								
	START OF SAMPLING BOREHOLE 17-2A								
76.6 13.7	CLAYEY SILT, trace sand, trace to some gravel Soft to stiff Grey Moist to wet	1	SS	14					
		2	SS	3					



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+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-2/2A

SHEET 2 OF 2

METRIC

PROJECT 1541610

G.W.P. 2453-13-00

LOCATION N 4779907.6; E 326069.7 MTM NAD 83 ZONE 10 (LAT. 43.158725; LONG. -79.238456)

ORIGINATED BY KN

DIST Central HWY 406

BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud

COMPILED BY DH/SK

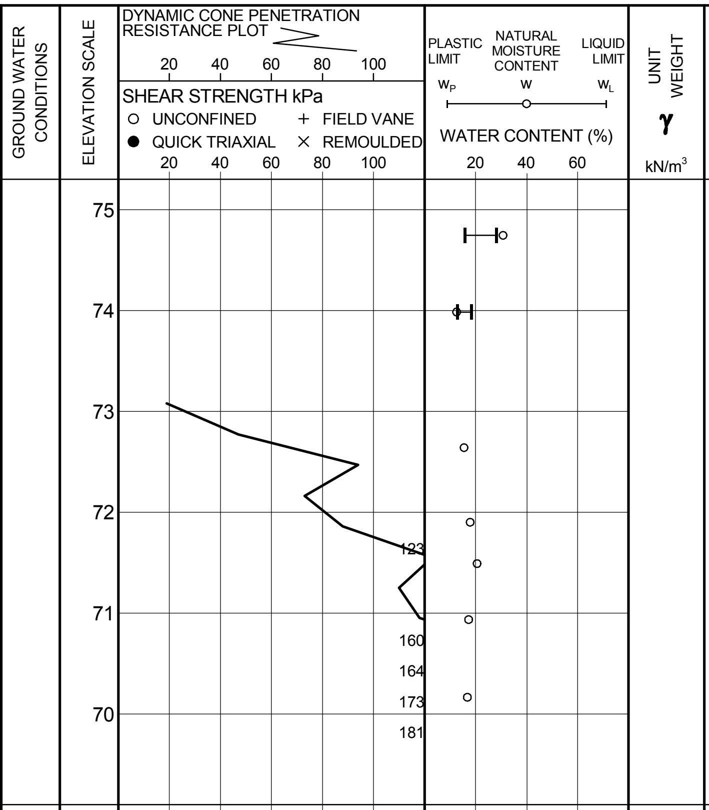
DATUM Geodetic

DATE October 31, 2017 and April 12 and 17, 2018

CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
74.4	CLAYEY SILT, trace sand, trace to some gravel Soft to stiff Grey Moist to wet	3	SS	10				
15.9	Sandy CLAYEY SILT, trace to some gravel (TILL) Stiff to hard Grey to brown Moist	4	SS	28	6	23	58	13
73.2 17.1	SILT, some sand, trace to some gravel, some silty clay seams Dense to very dense Grey Wet	5A 5B 5C	SS	36				
		6A 6B 6C	SS	60	6	15	64	15
72.1 18.2	SAND, some silt, trace clay Compact to dense Grey Wet - Clayey silt layer between depth of about 18.2 and 18.6 m	7A 7B	SS	30				
		8	SS	22	0	79	18	3
		9	SS	21				
		10	SS	24				
69.1 21.2	END OF BOREHOLE							



NOTES:

1. Borehole 17-2 was terminated at a depth of 8.2 m below ground surface (Elev. 82.1 m) on October 31, 2017. Borehole 17-2A was terminated at a depth of 21.2 m (Elev. 69.1 m) below ground surface on April 12, 2018.

2. Water level measurement in borehole at a depth of about 2.5 m below ground surface (Elev. 87.8 m) upon completion of drilling Borehole 17-2 on October 31, 2017.

3. Borehole 17-2 cave to a depth of about 4.3 m below ground surface (Elev. 86.0 m) upon completion of drilling on October 31, 2017.

4. Artesian conditions noted in Borehole 17-2A at a depth of about 4.6 m below ground surface (Elev. 85.7 m) on April 12, 2018. Water level measured at 0.4 m above ground surface.

5. In a seperate borehole casing advanced to Elev. 73.0 m then carried out a Dynamic Cone Penetration Test to refusal at Elev. 68.8 m.

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-3/3A

SHEET 1 OF 2

METRIC

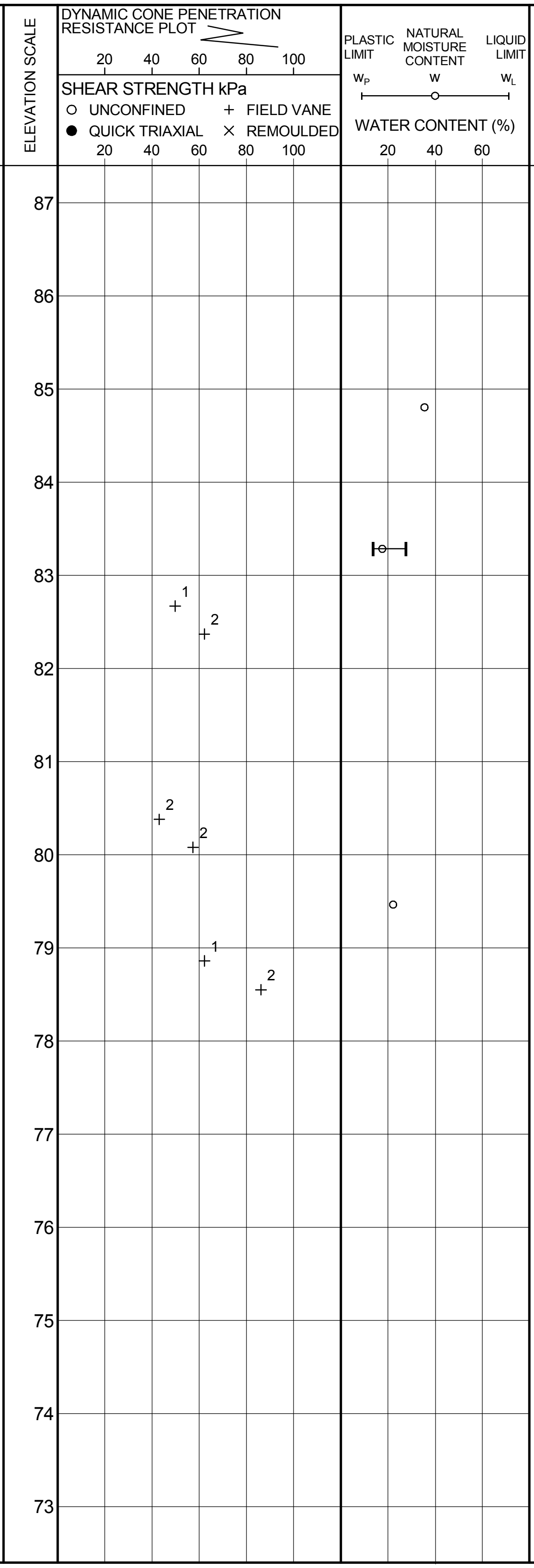
PROJECT 1541610
G.W.P. 2453-13-00
DIST Central HWY 406
DATUM Geodetic
LOCATION N 4779896.7; E 326090.3 MTM NAD 83 ZONE 10 (LAT. 43.158627; LONG. -79.238203)
BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE November 1, 2017 and April 12 and 18, 2018
ORIGINATED BY KN
COMPILED BY DH/SK
CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
87.4	GROUND SURFACE
0.0	Clayey silt, trace rootlets organics at surface, some gravel, containing concrete fragments (FILL) Hard Brown Moist
86.1 1.3	Gravel, some sand, some clay (FILL) Compact Brown Moist
85.2 2.2	CLAYEY SILT with SAND, trace gravel Firm to stiff Brown - grey Wet - Hydrocarbon odour from depths between 2.3 m and 2.9 m - Black between depths of 2.3 m and 2.9 m
78.6 8.8	END OF BOREHOLE 17-3 Advanced Borehole 17-3A 0.7 m southeast of Borehole 17-3.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
1	SS	100/0.05					
2	SS	100/0.08					
3	SS	19					
4	SS	6					
5	SS	11	∇				
6	SS	7		1	38	40	21
7	SS	3					
8	SS	8					



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-3/3A

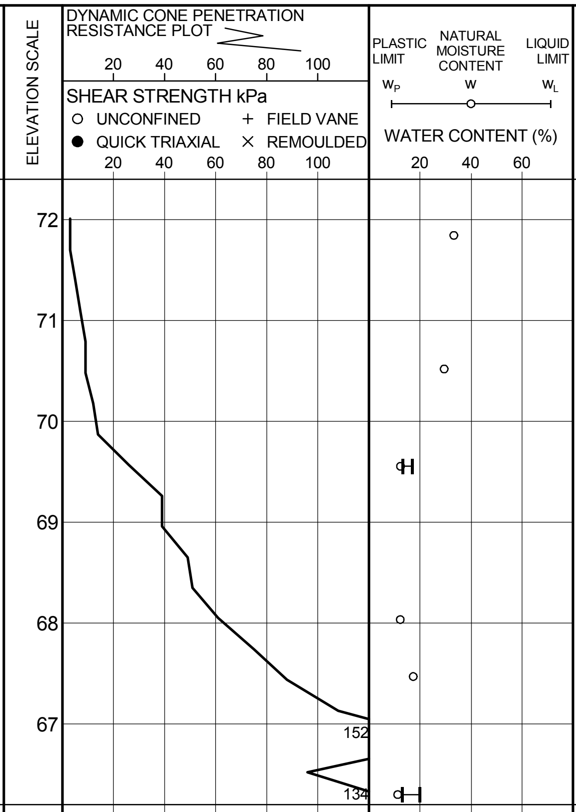
SHEET 2 OF 2

METRIC

PROJECT 1541610
G.W.P. 2453-13-00
DIST Central HWY 406
DATUM Geodetic
LOCATION N 4779896.7; E 326090.3 MTM NAD 83 ZONE 10 (LAT. 43.158627; LONG. -79.238203)
BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE November 1, 2017 and April 12 and 18, 2018
ORIGINATED BY KN
COMPILED BY DH/SK
CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
72.1 15.3	START OF SAMPLING BOREHOLE 17-3A									
	CLAYEY SILT, trace sand, trace gravel Firm Brown Moist to wet	1	SS	7						
		2	SS	5						
70.3 17.1		3A 3B	SS	8						
	Sandy SILT, trace to some clay, trace gravel (TILL) Loose to dense Brown Moist	4	SS	28			4	25	60	11
		5	SS	41						
67.7 19.7		6	SS	28						
	SILT, trace to some sand, trace to some clay, trace gravel Very dense Brown Moist	7A 7B	SS	61			3	5	85	7
66.5		8A	SS	39						
66.2 21.2	CLAYEY SILT, some sand, some gravel Hard Brown Moist	8B								
	END OF BOREHOLE									



NOTES:

1. Borehole 17-3 was terminated at a depth of 8.8 m below ground surface (Elev. 78.6 m) on November 1, 2018. Borehole 17-3A was terminated at a depth of 21.3 m below ground surface (Elev. 66.1 m) on April 12, 2018.

2. Water level measurement recorded in borehole at a depth of about 3.4 m below ground surface (Elev. 84.0 m) upon completion of drilling Borehole 17-3.

3. Borehole caved to a depth of 6.7 m below ground surface (Elev. 80.7 m) upon completion of drilling Borehole 17-3 on November 1, 2017.

4. In a seperate borehole casing was advanced to Elev. 72.0 m, then a Dynamic Cone Penetration Test was carried out to refusal at Elev. 66.2 m.

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No 17-4

SHEET 1 OF 2

METRIC

PROJECT	1541610
G.W.P.	2453-13-00
DIST	Central
HWY	406
DATUM	Geodetic
LOCATION	N 4779889.0; E 326105.4 MTM NAD 83 ZONE 10 (LAT. 43.158557; LONG. -79.238018)
BOREHOLE TYPE	150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE	November 1, 2017 and April 19, 2018
ORIGINATED BY	KN
COMPILED BY	DH
CHECKED BY	SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
88.1	GROUND SURFACE
0.0	Sandy clayey silt to clayey silt, some gravel, trace rootlets, trace brick fragments (FILL) Firm to stiff Brown Moist
85.9 2.2	CLAYEY SILT, trace gravel, trace to some sand, sand layers Firm to stiff Brown to grey to reddish grey Moist to wet - Wood / organics from depths between 4.6 m and 5.2 m - Silty sandy gravel layer (50 mm thickness) at a depth of about 10.7 m
74.8 13.3	CLAYEY SILT, some sand, trace gravel (TILL) Hard Reddish brown Moist

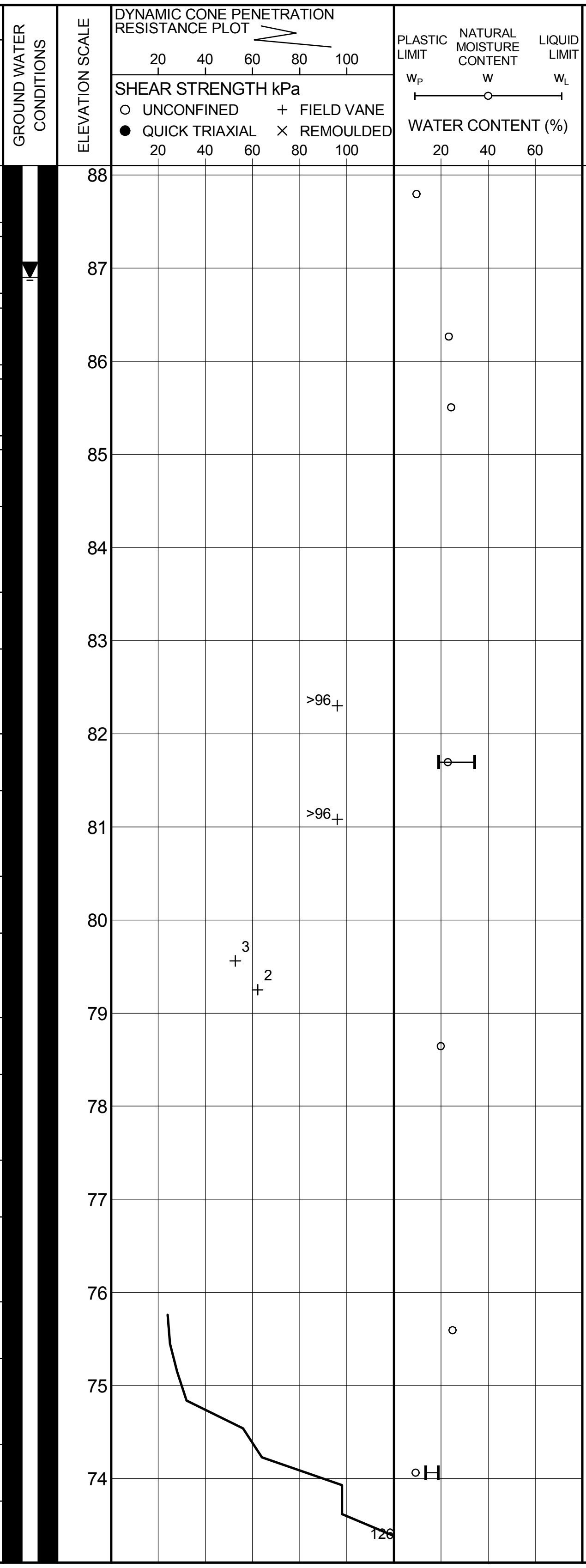
Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
1	SS	11				
2	SS	7				
3	SS	5				
4	SS	10				
5	SS	11				
6	SS	5				
7	SS	6	1	5	59	35
8	SS	3				
9	SS	11				
10	SS	11				
11	SS	10				
12	SS	40	9	19	60	12

REMARKS & GRAIN SIZE DISTRIBUTION (%)

UNIT WEIGHT γ kN/m³



+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-4

SHEET 2 OF 2

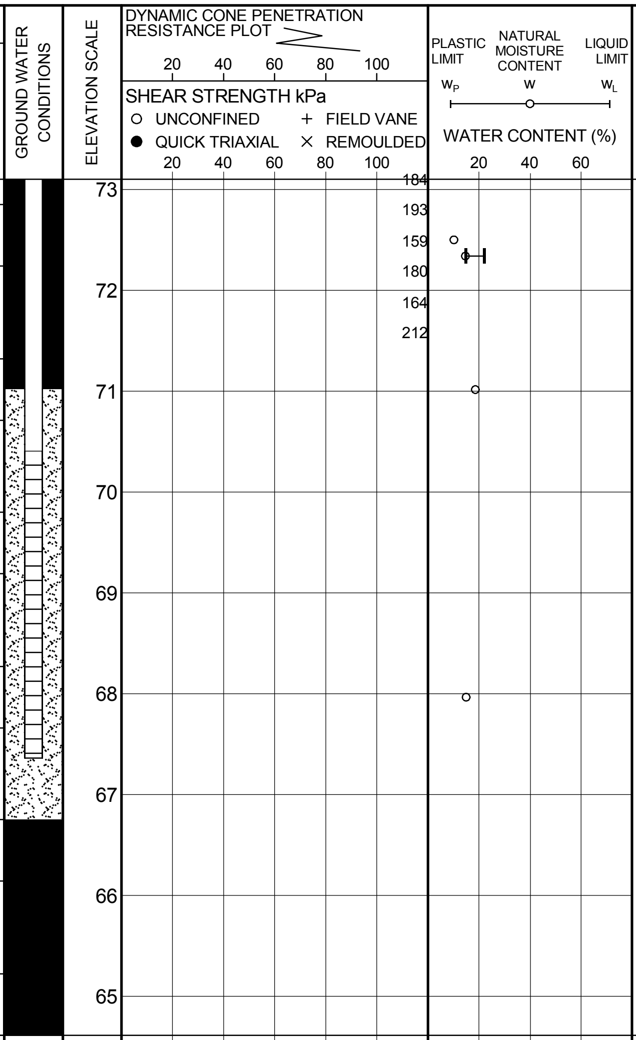
METRIC

PROJECT 1541610			
G.W.P. 2453-13-00	LOCATION N 4779889.0; E 326105.4 MTM NAD 83 ZONE 10 (LAT. 43.158557; LONG. -79.238018)	ORIGINATED BY KN
DIST Central HWY 406	BOREHOLE TYPE 150mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud	COMPILED BY DH
DATUM Geodetic	DATE November 1, 2017 and April 19, 2018	CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	SOIL PROFILE: DESCRIPTION
	CLAYEY SILT (TILL)
72.8 15.5	Gravelly SAND, some silt, trace clay Dense Grey Wet
71.8 16.3	Sandy CLAYEY SILT, trace gravel (TILL) Hard Brown Moist
	SILT, some sand, trace to some clay Dense Grey Wet
68.7 19.4	SILT and SAND, trace clay Compact to dense Reddish grey Wet
64.6 23.5	END OF BOREHOLE

SAMPLES: NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
13A 13B 13C	SS	37				
14	SS	32	0	17	76	7
15	SS	36				
16	SS	26	0	56	43	1
17	SS	32				
18	SS	39				



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

NOTES:

1. Water level at ground surface upon completion of drilling.

2. Water level measurements in standpipe piezometer

Date	Depth (m)	Elev. (m)
01/11/17	0	88.1
09/04/18	1.2	86.9
01/05/18	1.2	86.9

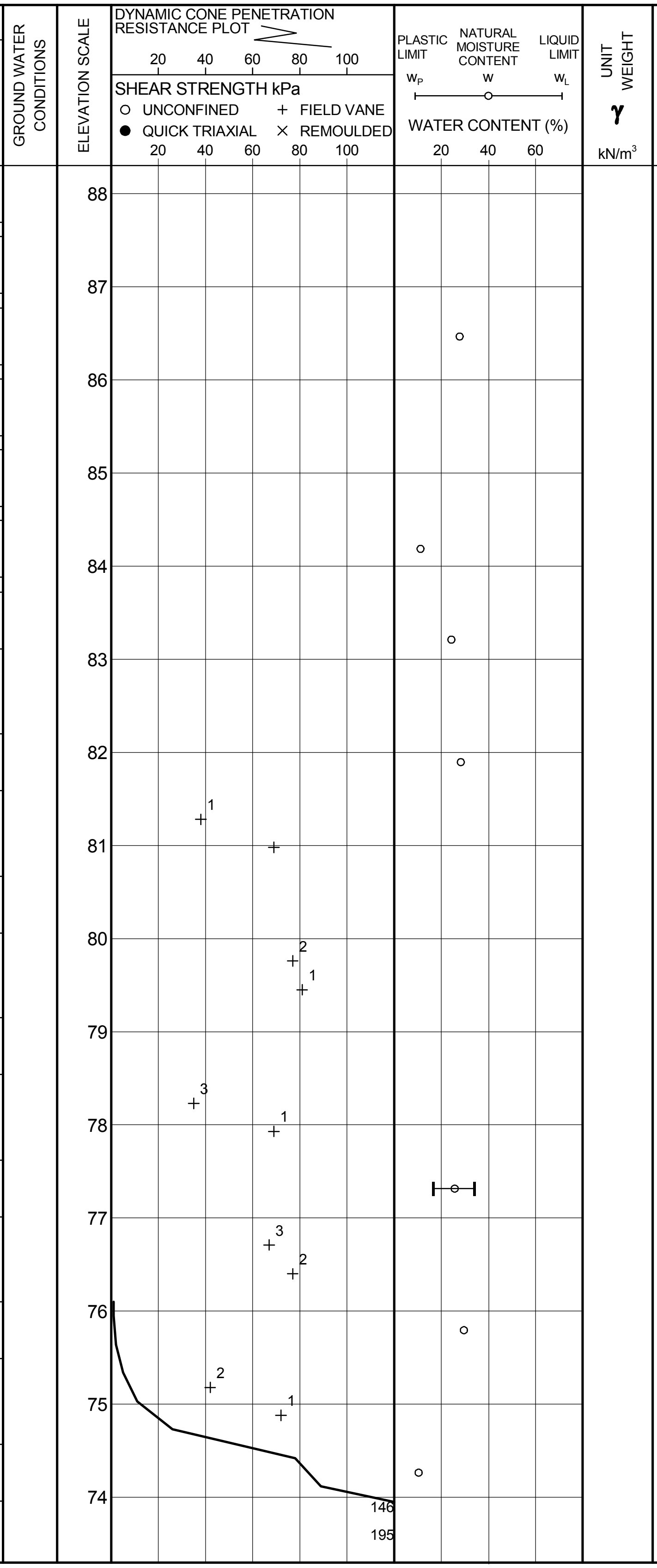
3. In a seperate borehole casing was advanced to Elev. 75.8 m, then a Dynamic Cone Penetration Test was carried out to refusal at Elev. 71.5 m.

4. Borehole advanced by mud-rotary, water level not representitive of in situ groundwater conditions.

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

PROJECT 1541610	**RECORD OF BOREHOLE No 17-5**	SHEET 1 OF 2	**METRIC**
G.W.P. 2453-13-00	LOCATION N 4779880.2; E 326126.3 MTM NAD 83 ZONE 10 (LAT. 43.158477; LONG. -79.237761)		ORIGINATED BY TP
DIST Central HWY 406	BOREHOLE TYPE 203mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud		COMPILED BY SK
DATUM Geodetic	DATE April 9 and 11, 2018		CHECKED BY SMM

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
88.3	GROUND SURFACE							
0.0	TOPSOIL (30 mm)							
0.0	Silty clay, some sand, trace gravel (FILL) Soft to firm Brown Moist to wet	1	SS	6				
		2	SS	8				
		3	SS	3				
		4	SS	8				
85.3 3.0	Sand and gravel, trace silt, trace clay (FILL) Compact Brown to black Wet	5	SS	17				
		6	SS	22	41	53	4	2
	- Hydrocarbon odour from depths between 3.0 m and 5.2 m	7A 7B	SS	15				
82.7 5.6	CLAYEY SILT, trace to some sand, trace gravel Firm to stiff Grey Moist to wet	8	SS	1				
		9	TO	PH				
		10	SS	5				
		11	SS	2	3	11	46	40
		12	SS	3				
75.0 13.3	CLAYEY SILT, trace to some sand, trace gravel (TILL) Hard Brown Moist	13	SS	49				



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

GTA-MTO 001 S:\CLIENTS\MTO\HWY_406\02_DATA\GINT\HWY_406.GPJ GAL-GTA.GDT 10/16/18

RECORD OF BOREHOLE No 17-5

SHEET 2 OF 2

METRIC

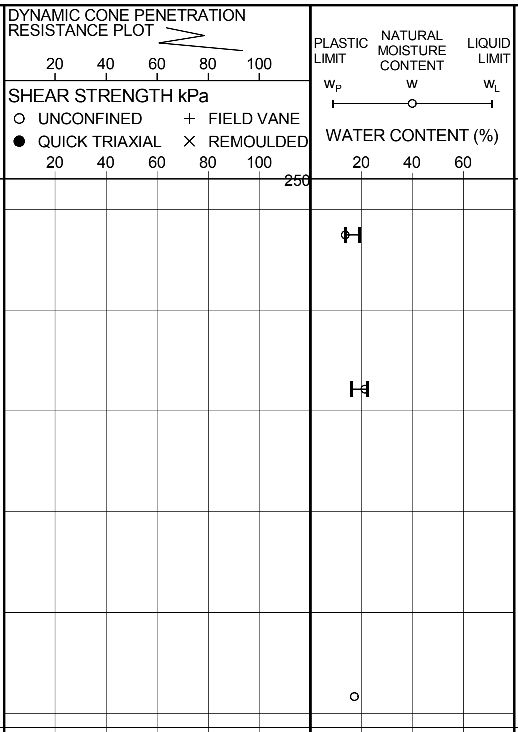
PROJECT 1541610
G.W.P. 2453-13-00
DIST Central HWY 406
DATUM Geodetic
LOCATION N 4779880.2; E 326126.3 MTM NAD 83 ZONE 10 (LAT. 43.158477; LONG. -79.237761)
BOREHOLE TYPE 203mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE April 9 and 11, 2018
ORIGINATED BY TP
COMPILED BY SK
CHECKED BY SMM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	--- CONTINUED FROM PREVIOUS PAGE ---											
	CLAYEY SILT, trace to some sand, trace gravel (TILL) Hard Brown Moist		14	SS	57		73		2	14	68	16
72.0 16.3	CLAYEY SILT, some sand Hard Grey Moist		15	SS	41		72 71					
70.5 17.8	SILT and SAND, trace clay Compact to dense Grey Wet		16A 16B 16C	SS	22		70 69					
			17	SS	44		68		Non-plastic 0	56	42	2
67.9 20.4	END OF BOREHOLE											

NOTES:

1. In a seperate borehole casing was advanced to Elev. 76.0 m, then a Dynamic Cone Penetration Test was carried out to refusal at Elev. 73.2 m.

2. Borehole advanced by mud-rotary, water level not representitive of in situ groundwater conditions.



+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 17-6

SHEET 1 OF 3

METRIC

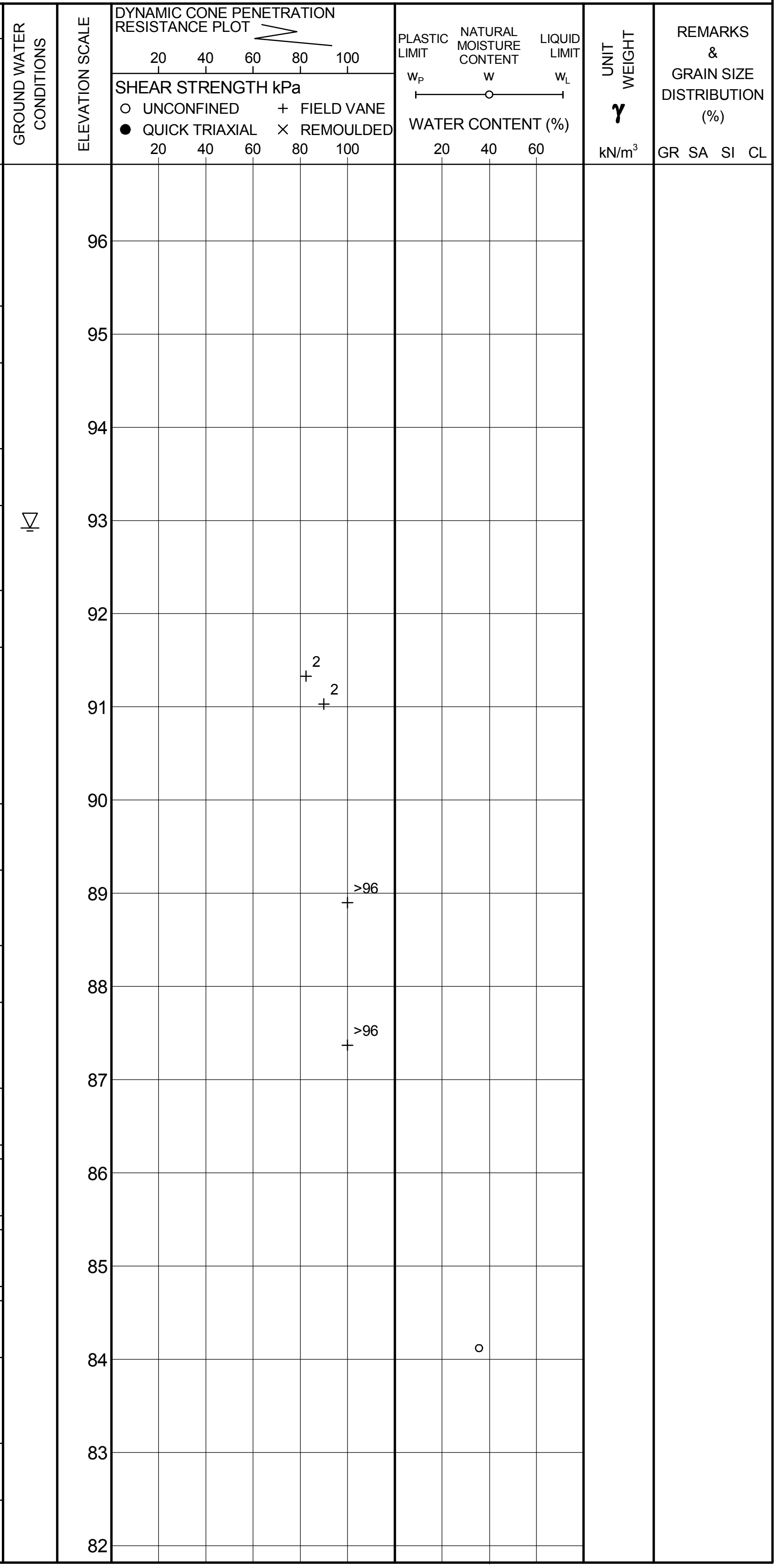
PROJECT 1541610
G.W.P. 2453-13-00
DIST Central HWY 406
DATUM Geodetic
LOCATION N 4779867.0; E 326151.9 MTM NAD 83 ZONE 10 (LAT. 43.158358; LONG. -79.237447)
BOREHOLE TYPE 178mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE April 30 and May 1, 2018
ORIGINATED BY LK
COMPILED BY KN
CHECKED BY SMM

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
96.8	GROUND SURFACE
0.0	ASPHALT (203 mm)
	CONCRETE with steel rebar (366 mm)
96.2 0.6	Sand and gravel, trace silt (FILL) Compact Grey Moist
94.2 2.6	Sand, trace gravel, trace silt (FILL) Compact Reddish brown Moist
92.2 4.6	Clayey silt, trace sand, trace gravel (FILL) Stiff to very stiff Brown Moist
85.6 11.2	Sand, some gravel (FILL) Compact Black Moist - Hydrocarbon odour from depths between 11.2 m and 12.7 m
84.1 12.7	CLAYEY SILT, trace sand, trace gravel Soft to stiff Grey to greyish brown Moist to wet

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	22
2	SS	13
3A 3B	SS	6
4	TO	PH
5	SS	17
6	SS	14
7A 7B	SS	25
8	SS	20
9A 9B	SS	10
10	SS	9



Continued Next Page

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

RECORD OF BOREHOLE No 17-6

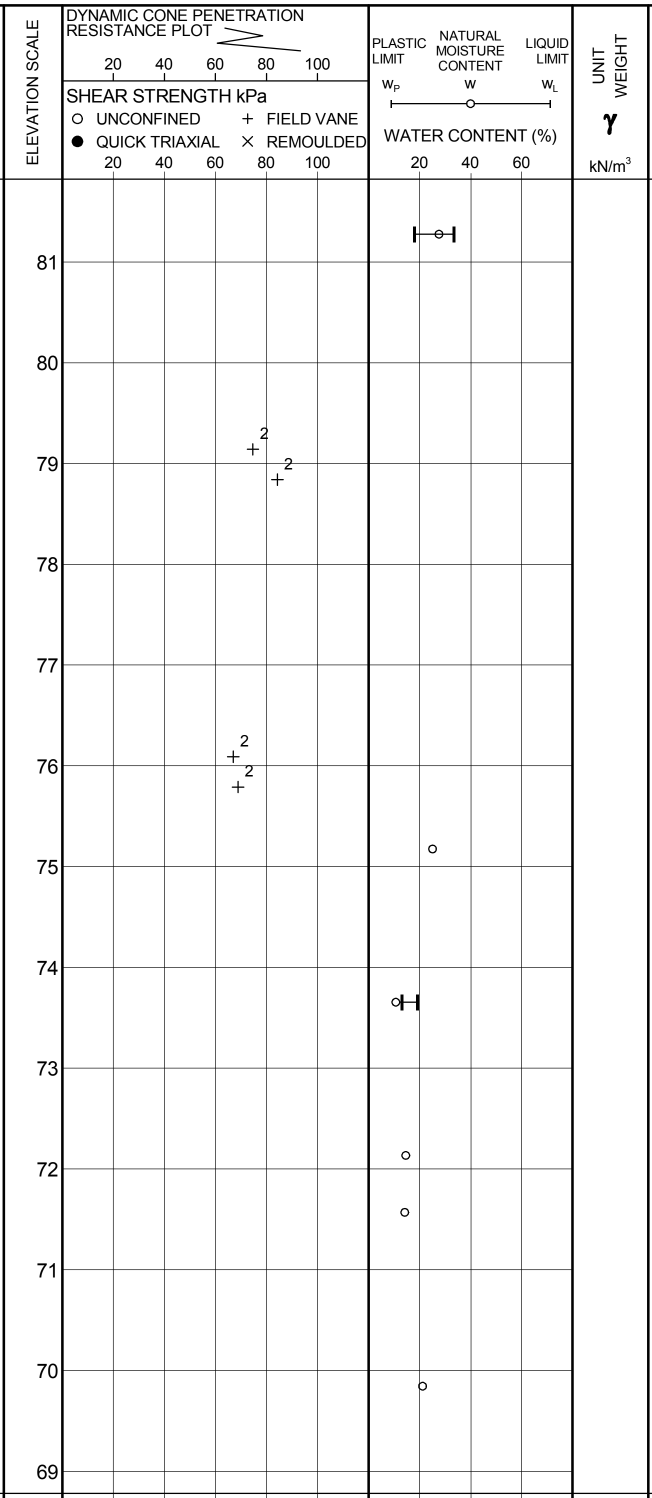
SHEET 2 OF 3

METRIC

PROJECT 1541610
G.W.P. 2453-13-00
DIST Central HWY 406
DATUM Geodetic
LOCATION N 4779867.0; E 326151.9 MTM NAD 83 ZONE 10 (LAT. 43.158358; LONG. -79.237447)
BOREHOLE TYPE 178mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE April 30 and May 1, 2018
ORIGINATED BY LK
COMPILED BY KN
CHECKED BY SMM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
	CLAYEY SILT, trace sand, trace gravel Soft to stiff Grey to greyish brown Moist to wet	11	SS	5	3	4	53	40
		12	SS	3				
		13	SS	4				
		14	SS	5				
		15	SS	6				
74.2 22.6	Sandy CLAYEY SILT, trace to some gravel (TILL) Hard Grey - Brown Moist	16	SS	49	4	23	59	14
		17	SS	35				
72.5 24.3	SILT and SAND, trace clay Very dense Grey Moist	18	SS	78	0	65	32	3
		19A 19B	SS	62				
71.0 25.8	SILT, clayey silt lenses, trace sand Dense Grey Moist	20	SS	35				
70.2 26.6	Sandy SILT, trace to some clay, trace gravel Compact Grey Moist to wet	21	SS	18	2	24	67	7
		22	SS	24				
68.8 28.0	END OF BOREHOLE NOTES: 1. Water level measurements in casing at the beginning of each work shift:							



Continued Next Page

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

PROJECT 1541610

RECORD OF BOREHOLE No 17-6

SHEET 3 OF 3

METRIC

G.W.P. 2453-13-00
LOCATION N 4779867.0; E 326151.9 MTM NAD 83 ZONE 10 (LAT. 43.158358; LONG. -79.237447)
ORIGINATED BY LK
DIST Central HWY 406
BOREHOLE TYPE 178mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
COMPILED BY KN
DATUM Geodetic
DATE April 30 and May 1, 2018
CHECKED BY SMM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION *--- CONTINUED FROM PREVIOUS PAGE ---*	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Date Depth(m) Elev.(m) 30/04/18 3.9 92.9 01/05/18 3.9 92.9 2. Borehole advanced by mud-rotary, water level not representitive of in situ groundwater conditions.										

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

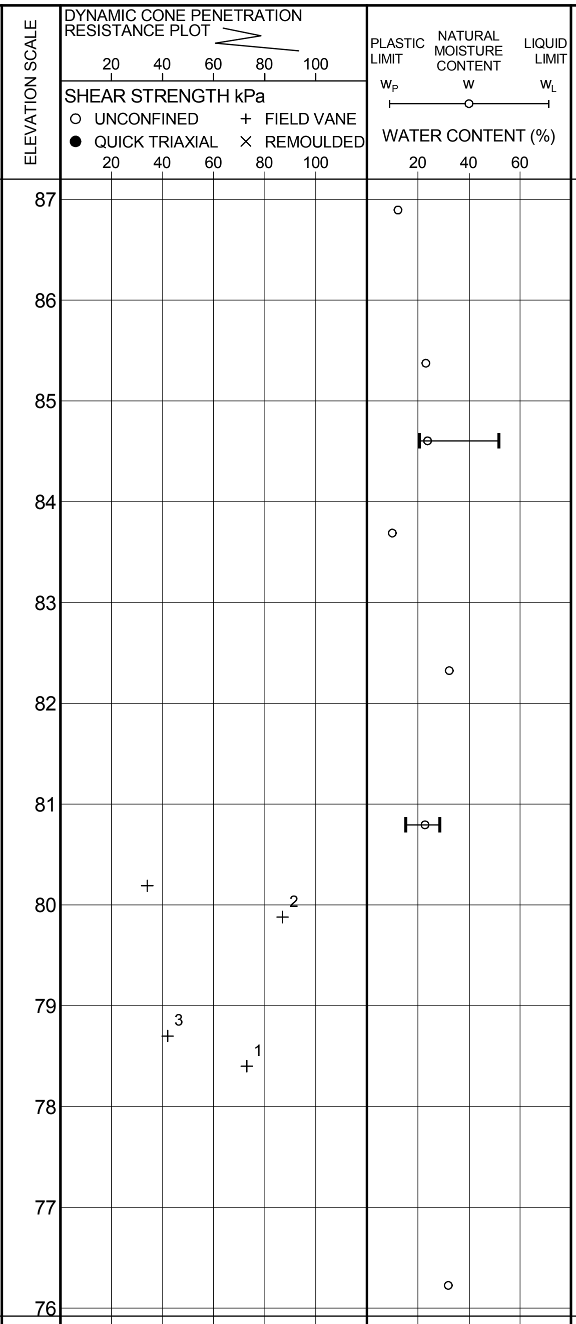
RECORD OF BOREHOLE No 17-7

SHEET 1 OF 1

METRIC

PROJECT	1541610		
G.W.P.	2453-13-00	LOCATION	N 4779898.7; E 326114.1 MTM NAD 83 ZONE 10 (LAT. 43.158644; LONG. -79.237910)
DIST	Central	HWY	406
DATUM	Geodetic	BOREHOLE TYPE	203mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud
DATE	April 18, 2018		
ORIGINATED BY	TP	COMPILED BY	DH
CHECKED BY	SMM		

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
87.2	GROUND SURFACE									
0.0	TOPSOIL (152 mm)									
0.2	Sandy clayey silt, trace gravel, trace rootlets, trace concrete fragments (FILL) Firm Brown Moist	1	SS	6						
86.5 0.7	CLAY, some silt, trace to some gravel, trace sand Firm to stiff Brown Moist	2	SS	12						
		3	SS	12						
		4	SS	6						
84.1 3.1	GRAVEL, some sand, trace to some silt, trace clay, inferred cobbles Dense Brown Wet	5A 5B	SS	31			73	19	6	2
83.5 3.7	CLAYEY SILT, trace to some sand, trace gravel Very soft to stiff Grey and brown Moist to wet	6	SS	7						
		7	SS	2						
		8	TO	PH						
		9	SS	1			2	11	54	33
		10	SS	6						
		11	SS	5						
		12	SS	3						
75.9 11.3	END OF BOREHOLE NOTE: 1. Borehole advanced by mud-rotary, water level not representitive of in situ groundwater conditions.									



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

RECORD OF BOREHOLE No 17-8

SHEET 1 OF 1

METRIC

PROJECT 1541610

G.W.P. 2453-13-00

LOCATION N 4779910.2; E 326105.8 MTM NAD 83 ZONE 10 (LAT. 43.158748; LONG. -79.238012)

ORIGINATED BY TP

DIST Central HWY 406

BOREHOLE TYPE 203mm O.D. Continuous Flight Solid Stem Auger & Wash Boring with Drilling Mud

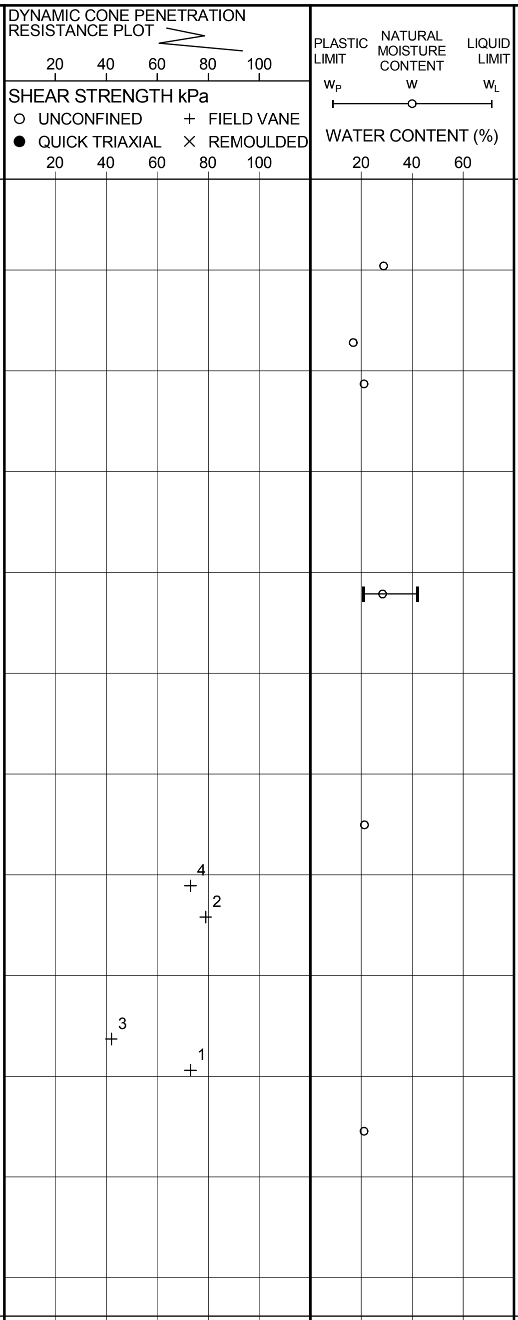
COMPILED BY DH

DATUM Geodetic

DATE April 16, 2018

CHECKED BY SMM

SOIL PROFILE		SAMPLES						REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
86.9	GROUND SURFACE										
0.0	TOPSOIL (152 mm)										
0.2	Sandy clayey silt, trace to some gravel (FILL) Firm Brown Moist	1	SS	6							
86.0 0.9	Sand and gravel, some silt, inferred cobbles (FILL) Loose to compact Grey Wet	2A 2B	SS	27		86		7	24	29	40
		3A 3B	SS	6		85					
85.1 1.8	SILTY CLAY, trace sand, trace gravel Soft to stiff Brown to grey and brown Moist	4	SS	2		84					
		5	SS	2		83					
		6	SS	7		82		1	2	49	48
		7	SS	6		81					
		8	SS	6		80					
		9	SS	5		79					
						78					
		10	SS	6		77					
		11	SS	7		76					
75.6 11.3	END OF BOREHOLE NOTE: 1. Borehole advanced by mud-rotary, water level not representitive of in situ groundwater conditions.										



+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

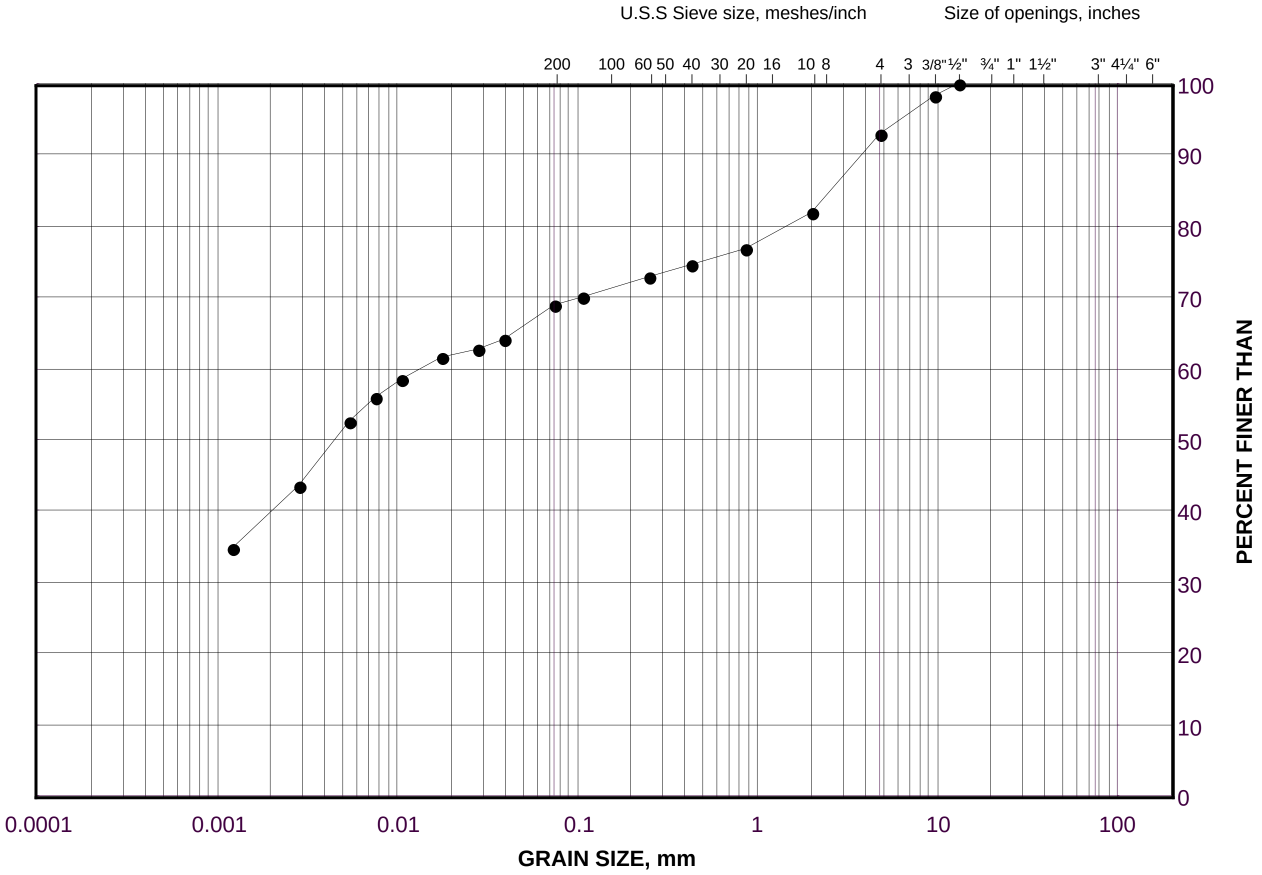
APPENDIX C

Appendix C – Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sandy 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)
●	17-8	2A	86.1

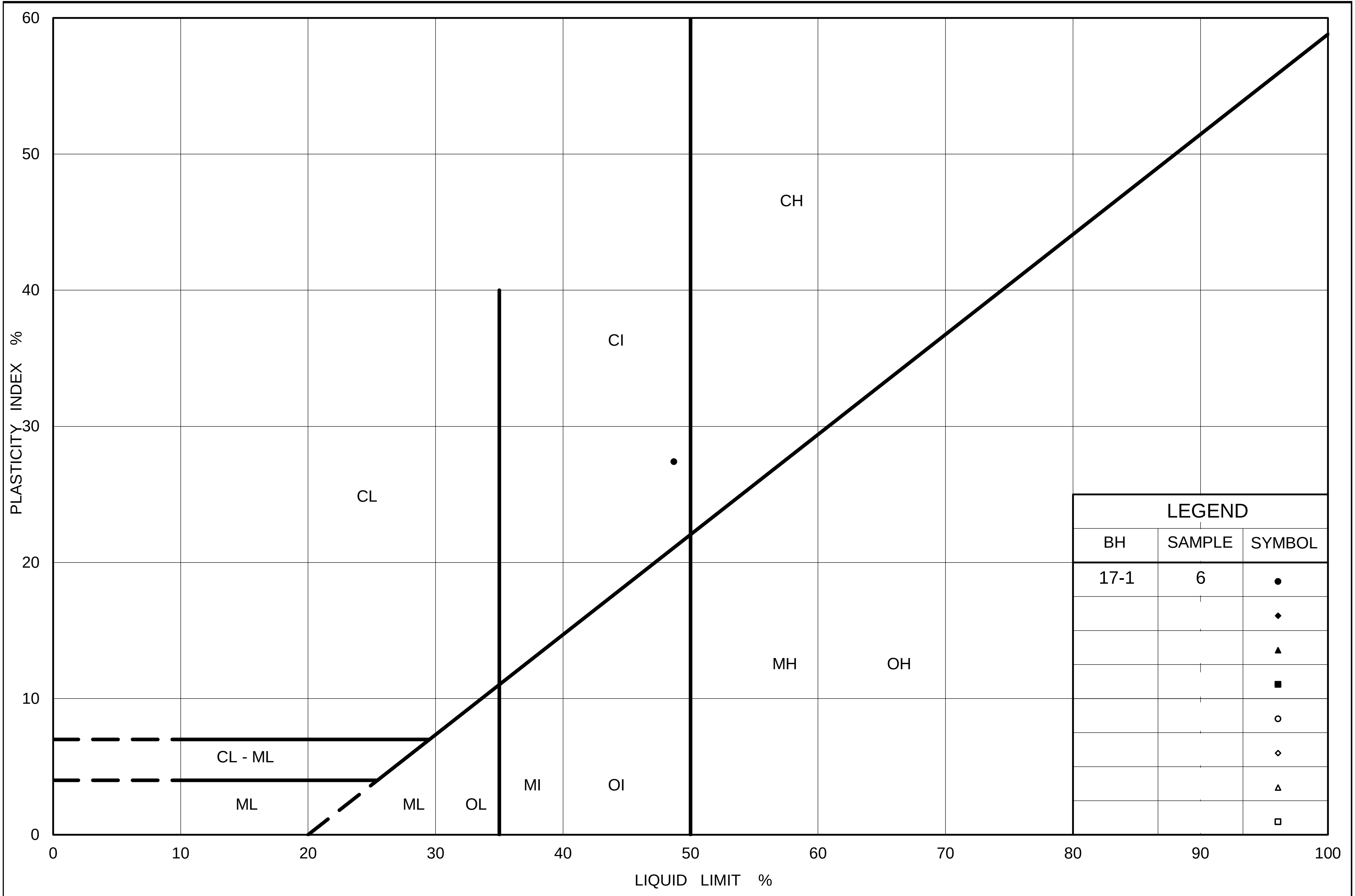
Project Number: 1541610

Checked By: SMM

Golder Associates

Date: 23-Jul-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay (Fill)

Figure No. C-2

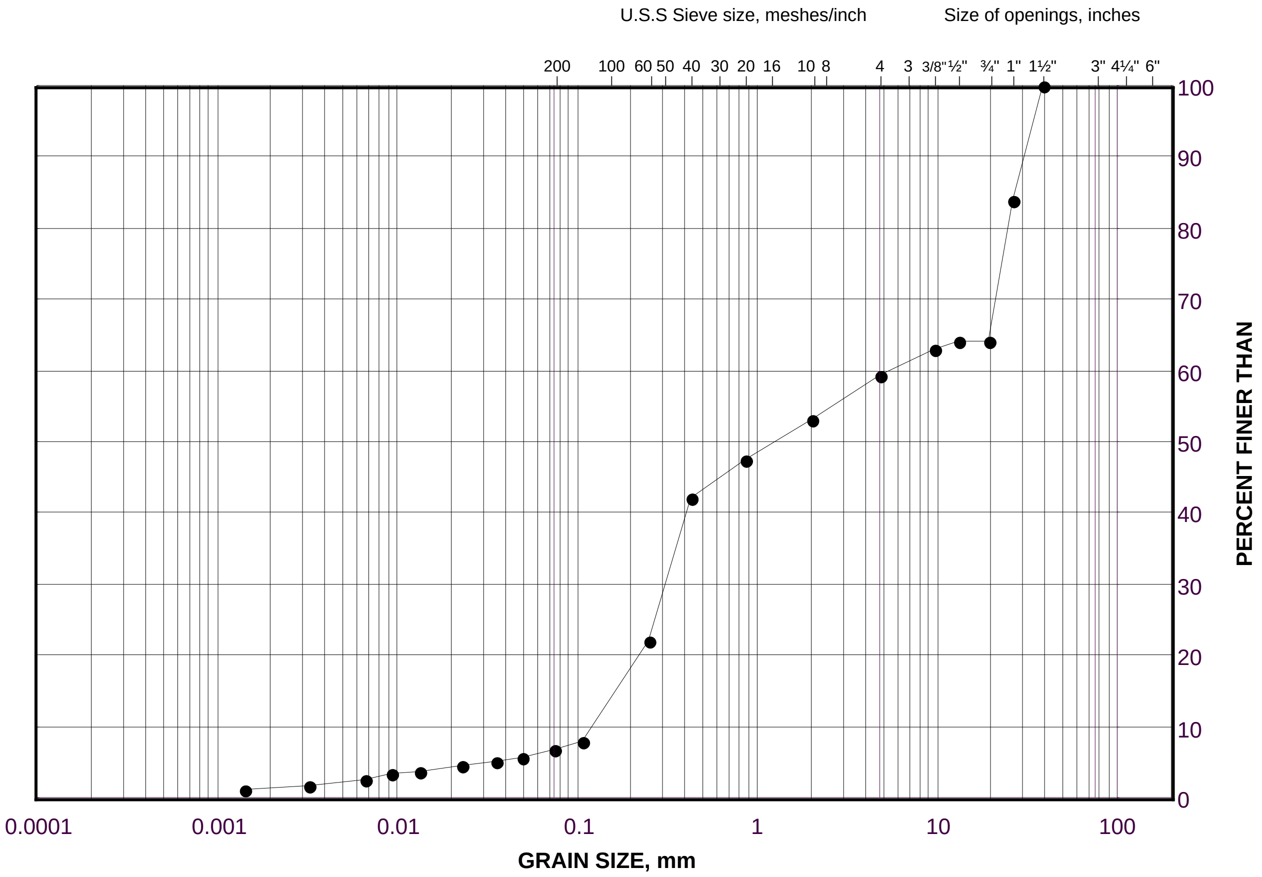
Project No. 1541610

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Fill)

FIGURE C-3



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-5	6	84.2

Project Number: 1541610

Checked By: SMM

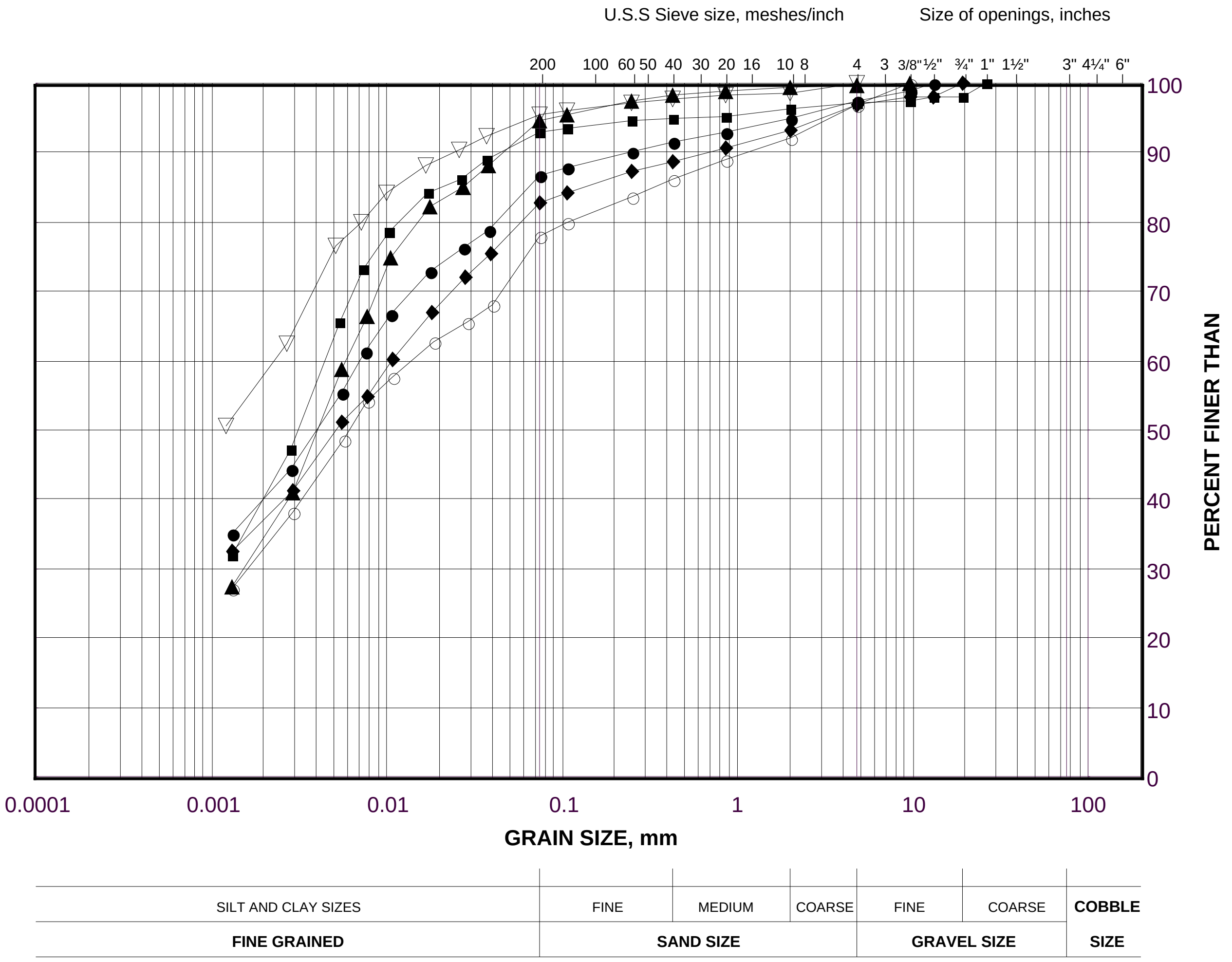
Golder Associates

Date: 23-Jul-18

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay to Clay

FIGURE C-4A



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-5	11	77.2
■	17-6	11	81.2
◆	17-1	15	77.9
▲	17-4	7	81.7
▽	17-2	8	83.9
○	17-1	9	88.6

Project Number: 1541610

Checked By: SMM

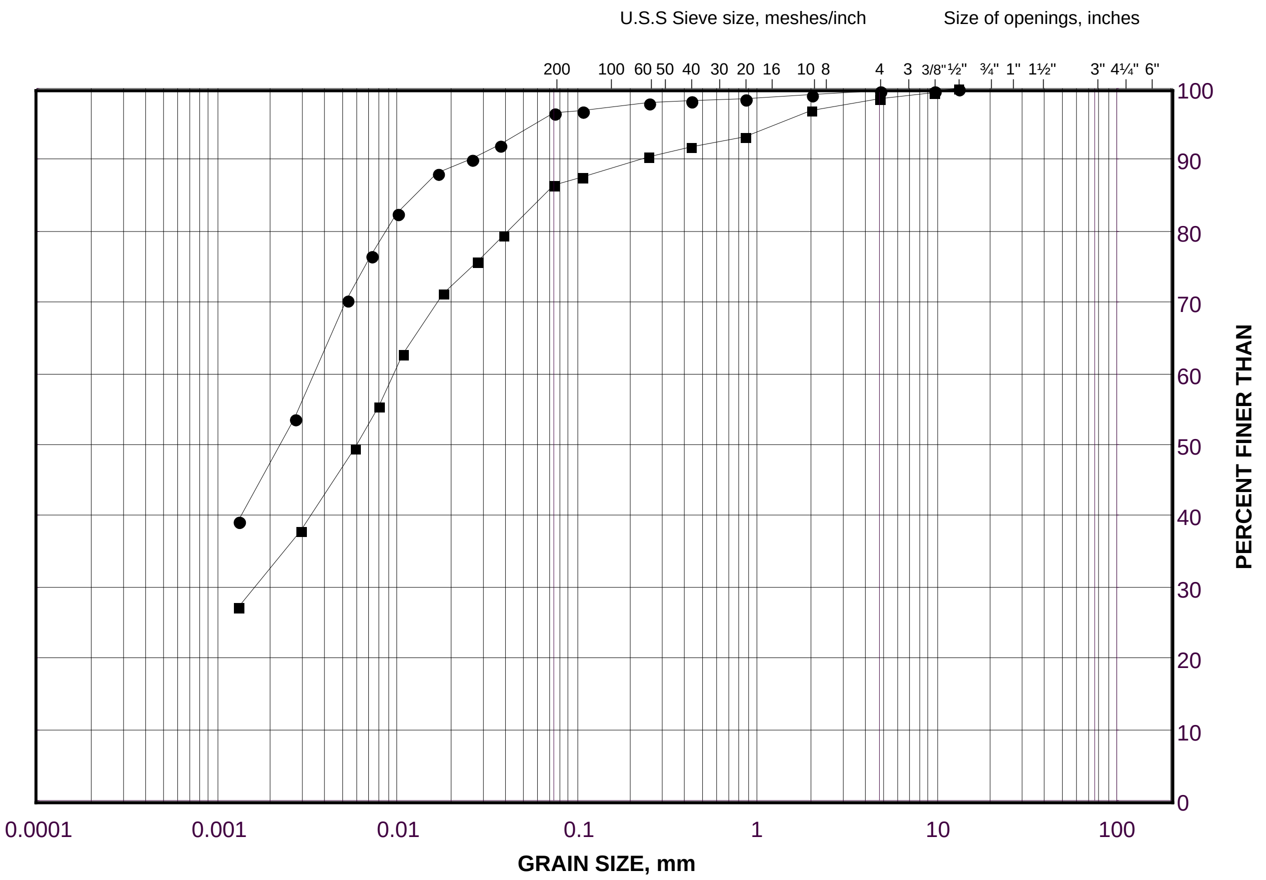
Golder Associates

Date: 23-Jul-18

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

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)
●	17-8	6	82.8
■	17-7	9	80.8

Project Number: 1541610

Checked By: SMM

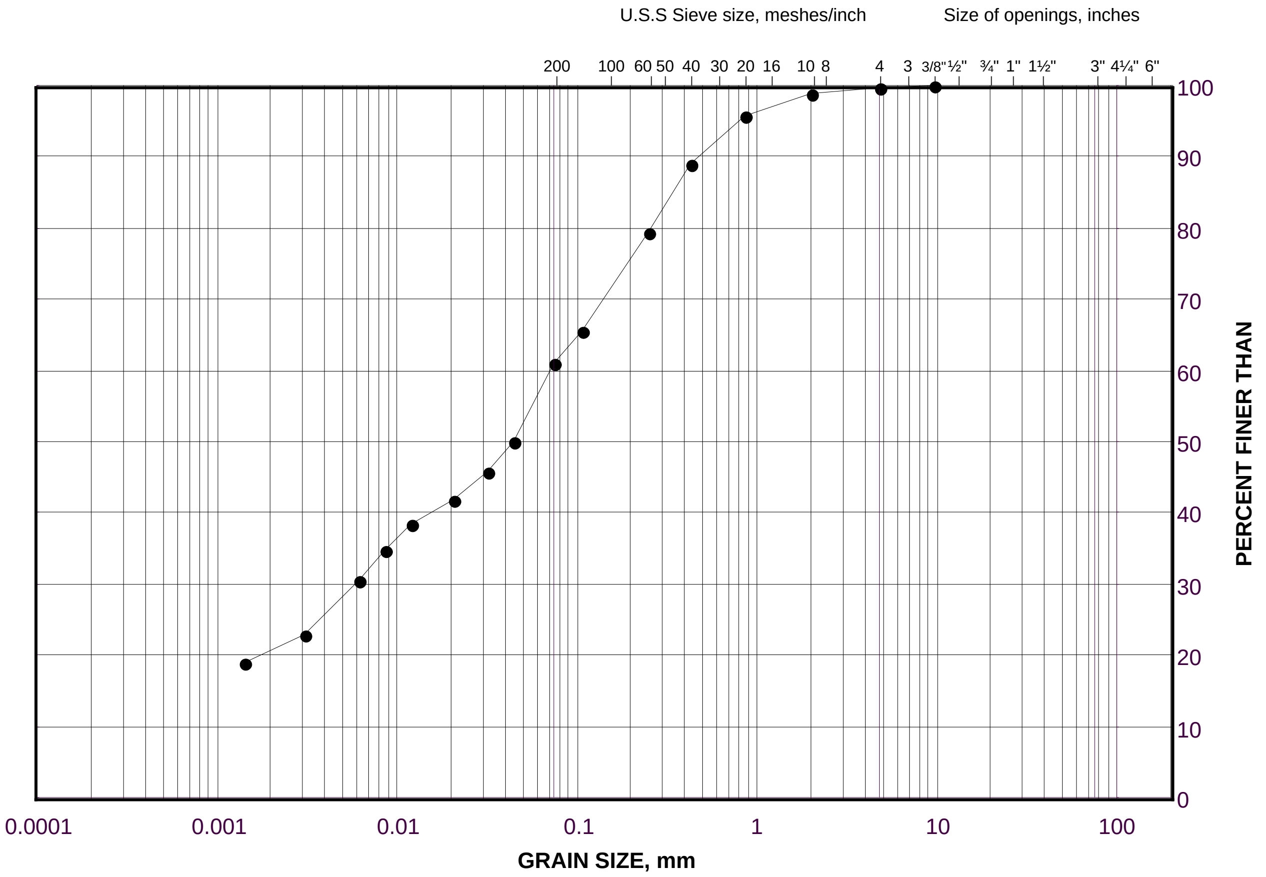
Golder Associates

Date: 23-Jul-18

GRAIN SIZE DISTRIBUTION

Clayey Silt with Sand

FIGURE C-4C



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-3	6	83.3

Project Number: 1541610

Checked By: SMM

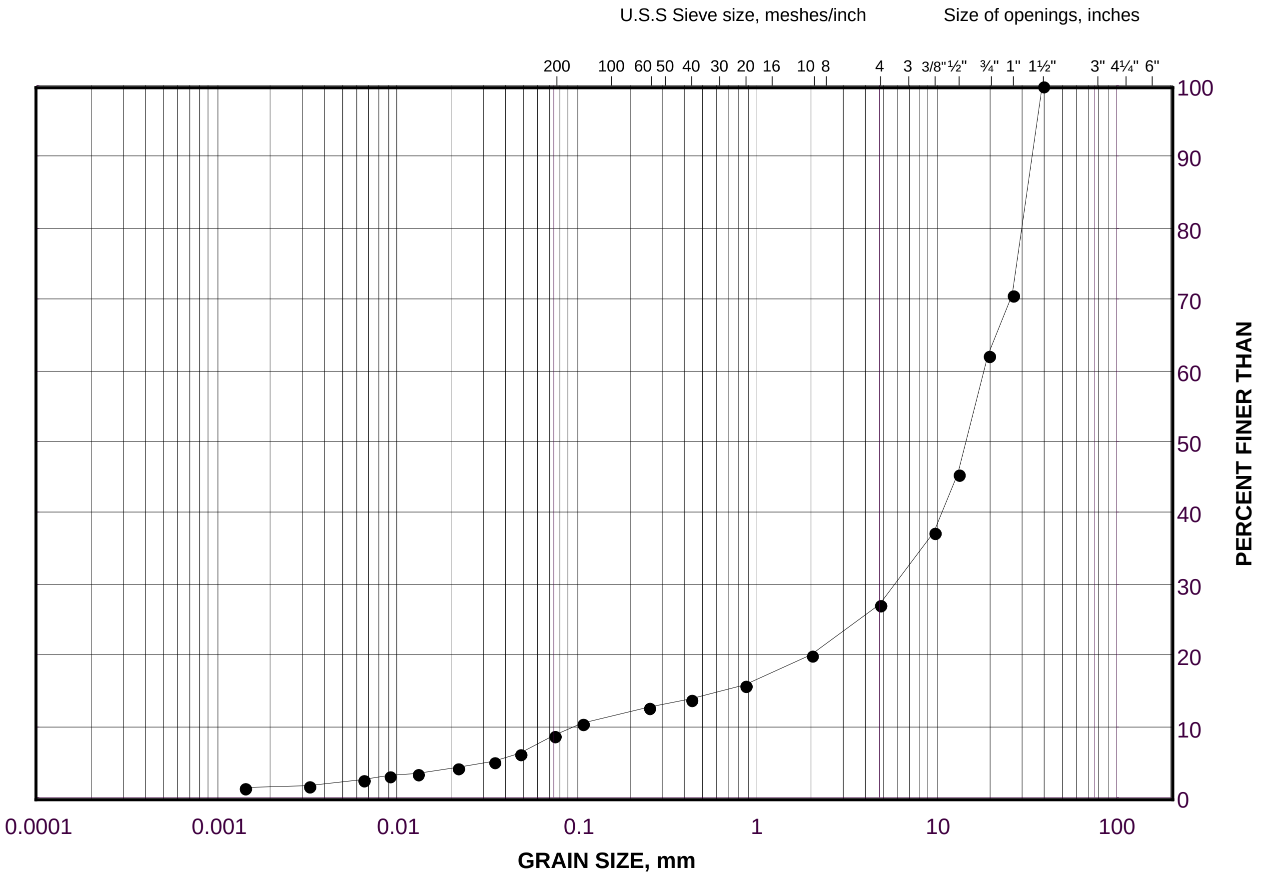
Golder Associates

Date: 23-Jul-18

GRAIN SIZE DISTRIBUTION

Gravel (Interlayer)

FIGURE C-5



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-7	5B	83.8

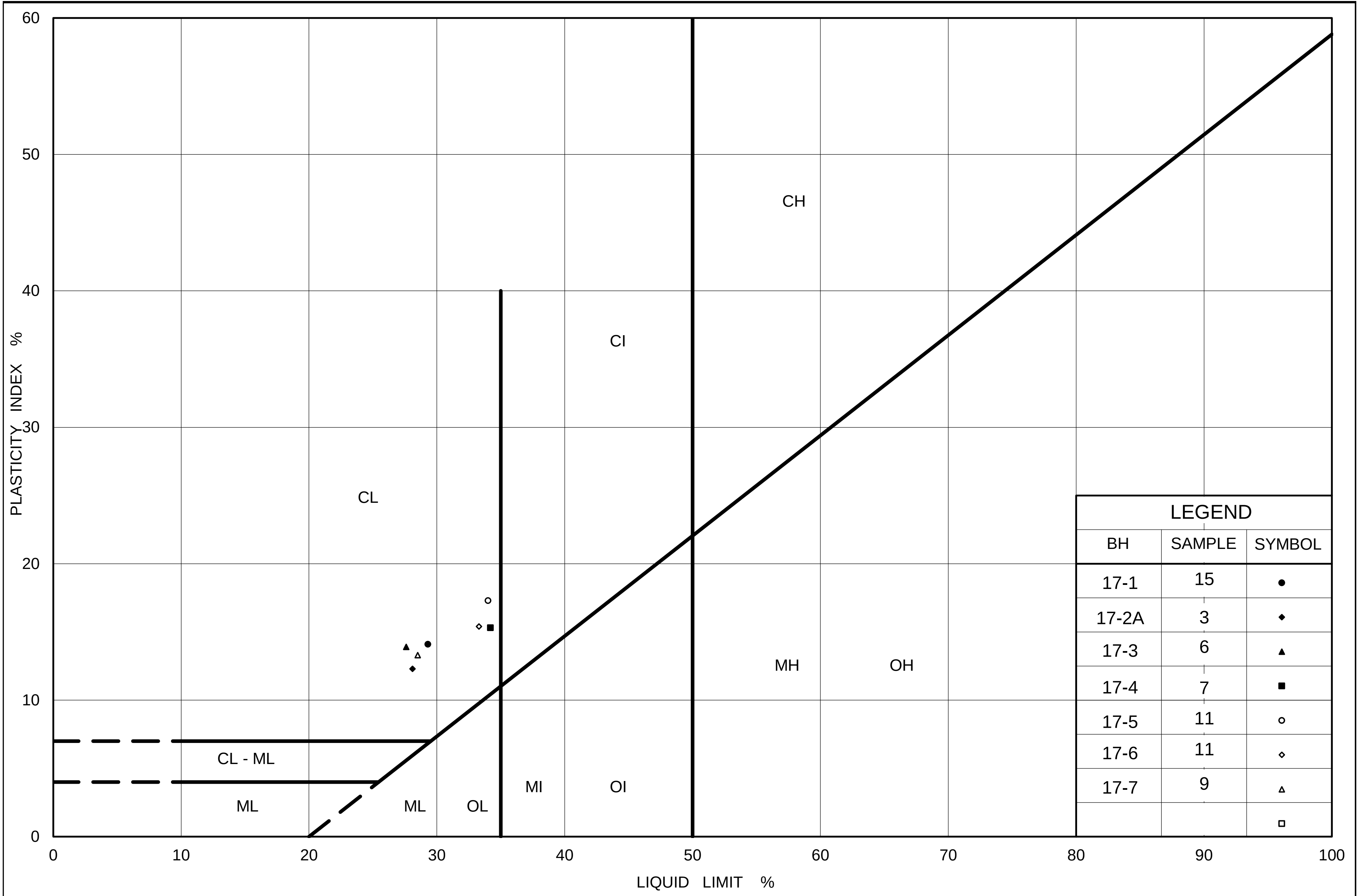
Project Number: 1541610

Checked By: SMM

Golder Associates

Date: 23-Jul-18

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
17-1	15	●
17-2A	3	◆
17-3	6	▲
17-4	7	■
17-5	11	○
17-6	11	◇
17-7	9	△
		□

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PLASTICITY CHART
Clayey Silt with Sand to Clayey Silt

Figure No. C-6A

Project No. 1541610

Checked By: SMM

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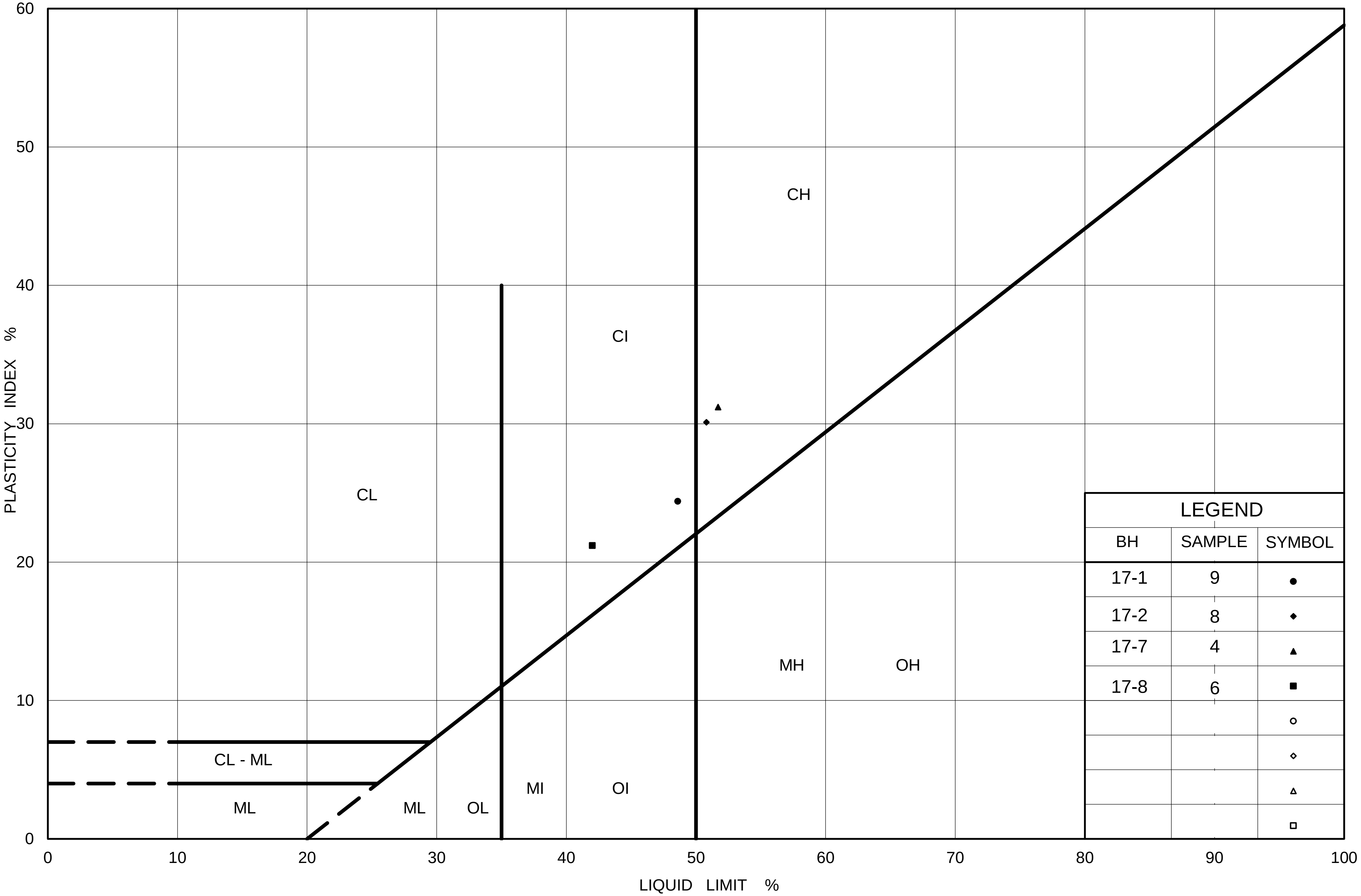

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PLASTICITY CHART
Silty Clay to Clay

Figure No. C-6B

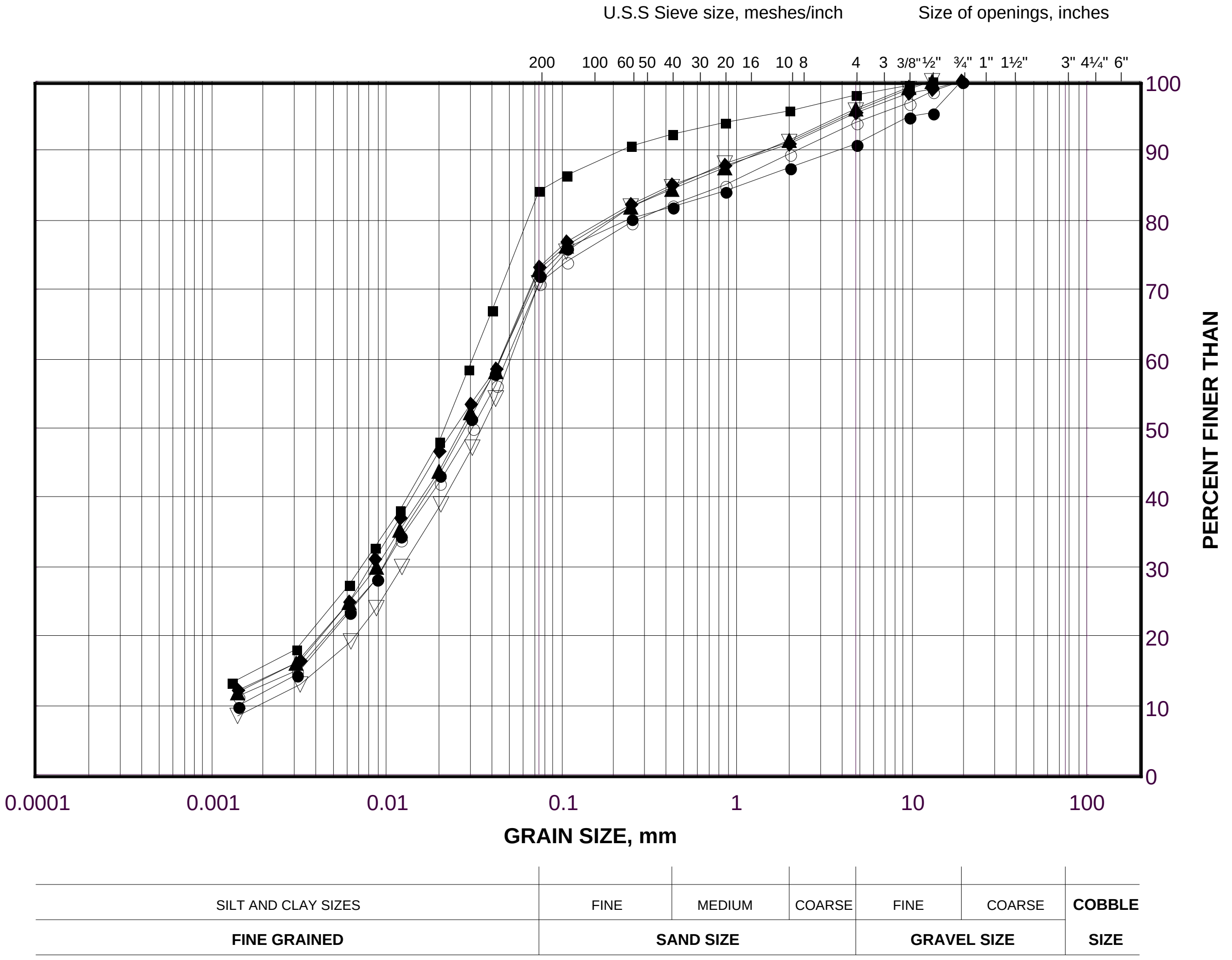
Project No. 1541610

Checked By: SMM

GRAIN SIZE DISTRIBUTION

Sandy Silt to Sandy Clayey Silt to Clayey Silt (Till)

FIGURE C-7



LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-4	12	74.1
■	17-5	14	72.8
◆	17-6	16	73.7
▲	17-1	17	74.8
▽	17-3A	4	69.6
❍	17-2A	4	74.0

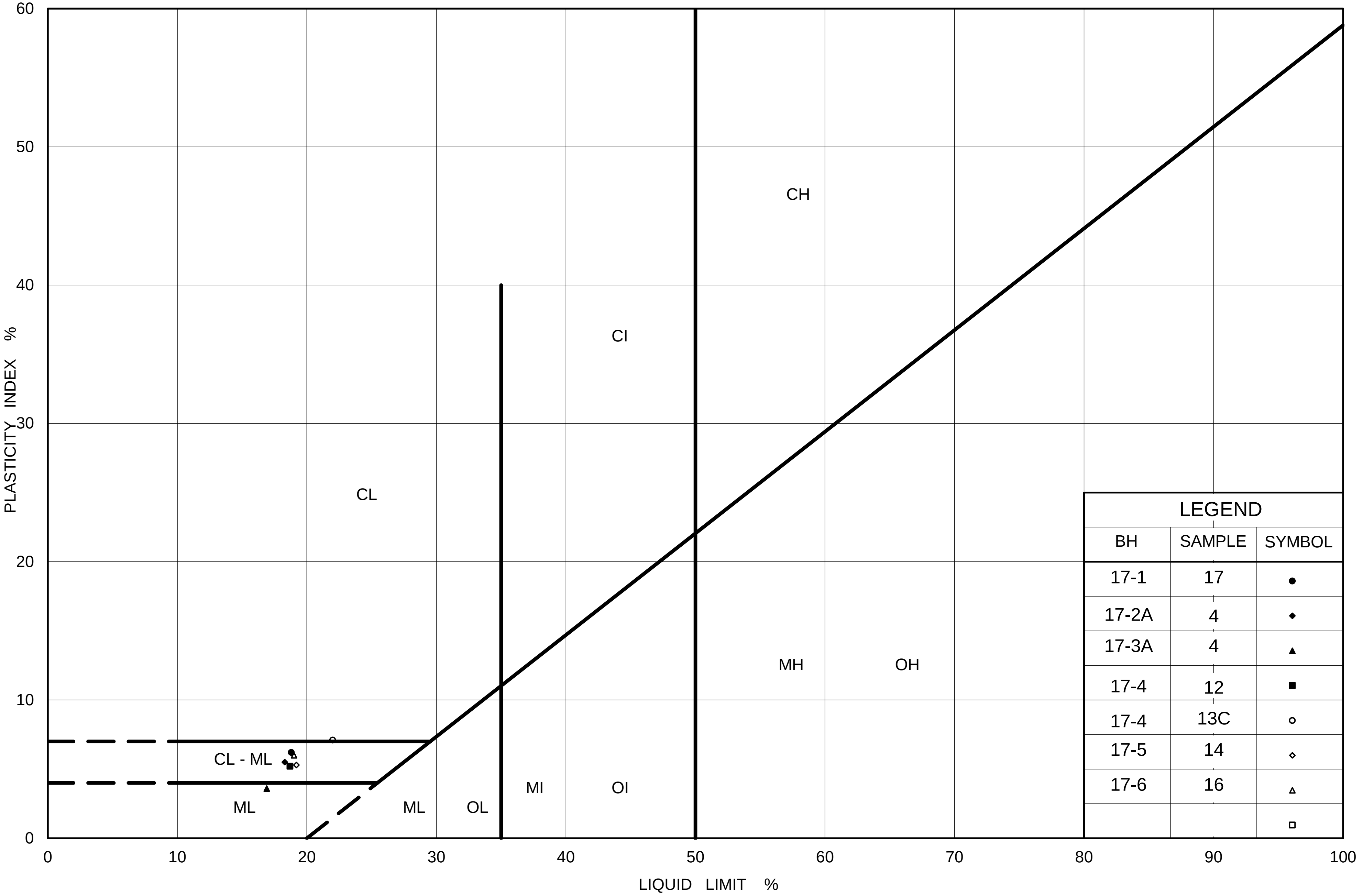
Project Number: 1541610

Checked By: SMM

Golder Associates

Date: 23-Jul-18

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
17-1	17	●
17-2A	4	◆
17-3A	4	▲
17-4	12	■
17-4	13C	○
17-5	14	◇
17-6	16	△
		□

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PLASTICITY CHART
Sandy Silt to Sandy Clayey Silt to Clayey Silt (Till)

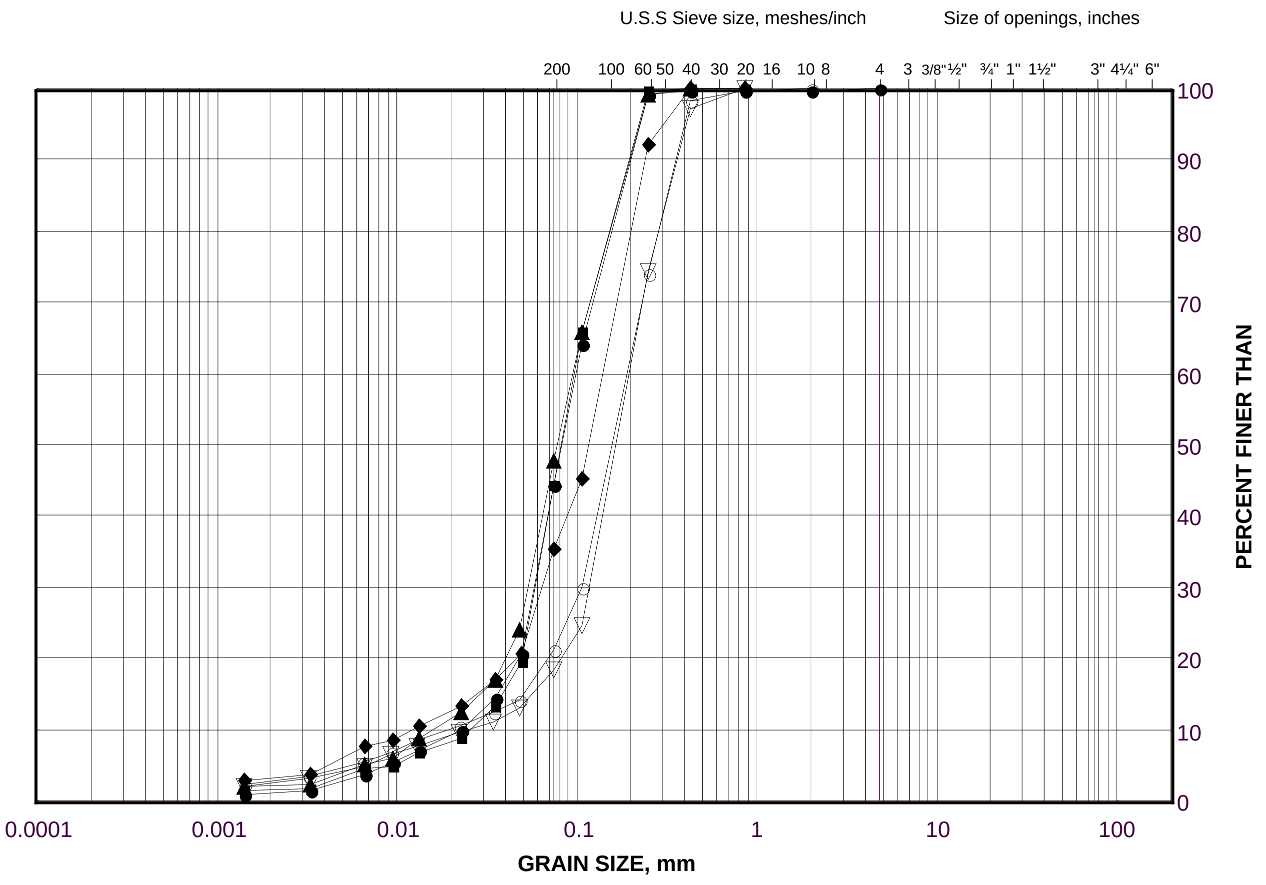
Figure No. C-8

Project No. 1541610

Checked By: SMM

GRAIN SIZE DISTRIBUTION
Silt and Sand to Sand

FIGURE C-9A



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-4	16	68.0
■	17-5	17	68.2
◆	17-6	18	72.1
▲	17-1	18	73.3
▽	17-1	20	70.3
❍	17-2A	8	70.9

Project Number: 1541610

Checked By: SMM

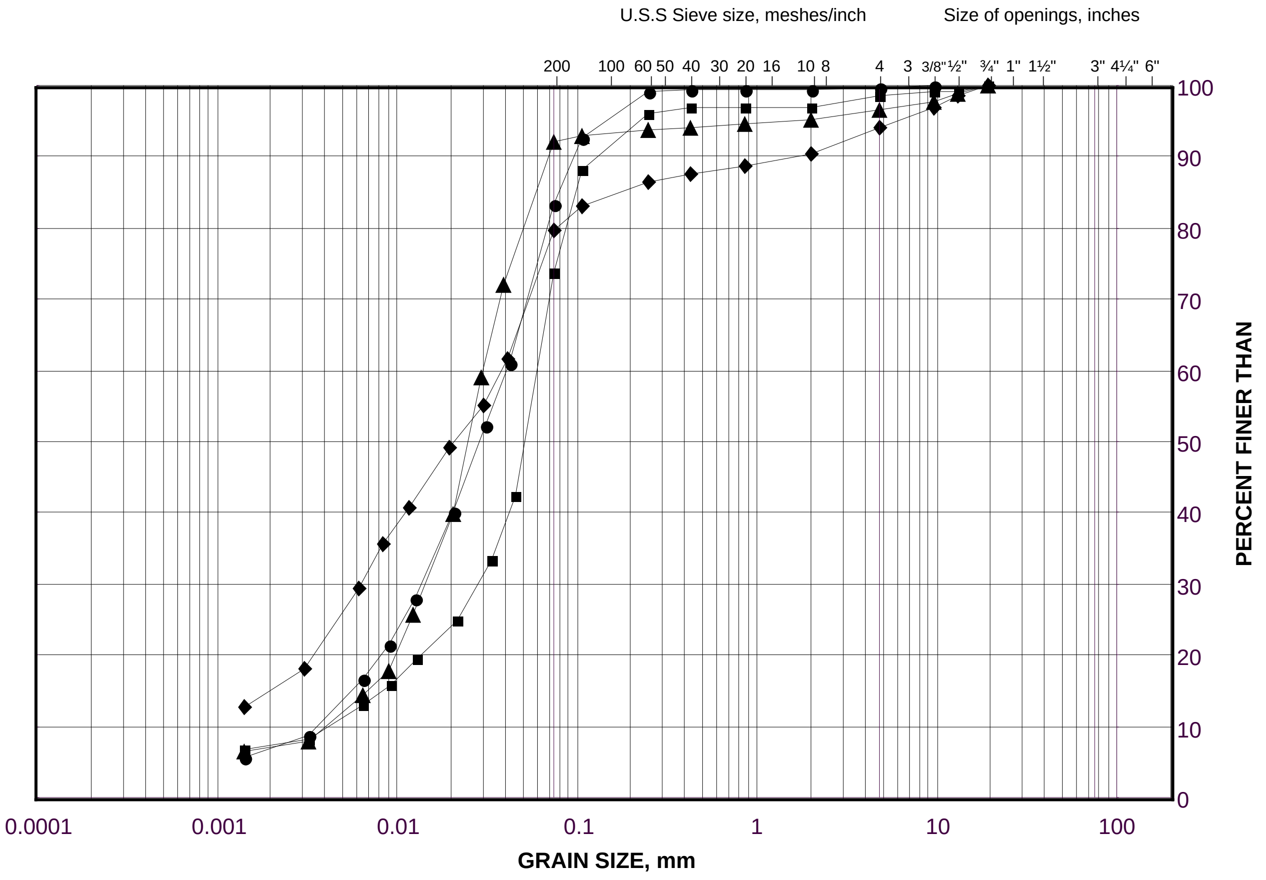
Golder Associates

Date: 23-Jul-18

GRAIN SIZE DISTRIBUTION

Silt to Sandy Silt

FIGURE C-9B



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

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	17-4	14	71.0
■	17-6	21	69.9
◆	17-2A	6A	72.7
▲	17-3A	7A	67.5

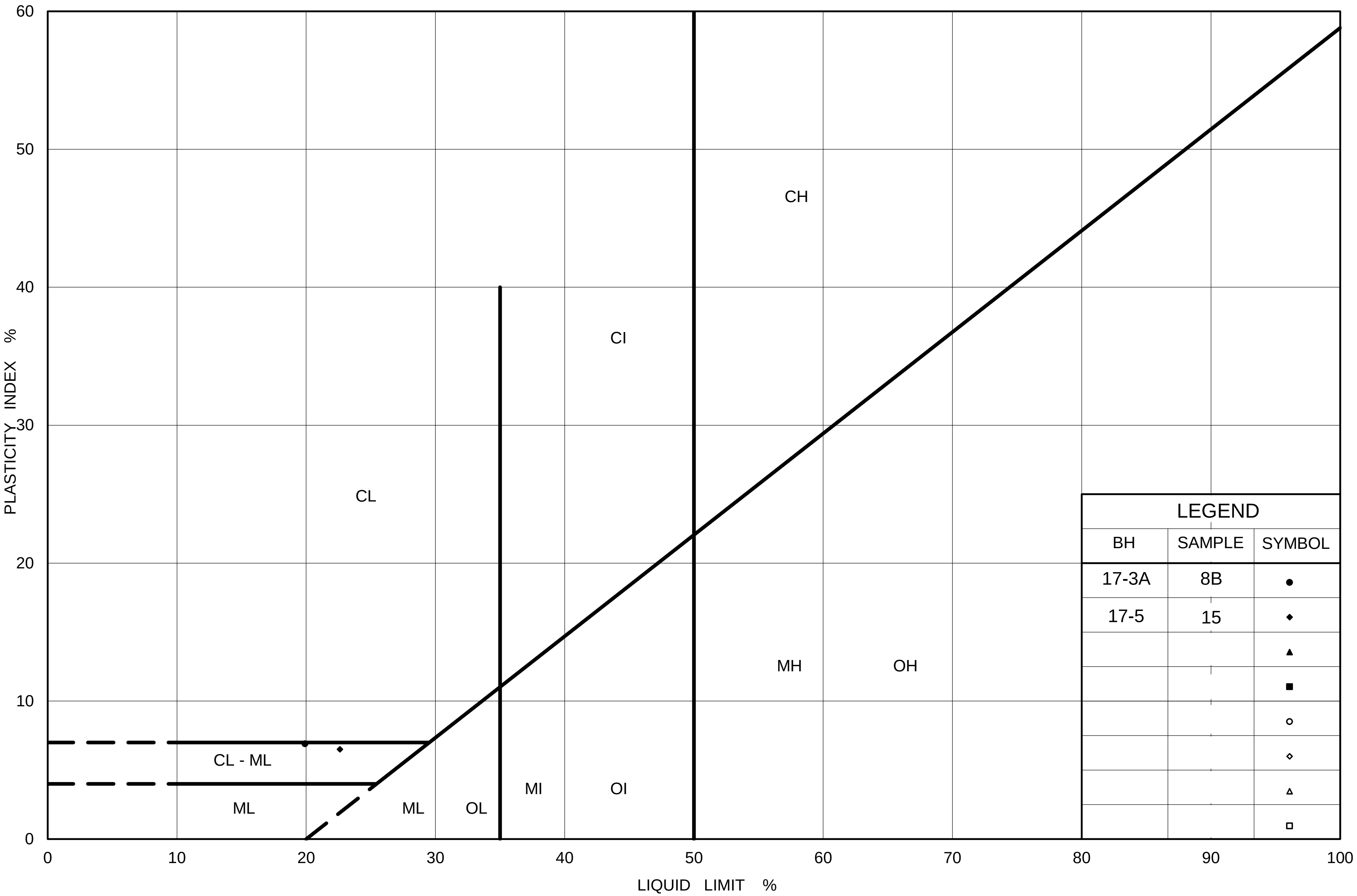
Project Number: 1541610

Checked By: SMM

Golder Associates

Date: 23-Jul-18

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
17-3A	8B	●
17-5	15	◆
		▲
		■
		○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt

Figure No. C-10

Project No. 1541610

Checked By: SMM

APPENDIX D

Analytical Laboratory – Results of Analysis of Soil

Your Project #: 1541610
Site Location: HWY 406
Your C.O.C. #: 81816

Attention:Sandra McGaghran

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
L5N 7K2

Report Date: 2017/11/14
Report #: R4857144
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7O8316
Received: 2017/11/06, 12:31

Sample Matrix: Soil
Samples Received: 3

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	3	N/A	2017/11/10	CAM SOP-00463	EPA 325.2 m
Conductivity	3	N/A	2017/11/09	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	3	2017/11/07	2017/11/07	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	3	2017/11/06	2017/11/09	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	3	N/A	2017/11/10	CAM SOP-00464	EPA 375.4 m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam’s profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix “m” indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Your Project #: 1541610
Site Location: HWY 406
Your C.O.C. #: 81816

Attention:Sandra McGaghran

Golder Associates Ltd
6925 Century Ave
Suite 100
Mississauga, ON
L5N 7K2

Report Date: 2017/11/14
Report #: R4857144
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7O8316
Received: 2017/11/06, 12:31

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B7O8316
Report Date: 2017/11/14

Golder Associates Ltd
Client Project #: 1541610
Site Location: HWY 406
Sampler Initials: KN

RESULTS OF ANALYSES OF SOIL

Maxxam ID		FML476	FML477	FML478	FML478		
Sampling Date		2017/11/01	2017/11/01	2017/11/01	2017/11/01		
COC Number		81816	81816	81816	81816		
	UNITS	**BH17-4-SS7**	**BH17-4-SS12**	**BH17-4-SS16**	**BH17-4-SS16 Lab-Dup**	**RDL**	**QC Batch**
Calculated Parameters							
Resistivity	ohm-cm	3200	2200	2200			5250950
Inorganics							
Soluble (20:1) Chloride (Cl)	ug/g	81	130	180	170	20	5257424
Conductivity	umho/cm	317	460	460		2	5257050
Available (CaCl2) pH	pH	7.71	8.42	8.64	8.66		5252165
Soluble (20:1) Sulphate (SO4)	ug/g	62	170	140	130	20	5257431

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

TEST SUMMARY

Maxxam ID: FML476
Sample ID: BH17-4-SS7
Matrix: Soil

Collected: 2017/11/01
Shipped:
Received: 2017/11/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5257424	N/A	2017/11/10	Deonarine Ramnarine
Conductivity	AT	5257050	N/A	2017/11/09	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5252165	2017/11/07	2017/11/07	Tahir Anwar
Resistivity of Soil		5250950	2017/11/09	2017/11/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5257431	N/A	2017/11/10	Deonarine Ramnarine

Maxxam ID: FML477
Sample ID: BH17-4-SS12
Matrix: Soil

Collected: 2017/11/01
Shipped:
Received: 2017/11/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5257424	N/A	2017/11/10	Deonarine Ramnarine
Conductivity	AT	5257050	N/A	2017/11/09	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5252165	2017/11/07	2017/11/07	Tahir Anwar
Resistivity of Soil		5250950	2017/11/09	2017/11/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5257431	N/A	2017/11/10	Deonarine Ramnarine

Maxxam ID: FML478
Sample ID: BH17-4-SS16
Matrix: Soil

Collected: 2017/11/01
Shipped:
Received: 2017/11/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5257424	N/A	2017/11/10	Deonarine Ramnarine
Conductivity	AT	5257050	N/A	2017/11/09	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5252165	2017/11/07	2017/11/07	Tahir Anwar
Resistivity of Soil		5250950	2017/11/09	2017/11/09	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5257431	N/A	2017/11/10	Deonarine Ramnarine

Maxxam ID: FML478 Dup
Sample ID: BH17-4-SS16
Matrix: Soil

Collected: 2017/11/01
Shipped:
Received: 2017/11/06

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5257424	N/A	2017/11/10	Deonarine Ramnarine
pH CaCl2 EXTRACT	AT	5252165	2017/11/07	2017/11/07	Tahir Anwar
Sulphate (20:1 Extract)	KONE/EC	5257431	N/A	2017/11/10	Deonarine Ramnarine

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	4.0°C

Results relate only to the items tested.

QUALITY ASSURANCE REPORT

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5252165	Available (CaCl2) pH	2017/11/07			99	97 - 103			0.15 (1)	N/A
5257050	Conductivity	2017/11/09			100	90 - 110	<2	umho/cm	1.4 (2)	10
5257424	Soluble (20:1) Chloride (Cl)	2017/11/10	NC (3)	70 - 130	103	70 - 130	<20	ug/g	4.1 (1)	35
5257431	Soluble (20:1) Sulphate (SO4)	2017/11/10	NC (3)	70 - 130	103	70 - 130	<20	ug/g	9.7 (1)	35

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

(1) Duplicate Parent ID [FML478-01]

(2) Duplicate Parent ID

(3) Matrix Spike Parent ID [FML478-01]

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Brad Newman, Scientific Service Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam
A Bureau Veritas Group Company

6740 Campobello Road, Mississauga, Ontario L5N 2L8
Phone: 905-817-5700 Fax: 905-817-5779 Toll Free: 800-563-6266
CAM FCD-01191/2

CHAIN OF CUSTODY RECORD 81816

Page 1 of 1

Invoice Information

Company Name: Golder Associates
Contact Name: Sandra McGaghran
Address: 6925 Century Avenue #100
Mississauga ON L5N 7K2
Phone: 905-567-4444 Fax: 905-567-6561
Email: Sandra_McGaghran@golder.com

Report Information (if differs from invoice)

Company Name:
Contact Name:
Address:
Phone: Fax:
Email:

Project Information (where applicable)

Quotation #:
P.O. #/ AFE#:
Project #: 1541610
Site Location: HWY 406
Site #:
Sampled By: K. Nero

Turnaround Time (TAT) Required

☒ Regular TAT (5-7 days) Most analyses

PLEASE PROVIDE ADVANCE NOTICE FOR RUSH PROJECTS

Rush TAT (Surcharges will be applied)

☐ 1 Day ☐ 2 Days ☐ 3-4 Days

Date Required:

Rush Confirmation #:

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153

☐ Table 1 ☐ Res/Park ☐ Med/ Fine
☐ Table 2 ☐ Ind/Comm ☐ Coarse
☐ Table 3 ☐ Agri/ Other
☐ Table ______
FOR RSC (PLEASE CIRCLE) Y / N

Other Regulations

☐ CCME ☐ Sanitary Sewer Bylaw
☐ MISA ☐ Storm Sewer Bylaw
☐ PWQO Region ______
☐ Other (Specify) ______
☐ REG 558 (MIN. 3 DAY TAT REQUIRED)

Include Criteria on Certificate of Analysis: Y / N

SAMPLES MUST BE KEPT COOL (< 10 °C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

LABORATORY USE ONLY

CUSTODY SEAL Y / [✓ N]		COOLER TEMPERATURES
Present	Intact	
		4/4/3

COOLING MEDIA PRESENT: Y [✓ N]

Analysis Requested

	SAMPLE IDENTIFICATION	DATE SAMPLED (YYYY/MM/DD)	TIME SAMPLED (HH:MM)	MATRIX	# OF CONTAINERS SUBMITTED	FIELD FILTERED (CIRCLE) Metals / Hg / CrVI	BTEX/ PHC F1	PHCs F2 - F4	VOCs	REFER TO BACK OF COC: REG 153 METALS & INORGANICS	REFER TO BACK OF COC: REG 153 ICPMS METALS	REFER TO BACK OF COC: REG 153 METALS (Hg, Cr VI, ICPMS Metals, HWS - B)	Corrosivity Package (pH, sulphate, Chloride, resistivity, electrical conductivity)	HOLD- DO NOT ANALYZE	COMMENTS
1	BH17-4-SS7	2017/11/01	AM	Soil	1								X		
2	BH17-4-SS12	"	AM	Soil	1								X		standard Corrosivity Package
3	BH17-4-SS16	"	AM	Soil	1								X		"
4															"
5															
6															
7															
8															
9															
10															

06-Nov-17 12:31
Ema Gitej
B708316
TSP ENV-612

RELINQUISHED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	RECEIVED BY: (Signature/Print)	DATE: (YYYY/MM/DD)	TIME: (HH:MM)	MAXXAM JOB #
Katie Nero / Katie Nero	2017/11/06	12:30		2017/11/06	12:31	

COC-1004 (10/14) - ENV. ENG.

White: Maxxam ~ Yellow: Client

APPENDIX E

Special Provisions

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision No. FOUN0003 March 8, 2018

Amendment to OPSS 902, November 2010

OPSS 902, November 2010, Construction Specification for Excavating and Backfilling – Structures, is amended as follows:

902.02 REFERENCES

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

Ontario Provincial Standard Specifications, Construction

OPSS 517	Dewatering
OPSS 805	Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

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

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

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

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 10 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

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

902.04.02.01 Working Drawings

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

902.04.02.02 Preconstruction Survey

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

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

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

902.04.02.03 Milestone Inspections

Clause 902.04.02.03 of OPSS 902 is deleted in its entirety.

902.07 CONSTRUCTION

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

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

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

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

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

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

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

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

Unwatering shall be carried out as necessary.

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

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

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

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

902.07.04.05 Removal

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

GEOTECHNICAL ASSESSMENT for (CRANE PAD / STOCKPILE AREAS - Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

The impact of the heavy equipment loads on the underlying firm to very stiff silty clay to clay soils, and existing bridge foundations must be considered during selection of the crane pad and material stockpile locations, and methodology and equipment employed for construction of the Highway 406 – Geneva Street N/S Ramp Bridge. For bidding purposes:

- At no time shall any heavy equipment (i.e. cranes, pile driving rigs, etc.) be parked or driven, or material stockpiles be placed in the immediate vicinity of the existing bridge.

- For a loaded area up to 4 m in wide, ground pressure on the existing fill or native soil subgrade due to construction traffic/equipment and/or material stockpiling, shall not exceed 100 kPa, and no construction traffic/equipment and/or material stockpiling shall be allowed within a horizontal distance from the crest of a slope / bank less than or equal to 1.5 times the height of the slope / bank (e.g. embankment slope, existing valley fill slope, channel / creek bank / slope).

2.0 REFERENCES

Foundation Investigation Report, Highway 406 – Geneva Street N/S Ramp Structure Site 18-168, from Fourth Avenue to Westchester Avenue, St. Catharines, Ontario, MTO GWP 2453-13-00, GEOCRES No. 30M3-306.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Design Requirements

Prior to commencement of construction, the Contractor shall retain a Geotechnical Consultant to select the crane pad location, assess the geotechnical suitability of the area to safely support the proposed equipment loads structure stockpile loads and impact of his construction methodology, and determine requirements and/or restrictions necessary to safely support the loads associated with his equipment and material stockpiles employed in the construction of the new bridge. All foundation engineering services required for this project shall be performed by a firm listed under MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

The geotechnical assessment carried out by the Contractor's Geotechnical Consultant shall include, but not be limited to, the following:

- Review of available geotechnical information and supplementing with additional subsurface information, as required, in the equipment pad/access material stockpiles and road areas;
- Determine appropriate and safe setbacks for heavy equipment and any material stockpiles from the crest(s) of any new and existing slopes, and from new and existing foundations;
- Determine the permissible ground pressure (with due consideration to both bearing capacity and global stability) that may be applied to the foundation soils and/or embankment fills by the equipment and material stockpiles;
- Provide recommendations for the distribution and support of all heavy equipment loads (including crane and pile-driving equipment loads) and material stockpiles to prevent foundation failure (either in bearing capacity or in global stability) at the crane pad / and material stockpile locations at any locations along the existing structure and slopes and access roads to the equipment pads, based on the proposed methodology of the Contractor.

4.1 Submission Requirements

The Contractor shall submit the geotechnical assessment report containing details of the proposed crane pad area for equipment and material stockpiles and construction methodology to the Contract Administrator for information purposes a minimum of two weeks prior to the start of construction.

10.0 Basis of Payment

Payment at the Contract price for the above tender items shall be full compensation for all labour to do the work.

Payment for costs associated with heavy construction equipment and materials necessary to complete the work, such as design and construction of temporary works, supply, mobilization/de-mobilization, and operation shall be made under the associated items.

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