February 11, 2015

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Lyons Creek Bridges (Site Nos. 36-66/1 and 36-66/2)
QEW Structure Replacements at Black Creek, Lyons Creek, Seventh Street and Tee Creek
Regional Municipality of Niagara
GWP 2177-08-00

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REPORT

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
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QEW STRUCTURE REPLACEMENTS AT BLACK CREEK, LYONS CREEK, SEVENTH STREET AND TEE CREEK,
REGIONAL MUNICIPALITY OF NIAGARA
G.W.P 2177-08-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement/rehabilitation of seven existing structures (Seventh Street, Lyons Creek, Tee Creek and Black Creek) on the Queen Elizabeth Way (QEW) highway in the Regional Municipality of Niagara, Ontario.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2011-E-0045 dated June 2011, and in Section 5.8 of the *Technical Proposal* for this assignment.

This report addresses the results of the subsurface investigation carried out for the proposed replacement of the existing Lyons Creek bridges.

This preliminary Foundation Investigation and Design Report is for planning purposes only and the Design/Build proponent shall satisfy himself as to the sufficiency of the available information and supplement the information as needed to meet the requirements for detail design.

2.0 SITE DESCRIPTION

The Lyons Creek bridges carry QEW southbound (Fort Erie bound) and northbound (Toronto bound) traffic over the Lyons Creek which is located south of the Lyons Creek Road underpass in the City of Niagara Falls, within the Regional Municipality of Niagara, Ontario.

Lyons Creek is a relatively shallow stream with its width approximately 15 m and it flows from west to east with the high water level at the existing bridge site at approximately Elevation 171.2 m.

In general, the topography along this section of the QEW is relatively flat. The existing ground surface at the borehole locations on the QEW ranges between Elevations 174.3 m and 175.0 m, referenced to Geodetic datum. The existing QEW embankments are up to about 5 m high at the north and south approaches. The areas adjacent to the bridges are sparsely treed and have been developed as residential and recreational properties.

Each of the existing bridges consists of a variable depth cast-in-place concrete T-beam structure with a 19.7 m centre span and two 5.3 m cantilevered end spans, for a total length of 30.3 m and a width of 13 m. Based on the detail design drawings for the piers of the Lyons Creek Bridges (Drawing No. 191-15-3), the foundations of the existing piers consist of sheet piles with Cruciform shaped cross sections driven to practical refusal. The soils inside the piles were then excavated or partially excavated following driving and the resulting void was filled with concrete. There are no design elevations shown on the design drawings and, therefore, the extent of the sheet piles (depth and width), the extent of the soil excavation inside of the piles and the thickness of the concrete fill are unknown at this time.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigations

As part of the QEW and Lyons Creek interchange construction in the late 1960's, a subsurface investigation was carried out as listed below:

MTO GEOCRES No. 30M03-111: Report titled "Foundation Investigation Report for Proposed S.-E.W. Ramp Crossing at Lyons Creek Q.E.W and Lyons Creek Interchange District No. 4 (Hamilton) W. J. 68-P-8 – W.P. 158-64-3", by Department of Highways – Ontario, dated March 20, 1968.

The above referenced previous investigation consisted of drilling five boreholes, designated as Boreholes 1 to 5 near the QEW-Lyons Creek exit ramp, at which 3 boreholes were extended into bedrock (refer to Section 4.2).

3.2 Current Investigation

The field work for this subsurface investigation was carried out between June 18 and 20, 2013 and between July 7 and 11, 2013, at which time four boreholes (Boreholes 13-03 to 13-06) were advanced adjacent to the existing abutment locations. The boreholes were advanced using a track-mounted CME-55 drill rig supplied and operated by Geo-Environmental Drilling Inc. of Milton, Ontario. The boreholes were advanced through the overburden using 108 mm inside diameter (I.D.) hollow stem augers. Soil samples were obtained at 0.75 m and 3.0 m intervals of depth using a 50 mm outside diameter (O.D.) split-spoon sampler driven by an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586, Standard Test Method for Standard Penetration Test).

The boreholes at the locations of the foundation elements were advanced to practical auger refusal on inferred bedrock to depths up to 38.1 m below the QEW pavement surface.

The groundwater conditions were observed within the hollow stem augers in selected boreholes during and upon completion of the drilling operations and the observed water levels are indicated on the Record of Borehole sheets contained in Appendix A. All boreholes were backfilled with bentonite pellets and capped with asphalt patches upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes in the field, completed utility clearances, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further visual examination and then to the Cambridge laboratory for testing. Index and classification tests consisting of water content and organic content determinations, Atterberg limits and grain size distribution were carried out on selected soil samples. The geotechnical laboratory testing was completed according to applicable MTO LS standards. The as-drilled borehole locations and ground surface elevations were determined in the field by Callon Dietz, Ontario Land Surveyors. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and drilled depth are summarized below and are shown on Drawing 1.

Foundation Element	Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m)	Easting (m)		
Lyons Creek Bridge SBL – Northwest Abutment	13-03	4,765,781.6	336,487.0	175.0	35.4
Lyons Creek Bridge SBL – Southwest Abutment	13-04	4,765,750.7	336,512.4	174.5	38.1
Lyons Creek Bridge NBL – Southeast Abutment	13-05	4,765,768.8	336,538.3	174.3	36.9
Lyons Creek Bridge NBL – Northeast Abutment	13-06	4,765,798.4	336,509.5	174.9	35.4

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of QEW is located in the Haldimand Clay Plain physiographic region as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[1].

The Haldimand Clay Plain physiographic region is a broad undulating plain of glaciolacustrine surface sediments which covers an area of about 3,500 square km. The region mostly contains lacustrine clay deposits overlying clay till which is turn underlain by shale and dolostone bedrock of the Salina formation.

4.2 Subsurface Conditions

As part of this subsurface investigation, four boreholes were advanced in the vicinity of the existing Lyons Creek bridge abutments. The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during the field investigation, together with the results of the in situ and laboratory tests carried out on selected soil samples are presented on the Record of Borehole sheets contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 to B17 contained in Appendix B. The results of the in situ field tests (i.e. SPT 'N'-values and field vane results) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profiles on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations. The interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions.

In summary, the subsoil conditions encountered at the site consist of cohesionless fill and organic layers underlain by a relatively thick deposit of clayey silt which has pockets of cohesive till within its lower portion. The cohesive deposit is underlain by deposits of silt to silt and sand to sand in places inter-bedded by a layer of or pockets of clayey silt. These non-cohesive deposits are underlain by a deposit of sand and gravel which is underlain by inferred bedrock, as evidenced by refusal to auger advancement in two boreholes. All of the boreholes were advanced to practical refusal either on deposits for which the "N" values are greater than 100 blows per 0.3 m of penetration or inferred bedrock.

[1] Chapman, L. J. and Putnam, D. F., 1984. The Physiography of Southern Ontario, Ontario Geological Society, Special Volume 2, Third Edition.

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

4.2.1 Asphalt/Concrete

An approximately 175 mm to 380 mm thick layer of asphalt was encountered immediately below the road level in Boreholes 13-03, 13-05 and 13-06 that were advanced through the existing pavement structure. The asphalt layer is underlain by 300 mm of concrete in Borehole 13-06. The surface of the asphalt layer ranges from Elevations 175.0 m to 174.3 m at the borehole locations.

4.2.2 Cohesionless Fill

A 0.9 m to 2.2 m thick layer of cohesionless fill comprised of brown to grey sand and gravel to brown to reddish-brown sand was encountered below the asphalt/concrete layer(s) in Boreholes 13-03, 13-05 and 13-06 and below the ground surface in Borehole 13-04 and extends to depths ranging between 1.5 m and 2.2 m (Elevations 173.5 m and 172.3 m). The deposit generally contains trace to some silt and trace clay.

The SPT 'N'-values measured within the cohesionless fill deposit range from 7 blows to 55 blows per 0.3 m of penetration, indicating that the cohesionless fill is loose to very dense.

The natural water content measured on five samples of the fill deposit ranges from about 3 per cent to 26 per cent. The results of grain size distribution tests completed on two samples of the fill are shown on Figure B1 in Appendix B.

4.2.3 Cohesive Fill

A 0.8 m to 4.0 m thick layer of cohesive fill comprised of silty clay to clay was encountered underlying the cohesionless fill in all boreholes. The deposit generally contains trace sand and trace organics. The deposit extends to depths ranging between 3.0 m and 5.5 m (Elevations 171.5 m and 169.4 m).

The SPT 'N'-values measured within the cohesive fill deposit range from 4 blows to 8 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strength ranging from about 86 kPa to greater than 96 kPa, with a sensitivity of 2. The SPT 'N'-values and field vane tests results suggest that the cohesive fill has a firm to stiff consistency.

The natural water content measured on samples obtained from the cohesive fill ranges from about 17 per cent to 42 per cent. The results of grain size distribution tests completed on two samples of the cohesive fill are shown on Figure B2 in Appendix B. Atterberg limits tests were carried out on three samples of the cohesive fill deposit and measured liquid limits ranging from about 47 per cent to 52 per cent, plastic limits ranging from about 22 per cent to 24 per cent and plasticity indices ranging from about 23 per cent to 30 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B3 in Appendix B, and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity.

4.2.4 Clayey Organic Silt

A 0.9 m to 2.5 m thick deposit of dark grey to black clayey organic silt was encountered underlying the cohesive fill deposit in Boreholes 13-03, 13-04 and 13-05. The deposit extends to depths of about 5.5 m (Elevations 169.5 m and 168.8 m).

The SPT 'N'-values measured within the clayey organic silt deposit range from 1 blow to 4 blows per 0.3 m of penetration. Two In situ field vane tests carried out within this deposit measured undrained shear strength of about 81 kPa and greater than 96 kPa, with a sensitivity of 2. The SPT 'N'-values and field vane tests results suggest that the clayey organic silt deposit has a very soft to stiff consistency.

The natural water content measured on four samples of the clayey organic silt deposit ranges from about 35 per cent to 120 per cent. The organic contents measured for two samples of this deposit are about 9 per cent and 20 per cent. The result of a grain size distribution completed on a sample of the clayey organic silt deposit is shown on Figure B4 in Appendix B. An Atterberg limits test was carried out on a sample of the clayey organic silt deposit and measured a liquid limit of about 52 per cent, a plastic limit of about 24 per cent and a corresponding plasticity index of about 28 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B5 in Appendix B indicating that the material is classified as clayey organic silt of high plasticity.

4.2.5 Clayey Silt

A deposit of brown to grey clayey silt was encountered below the cohesive fill deposit in Borehole 13-06 and underlying the clayey organic silt deposit in Boreholes 13-03 to 13-05. The thickness of the deposit varies between 12.3 m and 14.6 m, including the thickness of the clayey silt till pocket in Borehole 13-05, and extends to depths ranging from 17.8 m to 20.1 m below ground surface (Elevations 157.1 m to 154.8 m). The deposit generally contains trace to some sand, trace to some gravel and silt seams throughout. Within the lower portion of this deposit, an approximately 1.3 m thick pocket of clayey silt with sand till was encountered in Borehole 13-05 at about Elevation 157.8 m.

The SPT 'N'-values measured within this deposit range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strength ranging from about 47 kPa to greater than 96 kPa with sensitivity ranging between 1 and 3. The field vane tests results indicate that the silty clay deposit has a firm to stiff consistency.

The natural water content measured on thirty-three samples of this cohesive deposit ranges from about 22 per cent to 30 per cent. The results of grain size distribution tests completed on three samples of this cohesive deposit are shown on Figure B6 in Appendix B. Atterberg limits tests were carried out on nine samples of this cohesive deposit and measured liquid limits ranging from about 24 per cent to 35 per cent, plastic limits ranging from about 11 per cent to 18 per cent, and plasticity indices ranging from about 13 per cent to 18 per cent. The results of the Atterberg limits tests are shown on the plasticity chart on Figure B7 in Appendix B, and indicate that the material is classified as clayey silt of low plasticity.

4.2.6 Cohesive Till

Pockets of cohesive till comprised of sandy clayey silt to clayey silt with sand between approximately 1.3 m and 1.5 m thick were encountered within the lower portion of the clayey silt deposit at Elevation 157.8 m in Borehole 13-05 and underlying the clayey silt deposit at Elevation 156.2 m in Borehole 13-04. Grinding of the augers and bouncing of the split-spoon sampler were observed during the drilling operation and may be an indication of the presence of cobbles or boulders within this deposit.

SPT 'N'-values of 10 blows per 0.3 m of penetration and 50 blows per 0.1 m of penetration were measured within the till deposit, suggesting a stiff to hard consistency.

The natural water content measured on two samples of the till is about 10 per cent and 16 per cent. The results of grain size distribution test completed on two samples of the till deposit are shown on Figure B8 in Appendix B. An Atterberg limits test was carried out on a sample of the till deposit and measured a liquid limit of about 17 per cent, a plastic limit of about 12 per cent, corresponding to a plastic index of about 5 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B9 in Appendix B.

4.2.7 Silt to Sand

A deposit of non-cohesive soils comprised of silt to sandy silt to silt and sand to sand was encountered underlying the clayey silt deposit in all boreholes. The thickness of the deposit varies between 9.8 m and 13.7 m and the deposit extends to depths ranging from 29.3 m to 33.5 m below ground surface (Elevations 145.6 m to 141.0 m).

The SPT 'N'-values measured within the silt to sand deposit range from 3 blows to 52 blows per 0.3 m of penetration, indicating a very loose to very dense relative density.

The natural water content measured on selected samples of the cohesionless deposit ranges from about 19 per cent to 26 per cent. The results of grain size distribution tests completed on six samples of this deposit are shown on Figures B10 to B13 in Appendix B.

4.2.8 Sand and Gravel to Sandy Gravel

A deposit of sand and gravel to sandy gravel was encountered underlying the silt and sand deposit in Boreholes 13-03, 13-04 and 13-06 and below the clayey silt pocket in Borehole 13-04 and extends to borehole termination to depths of 35.4 m to 38.1 m (Elevations 139.6 m and 136.4 m). The deposit generally contains trace to some silt and trace to some clay. The split-spoon sampler bouncing was noted during drilling operation within this deposit which may be an indication of the presence of cobbles and boulders within this deposit.

The SPT 'N'-values measured within the cohesionless deposit ranges from 8 blows to 90 blows per 0.3 m of penetration, but generally greater than 23 blows per 0.3 m of penetration, indicating a generally compact to very dense relative density.

The natural water content measured on six samples of this deposit ranges from about 4 per cent to 9 per cent. The results of grain size distribution test completed on five samples of this deposit are shown on Figures B14 and B15 in Appendix B.

4.2.9 Clayey Silt to Clayey Silt with Sand Pockets

A pocket of clayey silt about 0.9 m thick and maybe up to about 1.9 m thick was encountered at Elevation 141.0, underlying the sand deposit in Borehole 13-04.

An approximately 0.3 m thick pocket of clayey silt with sand was encountered at Elevation 140.8 m, within the sand and gravel to sandy gravel deposit in Borehole 13-05.

A SPT 'N'-value of 41 blows per 0.3 m of penetration was measured within the clayey silt pocket in Borehole 13-04, suggesting a hard consistency and a SPT 'N'-value of 50 blows per 0.08 m of penetration was measured within the clayey silt with sand pocket in Borehole 13-05, suggesting a hard consistency.

The natural water content measured on a sample of the clayey silt pocket is about 19 per cent and on a sample of clayey silt with sand pocket is about 18 per cent. The result of a grain size distribution test completed on a sample of the clayey silt pocket from Borehole 13-04 is shown on Figure B12 and on a sample of the clayey silt with sand pocket from Borehole 13-05 is shown on Figure B16 in Appendix B. Atterberg limits testing was carried out on a sample of the clayey silt pocket and measured a liquid limit of about 27 per cent, a plastic limit of about 14 per cent and a corresponding plastic index of about 13 per cent. The result of the Atterberg limits test is shown on the plasticity chart on Figure B17 in Appendix B.

4.2.10 Refusal on Inferred Bedrock

The bedrock surface is inferred from refusal to further penetration of hollow stem augers at depths of about 35.4 m and 38.1 m below ground surface, corresponding to Elevations 139.5 m and 136.4 m, at two borehole locations. The 1968 investigation, from which the borehole records are presented in Appendix C. for the adjacent ramp site reported that the bedrock surface is between Elevations 136.2 m and 136.6 m, which is consistent with the results of the current investigation.

4.3 Groundwater Conditions

The soil samples obtained in the boreholes were generally moist to wet. During the drilling operation, sand heave inside the hollow stem augers to a depth of 12.2 m below ground surface (Elev. 162.7 m) while advancing the augers to a depth of about 18.9 m (Elev. 156.0 m) in Borehole 13-06. The observed water levels in the open boreholes during and upon completion of drilling are shown on the Record of Borehole sheets and are summarized below.

Borehole*	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)	Date
13-05	174.3	7.3	167.0	July 20, 2013
13-06	174.9	9.1	165.8	June 18, 2013

* The depth to the water level was not recorded in Boreholes 13-03 and 13-04.

The water levels presented above and on the Record of Borehole sheets may not represent stabilized groundwater conditions at the time of the investigation.

The groundwater level is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the Spring and periods of precipitation.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Al Varshoi, M.E.Sc., and reviewed by Mr. Mehdi Mostakhdemi, P.Eng., a geotechnical engineer with Golder. Mr. Ty Garde, P.Eng., and subsequently Mr. Jorge M. Costa, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, conducted an independent review of this report.

Report Signature Page

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
Lyons CREEK BRIDGES SITE No. 36-66/1 AND 36-66/2
QEW STRUCTURE REPLACEMENTS AT BLACK CREEK, LYONS CREEK, SEVENTH STREET AND TEE CREEK, REGIONAL MUNICIPALITY OF NIAGARA
G.W.P 2177-08-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing QEW bridges over Lyons Creek. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

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

This preliminary Foundation Design Report is for planning purposes only and the Design/Build proponent shall satisfy himself as to the sufficiency of the available information and supplement the information as needed to meet the requirements for detail design. The Design/Build proponent is solely responsible for selecting the appropriate foundation alternatives for replacement/rehabilitation of the Lyons Creek bridges.

6.2 Foundation Options

Based on the planning study completed to date for the replacement of the Lyons Creek bridges, it is understood that the future works will include replacement of the existing three span bridges with single span structures, with the new abutments to be placed behind the existing piers. It is further understood that a re-alignment and/or grade change of the QEW at the location of the bridges are not under consideration at this time.

Based on the subsurface conditions at this site, both shallow and deep foundation options have been considered for support of the abutments for the new Lyons Creek bridges. The as-built information of the existing pier foundations are unknown at this time. The original design drawings indicate that the existing pier foundations consist of mass concrete (surrounded by sheetpiles) supported on soils where the sheetpiles were driven to practical refusal. Based on the results of the current investigation, the subsoils within the upper 18 m to 20 m depth below road level are considered unsuitable to support shallow or mass concrete raft footings. Therefore, the validity of the original design drawings compared to the actual subsoil conditions should be verified during the detail design.

It is also possible that the sheetpiles were driven outside of their design locations. The location of the new abutment foundations should be selected to avoid interference from the existing foundation elements. For this reason, further investigation is recommended during the detail design to confirm the as-built configuration and location of the existing foundations system.

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

- **Spread footings:** Due to the presence of very soft zones within the clayey silt deposits within the upper 18 m to 20 m of the overburden, the preliminary geotechnical resistances are not sufficient to support the replacement structures on strip or spread footings constructed at shallow depths. Therefore, spread footings are not considered as a feasible option and not discussed further in this report.

- **Steel H-piles driven to found on the bedrock:** Driven steel H-piles are suitable and feasible for support of new abutments (and would permit integral abutment design) and associated wingwalls/retaining walls at this site. It is assumed that the new pile caps would be "perched" within the approach embankments above the floodplain grade, thus minimizing the depth of excavation and associated requirements for temporary protection systems and dewatering. There is a relatively minor risk associated with penetrating through or the piles "hanging up" on cobbles or boulders (although further investigation is recommended in this regard at the detail design stage).

- **Steel pipe piles driven to found on the bedrock:** Driven steel pipe piles could also be considered as a deep foundation option for support of new abutments (would permit semi-integral abutment design but are not normally accepted by MTO for integral abutment design but are not normally accepted by MTO for integral abutment design) and associated wingwalls/retaining walls at this site. It is assumed that the abutment pile caps would be "perched" within the QEW approach embankments, minimizing the depth of excavation and associated requirements for temporary protection and dewatering. Pipe piles are considered to have a slightly higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the sand and gravel deposit above the bedrock at this site.

- **Caissons founded in the bedrock:** Caissons founded in the bedrock are feasible for support of the new abutments (although they would preclude integral abutment design) at this site. Temporary or permanent liners would be required during caisson construction given the risk of running/flowing soil when excavating through the water-bearing sand and gravel deposits. In addition, coring and/or churn drilling techniques are expected to be required to penetrate into the bedrock to the target founding levels.

The following sections provide recommendations for driven steel H-pile or pipe pile foundations, and caisson foundations to support the proposed bridge replacement. Based on the subsurface conditions at the site and the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the new structure on steel H-piles driven to found on the bedrock, in an integral abutment configuration. Deep foundations whether H-piles or caissons, should be constructed in accordance with OPSS.PROV 903 (Deep Foundations).

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

6.3.1 Founding Elevations

The new abutments and associated wingwalls may be supported on steel H-piles or steel pipe (tube) piles driven to found on or in the bedrock. The surface elevation of the bedrock, as encountered in the 1968 investigations and in the current boreholes, although generally consistent does vary between the boreholes. Further, the strength characteristics of the bedrock as determined by unconfined compressive strength tests of core samples obtained during the 1968 investigation varies in the boreholes, and further investigation will be required at the detail design stage to confirm the preliminary founding elevations recommended below. The following pile tip elevations may be used for preliminary design purposes, assuming termination on or just into the bedrock:

Structure	Foundation Element	Borehole Number	Estimated Design Pile Tip Elevation (m)
Fort Erie Bound	Lyons Creek Bridge SBL – Northwest Abutment	13-03	139.5
	Lyons Creek Bridge SBL – Southwest Abutment	13-04	136.0
Toronto Bound	Lyons Creek Bridge NBL – Southeast Abutment	13-05	137.0
	Lyons Creek Bridge NBL – Northeast Abutment	13-06	139.0

The pile caps should be placed at a minimum depth of 1.2 m below final grade for frost protection purposes as per OPSD 3.50.101 (Foundations Frost Penetration Depths). The elevations of the underside of the new pile caps are not known at this time.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the till layers/pockets and non-cohesive soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of "hanging up" or being deflected away from their orientation during installation, due to their larger end (tip) area. The piles should be reinforced at the tip with driving shoes to reduce the potential for damage to the piles during driving.

As discussed further in Section 6.6 (Construction Considerations), vibration monitoring is not anticipated to be required during deep foundation construction activities, either at the existing bridges or at the nearest buildings.

The long-term settlement associated with the consolidation of the soft to stiff clayey deposits will induce a downward movement of the soils adjacent to the piles due to creep of the cohesive stratum and due to the new loading associated with the placement of approach embankment immediately behind the abutments. Hence, negative skin friction will develop along portions of the pile shafts embedded within or above the soft to stiff clayey layer. For preliminary design purposes, factored downdrag loads of 600 kN for HP 310x110 piles (assuming a negative skin friction factor of 0.25) should be considered in the preliminary design of the piles. The structural capacity of the pile must be sufficient to withstand the combined permanent load plus the downdrag load (if the downdrag loads are greater than the live loads). The magnitude and duration of the settlement and the magnitude of the downdrag loads should be reassessed during detail design, following completion of additional investigation and testing.

Alternatively, the portion of the approach embankment adjacent to the abutments could be constructed to design grade and preloaded for a period of approximately nine months (with the duration to be confirmed during detail design). This latter method of settlement mitigation is preferred, as it would address concerns with both differential settlement in the immediate vicinity of the abutment and potential downdrag loads on the piles. If there is no preload, the embankment may have to be constructed using lightweight fill to eliminate the differential settlement.

6.3.2 Geotechnical Axial Resistance/Reaction

For preliminary design for HP 310x110 piles driven to the estimated tip elevations provided in Section 6.3.1, the factored geotechnical axial resistance at ULS may be taken as 1,700 kN, and the geotechnical axial reaction at SLS (for approximately 10 mm of settlement) may be taken as 1,500 kN. Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the new foundation elements.

6.4 Caissons

As an alternative to steel H-piles or pipe piles, caissons could be considered for support of the new abutments. Temporary or permanent liners will be required during caisson construction because of the water-bearing non-cohesive soils that are present at this site. For the installation of caissons, consideration must be given to the potential presence of cobbles and boulders within the till and non-cohesive soil deposits.

6.4.1 Founding Elevations

As the surface of the bedrock varies, based on the refusal condition encountered in the boreholes of the current investigation and the borehole results of the DOH 1968 investigation, and to accommodate some weathering in the upper portion of the bedrock, socketting into the bedrock is recommended. The recommended caisson founding levels for preliminary design are founded below:

Structure	Foundation Element	Borehole Number	Design Caisson Founding Elevation (m)
Fort Erie Bound	Lyons Creek Bridge SBL – Northwest Abutment	13-03	138.5
	Lyons Creek Bridge SBL – Southwest Abutment	13-04	135.0
Toronto Bound	Lyons Creek Bridge NBL – Southeast Abutment	13-05	136.0
	Lyons Creek Bridge NBL – Northeast Abutment	13-06	138.0

It is expected that the sockets would have to be advanced into the rock by coring and/or churn drilling.

6.4.2 Axial Geotechnical Resistance/Reaction

For preliminary design, caissons socketted at least 1 m into the bedrock may be designed based on end-bearing resistance, using a factored geotechnical axial resistance at ULS of 5 MPa; for a 1 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of 4,000 kN. The geotechnical reaction at SLS (for less than 15 mm of settlement) may be taken as 3,000 kN.

6.5 Approach Embankments

6.5.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic material or existing surficial fill materials be stripped from the footprint of the sections of the new approach embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the widened approach embankment areas.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod, in accordance with OPSS 802, OPSS.PROV 804 and OPSS 803, respectively, is recommended as soon as practicable after construction of the embankments.

6.5.2 Approach Embankment Stability

Preliminary slope stability analyses have been performed for the proposed new section of approach embankments adjacent to the abutments using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed approach embankment of the bridge replacement on this project, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses were completed for a maximum 5 m high approach embankment, based on the subsurface conditions as encountered in Boreholes 13-03 to 13-06. The following parameters have been used in the preliminary analyses, based on field and laboratory test data as well as accepted correlations:

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	34°	-
Very soft to stiff clayey organic silt	18	26°	25*
Firm to stiff clayey silt	20	28°	30
Stiff to hard cohesive till	21	32°	-
Very Loose to very dense silt to sand	19	30°	-
Compact to very dense sand and gravel to sandy gravel	21	32°	-

* Lower range of underained shear strength suggested by SPT “N” values.

The preliminary stability analysis results indicate that a 5 m high embankment with side slopes no steeper than 2H:1V will have a factor of safety of at least 1.3 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example of the results from the static global stability analyses is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the additional borehole information obtained within the proposed footprint for the widened QEW approach embankments during detail design.

The approach embankments were analyzed for cross sections perpendicular to the QEW alignment. Once the design configuration of the new bridges is known, global stability of the front slopes of the bridges should be assessed.

6.5.3 Approach Embankment Settlement

The new Lyons Creek bridges are proposed to be constructed at the location of the existing structures. Preliminary settlement analyses for the anticipated soil conditions below the new/widened sections of the approach embankments adjacent to the new abutments were carried out using the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli as given in the table below, based on correlations with the SPT “N” values, undrained shear strengths and Atterberg limits testing (Bowels, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974) and engineering judgement from experience with similar soils in this region of Ontario.

Soil Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)	Preconsolidation Pressure (kPa)	C_c	C_r
Embankment fill	21	-	-	-	-
Very soft to stiff clayey organic silt	18	5	150	0.36	0.07
Firm to stiff clayey silt	20	10	150*	0.24*	0.05*
Stiff to hard cohesive till	21	50	-	-	-
Very Loose to very dense silt to sand	19	20	-	-	-
Compact to very dense sand and gravel to sandy gravel	21	75	-	-	-

* Based on the results of two consolidation tests from previous investigation

Based on this preliminary assessment, the settlement of the foundation soils under new 5 m high section of the approach embankments adjacent to the abutments is estimated to be up to about 350 mm. Approximately 150 mm of this settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments. However, approximately 200 mm of this settlement is associated with longer-term consolidation of the soft to firm portion of the clayey deposits under the new/widened approach embankment loading; it is anticipated that the majority of this settlement would be completed within approximately nine months. This estimated magnitude and duration of settlement should be reassessed following additional investigation (including consolidation testing) during detail design.

The above preliminary settlement estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 per cent to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material’s standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

6.6 Construction Considerations

The following subsections identify future construction considerations that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during the detail design stage of the project for incorporation into the Contract Documents.

6.6.1 Excavation and Temporary Protection Systems

The foundation excavations for pile caps would extend through the existing fill and into the very soft to stiff clayey deposit. If space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and soft/stiff soils are classified as Type 4 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) through these materials should be made with side slopes no steeper than 3H:1V, assuming that appropriate groundwater control is in place.

The selection and design of the protection system will be the responsibility of the Contractor. However, for conceptual/planning purposes, the temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 *(Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539. It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the temporary excavation support at the abutments.

6.6.2 Groundwater Control

While new abutment pile caps would be maintained above the groundwater level at the site, excavations for new pile caps would extend below the groundwater level.

Due to the proximity of the abutments to the edge of the Lyons Creek, a groundwater cut-off system (cofferdam or similar measure) is recommended to minimize dewatering requirements and potential environmental impacts. The selection and design of the groundwater control system is the responsibility of the contractor.

6.6.3 Bedrock Excavation and/or Socket Formation

If caissons are the selected foundation option and rock sockets are required to provide the necessary foundation capacity, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor of the bedrock quality and strength. Further, it is expected that socket formation would require coring or churn drilling to advance the hole.

It is recommended that an NSSP be developed at the detail design stage and included in the Contract Documents to warn the contractor that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation must not disturb the existing bridge foundations.

6.6.4 Obstructions

The soils at this site are glacially or glacio-fluvially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations. Further observation is recommended in the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.6.5 Vibration Monitoring During Construction

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities such as pile driving or coring/churn drilling, however, given that one of the existing bridges is likely to continue in operation during construction of the adjacent bridge and given the closeness of the newly constructed bridge to the adjacent bridge under construction it is recommended that vibration monitoring be required during construction of the adjacent structure.

Existing residential buildings are located to the northeast and southwest of the structure site, approximately 200 m from the Lyons Creek bridges. Although a lower PPV threshold of 50 mm/s is generally considered applicable for vibration impacts on buildings, the construction zone of influence would likely be less than 100 m. Therefore, vibration monitoring is not expected to be required at the existing buildings adjacent to the bridge site.

6.7 Recommendations for Further Work During Detail Design

Additional boreholes will be required at each of the foundation elements and within the approach embankment areas during the detail design stage of the project, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- Abutments:
  - Assessment of the presence of any cohesionless soil lenses or interlayers within the cohesive deposits at the site, which could impact groundwater control requirements for foundation excavations.
  - Observation of the presence and frequency of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile foundations.
  - Assessment of vibration thresholds for the nearby residential buildings, and if warranted development of an NSSP for a vibration monitoring plan.
  - Further assessment of the depth and strength of the bedrock at the location of the new abutments.
  - Further assessment of the groundwater conditions at the location of each foundation element where excavation would be required.
  - Further assess the as built configuration of the foundations of the existing bridges, determine the as-built location and configuration of the sheetpile foundations (Cored boreholes through the mass concrete foundations within the sheetpiled area; file search at MTO structural office for available information on design and construction details of the foundation elements; down-hole magnetometer survey adjacent to existing foundations).
- Approach embankments:
  - Assessment of the depth and extent of stripping of topsoil/organics, fill materials and loosened or softened native soils within the footprint of the new approach embankments.
  - Further assessment of the thickness and consolidation/elastic compression properties of the soils within the footprint of the new sections of the approach embankments, to confirm the settlement estimates.

- Further assessment of the engineering parameters and global slope stability of the new sections of the approach embankments.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Mehdi Mostakhdemi, M.Sc., P.Eng. Mr. Ty Garde, P.Eng., carried out a technical review of the report and Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent quality review of this report.

Report Signature Page

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REFERENCES

Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.

Kulhawy, F.H. and Mayne, P.W., 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.

Peck, R.B., Hanson, W.E., and Thornburn, T.H., 1974. *Foundation Engineering*, Second Edition, John Wiley and Sons, New York.

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding

Ontario Provincial Standard Drawings (OPSD)

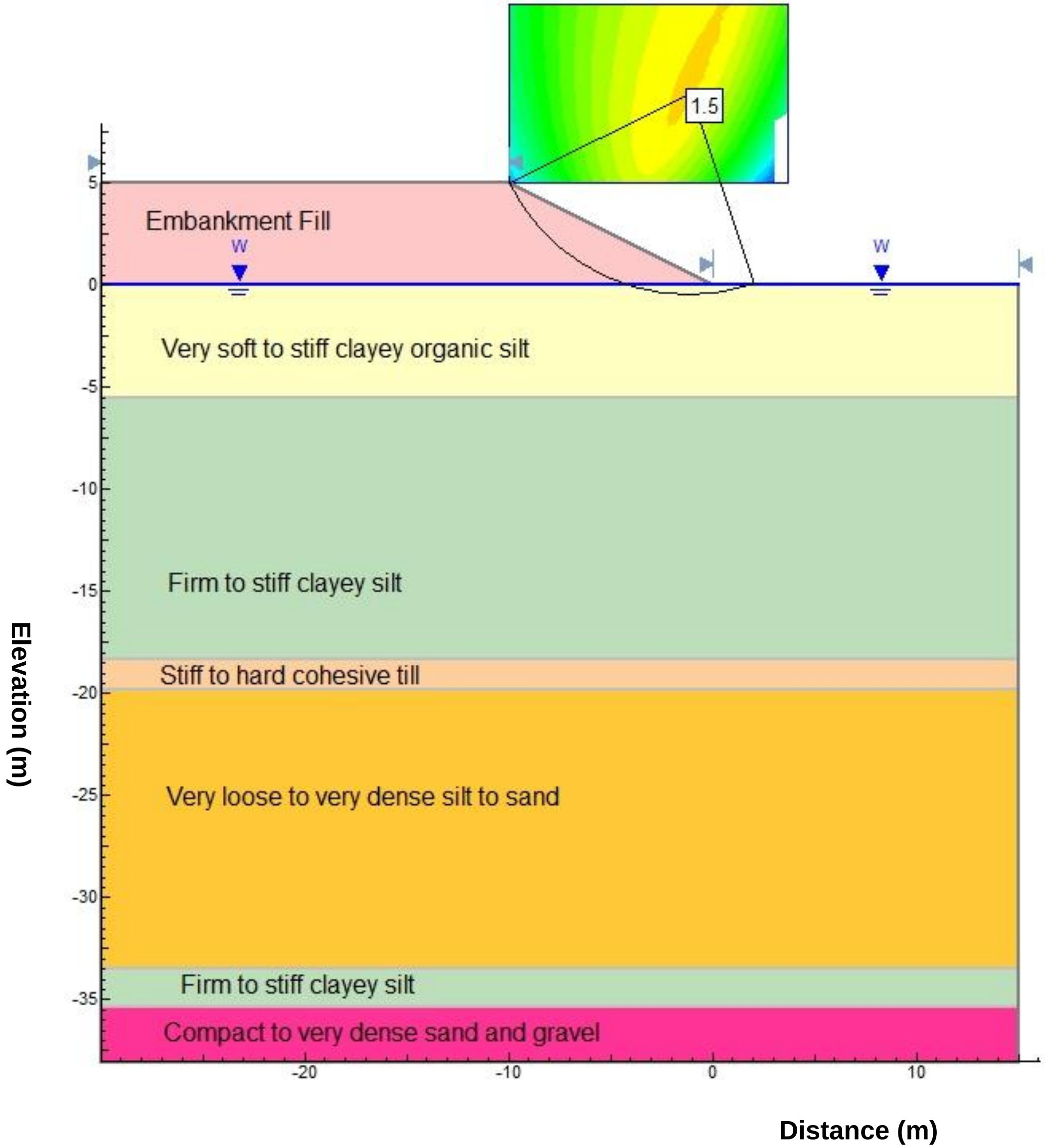
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario

TABLE 1 – COMPARISON OF FOUNDATION OPTIONS
LYONS CREEK BRIDGES

Foundation Option	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings	• Not feasible due to low geotechnical resistances associated with the very soft zones of the clayey silt deposit	• N/A	• N/A	• N/A
Steel H-piles driven to found on bedrock	• Pile caps can be constructed above the ground surface (i.e., within the approach embankment fill on a granular pad), reducing depth of excavation and temporary protection system requirements adjacent to QEW • Allows for integral abutment construction	• Requires excavation through fill materials and likely to below the creek water level for adequate cover from frost penetration, groundwater control maybe required	• Conventional construction methods • Risk of encountering obstructions (cobbles, boulders and/or existing sheetpile foundations) during pile driving; this could result in piles “hanging up” and lower geotechnical resistances	• Lower relative cost compared with caisson option • Estimated cost is approximately $250/m length for pile installation and $600/m^3 for pile cap construction, plus cost of any temporary protection systems
Steel pipe (tube) piles, driven to found on bedrock	• Pile caps can be constructed above the ground surface (i.e., within the approach embankment fill on a granular pad), reducing depth of excavation and temporary protection system requirements adjacent to QEW	• Not normally accepted to MTO for integral abutment design • More difficult to install given the required displacement of soil	• Conventional construction methods • Greater risk than for steel H-pile foundations of encountering obstructions (cobbles, boulders and/or existing sheetpile foundations) during driving; this could result in piles “hanging up” and lower geotechnical resistances	• Costs for steel pipe (tube) piles similar to but slightly higher than those for H-piles
Caissons founded in bedrock	• Abutment pile caps could be constructed at the level of the underside of the bridge, reducing depth of excavation and temporary excavation support requirements adjacent to QEW embankment • Higher capacity than piles will require fewer foundation elements	• Temporary or permanent liners would be required due to risk of running/flowing soils in water-bearing sand and gravel deposits • Coring and/or churn drilling techniques required to penetrate into the bedrock • Precludes use of integral abutments	• Conventional construction methods with temporary liners required • Greater risk than steel piles of encountering obstructions (cobbles, boulders and/or existing sheetpile foundations) during installation; this could result in caissons not achieving desired elevations and/or lower geotechnical resistances	• Higher cost compared with shallow foundations or steel H-piles

Static Global Stability – Lyons Creek Bridges
Effective Stress Analysis

Figure 1



Safety Factor: 0.0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5, 6.0+

Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Very soft to stiff clayey organic silt		18	0	26
Firm to stiff clayey silt		20	0	28
Stiff to hard cohesive till		21	0	32
Very loose to very dense silt to sand		19	0	30
Embankment Fill		21	0	34
Compact to very dense sand and gravel		21	0	32

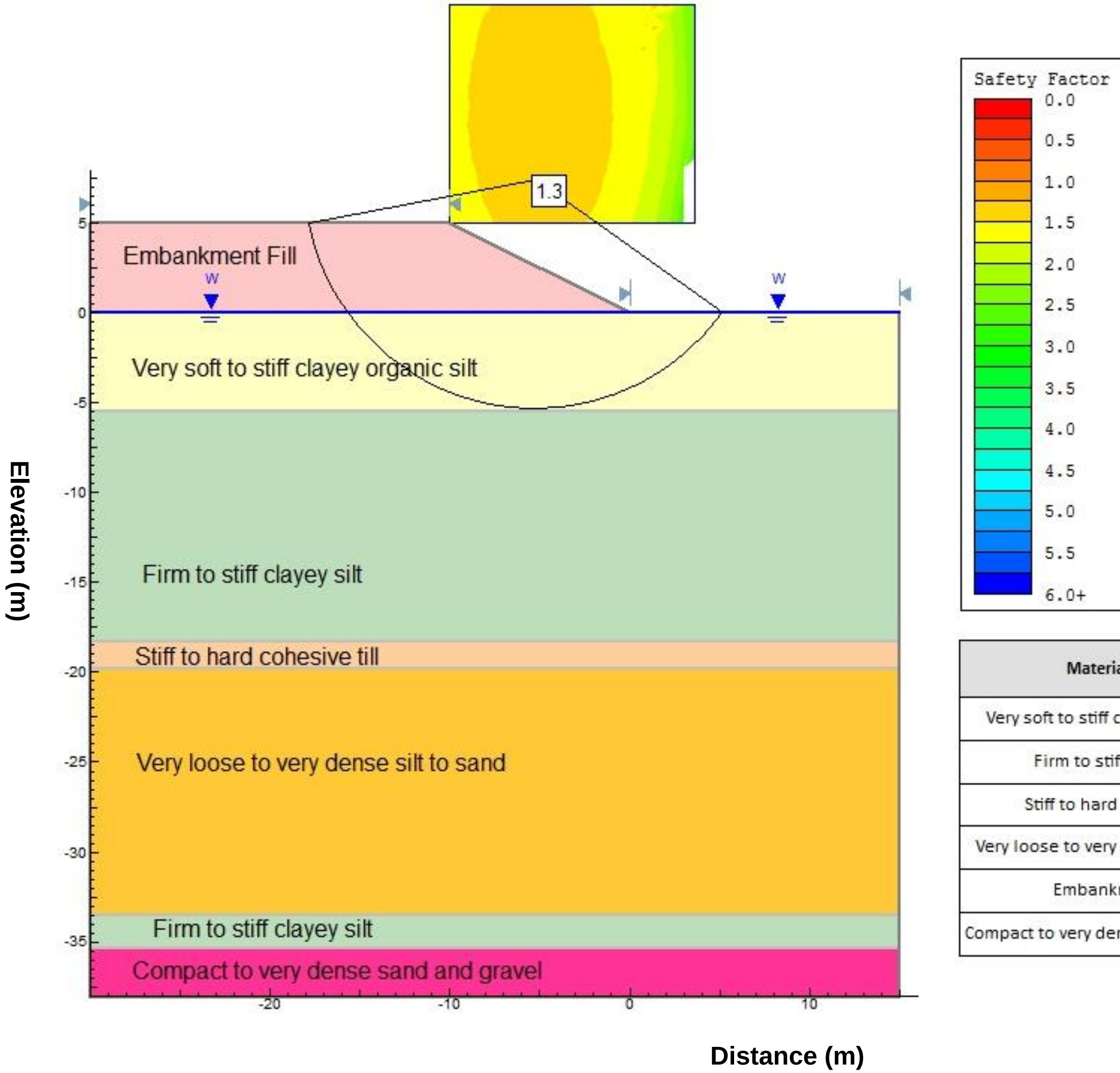
Date: October 2013
Project No: 12-1111-0088-3

Analysis By: MM **Reviewed By:** TJG

Golder Associates

Static Global Stability – Lyons Creek Bridge
Total Stress Analysis

Figure 2



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kN/m2)	Phi
Very soft to stiff clayey organic silt		18	25	
Firm to stiff clayey silt		20	30	
Stiff to hard cohesive till		21	0	32
Very loose to very dense silt to sand		19	0	30
Embankment Fill		21	0	34
Compact to very dense sand and gravel		21	0	32

Date: October 2013
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MINISTRY OF TRANSPORTATION, ONTARIO

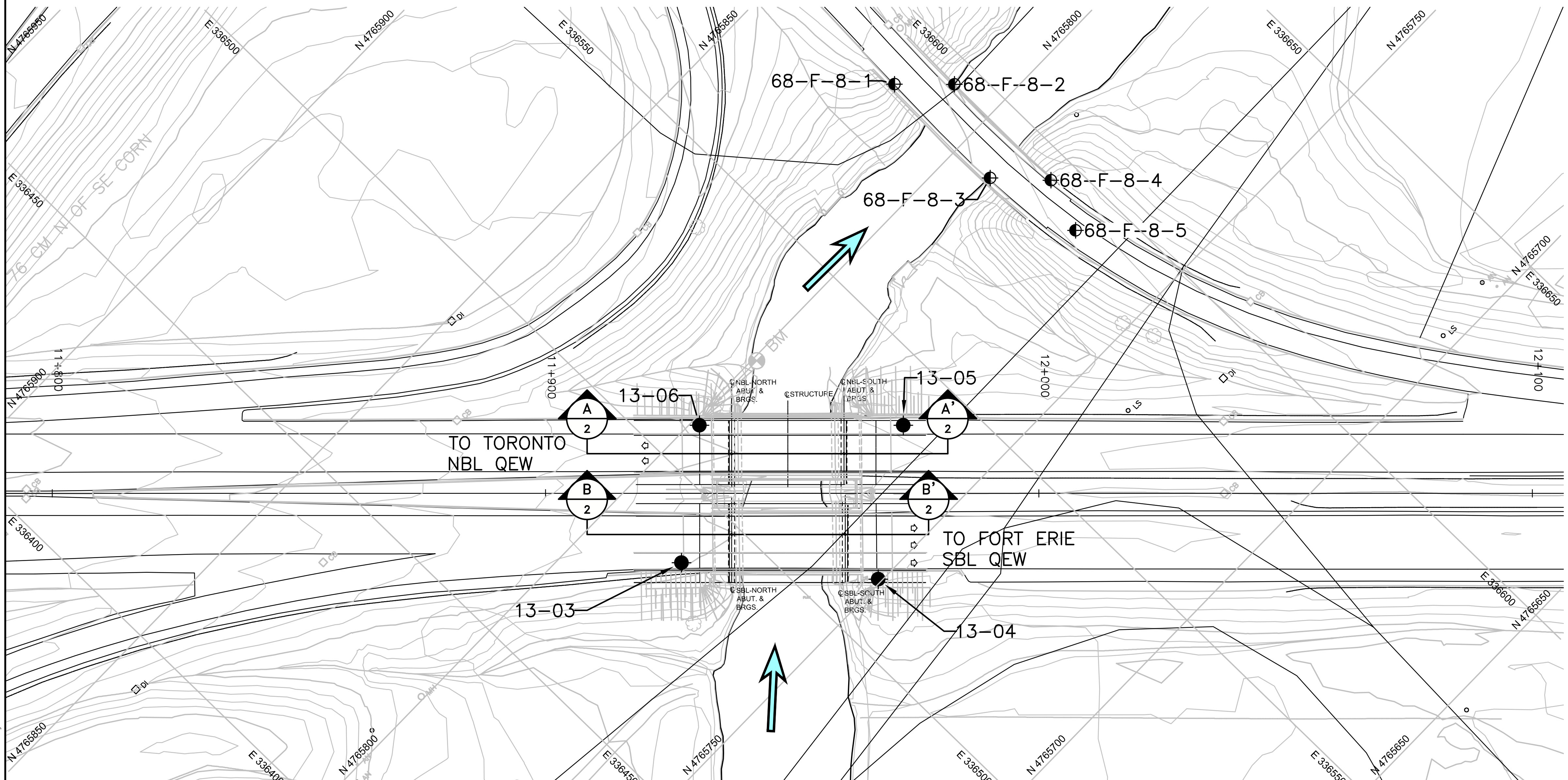
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2177-08-00

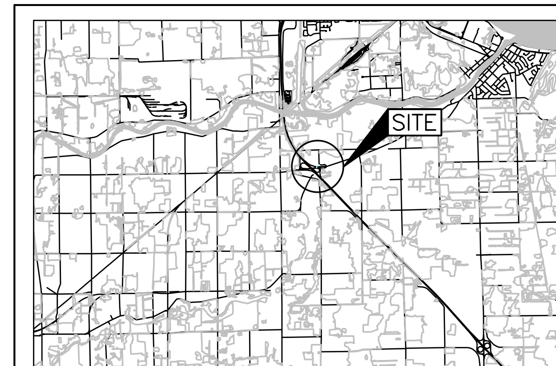
SHEET

QEW STRUCTURE REPLACEMENT
LYONS CREEK BRIDGES
BOREHOLE LOCATIONS

Golder Associates



PLAN
SCALE
10 0 10 20 m



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

Borehole – Current Investigation

Borehole – Geocres No. 68-F-8
Location is approximate

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-03	175.0	4765781.6	336487.0
13-04	174.5	4765750.7	336512.4
13-05	174.3	4765768.8	336538.3
13-06	174.9	4765798.4	336509.5

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 design configuration as shown elsewhere in the Preliminary Design Report.

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

The complete Preliminary Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base-All.dwg and X-Contours.dwg, received July 30, 2013 on GA and Profile file No. Draft_Lyons Creek_GA.dwg, received October 17, 2013.

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NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 30M3-279

HWY. QEW	PROJECT NO. 12-1111-0088		DIST.
SUBM'D. MM	CHKD. TJG	DATE: 12/24/2014	SITE: 36-66
DRAWN: JFC	CHKD. MM	APPD. JMAC	DWG. 1

PLOT DATE: January 9, 2015
FILENAME: T:\Projects\2012\12-1111-0088 (URS, Various Structure Replacement, QEW)\-BA- Lyons Creek\1211110088BA001.dwg

MINISTRY OF TRANSPORTATION, ONTARIO

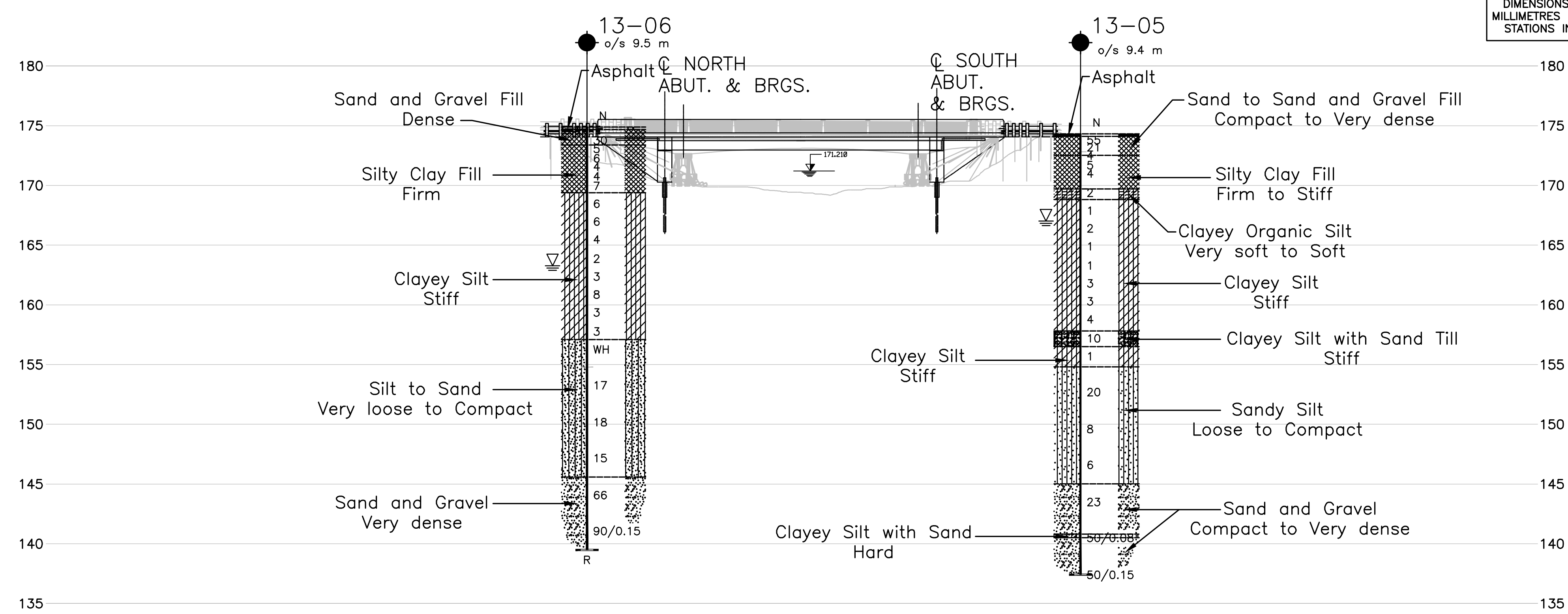
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2177-08-00

QEW STRUCTURE REPLACEMENT
LYONS CREEK BRIDGES
SOIL STRATA

SHEET

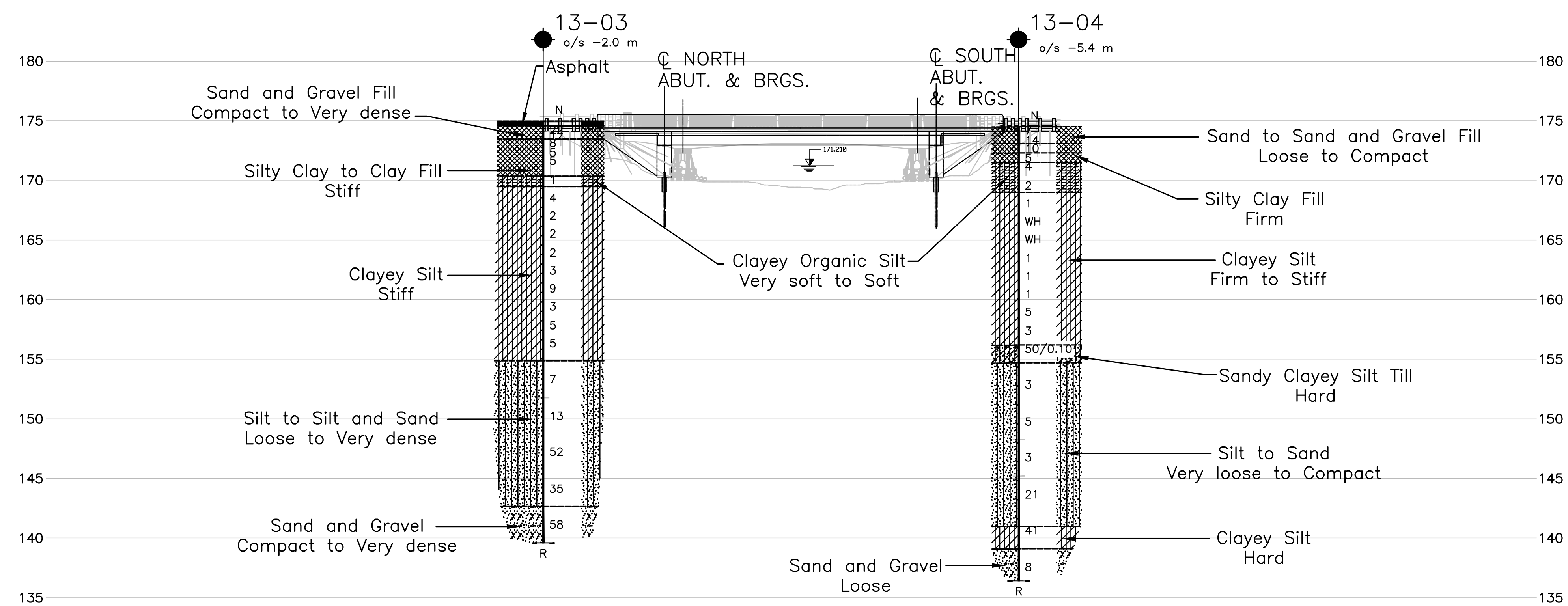
Golder Associates



PROFILE – NBL (TORONTO BOUND LANES)

A-A' / 1

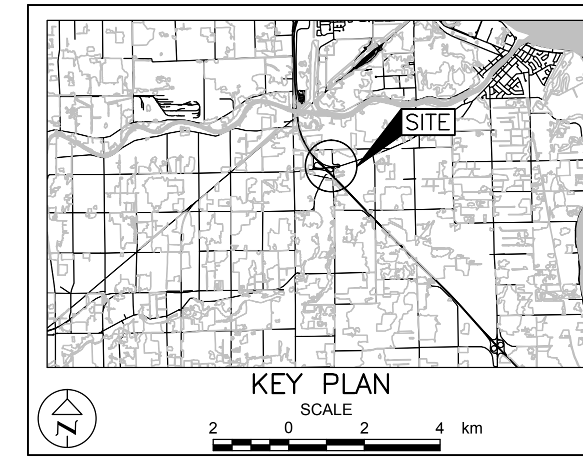
SCALE 5 0 5 10 m



PROFILE – SBL (FORT ERIE BOUND LANES)

B-B' / 1

SCALE 5 0 5 10 m



KEY PLAN

SCALE 2 0 2 4 km

LEGEND

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

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-03	175.0	4765781.6	336487.0
13-04	174.5	4765750.7	336512.4
13-05	174.3	4765768.8	336538.3
13-06	174.9	4765798.4	336509.5

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LICENSED PROFESSIONAL ENGINEER J. M. A. COSTA Jan 9, 2015 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	

Geocres No. 30M3-279

HWY. QEW		PROJECT NO. 12-1111-0088	DIST.
SUBM'D.	CHKD. MM	DATE: 1/06/2015	SITE: 36-66
DRAWN: JFC	CHKD. MM	APPD. JMAC	DWG. 2

PLOT DATE: January 9, 2015

APPENDIX A

Record of Borehole Sheets

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 $= \sigma'_p / \sigma'_{vo}$

(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

Density Index **Relative Density**	**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

Golder Associates

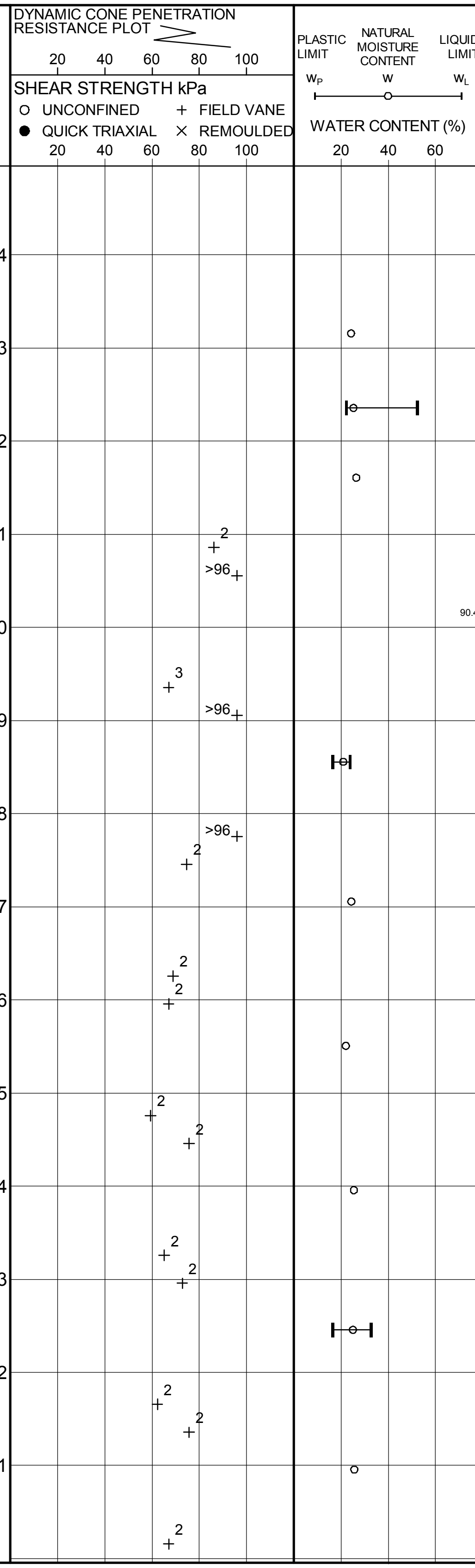
RECORD OF BOREHOLE No 13-03

SHEET 1 OF 3

METRIC

PROJECT 12-1111-0088			
W.P. 2177-08-00	LOCATION N 4765781.6 ;E 336487.0		ORIGINATED BY SB
DIST Central HWY QEW	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV
DATUM Geodetic	DATE July 7 to 9, 2013		CHECKED BY MM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
175.0	GROUND SURFACE					
0.0	ASPHALT (380 mm)					
174.6 / 0.4	Sand and gravel, trace fines (FILL) Compact Brown Moist	1	SS	22		
174.1 / 0.9	Sand (FILL) Compact Reddish brown Moist	2	SS	12	174	
173.5 / 1.5	Silty clay to clay, trace sand (FILL) Stiff Mottled brown and grey Wet	3	SS	8	173	
		4	SS	5		0 2 34 64
		5	SS	5	172	
					171	
170.4 / 4.6	Clayey ORGANIC SILT Very soft to soft Dark grey to black Wet	6	SS	1	170	
169.5 / 5.5	CLAYEY SILT, trace to some sand Stiff Grey Wet				169	
		7	SS	4		
					168	
		8	SS	2	167	
					166	
		9	SS	2		
					165	
		10	SS	2	164	
					163	
		11	SS	3		0 7 52 41
					162	
		12	SS	9	161	



UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No 13-03

METRIC

PROJECT 12-1111-0088			
W.P. 2177-08-00	LOCATION N 4765781.6 ;E 336487.0	ORIGINATED BY SB	
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers	COMPILED BY AV
DATUM Geodetic	DATE July 7 to 9, 2013	CHECKED BY MM	

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT / REMARKS (GR SA SI CL)
	CLAYEY SILT, trace to some sand Stiff Grey Wet	13	SS	3	
		14	SS	5	
		15	SS	5	
154.9 20.1	SILT, trace to some clay, trace sand Loose Brown Wet	16	SS	7	0 5 87 8
151.8 23.2	SILT and SAND, trace clay Compact to very dense Brown Wet	17	SS	13	
		18	SS	52	0 52 43 5

Field vane shear strength annotations: 2 (near elev. 159.9); >96 (near elev. 158.7); 3 and 2 (near elev. 155.5).

Elevation scale: 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O 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

Continued Next Page

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

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RECORD OF BOREHOLE No 13-03

METRIC

PROJECT 12-1111-0088
W.P. 2177-08-00
LOCATION N 4765781.6 ;E 336487.0
ORIGINATED BY SB
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY AV
DATUM Geodetic
DATE July 7 to 9, 2013
CHECKED BY MM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	SILT and SAND, trace clay Compact to very dense Brown Wet	19	SS	35	144	~20				
					143					
142.7 32.3	SAND and GRAVEL, trace to some silt, trace clay Very dense Brown Moist				142					
		20	SS	58	141	~13	47	37	12	4
					140					
139.6 35.4	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Depth to groundwater level was not measured upon completion of drilling.									

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; 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

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

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RECORD OF BOREHOLE No 13-04

METRIC

PROJECT 12-1111-0088			
W.P. 2177-08-00	LOCATION N 4765750.7 ;E 336512.4	ORIGINATED BY SB
DIST Central HWY QEW	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers	COMPILED BY AV
DATUM Geodetic	DATE July 9 to 11, 2013	CHECKED BY MM

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
174.5	GROUND SURFACE							
0.0	Sand and gravel, some silt, trace clay (FILL) Loose to compact Brown Moist		1	SS	7		174	
			2	SS	14			40 44 13 3
173.1 / 1.4	Sand (FILL) Compact Brown Wet		3	SS	10		173	
172.3 / 2.2	Silty clay, trace organics (FILL) Firm Grey Wet		4	SS	5		172	
171.5 / 3.0	Clayey ORGANIC SILT, trace sand, trace rootlets Soft to stiff Dark grey to black Wet		5	SS	4		171	OC = 9.2%; 0 4 41 55
			6	SS	2		170	
169.0 / 5.5	CLAYEY SILT, trace sand, trace gravel Soft to stiff Grey Wet						169	
			7	SS	1		168	
			8	SS	WH		167	
			9	SS	WH		166, 165	
			10	SS	1		164	
			11	SS	1		163, 162	
			12	SS	1		161	3 4 56 37
							160	

Plot headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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

Field vane values (sensitivity): 2, >96, 2, 2, 2, 3, 2, 1, 2, 2, 2, 2, 2, 2, 2. Water content note: 116.5

Continued Next Page

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

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RECORD OF BOREHOLE No 13-04

METRIC

PROJECT 12-1111-0088			
W.P. 2177-08-00	LOCATION N 4765750.7 ;E 336512.4		ORIGINATED BY SB
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers	COMPILED BY AV
DATUM Geodetic	DATE July 9 to 11, 2013		CHECKED BY MM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace sand, trace gravel Soft to stiff Grey Wet	13	SS	5	
	Trace silt seams below a depth of 16.8 m.	14	SS	3	
156.2 18.3	Sandy CLAYEY SILT, some gravel (TILL) Hard Grey Wet	15	SS	50/0.10	13 21 41 25
154.7 19.8	SILT and SAND, trace clay Very loose to loose Brown Wet	16	SS	3	0 45 52 3
		17	SS	5	
148.3 26.2	Silt, some clay, trace sand Very loose Brown Wet	18	SS	3	0 3 79 18
145.2 29.3	SAND, some gravel Compact Grey Wet				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (159 to 145); 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.

Continued Next Page

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

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RECORD OF BOREHOLE No 13-04

SHEET 3 OF 3 **METRIC**

PROJECT 12-1111-0088
W.P. 2177-08-00 LOCATION N 4765750.7 ;E 336512.4 ORIGINATED BY SB
DIST Central HWY QEW BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers COMPILED BY AV
DATUM Geodetic DATE July 9 to 11, 2013 CHECKED BY MM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
	SAND, some gravel Compact Grey Wet	19	SS	21							
141.0 / 33.5	CLAYEY SILT, trace to some sand Hard Grey Wet	20	SS	41				0	8	53	39
139.1 / 35.4	SAND and GRAVEL, trace to some silt, trace clay Loose Brown Wet	21	SS	8				34	57	6	3
136.4 / 38.1	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Depth to groundwater level was not measured upon completion of drilling.										

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O 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; REMARKS & GRAIN SIZE DISTRIBUTION (%).

Elevation scale: 144, 143, 142, 141, 140, 139, 138, 137

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

RECORD OF BOREHOLE No 13-05

SHEET 1 OF 3 — **METRIC**

PROJECT	12-1111-0088		
W.P.	2177-08-00	LOCATION	N 4765768.8 ;E 336538.3
DIST	Central	HWY	QEW
BOREHOLE TYPE	108 mm I.D. Continuous Flight Hollow Stem Augers		
DATUM	Geodetic	DATE	June 19 and 20, 2013
ORIGINATED BY	SB	COMPILED BY	AV
CHECKED BY	MM		

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
174.3	GROUND SURFACE				
0.0	ASPHALT (175 mm)				
0.2	Sand and gravel, trace to some fines (FILL) Compact to very dense Brown Moist	1	SS	55	
		2	SS	21	51 38 (11)
172.8	Sand, trace silt (FILL) Loose Brown Wet	3A / 3B	SS	4	
172.5 / 1.8	Silty clay, trace sand, trace organics (FILL) Firm to stiff Grey Wet	4	SS	5	
		5	SS	4	
169.7 / 4.6	Clayey ORGANIC SILT, trace sand Very soft to soft Dark brown to black Wet	6	SS	2	OC = 20.5%
168.8 / 5.5	CLAYEY SILT, trace sand Stiff Grey Wet	7	SS	1	
		8	SS	2	
		9	SS	1	
		10	SS	1	
		11	SS	3	
		12	SS	3	

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O 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

Field vane sensitivity labels: 2, >96, 3, 2, 2, 2, 2, 2, 2, 2, 2, >96, 1, 2. Water content annotation: 119.7

Continued Next Page

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

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Foundation Design

RECORD OF BOREHOLE No 13-05

METRIC

PROJECT: 12-1111-0088			
W.P.: 2177-08-00	LOCATION: N 4765768.8 ;E 336538.3		ORIGINATED BY: SB
DIST: Central	HWY: QEW	BOREHOLE TYPE: 108 mm I.D. Continuous Flight Hollow Stem Augers	COMPILED BY: AV
DATUM: Geodetic	DATE: June 19 and 20, 2013		CHECKED BY: MM

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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)

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION
	CLAYEY SILT, trace sand Stiff Grey Wet
157.8 / 16.5	CLAYEY SILT with SAND, some gravel (TILL) Stiff Grey Wet
156.5 / 17.8	CLAYEY SILT, trace sand, trace gravel Stiff Grey Wet
154.8 / 19.5	Sandy SILT, trace to some clay Loose to compact Brown Wet
145.0 / 29.3	

NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
13	SS	4				
14	SS	10	18	32	33	17
15	SS	1				
16	SS	20				
17	SS	8	0	27	65	8
18	SS	6				

Elevation scale: 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145

Field vane results (sensitivity labels): at approx. El. 158.0: 2, >96; at approx. El. 154.9: 2, >96

Continued Next Page

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

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RECORD OF BOREHOLE No 13-05

METRIC

PROJECT 12-1111-0088
W.P. 2177-08-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4765768.8 ;E 336538.3
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
DATE June 19 and 20, 2013
ORIGINATED BY SB
COMPILED BY AV
CHECKED BY MM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, trace to some fines, Compact, Brown, Wet	19	SS	23	144	o	48 44 (8)
140.8					143, 142, 141		
140.5 / 33.8	CLAYEY SILT with SAND, trace gravel, Hard, Brown, Wet	20A / 20B	SS	50/0.08		o	2 55 31 12
	Sandy GRAVEL, trace to some fines, Dense, Brown, Wet				140, 139, 138		
137.4		21	SS	50/0.15		o	59 28 (13)
36.9	END OF BOREHOLE						

NOTE:

1. Water level inside auger at a depth of 7.3 m below ground surface (Elev. 167.0 m) upon completion of drilling.

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

RECORD OF BOREHOLE No 13-06

SHEET 1 OF 3

METRIC

PROJECT 12-1111-0088			
W.P. 2177-08-00	LOCATION N 4765798.4 ;E 336509.5	ORIGINATED BY SB	
DIST Central	HWY QEW	BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers	COMPILED BY AV
DATUM Geodetic	DATE June 18 and 19, 2013	CHECKED BY MM	

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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
174.9	GROUND SURFACE
0.0	ASPHALT (200 mm)
	CONCRETE (300 mm)
174.4 / 0.5	Sand and gravel (FILL) Dense Brown Moist
173.4 / 1.5	Silty clay, trace sand, trace organics (FILL) Firm Mottled grey and brown Wet Black staining below a depth of 3.8 m.
169.4 / 5.5	CLAYEY SILT, trace to some sand, trace to some gravel Stiff Brown becoming grey below a depth of 10.7 m Wet Silt seams at a depth of 13.7 m.

NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
1	SS	30	
2	SS	5	
3	SS	6	
4	SS	4	
5	SS	4	0 2 33 65
6	SS	7	
7	SS	6	
8	SS	6	
9	SS	4	
10	SS	2	
11	SS	3	
12	SS	8	

Elevation scale: 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160

Field vane annotations: >96, >96, >96, 2, 2, 2, 2, 2, >96, 3

Continued Next Page

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

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RECORD OF BOREHOLE No 13-06

METRIC

PROJECT 12-1111-0088

W.P. 2177-08-00 | LOCATION N 4765798.4 ;E 336509.5 | ORIGINATED BY SB

DIST Central HWY QEW | BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers | COMPILED BY AV

DATUM Geodetic | DATE June 18 and 19, 2013 | CHECKED BY MM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	DCPT / SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS GR SA SI CL
	CLAYEY SILT, trace to some sand, trace to some gravel Stiff Brown becoming grey below a depth of 10.7 m Wet	13	SS	3		+ 1		9 6 45 40
					159	+ 2, + 2		
		14	SS	3	158		o	
157.1 / 17.8	SILT, some sand, trace to some clay Very loose Brown Wet				157	>96 +		
		15	SS	WH	156		o	0 13 80 7
154.8 / 20.1	SAND, some silt, trace to some clay Compact Brown Wet				155			
					154			
		16	SS	17	153		o	
					152			
					151			
		17	SS	18	150		o	0 78 14 8
					149			
					148			
		18	SS	15	147		o	
					146			
145.6 / 29.3					145			

Continued Next Page

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

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RECORD OF BOREHOLE No 13-06

SHEET 3 OF 3 **METRIC**

PROJECT 12-1111-0088
W.P. 2177-08-00
LOCATION N 4765798.4 ;E 336509.5
ORIGINATED BY SB
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY AV
DATUM Geodetic
DATE June 18 and 19, 2013
CHECKED BY MM

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and GRAVEL, some silt, trace to some clay Very dense Brown Wet	19	SS	66	144	o	
					143		
					142		
		20	SS	90/0.15		o	42 37 15 6
	Split-spoon sampler bouncing.				141		
					140		
139.5 35.4	AUGER REFUSAL END OF BOREHOLE						

NOTES:

1. Sand blown up inside auger to a depth of 12.2 m (Elev. 162.7 m) during drilling at a depth of 18.9 m (Elev. 156.0 m).

2. Water level inside auger at a depth of 9.1 m below ground surface (Elev. 165.8 m) during drilling.

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

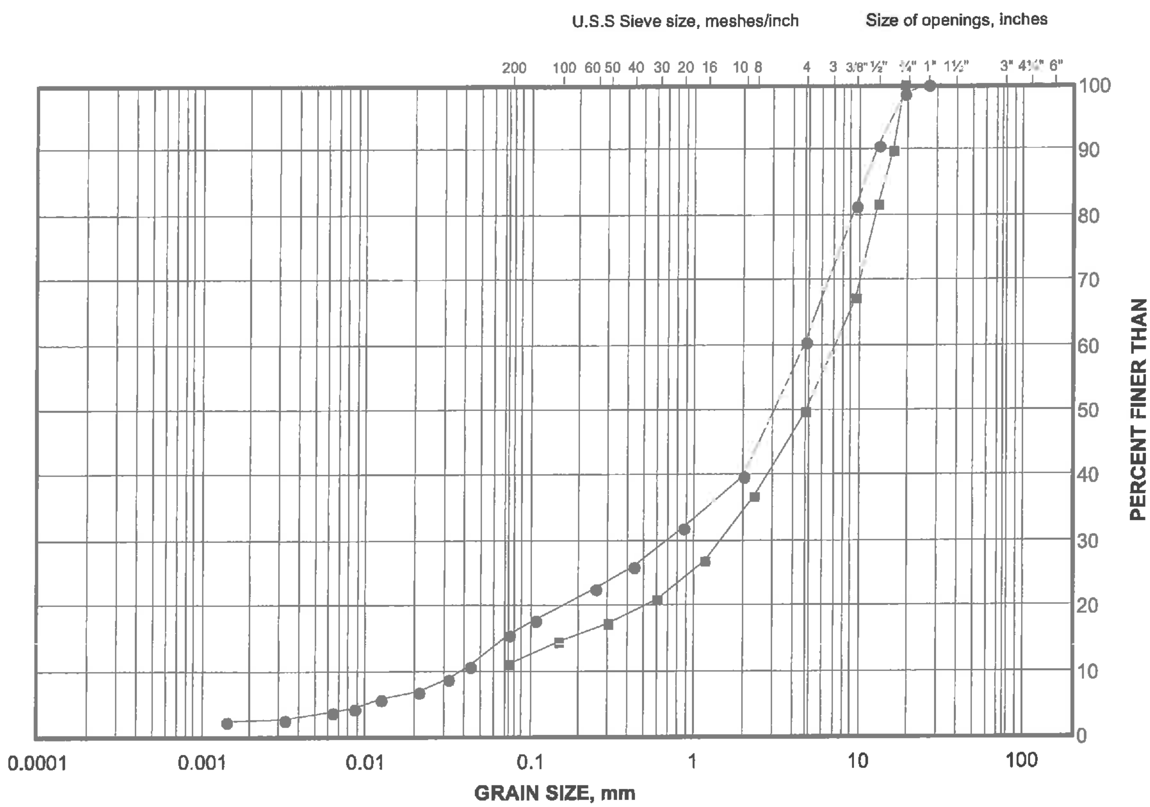
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sand and Gravel Fill

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-04	2	173.4
■	13-05	2	173.2

Project Number: 12-1111-0088

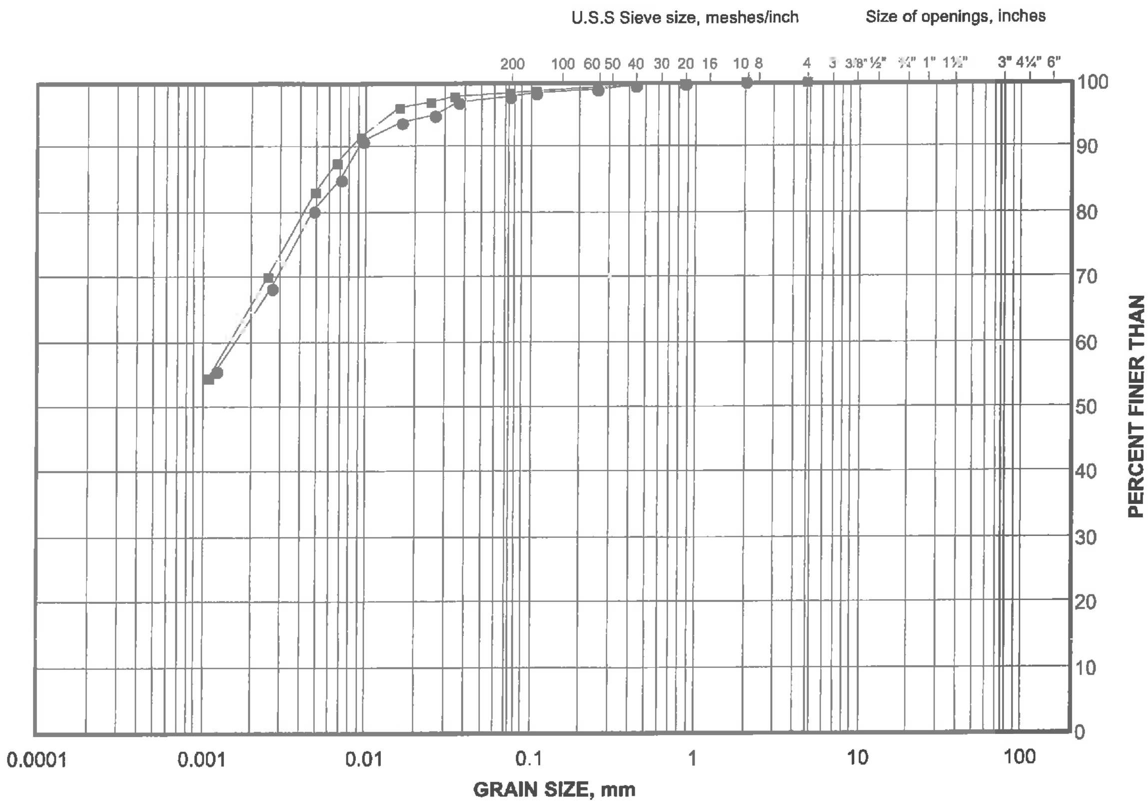
Checked By: [signature]

Golder Associates

Date: 22-Oct-13

GRAIN SIZE DISTRIBUTION
Silty Clay to Clay Fill

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-03	4	172.4
■	13-06	5	170.8

Project Number: 12-1111-0088

Checked By: ________

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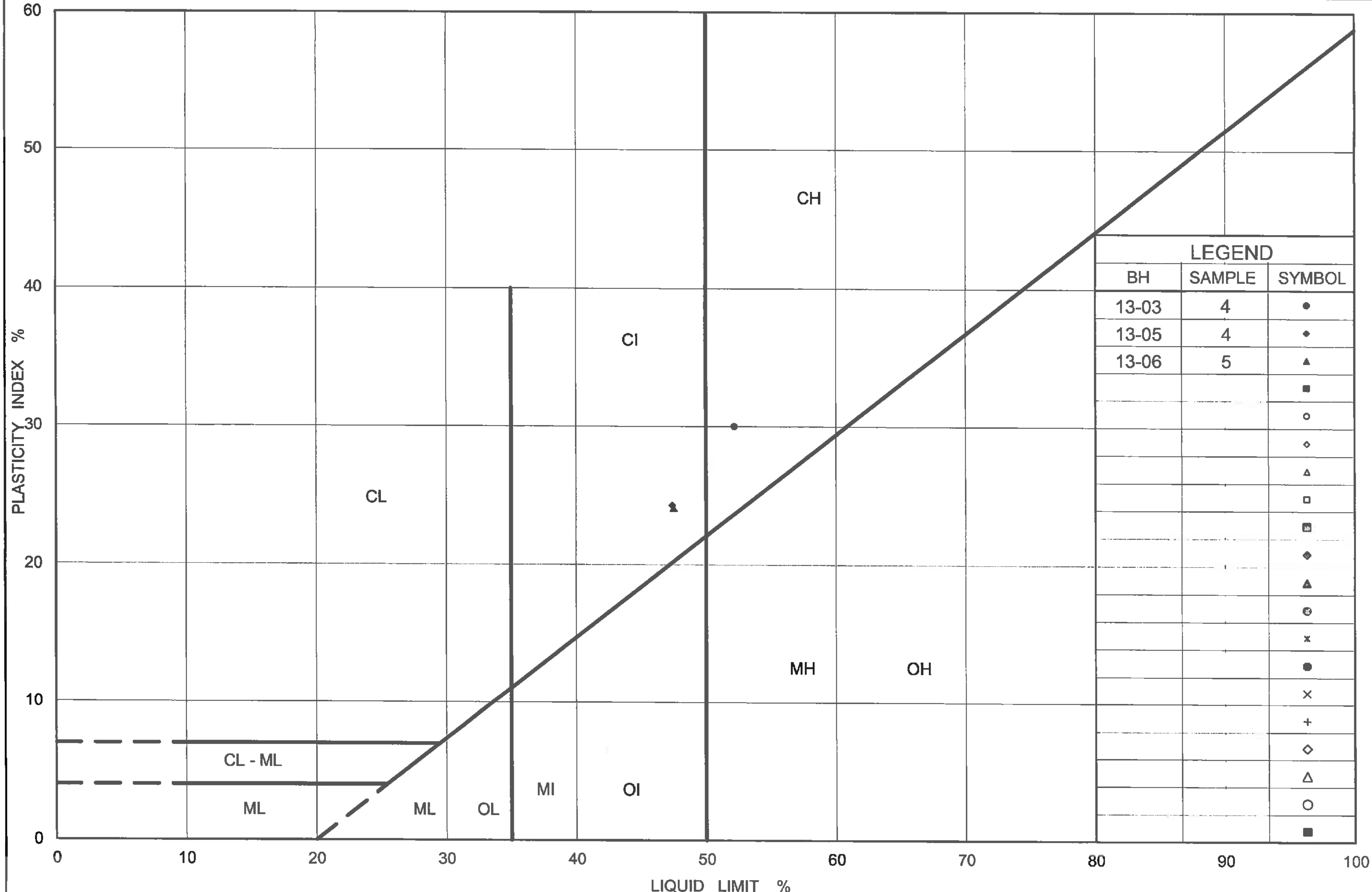
Date: 22-Oct-13



Ministry of Transportation

Ontario

PLASTICITY CHART

Silty Clay to Clay Fill

Figure No. B3

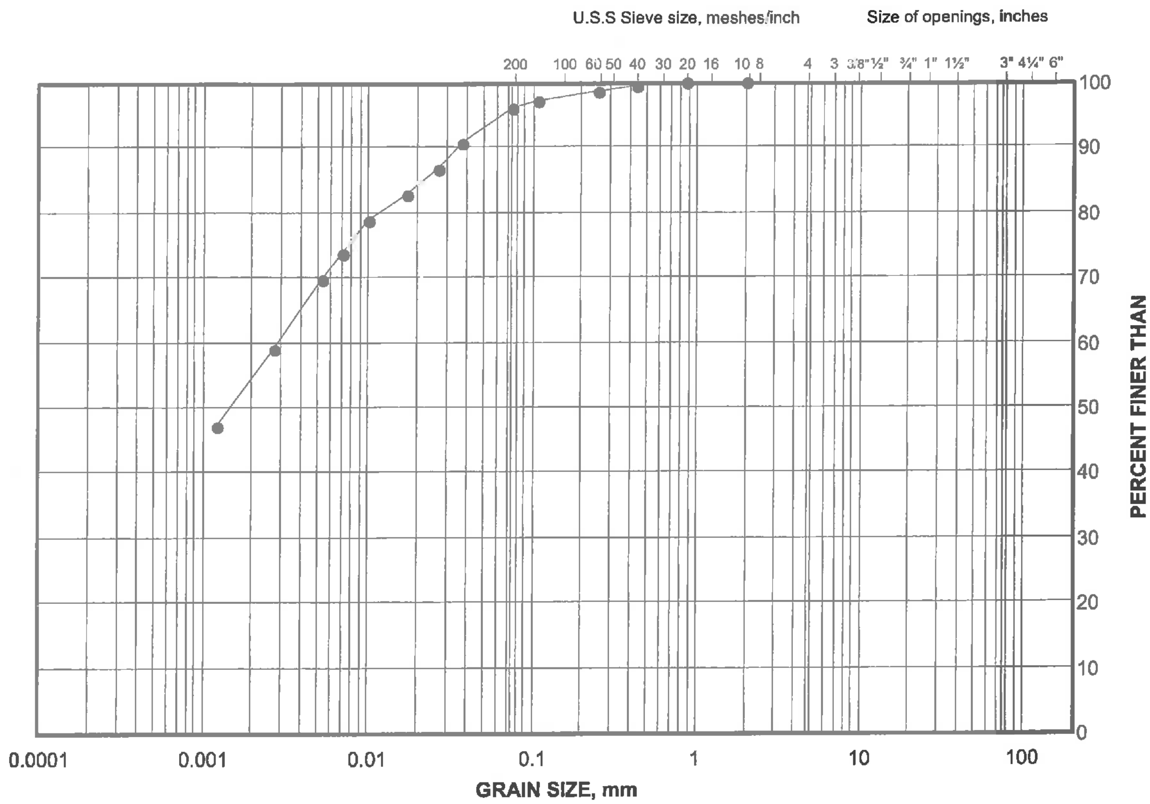
Project No. 12-1111-0088

Checked By:

GRAIN SIZE DISTRIBUTION

Clayey Organic Silt

FIGURE B4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-04	5	171.1

Project Number: 12-1111-0088

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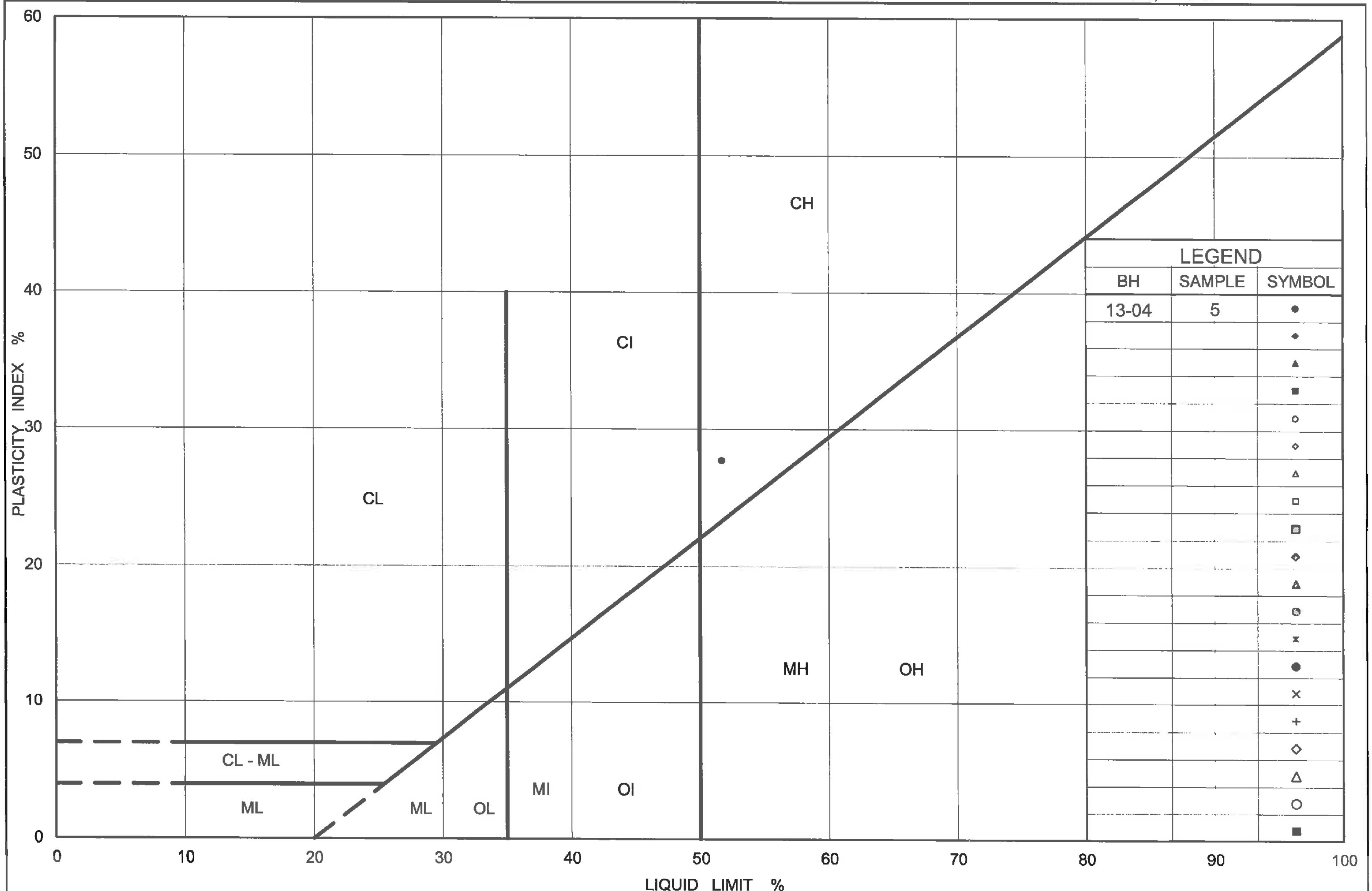
Date: 22-Oct-13

Oct 75, FF-S-21



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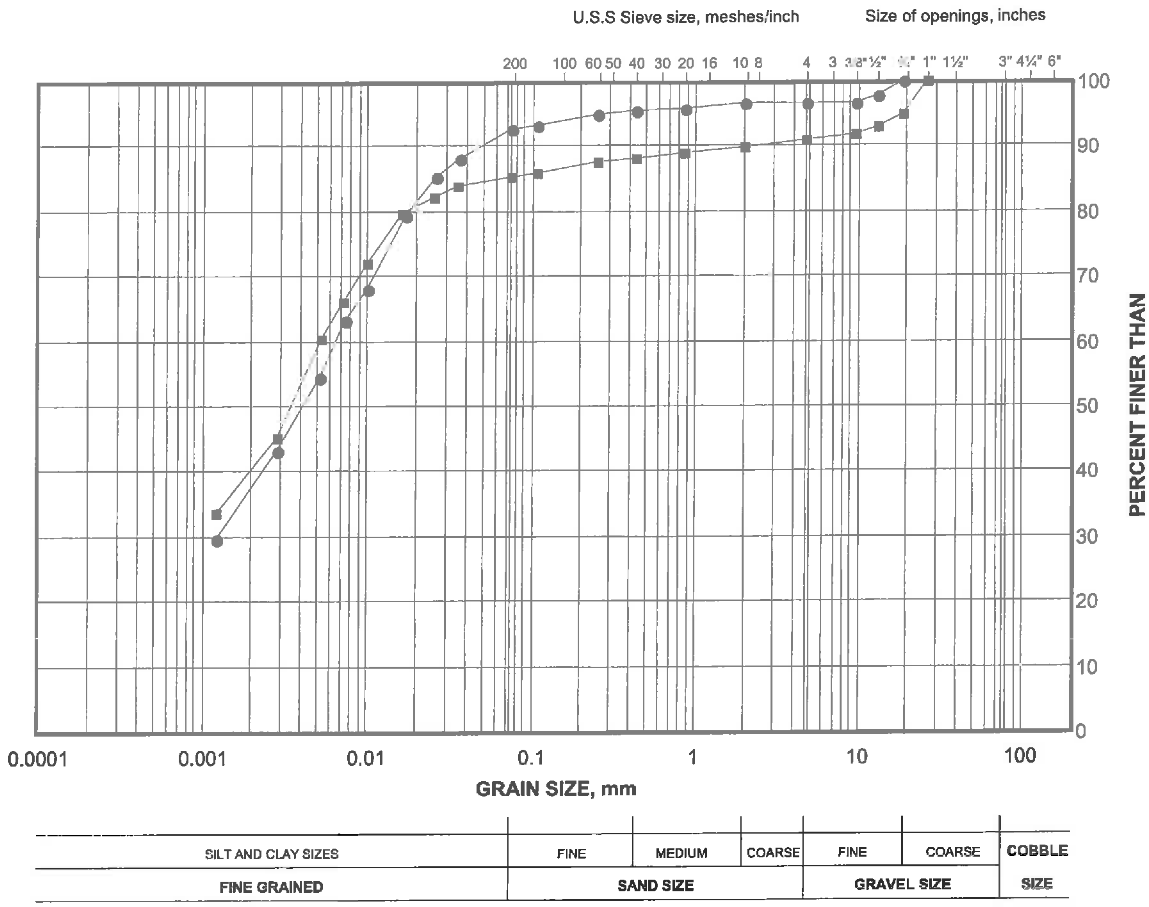
Ontario

PLASTICITY CHART
Clayey Organic Silt

Figure No. B5	
Project No. 12-1111-0088	
Checked By:	

GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIGURE B6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-04	12	160.5
■	13-06	13	159.4

Project Number: 12-1111-0088

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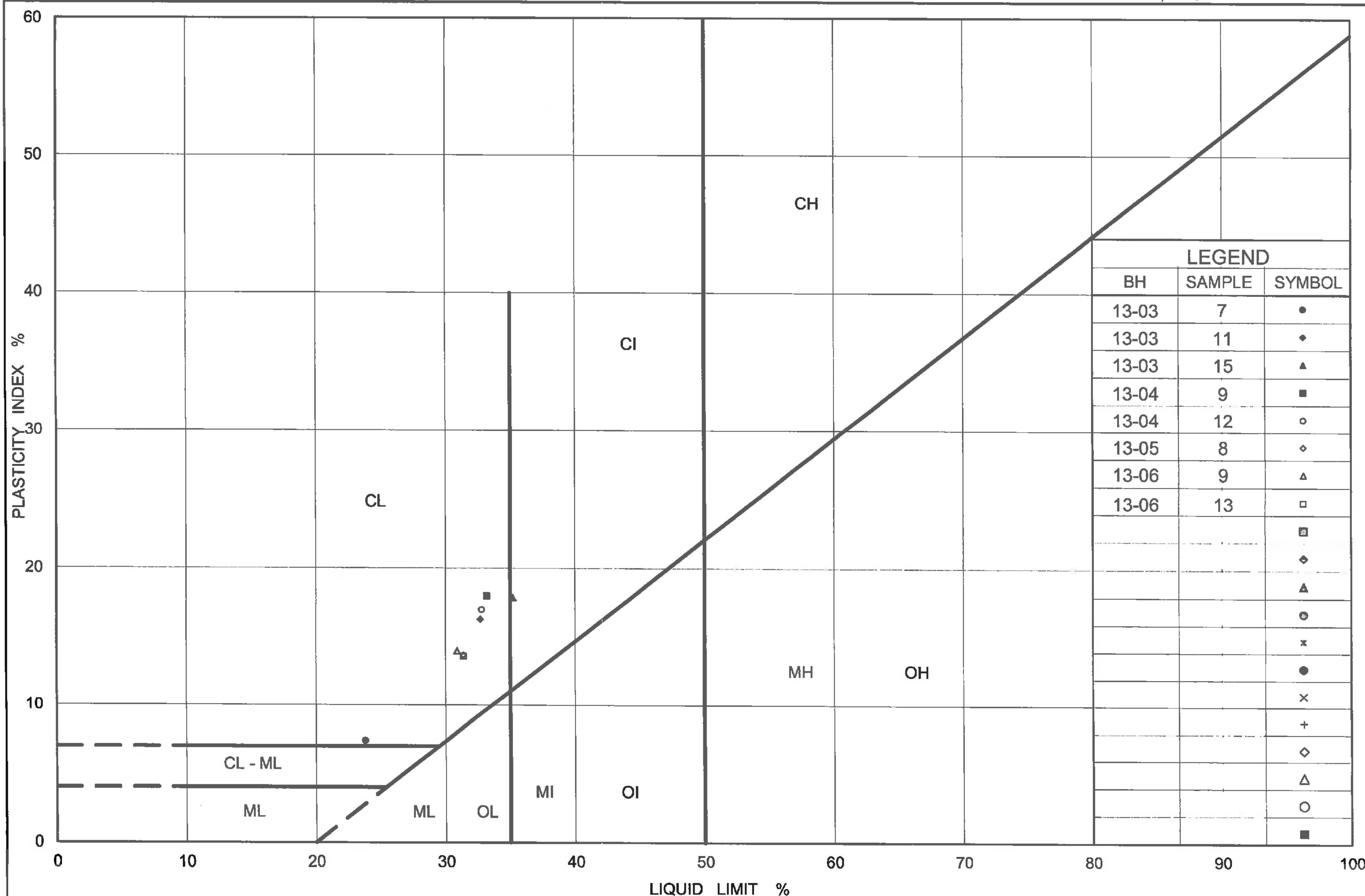

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PLASTICITY CHART
CLAYEY SILT

Figure No. B7

Project No. 12-1111-0088

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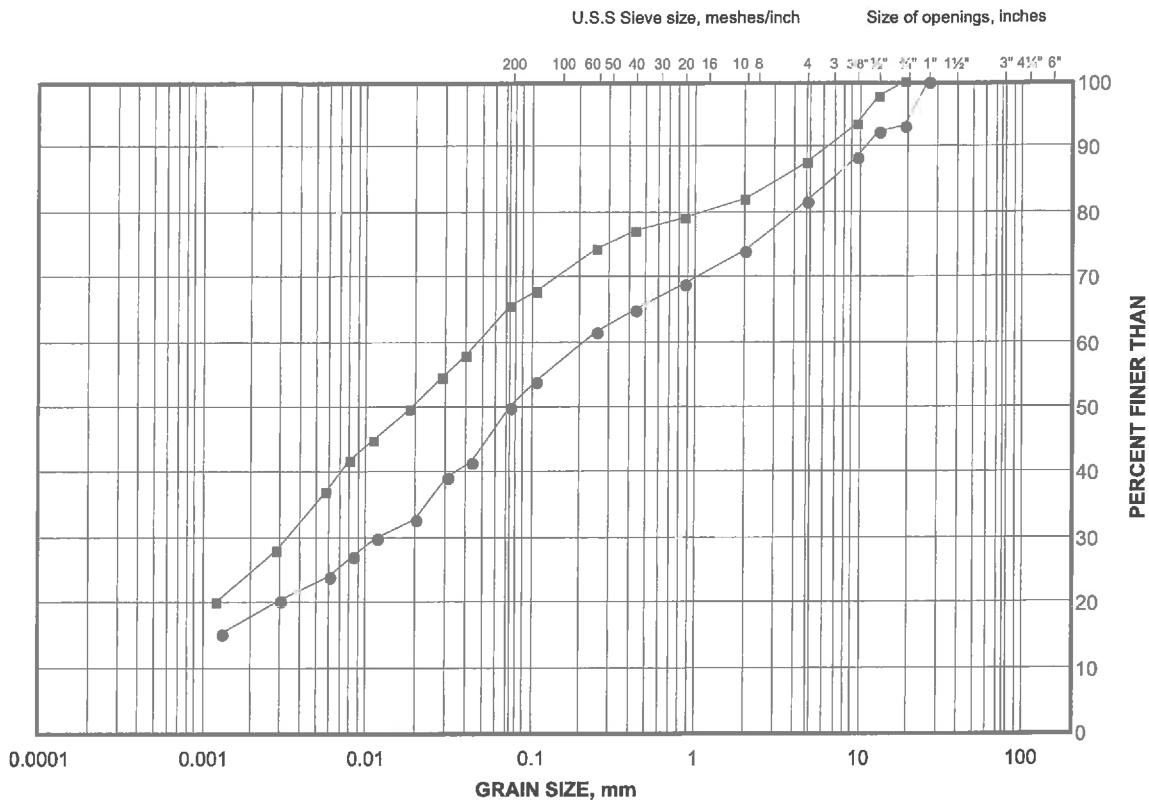
GRAIN SIZE DISTRIBUTION CLAYEY SILT TILL	FIGURE B8



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-05	14	157.2
■	13-04	15	155.9

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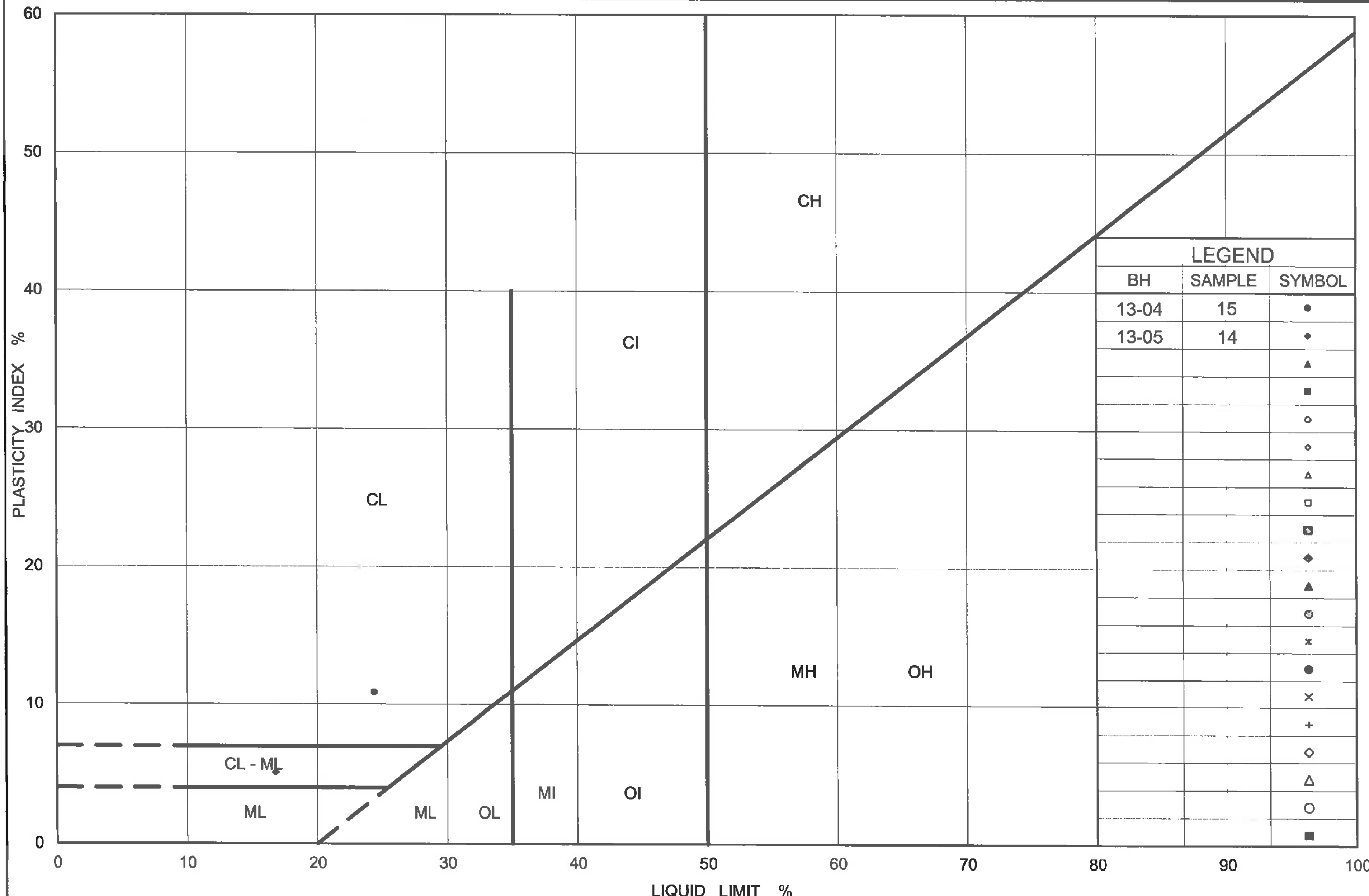

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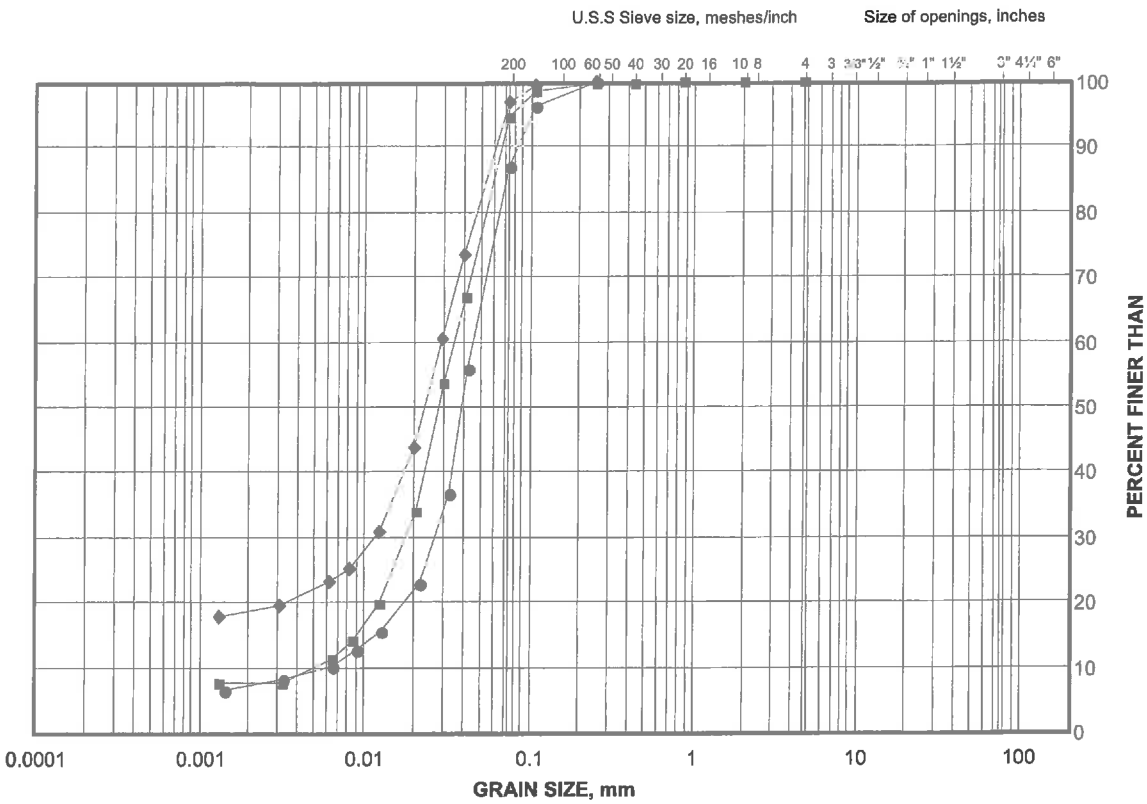
PLASTICITY CHART
CLAYEY SILT TILL

Figure No. B9	
Project No. 12-1111-0088	
Checked By:	

GRAIN SIZE DISTRIBUTION
SILT

FIGURE B10



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-06	15	156.3
■	13-03	16	153.4
◆	13-04	18	146.8

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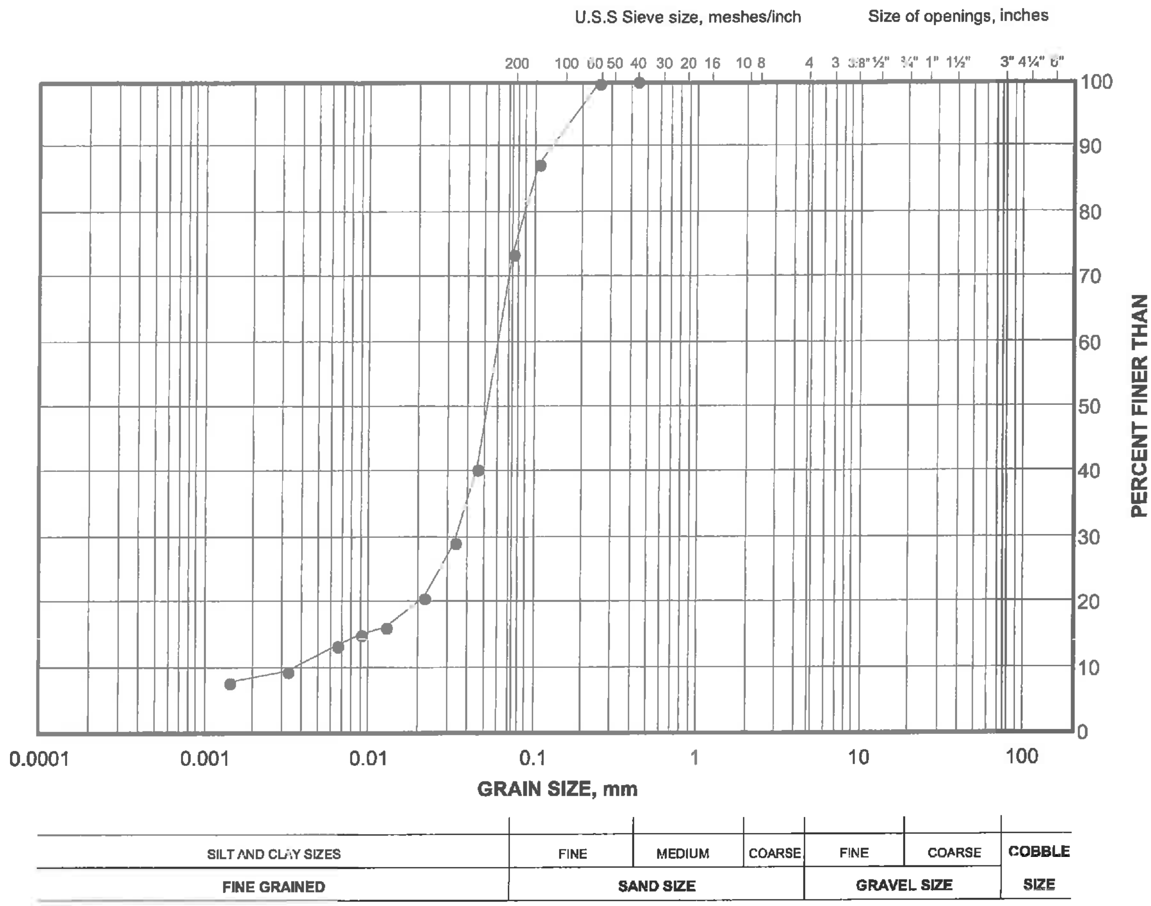
GRAIN SIZE DISTRIBUTION

Sandy SILT

FIGURE B11



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-05	17	149.6

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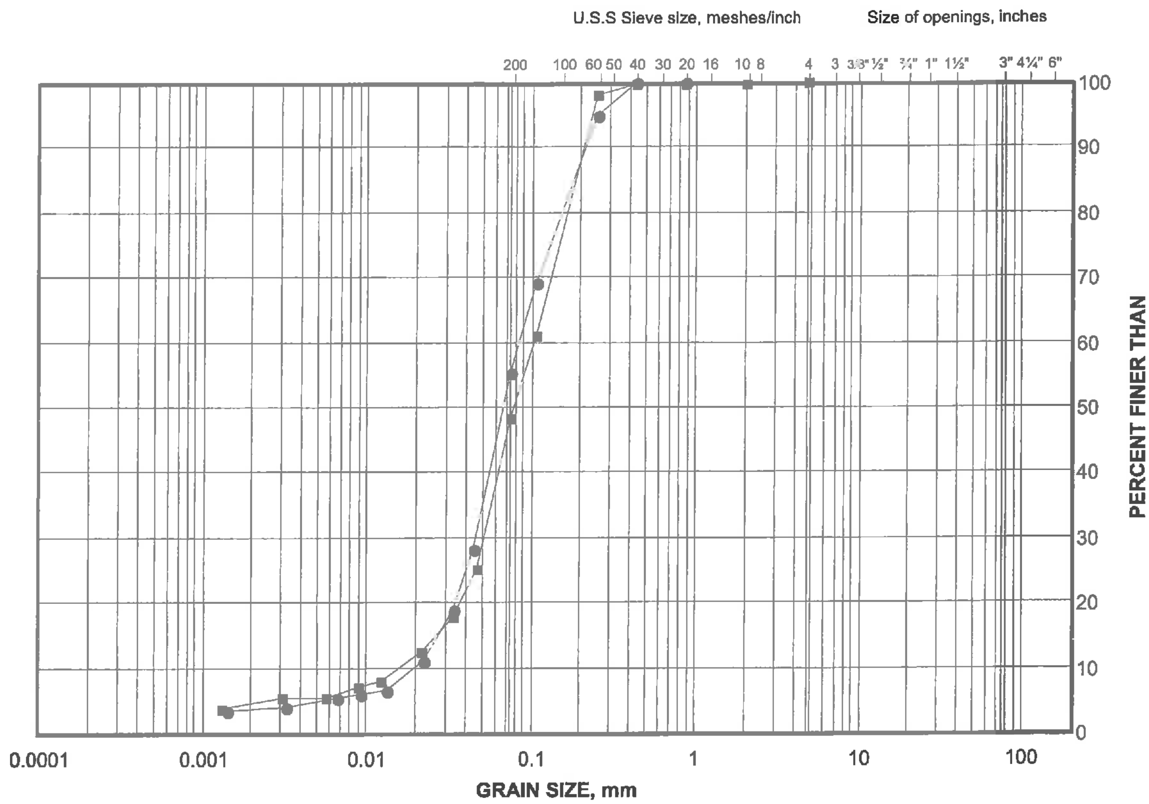
GRAIN SIZE DISTRIBUTION
SILT and SAND

FIGURE B12



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-04	16	152.9
■	13-03	18	147.3

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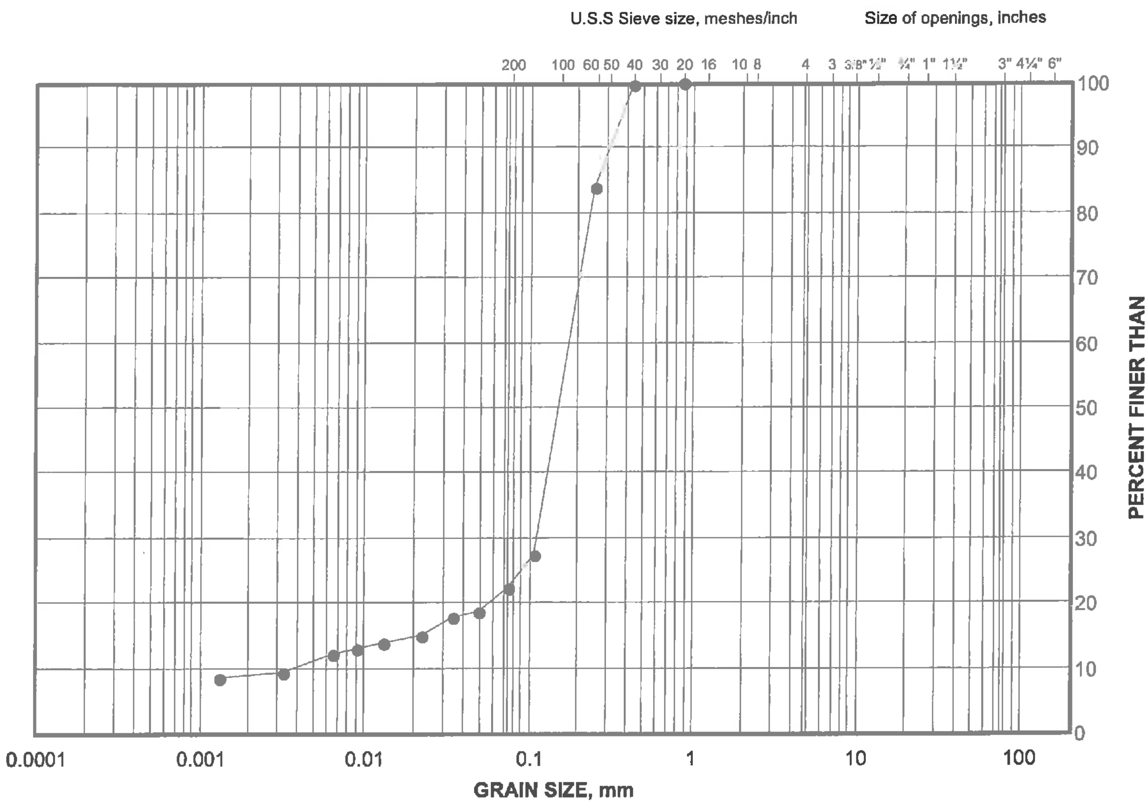
GRAIN SIZE DISTRIBUTION SAND	FIGURE B13



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-06	17	150.2

Project Number: 12-1111-0088

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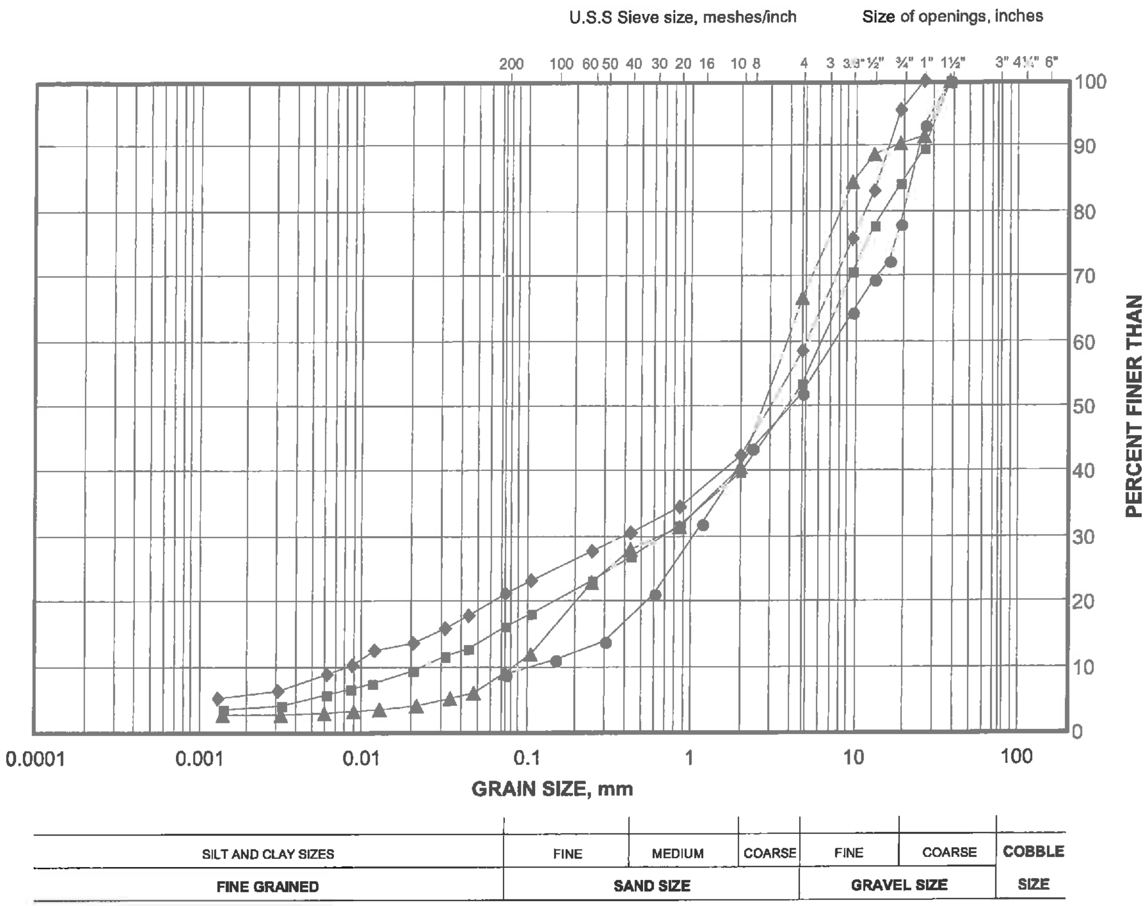
GRAIN SIZE DISTRIBUTION SAND and GRAVEL	FIGURE B14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-05	19	143.5
■	13-03	20	141.2
◆	13-06	20	141.3
▲	13-04	21	137.6

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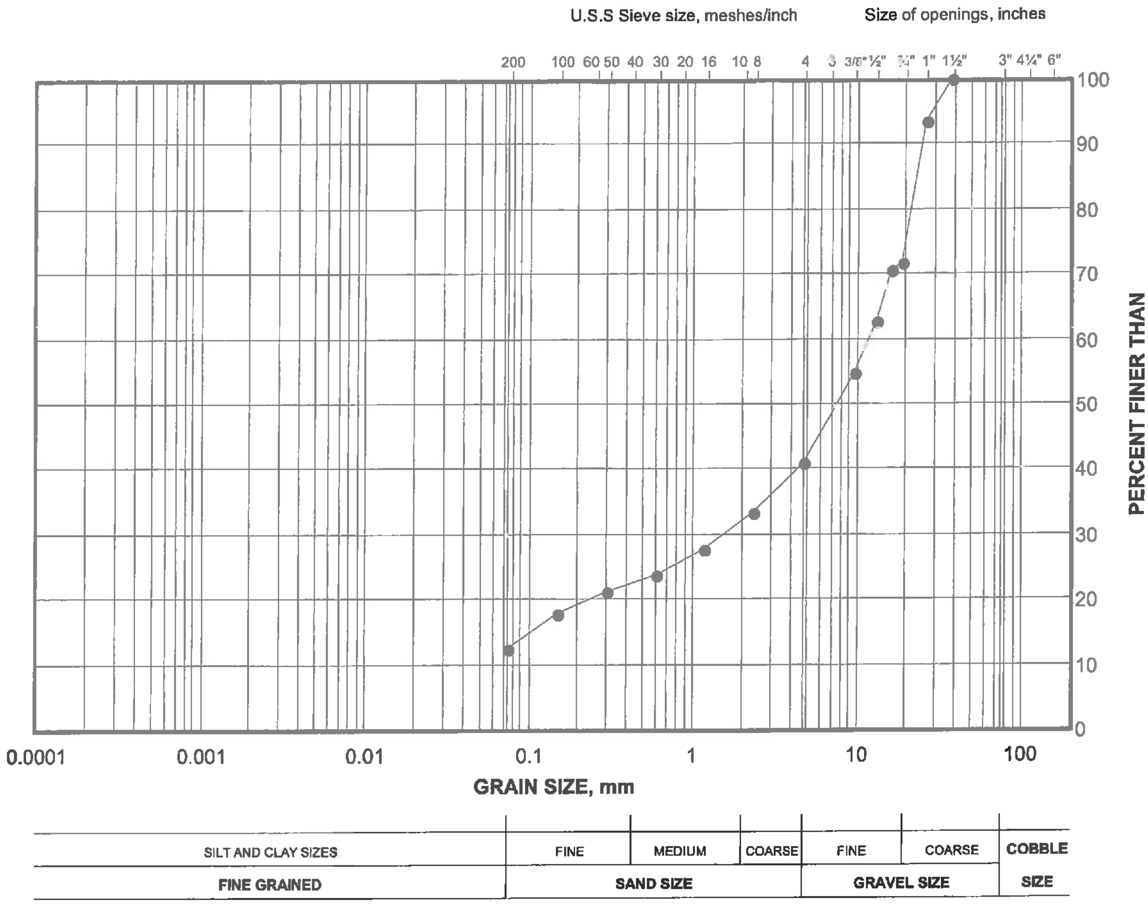
GRAIN SIZE DISTRIBUTION
Sandy GRAVEL

FIGURE B15



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-05	21	137.6

Project Number: 12-1111-0088

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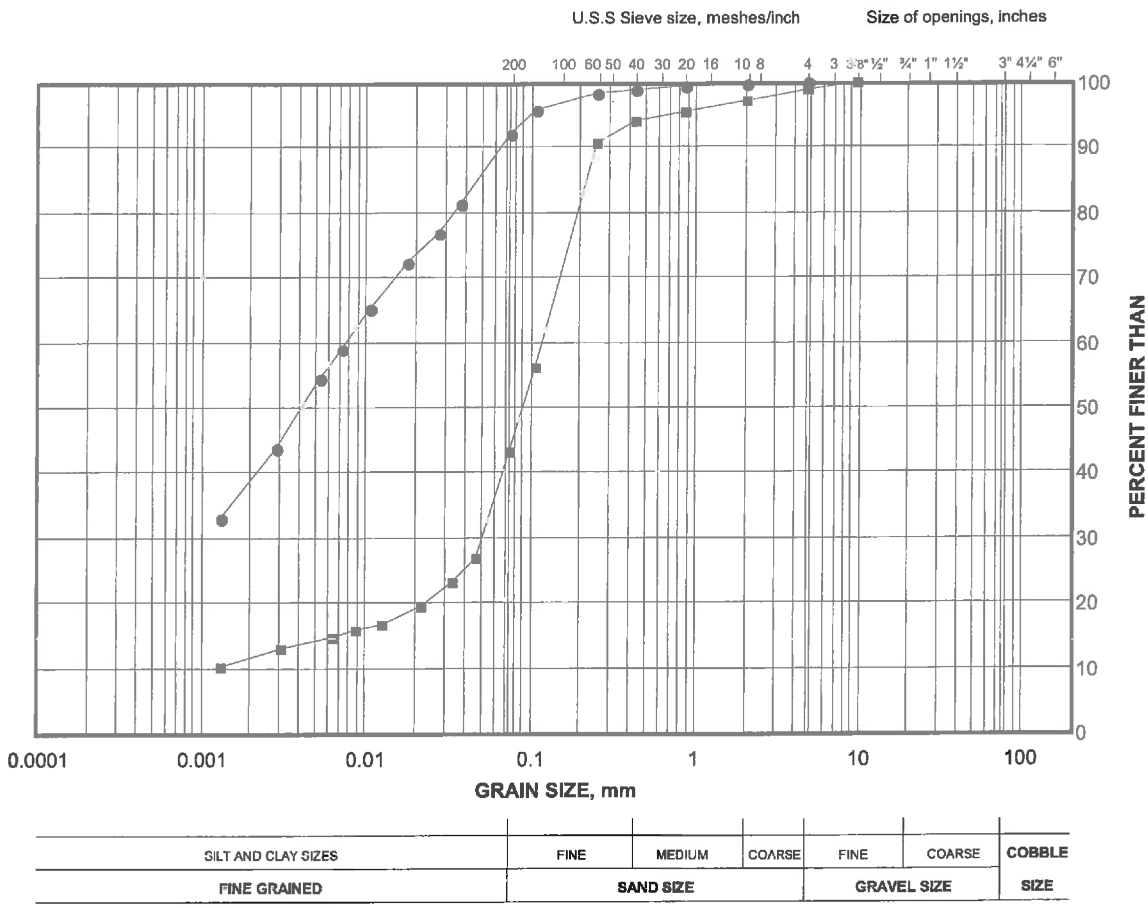
Date: 22-Oct-13

GRAIN SIZE DISTRIBUTION

CLAYEY SILT to CLAYEY SILT with SAND

FIGURE B16



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-04	20	140.7
■	13-05	20B	140.6

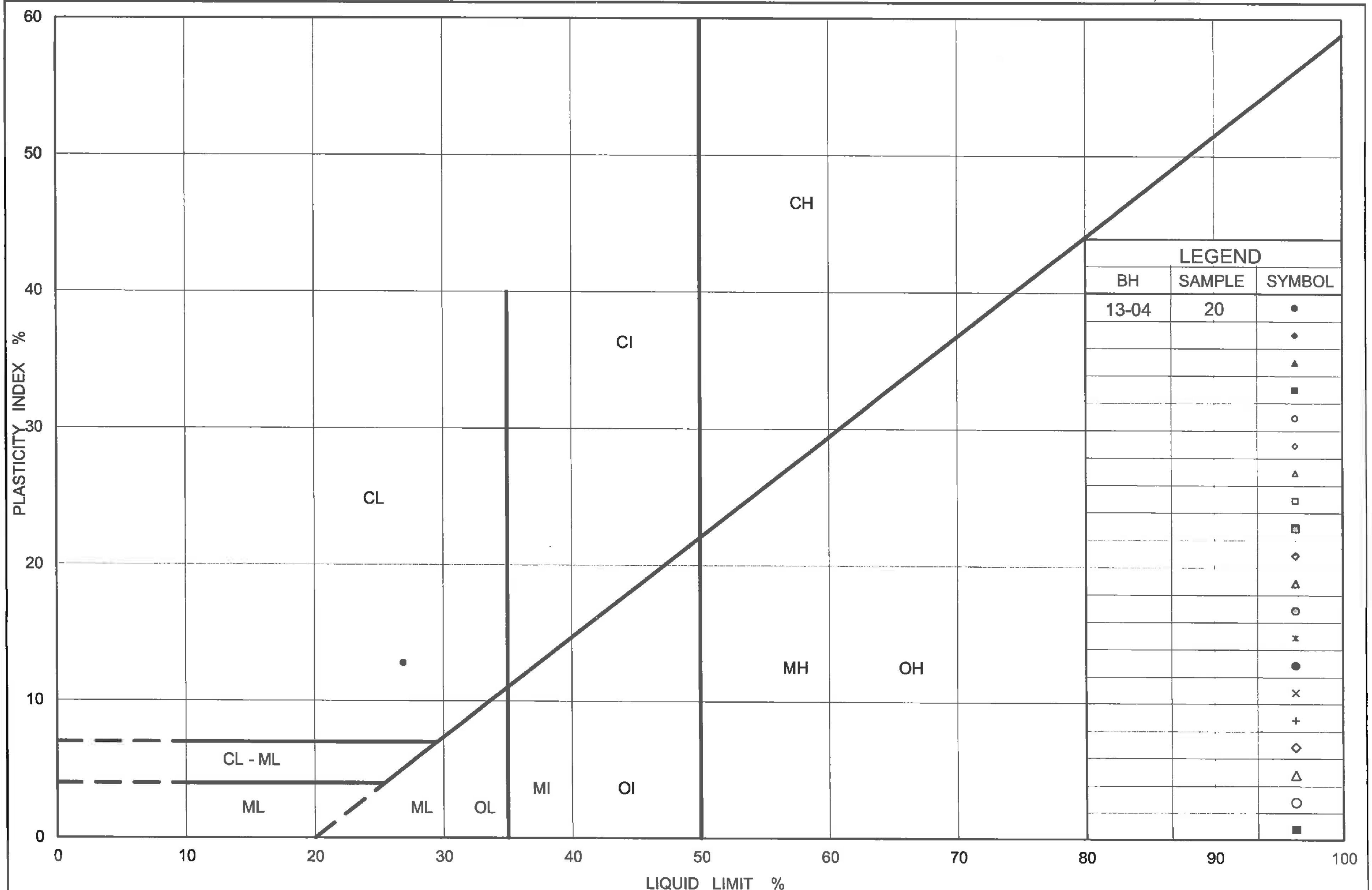
Project Number: 12-1111-0088

Checked By:

Golder Associates

Date: 22-Oct-13

Oct 75, FF-S-21



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PLASTICITY CHART
CLAYEY SILT to CLAYEY SILT with SAND

Figure No. B17

Project No. 12-1111-0088

Checked By:

APPENDIX C

Record of Borehole sheets from Previous Investigation (GEOCRES NO. 30M03-111)

FORM 08-MT-126
66-4346

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS · ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 68-F-8 LOCATION Q.E.W. & Lyons Creek (Ramp S-EW) Sta. 5 + 47 10.5' Lt. ORIGINATED BY VK
W.P. 158-64-3 BORING DATE Feb. 1, 1968 COMPILED BY VK
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX, AX Casing, AXT Core CHECKED BY

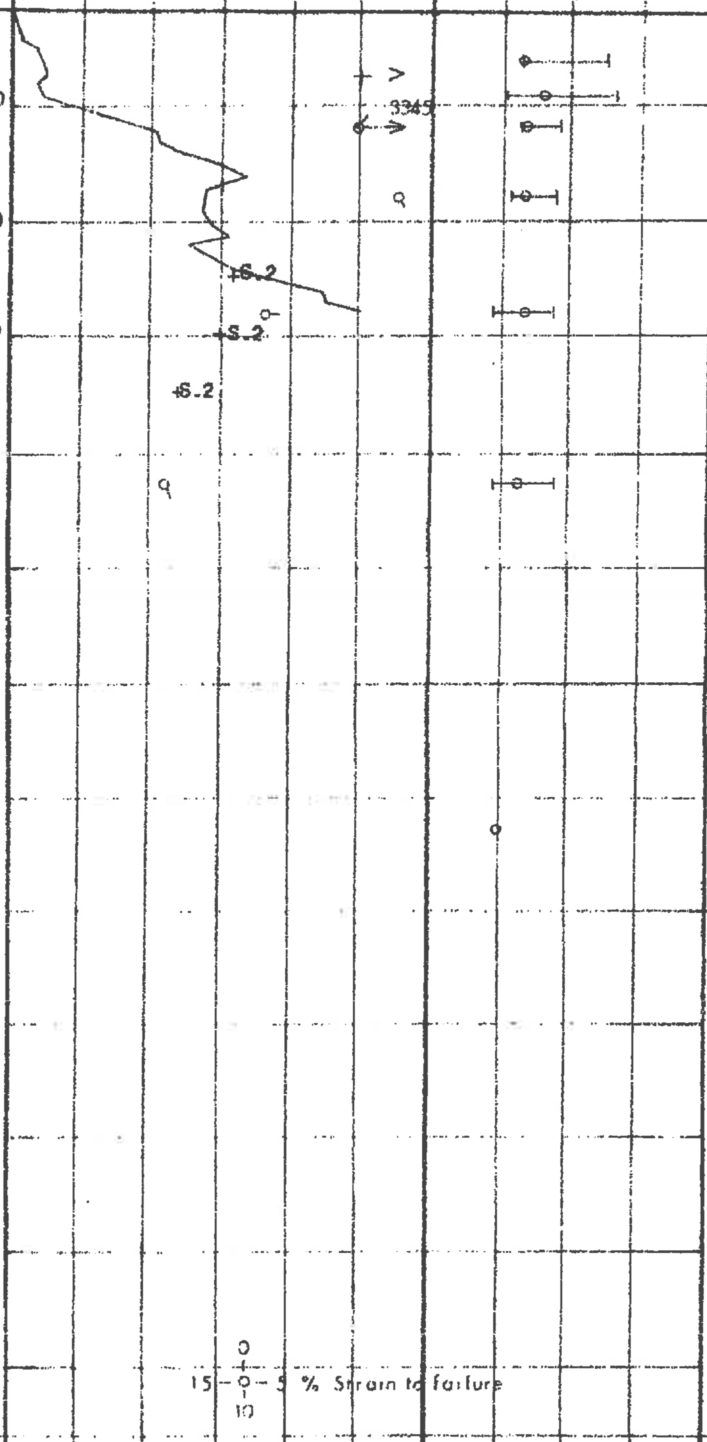
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS/FOOT	REMARKS (Gr.Sa.Si.Cl)
568.0	Ground Level					
0.0	Desiccated (mottled brown to grey-brown) Very stiff		1	SS	28	0 1 27 72
			2	SS	15	0 4 61 35
			4	TW	PM	▼556.5
			5	TW	PM	
			6	TW	PM	
552.0 / 16.0	Silty clay to clayey silt, trace of sand & occasional gravel (occasional seams of silt up to 2" thick throughout) (grey-brown)		7	TW	PM	
			8	TW	PM	
			9	TW	PM	
			10	TW	PM	
	Firm to stiff		11	SS	14	
510.0 / 58.0	Sandy silt to silty fine sand occasional boulders up to 6 to 12" in size below about elev. 477		12	SS	18	0 31 63 6
	Compact to very dense		13	SS	59	
			14	AXT	39%	
			15	AXT	16%	
			16	AXT	11%	
			17	AXT	20%	
			18	AXT	13%	
448.0 / 120.0	Dolomitic Limestone Bedrock (sound)		19	AXT	100%	
444.0 / 124.0	End of Borehole					

ELEV SCALE: 560, 550, 540, 530, 520, 510, 500, 490, 480, 470, 460, 450

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. o Unconfined + Field Vane 400 800 1200 1600 2000

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w
WATER CONTENT % 20 40 60

BULK DENSITY γ P.C.F.: 124, 121, 126, 126



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 2

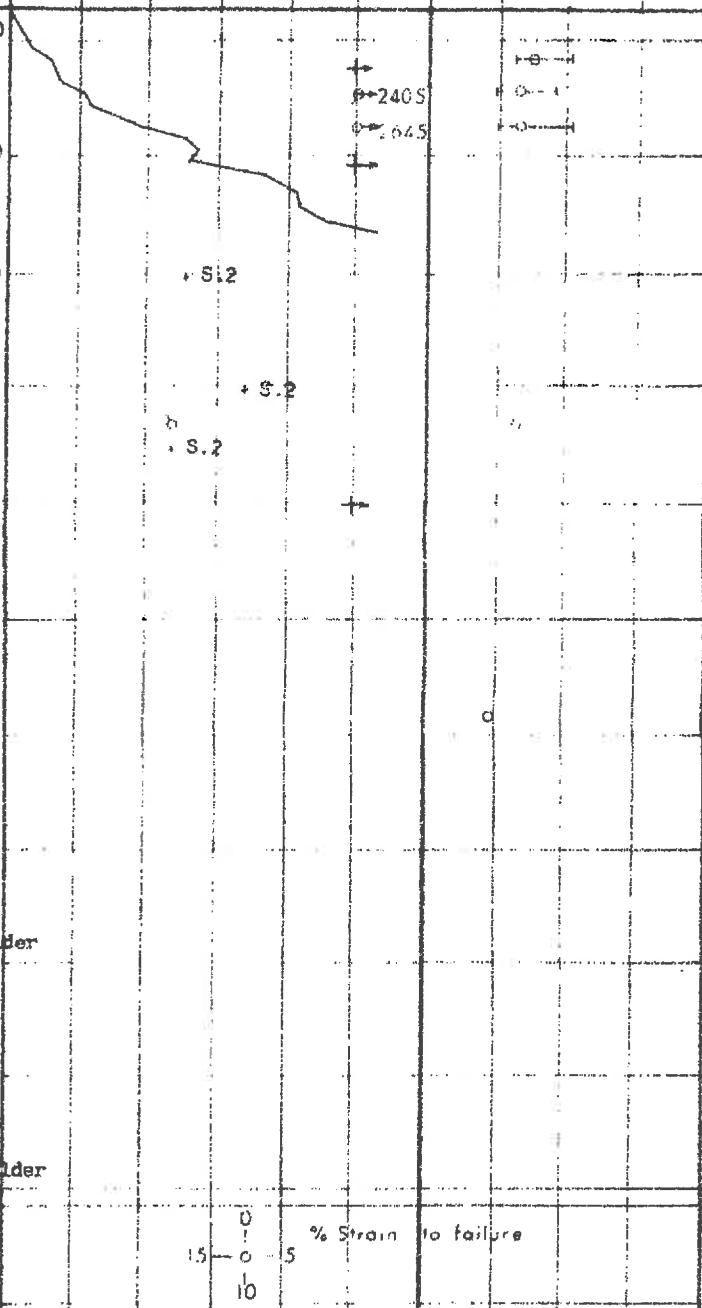
FOUNDATION SECTION

JOB 68-F-8
W P 158-64-3
DATUM Geodetic
LOCATION Sta. 5 + 12 (Ramp S-EW) 24.0' Rt.
BORING DATE Feb. 8, 1968
BOREHOLE TYPE Diamond Drill - NX, BX Casing - BXL Core
ORIGINATED BY VK
COMPILED BY VK
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY P.C.F.	REMARKS
562.5	Ground Level					
0.0	Desiccated (mottled red-brown & grey-brown) Very stiff.	1	TW	PM		Org. 6.7%
		2	TW	PM	124	
		3	SS	23		
		4	TW	PM	125	
		5	SS	15		
545.5 / 17.0	Silty clay to clayey silt, trace of sand and occasional gravel (brown to grey-brown) Firm to stiff	6	TW	PM		
		7	TW	PM		
		8	TW	PM		
		9	TW	PM		
		10	TW	PM	128	
		11	TW	PM		
511.5 / 51.0	Sandy silt to silty fine sand. Occasional boulders up to 6 - 12" in size below about elev. 482. Compact to dense.	12	SS	17		
		13	SS	31		Gr. 0, Sa. 32, Si. 66, Cl. 2
		14	SS	28		
		15	SS	100/5" Boulder		
		16	SS	44		
		17	SS	100/5" Boulder		
458.5 / 104.0	End of Borehole					

ELEV SCALE: 560, 550, 540, 530, 520, 510, 500, 490, 480, 470, 460, 450

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20, 40, 60, 80, 100
SHEAR STRENGTH P.S.F.: o Unconfined, + Field Vane: 400, 800, 1200, 1600, 2000
LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %: 20, 40, 60



FORM DB-MT 126
65-4516

OFFICE REPORT ON SOIL EXPLORATION

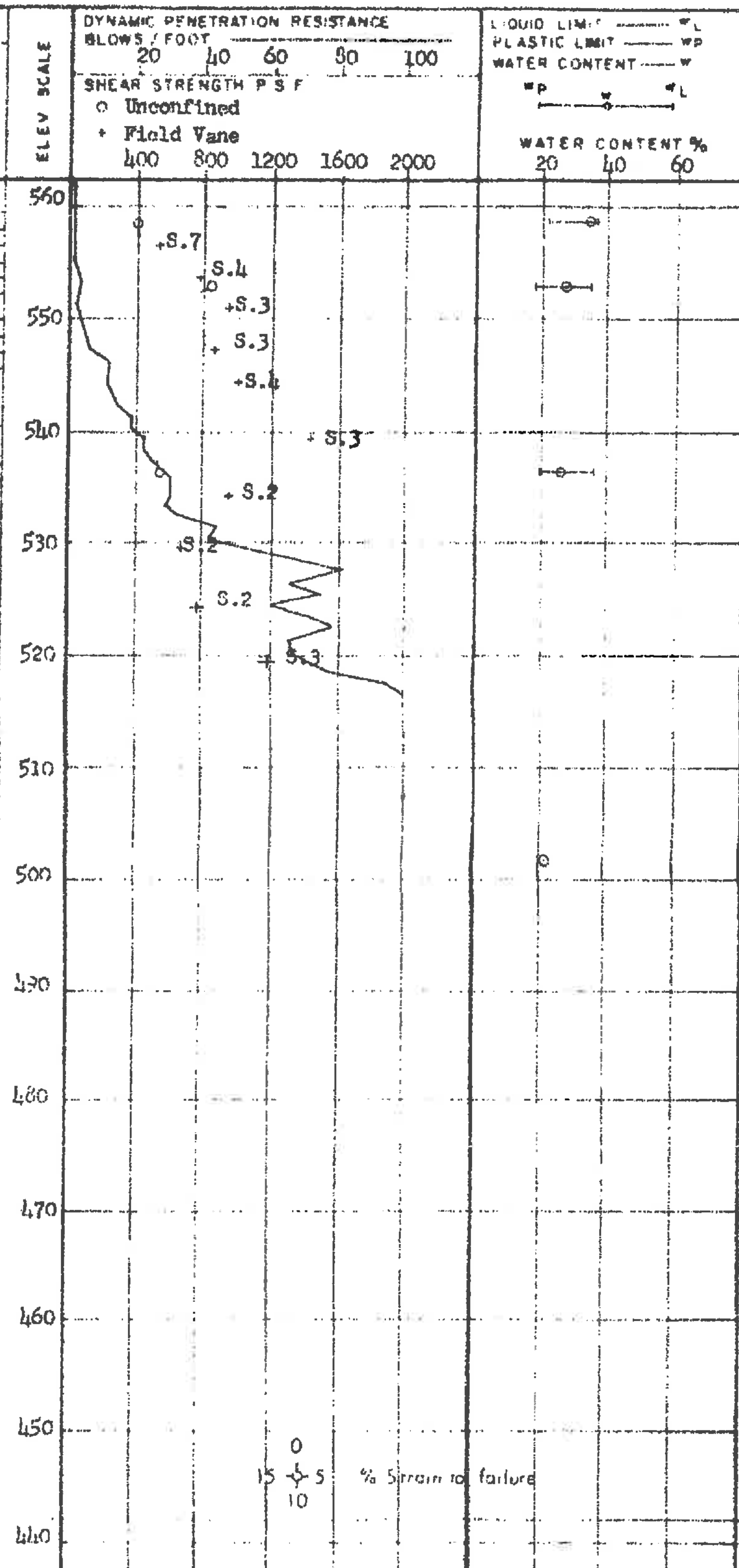
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 68-F-8 — LOCATION (Ramp N-EW) Sta. 4 + 37 10.5' Lt. — ORIGINATED BY VK
W.P. 158-64-3 — BORING DATE Feb. 14, 1968 — COMPILED BY VK
DATUM Geodetic — BOREHOLE TYPE Diamond Drill - NX Casing, AXT Core — CHECKED BY

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
562.0	Ground Level					
0.0	Organic clayey silt. (Black) Soft	1	TW	PM	117	Org. 26.8%
558.5 / 3.5	Silty clay to clayey silt, trace of sand & occasional gravel (brown to grey) Firm to stiff	2	TW	PM		
		3	TW	PM	124	
		4	TW	PM		
		5	TW	PM		
		6	TW	PM		
		7	TW	PM	124	
		8	TW	PM		
		9	TW	PM		
		10	TW	PM		
510.0		11	TW	PH		
52.0	Sandy silt to silty fine sand, occasional boulders up to 6-12" in size below about elev. 482. (brown to grey) Compact to dense	12	SS	23		1 38 58 3
		13	SS	38		
447.0 / 115.0	Dolomitic Limestone Bedrock (sound)	14	AXT	100%		
442.0 / 120.0	End of Borehole					



DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 4

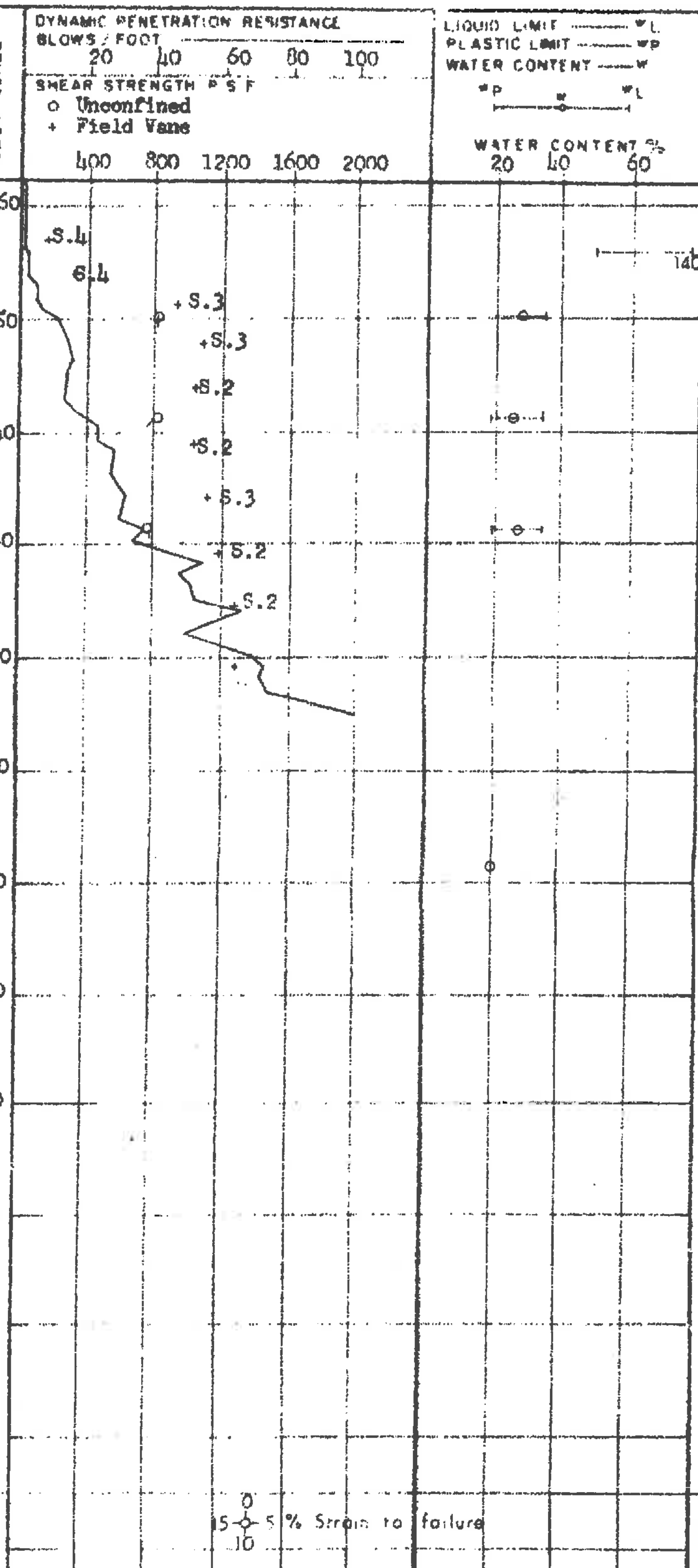
FOUNDATION SECTION

JOB 68-F-8 | LOCATION (Ramp S-EW) Sta. 4 + 01 22.5' Rt. | ORIGINATED BY VK

W P 158-64-3 | BORING DATE Feb. 19, 1968 | COMPILED BY VK

DATUM Geodetic | BOREHOLE TYPE Diamond Drill – NX, BX Casing – AXT Core | CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS
562.0	Ground Level					W.L.
0.0	Organic clayey silt. (black) Very soft to soft	1	TW	PM		
		2	TW	PM	172	Org. 13.3%
553.5		3	TW	PM		
8.5	Silty clay to clayey silt, trace of sand and occasional gravel (grey-brown) Firm to stiff	4	TW	PM	124	
		5	TW	PM		
		6	TW	PM	122	
		7	TW	PM		
		8	TW	PM	125	
		9	TW	PM		
		10	TW	PM		
516.0 / 46.0	Sandy silt to silty fine sand; occasional boulders up to 6-12" in size below about elev. 482. Compact to dense.	11	SS	13		
		12	SS	26		Gr.0, Sa.19 Si.76, Cl.5
		13	SS	35		
447.0 / 445.0	Dolomitic Limestone Bedrock (sound)	14	AXT	100%		
417.0	End of Borehole					



FORM 08-MT-126

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 5

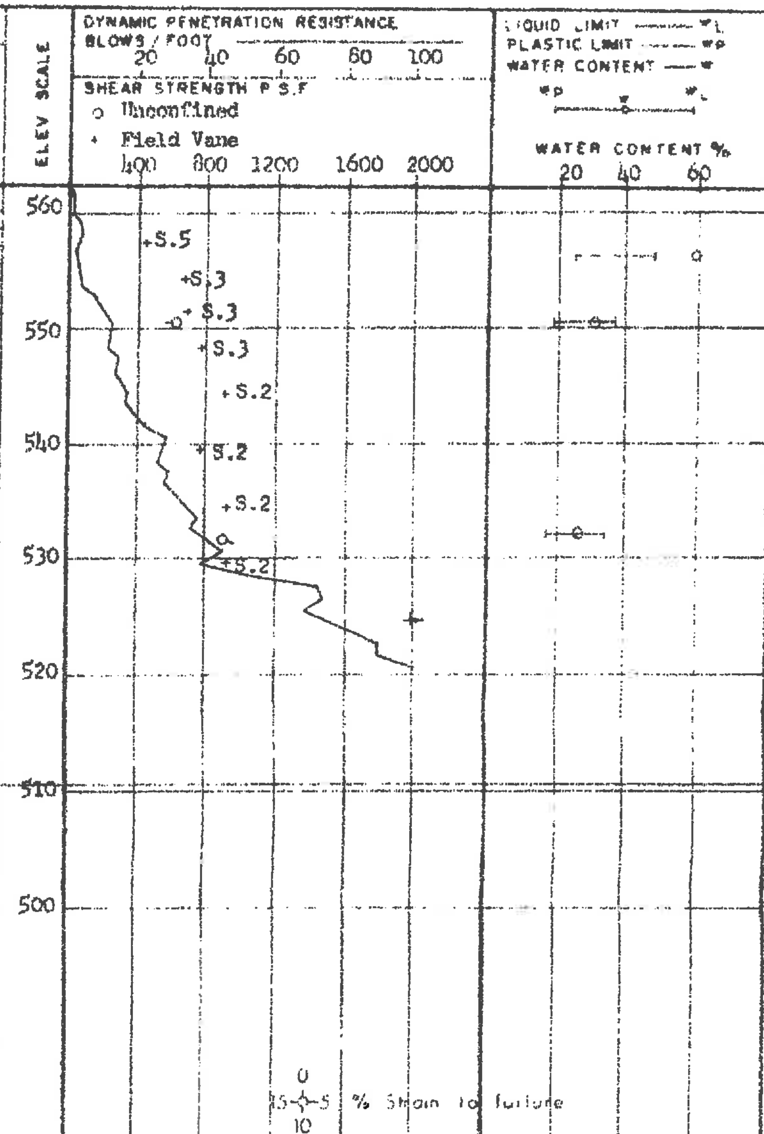
FOUNDATION SECTION

JOB 68-F-8 LOCATION (Ramp S-EW) Sta. 3 + 61 4.5' Rt. ORIGINATED BY VK

W. P. 158-64-3 BORING DATE Feb. 23, 1968 COMPILED BY VK

DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX Casing CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	BULK DENSITY γ P.C.F.	REMARKS
562.0	Ground Level					
0.0	Organic clayey silt (Black)	1	TW	PM		
554.5	Very soft to soft.	2	TW	PM		Org. 4.5%
7.5	Silty clay to clayey silt, trace of sand and occasional gravel (grey-brown) Firm	3	TW	PM		
		4	TW	PM	118	
		5	TW	PM		
		6	TW	PM		
		7	TW	PM		
		8	TW	PM	125	
		9	TW	PM		
		10	TW	PM		
516.0 / 46.0	Sandy silt to silty fine sand.					
510.5	(Brown) Very dense.	11	SS	52		
51.5	End of Borehole					



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