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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Black Creek Bridge Replacement Structure Site Nos. 34-128/1 and 34-128/2 Queen Elizabeth Way (QEW) Fort Erie, Regional Municipality of Niagara GWP 2177-08-00

Submitted to:
URS Canada Inc.
30 Leek Crescent, 4th Floor
Richmond Hill, Ontario
L4B 4N4



REPORT

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(GEOCRES No. 30L14-031)**

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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
BLACK CREEK BRIDGE REPLACEMENT
STRUCTURE SITE NOS. 34-128/1 AND 34-128/2
QUEEN ELIZABETH WAY (QEW)
FORT ERIE, 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 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 Black Creek bridges (MTO Structure Site Nos. 34-128/1 and 34-128/2).

2.0 SITE DESCRIPTION

The Black Creek bridges carry the southbound (Fort Erie-bound) and northbound (Toronto-bound) QEW over Black Creek, in the immediate vicinity of the Townline Road underpass in the Town of Fort Erie, within the Regional Municipality of Niagara, Ontario. Residential development and recreational properties are present in the vicinity of the site.

In general, the topography along this section of the QEW is relatively flat. The natural ground surface at the bridge site varies from about Elevation 172.8 m to 175.5 m, sloping downward toward the Black Creek channel which has its base at approximately Elevation 170 m. Black Creek is a relatively shallow watercourse, flowing from south to north, and it is approximately 20 m wide at the location of the QEW bridges. The water level at the time of the previous (1968) investigation was at approximately Elevation 171.1 m, and it is understood that the high water level at the bridge sites is at approximately Elevation 172.2 m.

The QEW embankments are up to about 4 m high at the north and south approaches to the Black Creek bridges, with the pavement grade between Elevation 175.6 m and 175.8 m. Townline Road crosses above the existing QEW lanes and Black Creek bridges at a skew, as shown on the plan on Drawing 1. The existing Townline Road underpass grade above the QEW is at about Elevation 182.3 m.

The existing QEW-Black Creek bridges are variable depth cast-in-place concrete T-beam structures 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 14.5 m each. Based on the design drawings, the foundations of the existing piers consist of interlocking sheet piles arranged in a cruciform-shaped cross-section, driven to practical refusal. The soil inside the sheet pile area was excavated or partially excavated following driving, and filled with concrete. There are no design elevations shown on the design drawings, and no as-built drawings have been found. Therefore, the extent of the sheet piles (depth and width), the extent of soil excavation inside the sheet piles and the thickness of the concrete backfill are unknown at this time.



3.0 INVESTIGATION PROCEDURES

3.1 Previous Investigations

As part of the QEW widening in the vicinity of Townline Road and Black Creek in the late 1960s, two subsurface investigations were carried out as listed below:

MTO GEOCRES No. 30L14-031: Report titled “Foundation Investigation Report for Proposed Underpass at the Crossing of Queen Elizabeth Way and Revised Townline Road, Twp. Of Willoughby – Co. of Welland, District No. 4 (Hamilton), W. J. 68-F-31 – W.P. 167-64-01”, by Department of Highways – Ontario, dated July 12, 1968.

MTO GEOCRES No. 30L14-032: Report titled “Foundation Investigation Report for Proposed Service Road No. 8 and Black Creek Crossing, Twp. Of Willoughby – Co. of Welland, District No. 4 (Hamilton), W. J. 68-F-32 – W.P. 167-64-03”, by Department of Highways – Ontario, dated July 25, 1968.

The GEOCRES-sourced boreholes referenced in the text of this report have been re-numbered to include the MTO GEOCRES No. followed by the original borehole designation to differentiate them from the boreholes advanced for the current investigation. Therefore, the boreholes from MTO GEOCRES No. 30L14-031 have been renamed to “031-X”, where X is the original borehole number.

3.2 Current Investigation

The field work for this subsurface investigation was carried out in June 2013, at which time four boreholes (Boreholes 13-11 to 13-14) were advanced through the QEW lane/shoulder, behind the existing abutments. 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 hollow stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth using a 50 mm outside diameter 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 were advanced to practical auger refusal at depths ranging from 6.7 m to 8.6 m below the QEW pavement grade.

The groundwater conditions were observed in the open boreholes during and upon completion of the drilling operations, and water level readings are indicated on the borehole records contained in Appendix A. All boreholes were backfilled with bentonite pellets and covered with asphalt patches upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised 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 Golder’s Cambridge laboratory for testing. Index and classification tests consisting of water contents, Atterberg limits and grain size distributions 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), the



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ground surface elevations (referenced to Geodetic datum) and the drilled depths are summarized below and are shown on Drawing 1.

Structure	Foundation Element	Borehole No.	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)
			Northing (m)	Easting (m)		
Fort Erie (South) Bound	North (Northwest) Abutment	13-11	4,758,285.1	343,768.6	175.7	8.5
	South (Southwest) Abutment	13-12	4,758,255.1	343,799.6	175.6	6.7
Toronto (North) Bound	South (Southeast) Abutment	13-13	4,758,274.2	343,818.4	175.8	6.6
	North (Northeast) Abutment	13-14	4,758,302.1	343,785.7	175.8	8.6

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This site 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 is a broad undulating plain that covers the area lying between the Niagara Escarpment and Lake Erie, thus occupying all of the Niagara Peninsula except that portion below the Escarpment. The region mostly contains glacio-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 Black Creek bridges. 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 borehole records contained in Appendix A. The results of the in situ field tests (i.e., SPT 'N'-values and field vane results) as presented on the borehole records and in Section 4.2 are uncorrected. The results of geotechnical laboratory testing are also presented on Figures B1 to B7 contained in Appendix B. Selected borehole records from the previous investigation at the site (GEOCRES No. 30L14-031) are presented in Appendix C. The results of the in situ field tests (i.e., SPT 'N'-values and field vane results) as presented on the borehole records and in Section 4.2 are uncorrected.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic section on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact

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



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 non-cohesive fill overlying deposits of clayey organic silt and/or silty clay which are in turn underlain by deposits of sandy clayey silt till and/or sand and gravel till. All of the boreholes were advanced to practical refusal on inferred bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

Approximately 250 mm to 300 mm of asphalt was encountered immediately below the road level in Boreholes 13-11 to 13-14. The surface of the asphalt layer varies from about Elevation 175.6 m to 175.8 m at the borehole locations.

4.2.2 Non-Cohesive Fill

A 0.5 m to 1.2 m thick layer of non-cohesive fill was encountered below the asphalt layer in Boreholes 13-11 to 13-14, and extends to depths of between 0.8 m and 1.4 m (Elevation 174.2 m to 175.0). The fill consists of brown to grey sand and gravel, generally containing trace to some silt and trace clay.

The SPT 'N'-values measured within the non-cohesive fill range from 5 blows to 30 blows per 0.3 m of penetration, indicating that the fill has a loose to dense relative density.

The natural water content measured on six selected samples of the fill range from about 3 per cent to 8 per cent. The result of a grain size distribution test completed on a selected sample of the fill is shown on Figure B1 in Appendix B.

4.2.3 Clayey Organic Silt

A 2.1 m to 4.4 m thick deposit of dark grey to black clayey organic silt, containing trace sand as well as occasional sand pockets, was encountered below the fill materials in Boreholes 13-11 to 13-13. The deposit extends to depths of 2.9 m to 4.5 m (Elevation 169.8 m to 172.8 m).

The SPT 'N'-values measured within the clayey organic silt deposit range from 3 blows to 10 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strengths ranging from about 73 kPa to greater than 96 kPa, with sensitivities of 1 and 2. Given the presence of sand pockets within the clayey organic silt deposit, the field vane results greater than 96 kPa may not fully represent the consistency of the clayey organic silt deposit. The field vane tests results indicate that the clayey organic silt deposit has a stiff consistency, while the SPT 'N'-values suggest a soft to stiff consistency.

Atterberg limits tests were carried out on selected samples of the clayey organic silt deposit and measured plastic limits of about 30 to 34 per cent, liquid limits of about 58 to 60 per cent, and plasticity indices of about 26 to 28 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B2 in Appendix B, and indicate that the material is classified as clayey organic silt of high plasticity. The natural water content measured on selected samples of the clayey organic silt deposit ranges from about 26 to 34 per cent, near the plastic limit for this material. The organic content measured on a sample of this deposit is about 10 per cent.



4.2.4 Silty Clay to Clay

A 1.1 m to 4.8 m thick deposit of brown to grey silty clay to clay was encountered below the fill materials in Borehole 13-14 and below the clayey organic silt deposit in Boreholes 13-11 and 13-13. This deposit, which generally contains trace to some sand, trace to some gravel, and trace organics, extends to depths between 4.6 m and 5.6 m (Elevations 171.1 m and 170.2 m).

The SPT 'N'-values measured within this deposit range from 2 blows to 16 blows per 0.3 m of penetration. In situ field vane tests carried out within this deposit measured undrained shear strength ranging from about 74 kPa to greater than 96 kPa, with sensitivities between 1 and 2. The field vane test results indicate that the silty clay deposit has a stiff consistency, while the SPT 'N'-values suggest a soft to stiff consistency.

The results of grain size distribution completed on selected samples of this deposit are shown on Figure B3 in Appendix B. Atterberg limits tests were carried out on selected samples of this deposit and measured plastic limits between about 20 and 24 per cent, liquid limits between about 40 and 52 per cent, and plasticity indices between about 20 and 28 per cent. The results of the Atterberg limits tests are shown on a plasticity chart on Figure B4 in Appendix B, and indicate that the material is classified as silty clay of intermediate plasticity to clay of high plasticity. The natural water content measured on selected samples of this cohesive deposit ranges from about 17 to 38 per cent. The organic content measured on a selected sample of this layer is about 4 per cent.

4.2.5 Sandy Clayey Silt Till

Pockets of sandy clayey silt till were encountered below the silty clay to clay deposit in Boreholes 13-11 and 13-14, extending to depths of 7.0 m and 7.2 m (Elevations 168.7 m and 168.6 m), respectively. This till deposit was logged as "clayey silt with sand" in the 1968 boreholes (see Appendix C), and was noted to contain cobbles and boulders.

The SPT 'N'-values measured within the cohesive till deposit in the current boreholes range from 20 blows to 48 blows per 0.3 m of penetration. The SPT 'N'-values measured in the 1968 borehole investigation ranged from 18 blow to 68 blows per 0.3 m of penetration, with numerous samples having SPT 'N'-values of greater than 100 blows per 0.3 m of penetration. These results suggest that the clayey silt till deposit has a very stiff to hard consistency.

The result of a grain size distribution test completed on a sample of this till is shown on Figure B5 in Appendix B. An Atterberg limits test was carried out on one sample of the till and measured a liquid limit of about 19 per cent, a plastic limit of about 12 per cent, and a plasticity index of about 7 per cent. The result of the Atterberg limits test is shown on a plasticity chart on Figure B6 in Appendix B, and indicates that the material is classified as clayey silt of low plasticity. The natural water content measured on three samples of the clayey silt till ranges from about 9 to 11 per cent, near or below the plastic limit for this material.

4.2.6 Sand and Gravel Till

A 1.1 m to 1.5 m thick deposit of sand and gravel till was encountered below the clayey organic silt deposit in Borehole 13-12, below the silty clay deposit in Borehole 13-13 and below the cohesive till deposit in Boreholes 13-11 and 13-14. This deposit extended to the borehole termination depths of 6.7 m to 8.6 m (Elevation 167.2 m to 169.1 m). The sand and gravel till deposit generally contains trace to some silt and trace to some clay.



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The SPT 'N'-values measured within the sand and gravel till deposit range from 7 blows to 72 blows per 0.3 m of penetration, indicating a loose to very dense relative density.

The results of grain size distribution tests completed on three selected samples of the sand and gravel till are shown on Figure B7 in Appendix B. Atterberg limits tests were completed on two samples of the fine particles of the sand and gravel till, and the results are shown on a plasticity chart on Figure B8 in Appendix B. This testing indicates that the fine matrix material is classified as clayey silt of low plasticity to silt of slight plasticity. The natural water content measured on four samples of this deposit ranges from about 8 to 14 per cent, near the plastic limit for this material.

4.2.7 Bedrock

The bedrock surface is inferred from refusal to auger advance in Boreholes 13-11 to 13-14 in the current investigation. Based on refusal, the inferred bedrock surface is at depths of between 6.7 m and 8.6 m below the QEW grade, corresponding to Elevations between 169.1 m and 167.2 m at the borehole locations. The following table summarizes the bedrock surface elevation, as inferred from auger refusal in the current investigation and based on bedrock coring in boreholes from the 1968 investigation.

Borehole No.	Bedrock Surface Elevation (m)
13-11	167.2 (Inferred)
13-12	168.9 (Inferred)
13-13	169.2 (Inferred)
13-14	167.2 (Inferred)
031-4	165.9 (Cored)
031-5	166.8 (Cored)
031-6	166.9 (Cored)
031-7	168.2 (Cored)
031-8	167.5 (Cored)

The bedrock is described in the 1968 boreholes (see Appendix C) as interbedded shale and dolomite, containing occasional gypsum inclusions and/or seams.

4.3 Groundwater Conditions

The water levels observed in the open boreholes following completion of drilling in June 2013 are shown on the borehole records contained in Appendix A, and are summarized below, together with the water level observed in open boreholes following overburden drilling in the 1968 investigation (see Appendix C).

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)
13-11	175.7	4.1	171.6
13-12	175.6	3.9	171.7
13-13	175.8	4.2	171.6



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Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Groundwater Elevation (m)
13-14	175.8	4.3	171.5
031-7	175.3	3.2	172.1
031-10	174.4	3.0	171.4

The water levels presented above and on the borehole records may not represent the long-term, stabilized groundwater level at the site. 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. Mehdi Mostakhdemi, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge Costa, P.Eng., a Principal of Golder and a Designated MTO Foundations Contact, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.

Mehdi Mostakhdemi, M.Sc., P.Eng.
Geotechnical Engineer



Lisa C. Coyne, P.Eng.
Senior Geotechnical Engineer, Principal



Jorge Costa, P.Eng.
Designated MTO Foundations Contact, Principal

AV/MM/LCC/JMAC/sm

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
BLACK CREEK BRIDGE REPLACEMENT
STRUCTURE SITE NOS. 34-128/1 AND 34-128/2
QUEEN ELIZABETH WAY (QEW)
FORT ERIE, 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 QEW bridges over Black Creek in Fort Erie, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation, together with the results from the 1968 investigation at this site. 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 replacement structure foundations.

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

The 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 alternative(s) for replacement of the Black Creek bridges.

6.2 Foundation Options

Based on the preliminary design completed to date, it is understood that the existing bridges are proposed to be replaced with single-span structures with the new abutments located immediately behind the existing piers. It is further understood that no widening/re-alignment or grade change of the QEW at the structure site is proposed at this time.

The location of the new abutment foundations should be selected to avoid interference from the existing pier foundations, which were designed to incorporate steel sheet piles driven in a cruciform shape; the soil was subsequently excavated from within the sheet piles, and the foundation element was backfilled with concrete. The design drawing (Drawing Nos. 191-15-1 to 191-15-3) for the piers of the existing structures is presented in Appendix D. No as-built information is available for the existing piers, so the founding elevation, extent of subexcavation and concrete backfill are not known. In addition, it is possible that the sheet piles were driven outside of their design locations. For this reason, further investigation is recommended to confirm the as-built configuration and location of the existing pier foundation system, to optimize the location for the new abutments. The proximity of the existing Townline Road underpass to the proposed bridges should also be taken into consideration in the selection of the preferred foundation option.

Based on the presence of the clayey organic silt and compressible silty clay to clay deposits at this site, spread footings are not considered to be a preferred option for support of the abutments for the replacement bridges. Spread footings would need to be founded below these deposits, on the hard till deposit or on bedrock, at depths of approximately 6 m to 7 m below the existing QEW grade, or on the order of 4 m below the ground surface adjacent to the existing piers. The abutments for the new structure should be supported on deep foundations that extend to or into the bedrock. The preferred option from a geotechnical/foundations perspective is to support the abutments on driven steel H-piles founded on the bedrock, in an integral abutment configuration.



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However, if it becomes necessary to core into the bedrock to achieve a minimum pile length/pile toe fixity for integral abutment design, it is recommended that consideration be given to advancing the deep foundation elements using drilled steel casings in order to penetrate into the shale and dolostone bedrock at this site.

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

- **Spread footings founded on the very stiff to hard till at the north abutments and on the bedrock at the south abutments:** The preliminary geotechnical resistances associated with the soft to stiff portion of the clayey deposits are not sufficiently high to permit design of the replacement structures on strip or spread footings supported at relatively shallow depths. Spread footings would have to be founded on the very stiff to hard till deposit at the new north abutments, and on the bedrock (or on compacted granular fill following subexcavation to bedrock) at the new south abutments. The depth of such excavation relative to the existing QEW grade would be on the order of 6 m to 7 m, requiring significant temporary protection systems/coffer dams.
- **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. Due to the relatively shallow depth to bedrock at the location of the abutments (within approximately 8.6 m below the existing QEW grade), it is recommended that the new pile caps be “perched” within the approach embankments above the floodplain grade; this would also minimize the depth of excavation and associated requirements for temporary protection systems and dewatering. If the pile caps are designed to be at lower elevations, pre-coring into the bedrock may be required in order to achieve a minimum pile length of 5 m for integral abutment design; in this case, it is recommended that consideration be given to the use of drilled steel casings to penetrate into the bedrock. There is a minor to moderate risk associated with driven piles penetrating through or “hanging up” on cobbles or boulders within the till deposits above the bedrock; this risk could be mitigated with the use of pre-coring or drilled steel casings. Where driven foundations are used, the piles should be fitted with rock points to protect them during driving through the till deposit, and to promote seating of the pile tips into the bedrock, which is sloping in the immediate vicinity of the Black Creek valley. Access and pile driving equipment set-up within or immediately adjacent to the valley must be considered for construction.
- **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 (but would permit semi-integral abutment design only) and associated wing walls/retaining walls at this site. As for the option of driven H-piles, 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. Driven 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 till deposits at this site. As for H-piles, it may be necessary to pre-core into the bedrock in order to achieve a minimum pile length and/or pile toe fixity, and in this case, drilled steel casings (which are typically on the order of 600 mm in diameter) could be considered as a means of penetrating the bedrock, or as an alternative to driven pipe piles.



- **Caissons founded in the bedrock:** Caissons founded in the bedrock are feasible for support of the new abutments (although they would preclude integral abutments) 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 till deposits. In addition, coring and/or churn drilling techniques are expected to be required to penetrate into the bedrock to the target founding levels. Penetrating through cobbles/boulders and into medium strong bedrock is feasible for caissons on the order of 0.9 m to 1.2 m in diameter; however, smaller diameters (such as 600 mm diameter drilled steel casings) would be more readily constructable.

The following sections provide recommendations for spread footings, driven steel H-pile or pipe pile foundations, and caisson foundations to support the proposed bridge replacement. Deep foundations, whether H-piles or caissons, should be constructed in accordance with OPSS.PROV 903 (Deep Foundations).

6.3 Spread Footings

6.3.1 Founding Elevations

Although footings are not preferred from a foundations perspective, if they are adopted for support of the new abutments, associated wingwalls or retaining walls, strip or spread footings should be founded below any fill and soft to stiff cohesive soils on the very stiff to hard cohesive deposits at the north abutments and on the bedrock at the south abutments. Alternatively, the foundations can be perched on compacted granular pads following sub-excavation of any fill, soft to stiff and/or loose to compact native soils at the locations of the north and south abutments. The footing founding elevations should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration. For preliminary design purposes, the maximum (highest) founding elevations for spread footings at the locations of the north and south abutments may be taken as given in the table below:

Structure	Foundation Element	Borehole No.	Founding Stratum	Maximum Founding Elevation (m)
Fort Erie (South) Bound	North Abutment	13-11	Very stiff silty clay	171.5
	South Abutment	13-12	Bedrock	168.9
Toronto (North) Bound	North Abutment	13-14	Very stiff clayey silt till	169.5
	South Abutment	13-13	Bedrock	169.1

It is noted that with the QEW grade at approximately Elevation 175.7 m, the above recommended founding elevations will be approximately 4.1 m to 8.5 m below the current highway grade. As an alternative to founding directly on the native soil and/or bedrock, subexcavation could be carried out to the elevation identified in the table above, then backfilled with compacted Ontario Provincial Standard Specification (OPSS.PROV 1010) Granular A or Granular B Type II fill prior to construction of the footings at a higher elevation, at a minimum depth of 1.2 m below the lowest surrounding grade. The compacted granular pad should extend at least 1 m



beyond the front and back edge of the new centre pier footing, then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS.PROV 501 (*Compacting*).

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill, loose or soft to stiff soils, or other unsuitable material have been removed. The founding soils will be susceptible to disturbance and should be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade.

6.3.2 Geotechnical Resistance/Reaction

Strip or spread footings placed on the properly prepared native soil or bedrock, at or below the design elevations given in the preceding section, should be designed based on the following factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS). These values assume a footing width of approximately 3 m to 4 m.

Structure	Foundation Element	Founded on Native Soils/Bedrock		Founded on Compacted Granular Pads	
		Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS	Factored Geotechnical Resistance at ULS	Geotechnical Reaction at SLS
Fort Erie (South) Bound	North Abutment	400 kPa	350 kPa*	600 kPa	400 kPa*
	South Abutment	800 kPa	600 kPa**	600 kPa	400 kPa*
Toronto (North) Bound	North Abutment	400 kPa	350 kPa*	600 kPa	400 kPa*
	South Abutment	800 kPa	600 kPa**	600 kPa	400 kPa*

* For 25 mm of settlement where supported on till deposit

** For 10 mm of settlement where supported on bedrock

The preliminary geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed new foundation elements.



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

6.4.1 Founding Elevations

The new abutments and associated wingwalls or retaining walls may be supported on steel H-piles or steel pipe (tube) piles driven to found on or in the bedrock. The surface elevation for the bedrock varies at the site, with a shallow “valley” within the rock below Black Creek, and further investigation is recommended to confirm these preliminary founding elevations. The following pile tip elevations may be used for preliminary design purposes, assuming termination on or just into the bedrock:

Structure	Foundation Element	Borehole No.	Estimated Design Pile Tip Elevation (m)
Fort Erie (South) Bound	North Abutment	13-11 031-5 031-6	166.5
	South Abutment	13-12 031-8	167.0
Toronto (North) Bound	North Abutment	13-14 031-5 031-6	166.5
	South Abutment	13-13 031-8	167.0

The pile caps should be placed at a minimum depth of 1.2 m below adjacent final grade for frost protection purposes. The elevation of the underside of the new pile caps is not known at this time. However, a minimum pile length of 5 m is required for integral abutment design, and if possible the structure span should be configured such that the pile caps can be placed high enough to eliminate the need for pre-coring into the bedrock.

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 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 vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with rock points to reduce the potential for damage to the piles during driving, and to facilitate “seating” of the piles onto the bedrock surface, which is relatively shallow and which may have local slopes or steps.

6.4.2 Geotechnical Axial Resistance/Reaction

For preliminary design for HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored geotechnical axial resistance at ULS may be taken as 1,600 kN, and the geotechnical axial reaction at SLS (for approximately 10 mm of settlement) may be taken as 1,400 kN. The same 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.).

As the abutments are proposed to be moved closer to Black Creek, some fill placement will be required behind the new abutments above the current ground surface. Although the design and construction of this backfilling



may be completed using lightweight fill materials to mitigate settlements, at this preliminary stage it is recommended that conventional granular fill materials be assumed. 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, and 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 50 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 for the settlement and the downdrag loads should be reassessed during detail design, when the abutment locations are confirmed and the construction staging and settlement mitigation approaches are known. In this context, it is recommended that allowances be made for the settlement of the clayey organic silt in the design.

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

6.5 Caissons

As an alternative to driven steel H-piles or pipe piles, caissons or drilled steel casings 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 soil deposits, and to penetrating into the medium strong to strong bedrock. Both drilled steel casings (typically on the order of 600 mm) and larger diameter caissons are capable of penetrating through obstructions, generally by coring, and socketting into the bedrock, generally by coring or churn drilling. From a constructability standpoint, smaller diameter drilled steel casings would have an advantage over larger diameter caissons both for penetrating obstructions and forming a socket within the bedrock.

6.5.1 Founding Elevations

As the surface of the bedrock varies based on the borehole results, and to accommodate some weathering in the upper portion of the bedrock, socketting for a minimum depth of 1 m into the bedrock is recommended. The table below provides caisson founding levels for preliminary design:

Structure	Foundation Element	Borehole No.	Design Caisson Founding Elevation (m)
Fort Erie (South) Bound	North Abutment	13-11 031-5 031-6	165.5
	South Abutment	13-12 031-8	166.0
Toronto (North) Bound	North Abutment	13-14 031-5 031-6	165.5
	South Abutment	13-13 031-8	166.0



6.5.2 Geotechnical Axial Resistance/Reaction

For preliminary design, drilled steel casings or caissons socketted approximately 1 m or more into the bedrock should 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 geotechnical axial 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.

6.6 Approach Embankments

It is understood that the existing QEW embankment will not be widened, nor will the grade be raised as part of the proposed structure replacement. However, if the new abutments for the replacement are located in front of the existing abutments (i.e., a shorter single-span structure), some additional fill will be required to be placed immediately behind the new abutments up to the QEW grade.

6.6.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic material be stripped from the footprint of the additional fill that is required behind the new abutment locations. Additional subexcavation of “steps” may be required to facilitate the placement of lifts of conventional (earth or granular) fill, lightweight fill or expanded polystyrene (EPS) fill behind the new abutments, with the choice of fill material based on minimizing post-construction settlement, mitigating downdrag loads, and improving the performance of the approach embankments, depending on construction staging requirements.

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.PROV 804, is recommended as soon as practicable after construction of the new portion of the approach embankments.

6.6.2 Approach Embankment Stability

Preliminary slope stability analyses have been performed for the approach embankments 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 existing/proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed 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 4 m high approach embankment, based on the subsurface conditions as encountered in Boreholes 13-11 to 13-14. 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/m³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment fill	21	34°	-
Soft to stiff clayey organic silt	20	26°	30
Soft to stiff silty clay to clay	20	26°	30
Very stiff to hard clayey silt till	21	32°	-
Loose to very dense sand and gravel till	21	30°	-



The preliminary stability analysis results indicate that a 4 m high embankment constructed of conventional granular fill with side slopes no steeper than 2H:1V will have a factor of safety of at least 1.3 against global instability. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed during detail design, based on the material types used adjacent to the new abutment locations.

The approach embankments were analyzed for cross sections perpendicular to the QEW alignment. The global stability of any abutment foreslope adjacent to the creek channel should also be checked during detail design, based on the detailed geometry.

6.6.3 Approach Embankment Settlement

As noted above, assuming the new abutments for the replacement structures are located in front of the existing abutments (i.e., shorter single-span structures), some additional fill will be required to be placed immediately behind the new abutments up to the QEW grade. It is understood that no other grade raise or embankment widening is planned at this time. Preliminary settlement analyses for the anticipated soil conditions below the new fill behind the abutments were carried out using the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli and consolidation parameters as given in the table below, based on correlations with the SPT “N” values, undrained shear strengths, Atterberg limits testing and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m ³)	Elastic Modulus (MPa)	Preconsolidation Pressure (kPa)	C _c	C _r
Embankment fill	21	-	-	-	-
Soft to stiff clayey organic silt	20	5	300	0.36	0.07
Soft to stiff clay to silty clay	20	5	350	0.32	0.06
Very stiff to hard clayey silt till	21	50	-	-	-
Loose to very dense sand and gravel till	21	75	-	-	-

Based on this preliminary assessment, the settlement of the foundation soils under the placement of up to 4 m of additional fill is estimated to be up to about 100 mm. Approximately 30 mm of this settlement is expected to occur relatively quickly during and immediately following construction of the approach embankments. However, approximately 70 mm of this settlement is associated with longer-term consolidation of the soft to firm portion of the clayey deposits if/where new fill is placed behind new abutment locations; it is anticipated that the majority of this settlement would be completed within approximately six months. This estimated magnitude and duration of settlement should be reassessed during detail design, based on the specific abutment location, the geometry of the new fill zone, the type of fill material used (i.e., conventional, lightweight or EPS), and any additional investigation and testing completed as part of detail design.

In order to mitigate this settlement, the use of lightweight fill materials could be considered; this could include lightweight or ultra-lightweight slag, tire-derived aggregate, or expanded polystyrene (EPS) blocks. Alternatively, depending on construction staging requirements, the zone of new backfill could be constructed to design grade and preloaded for a period of approximately six months (with the duration to be confirmed 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 compression for conventional fill materials may range from 0.5 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.

6.7 Construction Considerations

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

6.7.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings or pile caps would extend through the existing fill and soft to stiff clayey deposits, and (depending on the foundation option) into very stiff to hard silty clay till and/or dense to very dense sand and gravel till. 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 should be classified as Type 3 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 1H:1V, assuming that appropriate groundwater control is in place.

Based on the existing structure and site configurations, however, it is anticipated that there is insufficient space for open-cut excavations. Therefore, protection systems would be required to support any excavations, as well as to facilitate construction staging. 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 site. In the installation of protection systems, consideration must be given to the presence of rip-rap/boulder materials on the existing abutment foreslopes, and the potential presence of cobbles and boulders within the native soils at this site.

6.7.2 Groundwater Control

While new abutment pile caps would likely be maintained above the groundwater level at the site, excavations for new spread footings (if adopted) and for pile caps could extend below the groundwater level, into the water-bearing sand and gravel till or into the clayey silt till under the clayey organic silt.

Due to the proximity of the abutments to the edge of the Black Creek, a groundwater cut-off (cofferdam or similar measure) is recommended to minimize dewatering requirements and potential environmental impacts. A cut-off/cofferdam could consist of interlocking steel sheet piles driven to the bedrock surface, with appropriate consideration for groundwater cut-off (concreting or other measures) at the bedrock interface.



6.7.3 Socket Formation in Bedrock

If pre-coring is necessary to achieve the minimum pile length required for integral abutments, or if drilled steel casings or caissons are the selected foundation option, it is recommended that an NSSP be included in the Contract Documents to warn the Contractor that the bedrock ranges in strength from weak to strong. Further, it is expected that socket formation would require coring or churn drilling to advance the hole.

6.7.4 Subgrade Protection

The soil deposits and/or the bedrock that will be exposed at the foundation subgrade level for either pile caps or footings will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a 100 mm thick concrete working slab (of 20 MPa concrete) be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and with an NSSP, which can be developed during the detail design stage.

6.7.5 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations. 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 as this may affect the installation of protection systems and deep foundation elements.

6.7.6 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 will reach this threshold level. However, vibration monitoring on the existing Black Creek bridges and the existing Townline Road underpass is recommended during construction to confirm that the vibration levels produced by construction are maintained within tolerable levels.

Existing residential buildings are located to the northeast and southwest of the structure site, approximately 100 m to 200 m from the Black 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.

6.7.7 Limited Headroom Under Existing Townline Road Underpass

Much of the new south abutment for the southbound (Fort Erie-bound) bridge structure is proposed to be located beneath the existing Townline Road underpass structure, which will afford limited headroom for foundation construction activities. Equipment is available to install driven piles, drilled steel casings or caissons within limited headroom environments. This will require further assessment through detail design.



6.8 Recommendations for Further Work During Detail Design

Additional boreholes should be drilled during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided herein, as follows:

- **Abutments:**
 - 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 the depth/surface elevation and strength of the bedrock at the location of the new abutment locations, to confirm the pile lengths, pile tip elevations and any requirements for socketting into the bedrock to achieve minimum pile lengths if applicable.
 - Assessment of the as-built configuration of the foundations of the existing bridges, including determining the as-built location and configuration of the sheetpile foundations for the existing piers.
- **Approach embankments:**
 - Further assessment of the thickness and consolidation/elastic compression properties of the soft to stiff clayey soils, to confirm the settlement estimates once the detailed geometry of the new fill placement behind the new abutment locations is confirmed.
 - Further assessment of the engineering parameters and global slope stability based on the potential use of lightweight fill materials.



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7.0 CLOSURE

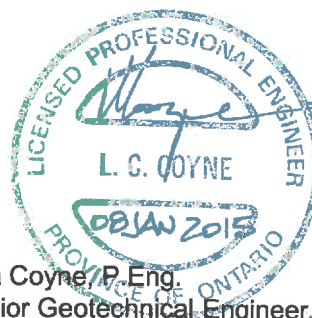
This Preliminary Foundation Design Report was prepared by Mr. Mehdi Mostakhdemi, M.Sc., P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Jorge Costa, P.Eng., a Principal of Golder and Designated MTO Foundations Contact, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.




m.m.

Mehdi Mostakhdemi, M.Sc., P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Senior Geotechnical Engineer, Principal



Jorge M.A. Costa, P.Eng.
Principal, Designated MTO Foundations Contact

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Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario



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**TABLE 1 – COMPARISON OF FOUNDATION OPTIONS
BLACK CREEK BRIDGES**

Foundation Option	Advantages	Disadvantages	Constructability/ Risk	Relative Costs
Spread footings founded on the very stiff to hard clayey silt till deposit (north abutments only) or on bedrock	<ul style="list-style-type: none">Allows for semi-integral abutments	<ul style="list-style-type: none">Significant excavations (to a depth of approximately 4.1 m to 8.5 m below QEW grade); temporary protection systems will be requiredGroundwater control will be required in the non-cohesive, water-bearing sand and gravel tillPrecludes use of integral abutments; potentially greater maintenance required at abutments	<ul style="list-style-type: none">Conventional excavation and construction techniques, but excavations will be quite deep relative to QEW grade and in creek valleyDeep excavations may impact the foundations of the Townline Road underpass in the vicinity of the new southbound bridgeMay be very difficult if they are required to be constructed in close proximity to the existing piers (with cruciform-shaped, sheetpiled foundations)Potential for differential settlements between abutments if south abutments are founded on bedrock	<ul style="list-style-type: none">Actual excavation is anticipated to be less expensive than deep foundations; however, additional costs will apply for temporary protection systems, dewatering and soil disposalStructure maintenance costs may be higher due to non-integral abutment configurationEstimated cost is about \$600/m³ for a concrete unit for construction of shallow foundations, excluding deeper excavation and temporary protection system



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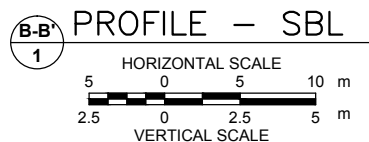
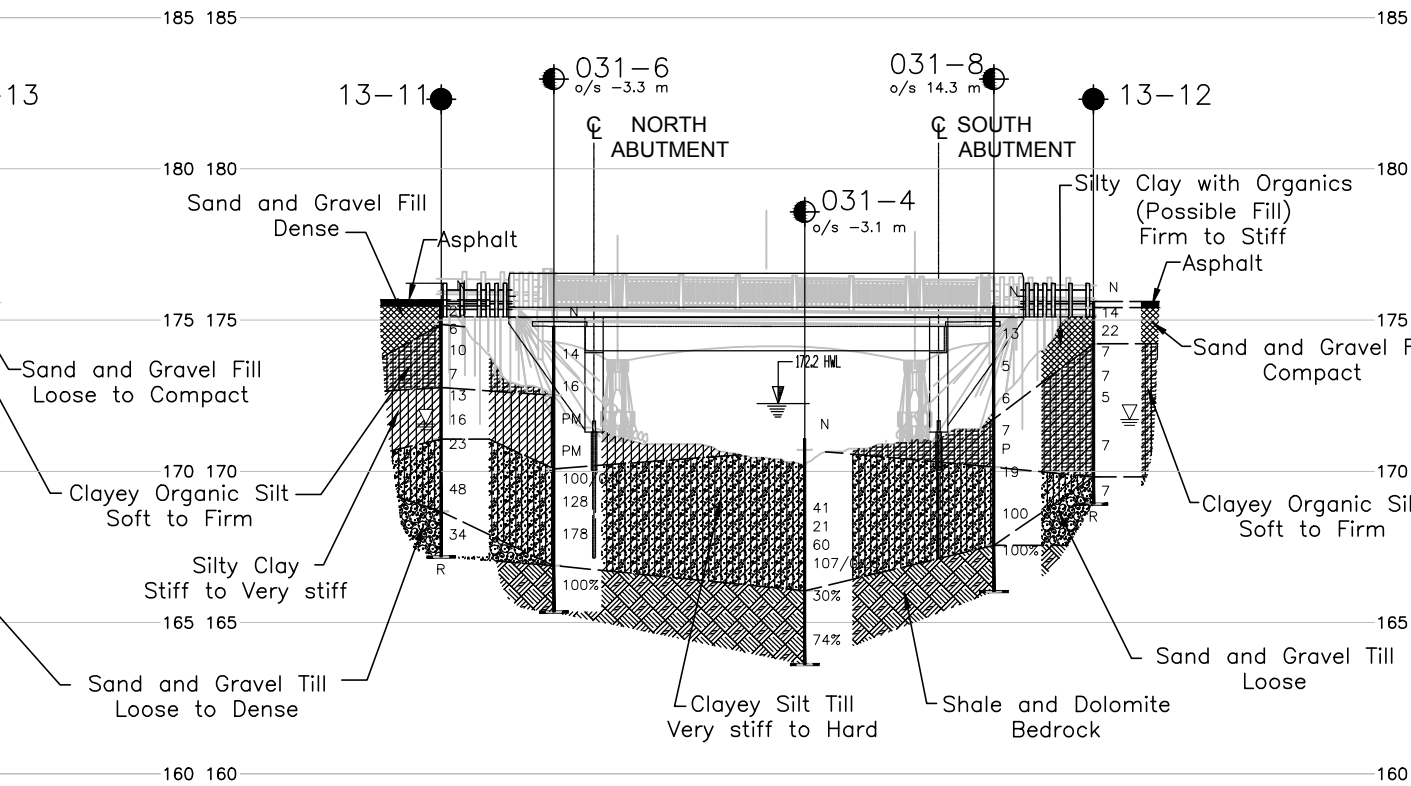
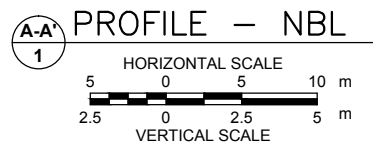
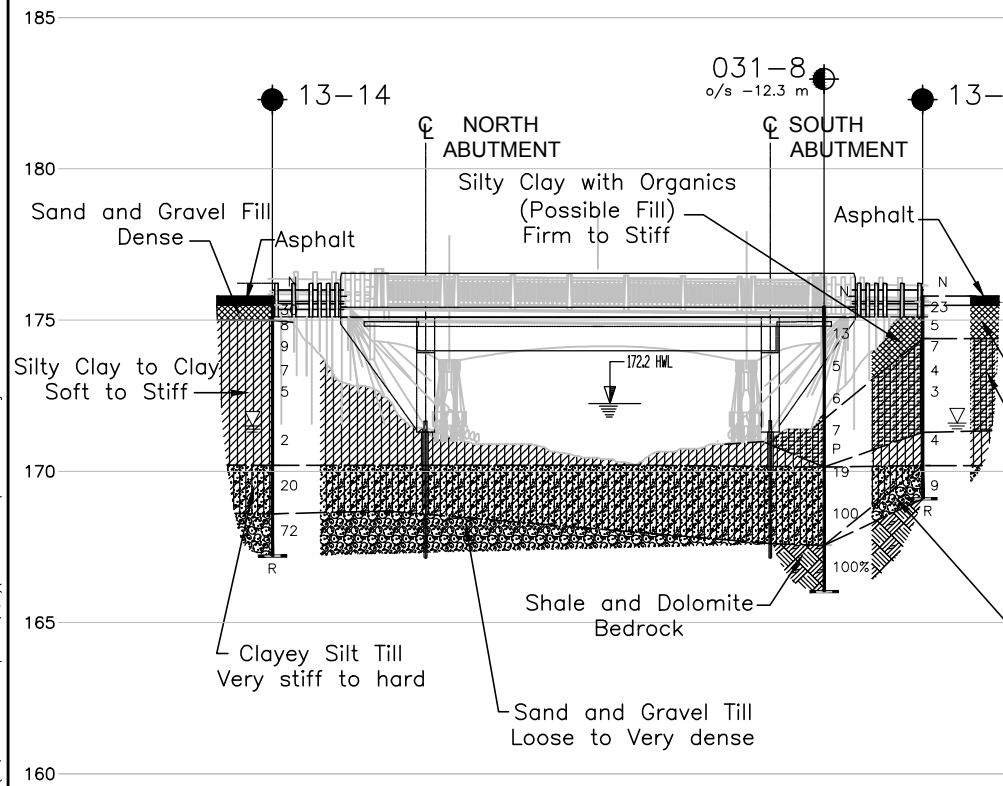
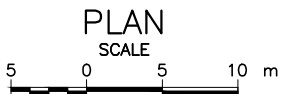
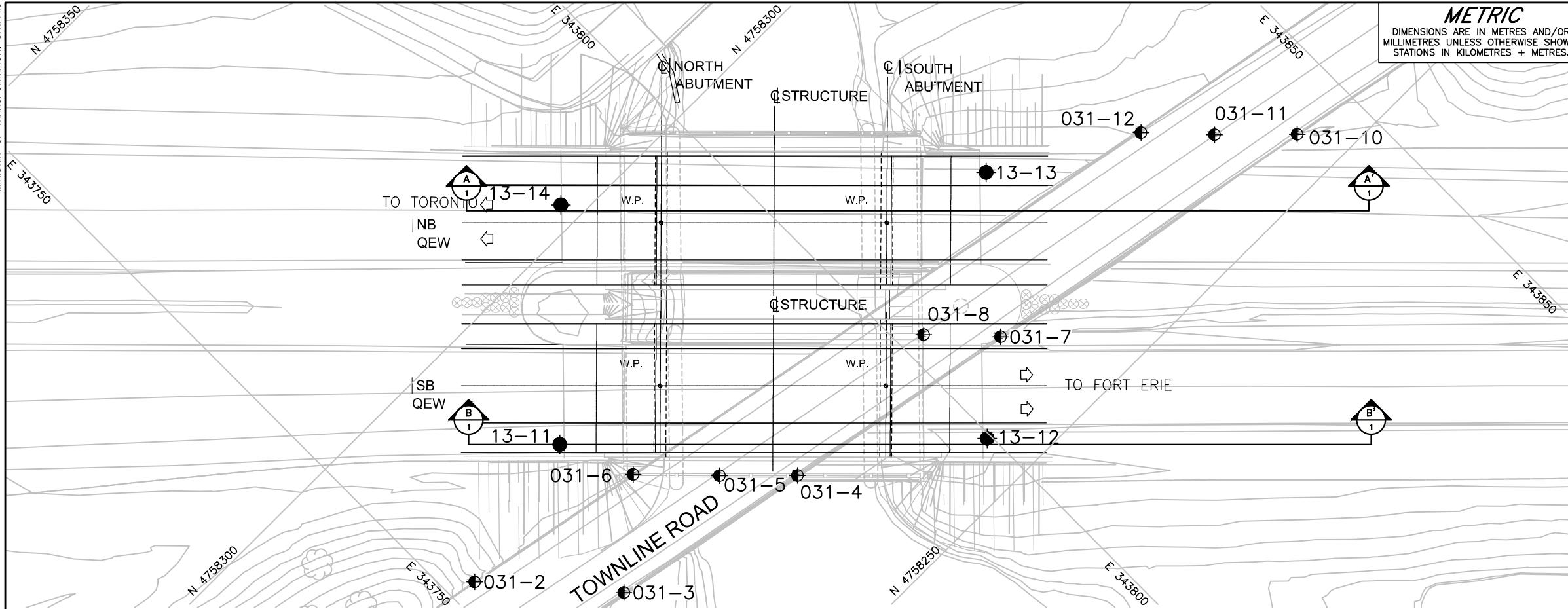
Foundation Option	Advantages	Disadvantages	Constructability	Estimated Costs
Spread/strip footings perched on compacted granular pad after excavation of loose/soft soils	<ul style="list-style-type: none"> Abutment footings could be maintained higher than footings founded on native soils and/or bedrock 	<ul style="list-style-type: none"> Precludes use of integral abutments; potentially greater maintenance required at abutments Proximity to the Creek and construction of granular pad (with front slope) will result in significant increase in span length 	<ul style="list-style-type: none"> Conventional excavation and construction techniques Relatively deep excavations to remove unsuitable soil may undermine/impact the foundations of the Townline Road underpass 	<ul style="list-style-type: none"> Anticipated to be less expensive than deep foundations, additional costs will apply for temporary protection systems, subexcavation, dewatering, and increased span length Structure maintenance costs may be higher due to non-integral abutment configuration Estimated cost is about \$600/m³ for a concrete unit for construction of shallow foundations, excluding deeper excavation and temporary protection system
Steel H-piles driven to found on bedrock	<ul style="list-style-type: none"> Pile caps could be maintained higher than spread footings, reducing depth of excavation and temporary protection system requirements adjacent to QEW Limited groundwater control required Allows for integral abutment construction 	<ul style="list-style-type: none"> Pre-augering or use of drilled steel casings to extend into the bedrock may be required if the pile caps cannot be constructed at an elevation high enough to provide a minimum pile length of 5 m for design of integral abutments 	<ul style="list-style-type: none"> 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, although drilled steel casings would be able to handle such obstructions Will require special equipment at the south abutment of the new southbound structure due to the limited headroom for pile driving beneath the Townline Road underpass 	<ul style="list-style-type: none"> Lower relative cost compared with caisson option Estimated cost is approximately \$250/m length for pile installation and \$600/m³ for pile cap construction, plus cost of any temporary protection systems; these costs will increase if coring/churn drilling into bedrock is required



PRELIMINARY FOUNDATION REPORT

QEW-BLACK CREEK BRIDGE REPLACEMENT, GWP 2177-08-00

Foundation Option	Advantages	Disadvantages	Constructability	Estimated Costs
Steel pipe (tube) piles, driven to found on bedrock	<ul style="list-style-type: none">• Pile caps could be maintained higher than spread footings, reducing depth of excavation and temporary protection system requirements adjacent to QEW• Limited groundwater control required	<ul style="list-style-type: none">• Not typically used for integral abutment design• Pre-augering or use of drilled steel casings to extend into the bedrock may be required if longer pile lengths are needed, or if a rock socket is required for pile toe fixity	<ul style="list-style-type: none">• 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, although drilled steel casings could be used instead of driving and these would be able to handle such obstructions• Will require specialized equipment at the south abutment of the southbound structure due to the limited headroom for pile driving beneath the Townline Road underpass	<ul style="list-style-type: none">• Costs for steel pipe (tube) piles similar to but slightly higher than those for H-piles; costs will be higher where coring into bedrock is required
Caissons founded in bedrock	<ul style="list-style-type: none">• Abutment pile caps could be constructed at the underside of the bridge structure, reducing depth of excavation and temporary excavation support requirements adjacent to QEW embankment• Higher capacity than piles, and will require less foundation elements	<ul style="list-style-type: none">• Temporary or permanent liners would be required due to risk of running/flowing soils in water-bearing sand and gravel till deposits• Coring and/or churn drilling techniques required to penetrate into the bedrock• Precludes use of integral abutments	<ul style="list-style-type: none">• Conventional construction methods with temporary liners required through water-bearing soil deposits• Moderate risk of encountering cobbles/boulders; also difficulties associated with larger diameter caissons in coring or churn drilling to penetrate into medium strong to strong portions of the bedrock• Will require specialized equipment at the south abutment of the southbound structure due to the limited headroom beneath the Townline Road underpass	<ul style="list-style-type: none">• Higher cost compared with shallow foundations or driven piles



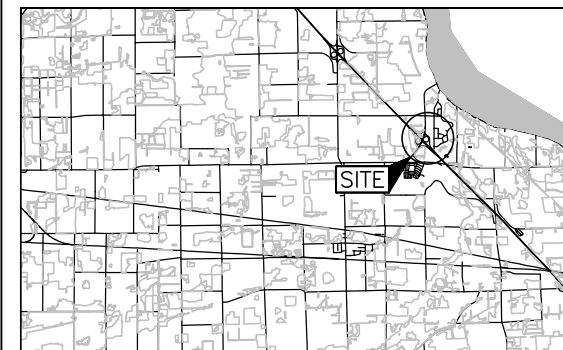
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 2177-08-00



BLACK CREEK BRIDGES
QUEEN ELIZABETH WAY
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole - Current Investigation
- ⊕ Borehole - (Geocres No. 30L14-031) - Location is approximate
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- R Refusal
- 0% Recovery of rock core
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
13-11	175.7	4758285.1	343768.6
13-12	175.6	4758255.1	343799.6
13-13	175.8	4758274.2	343818.4
13-14	175.8	4758302.1	343785.7

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 an GA and Profile file No. Draft_Black_Creek_GA.dwg, received October 9, 2013.

NO.	DATE	BY	REVISION

Geocres No. 30L14-57

HWY. QEW	PROJECT NO. 12-1111-0088	DIST. CENTRAL
SUBM'D. MM	CHKD. LCC	DATE: 12/22/2014
DRAWN: JFC	CHKD. LCC	APPD. JMAC
		DWG. 1





APPENDIX A

Records of Boreholes



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p) / I_p$
I_C	consistency index = $(w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-11		SHEET 1 OF 1		METRIC											
G.W.P. 2177-08-00		LOCATION N 4758285.1 ; E 343768.6		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 25, 2013		CHECKED BY MM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L 20 40 60				
175.7	GROUND SURFACE																
0.0	ASPHALT (250 mm)																
0.2	Sand and gravel, trace silt, trace clay (FILL)		1	SS	26												
174.9	Compact Grey Moist		2	SS	6												
0.8	CLAYEY ORGANIC SILT, some silt, trace sand		3	SS	10												
	Firm to stiff		4	SS	7												
	Dark grey to black																
	Wet																
172.8	SILTY CLAY, trace sand		5	SS	13												
2.9	Stiff to very stiff		6	SS	16												
	Brown and grey																
	Wet																
171.1	Sandy CLAYEY SILT, some gravel (TILL)		7	SS	23												
4.6	Very stiff to hard																
	Brown		8	SS	48												
	Wet																
168.7	SAND and GRAVEL, some silt, trace clay (TILL)		9	SS	34												
7.0	Dense Grey Wet																
167.2	END OF BOREHOLE AUGER REFUSAL																
8.5	NOTE: 1. Water level in open borehole at a depth of 4.1 m below ground surface (Elev. 171.6 m) upon completion of drilling.																

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-12		SHEET 1 OF 1		METRIC																										
G.W.P. 2177-08-00		LOCATION N 4758255.1 ; E 343799.6		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 25, 2013		CHECKED BY MM																												
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)																	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					WATER CONTENT (%) W _p W W _L			γ			GR SA SI CL													
175.6	GROUND SURFACE							20 40 60 80 100																								
0.0	ASPHALT (250 mm)																															
0.2	Sand and gravel, trace to some silt, trace clay (FILL) Compact Grey Moist		1	SS	14		175																									
			2	SS	22																											
174.2							174																									
1.4	CLAYEY ORGANIC SILT, some silt, trace sand, containing sand pockets Firm to stiff Dark grey to black Wet		3	SS	7																											
			4	SS	7		173																									
			5	SS	5		172																									
							171																									
			6	SS	7																											
169.8							170																									
5.8	SAND and GRAVEL, some silt, some clay (TILL) Loose Grey Wet		7	SS	7		169																									
168.9																																
6.7	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Water level in open borehole at a depth of 3.9 m below ground surface (Elev. 171.7 m) upon completion of drilling.																															

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-13		SHEET 1 OF 1		METRIC											
G.W.P. 2177-08-00		LOCATION N 4758274.2; E 343818.4		ORIGINATED BY SB													
DIST Central HWY QEW		BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers		COMPILED BY AV													
DATUM Geodetic		DATE June 25, 2013		CHECKED BY MM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L WATER CONTENT (%)			γ	GR SA SI CL
175.8	GROUND SURFACE							20 40 60 80 100									
0.0	ASPHALT (250 mm)																
0.2	Sand and gravel, trace silt, trace clay (FILL) Loose to compact Grey Moist		1	SS	23		175										
			2	SS	5												
174.4																	
1.4	CLAYEY ORGANIC SILT, some silt, trace sand, containing rootlets Soft to firm Dark grey to black Wet		3	SS	7		174										
			4	SS	4		173										
			5	SS	3		172										
171.3	SILTY CLAY, some sand, trace to some gravel Firm Grey Moist to wet		6	SS	4		171										6 20 39 35
170.2																	
5.6	SAND and GRAVEL, trace silt, trace clay (TILL) Loose Grey Wet		7	SS	9		170										58 32 5 5
169.2																	
6.6	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Water level inside auger at a depth of 4.2 m below ground surface (Elev. 171.6 m) upon completion of drilling. 2. Split spoon refusal at a depth of 6.6 m below ground surface (Elev. 169.2 m).																

PROJECT 12-1111-0088		RECORD OF BOREHOLE No 13-14		SHEET 1 OF 1		METRIC											
G.W.P. 2177-08-00		LOCATION N 4758302.1 ; E 343785.7		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 25, 2013		CHECKED BY MM													
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	20 40 60	kN/m ³			
175.8	GROUND SURFACE																
0.0	ASPHALT (300 mm)																
175.5																	
0.3	Sand and gravel, trace silt, trace clay (FILL)		1	SS	30												
175.0	Compact Brown Moist		2	SS	8												
0.8	SILTY CLAY to CLAY, trace sand, trace organics		3	SS	9												
	Soft to stiff		4	SS	7												
	Brown becoming grey below a depth of 1.4 m		5	SS	5												
	Wet		6	SS	2												
			7	SS	20												
170.2	Sandy CLAYEY SILT, some gravel (TILL)		8	SS	72												
5.6	Very stiff Brown Wet																
168.6	SAND and GRAVEL, some silt, trace to some clay (TILL)																
7.2	Very dense Grey Wet																
167.2	END OF BOREHOLE AUGER REFUSAL																
8.6	NOTE: 1. Water level in open borehole at a depth of 4.3 m below ground surface (Elev. 171.5 m) upon completion of drilling.																



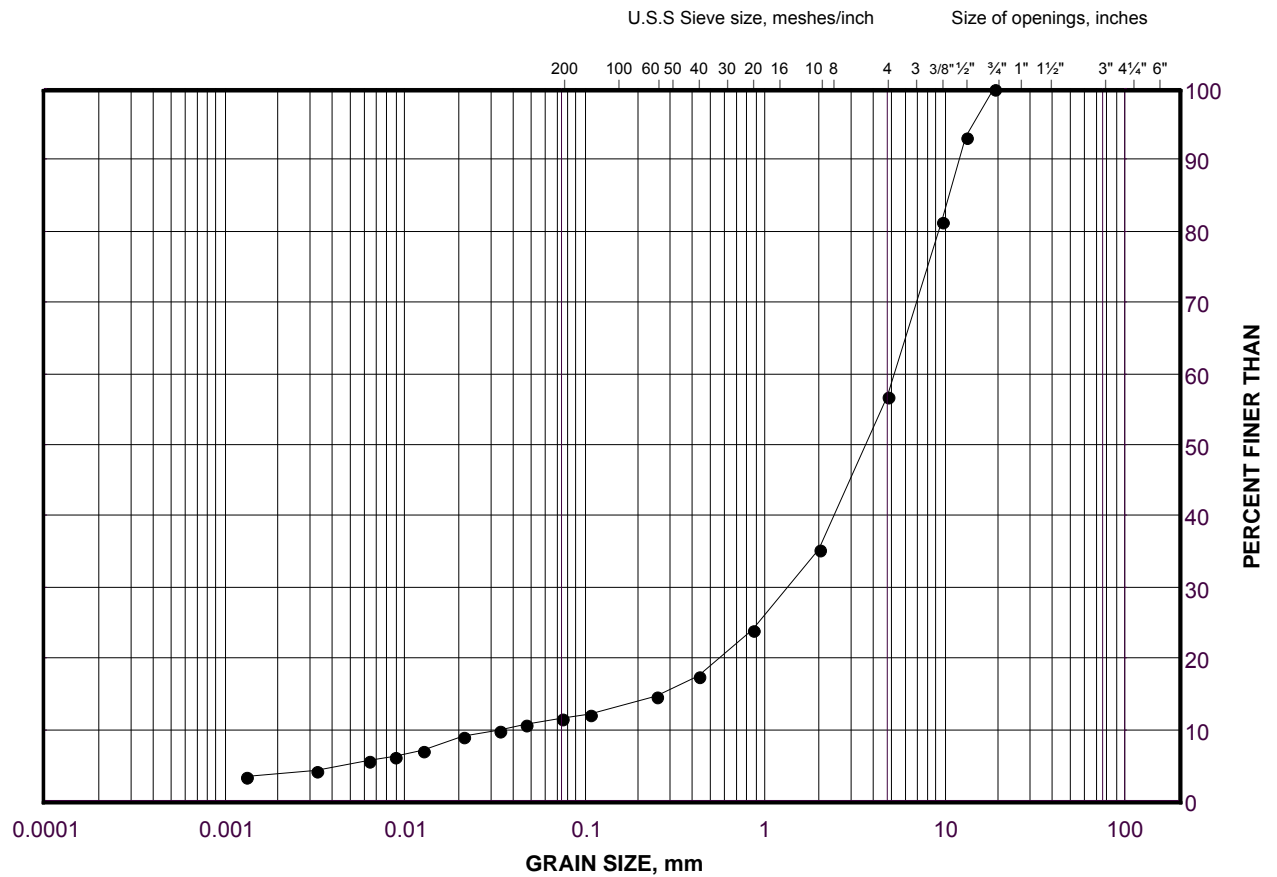
APPENDIX B

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

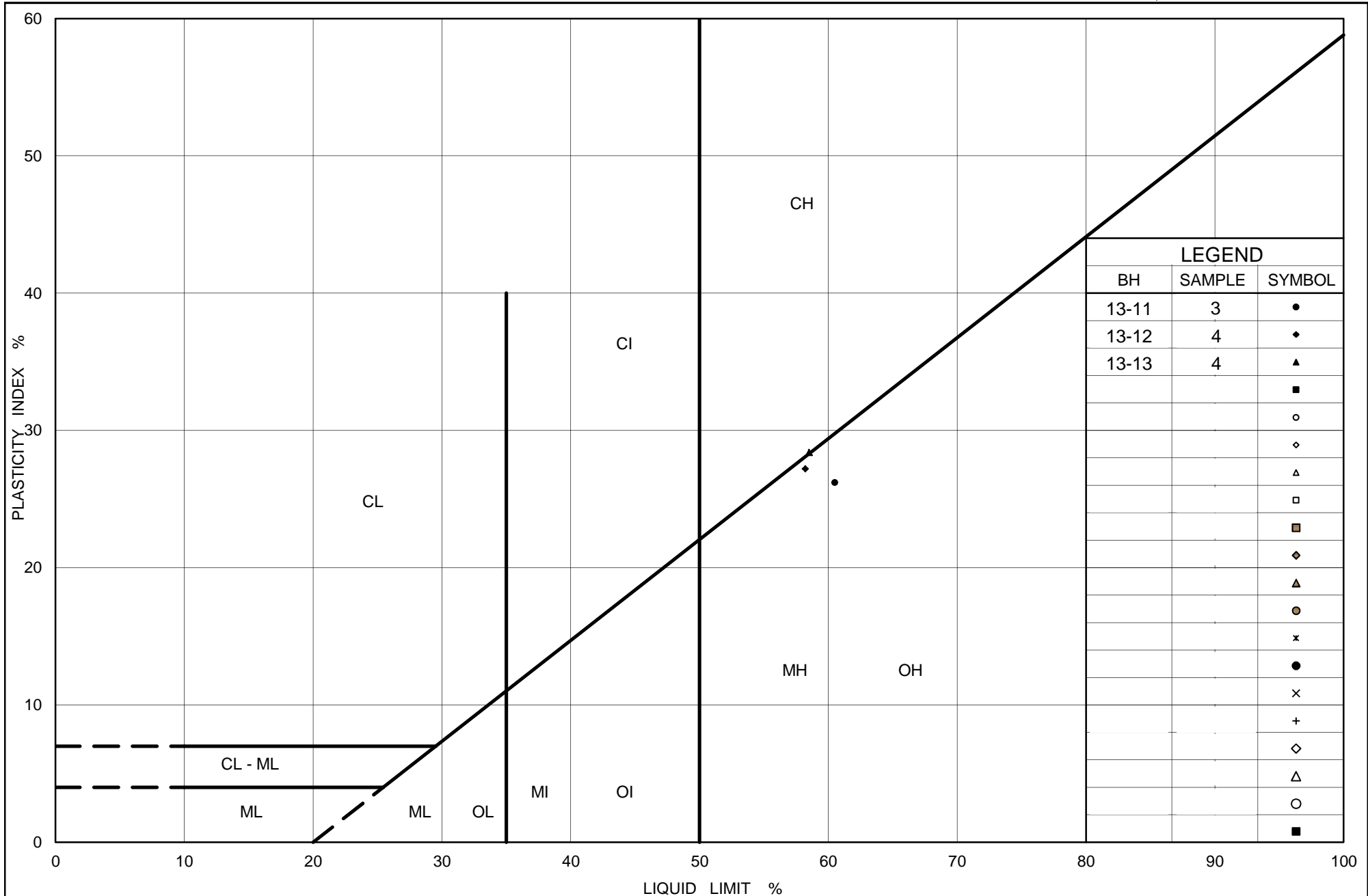
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	13-12	1	175.1

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
Transportation

Ontario

PLASTICITY CHART CLAYEY ORGANIC SILT

Figure No. B2

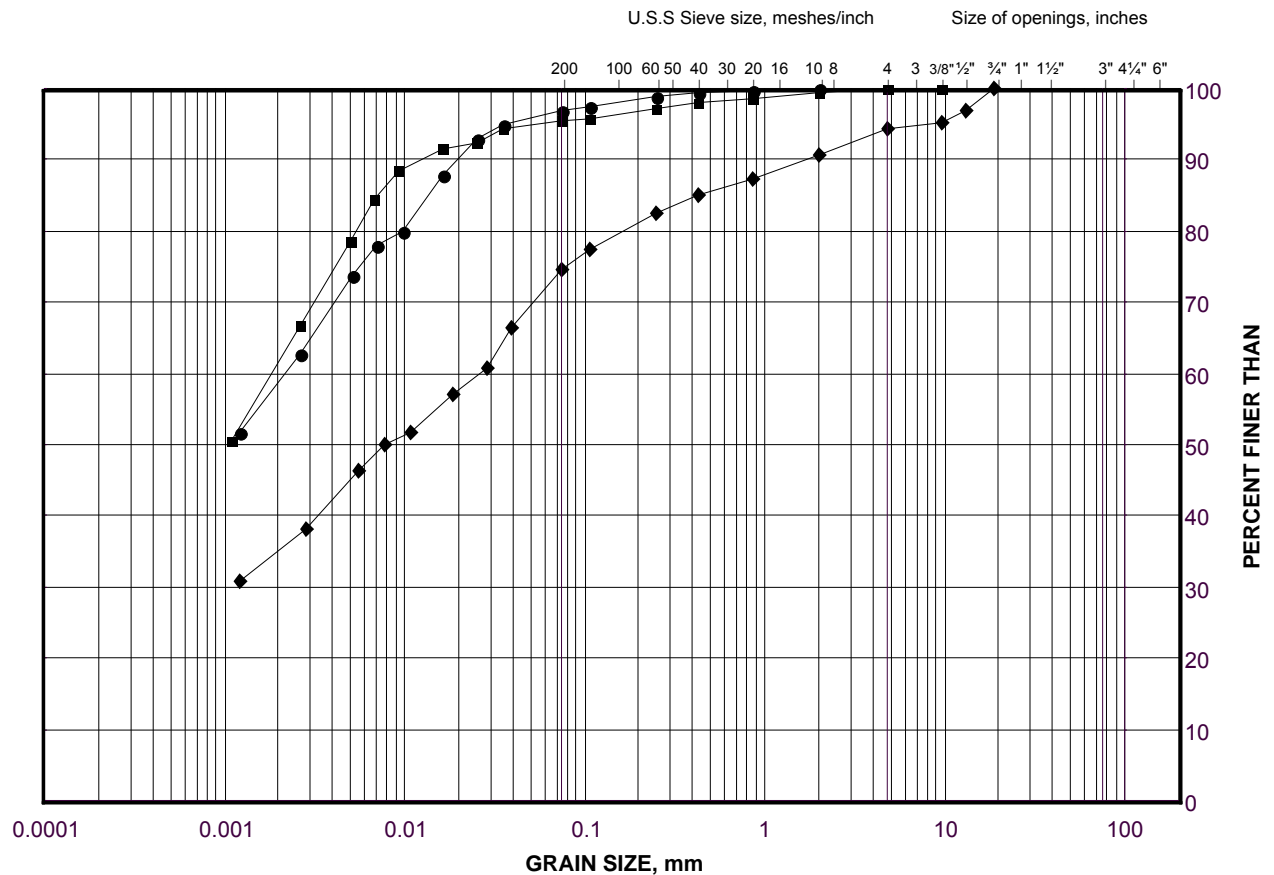
Project No. 12-1111-0088

Checked By: MM

GRAIN SIZE DISTRIBUTION

SILTY CLAY to CLAY

FIGURE B3



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

LEGEND

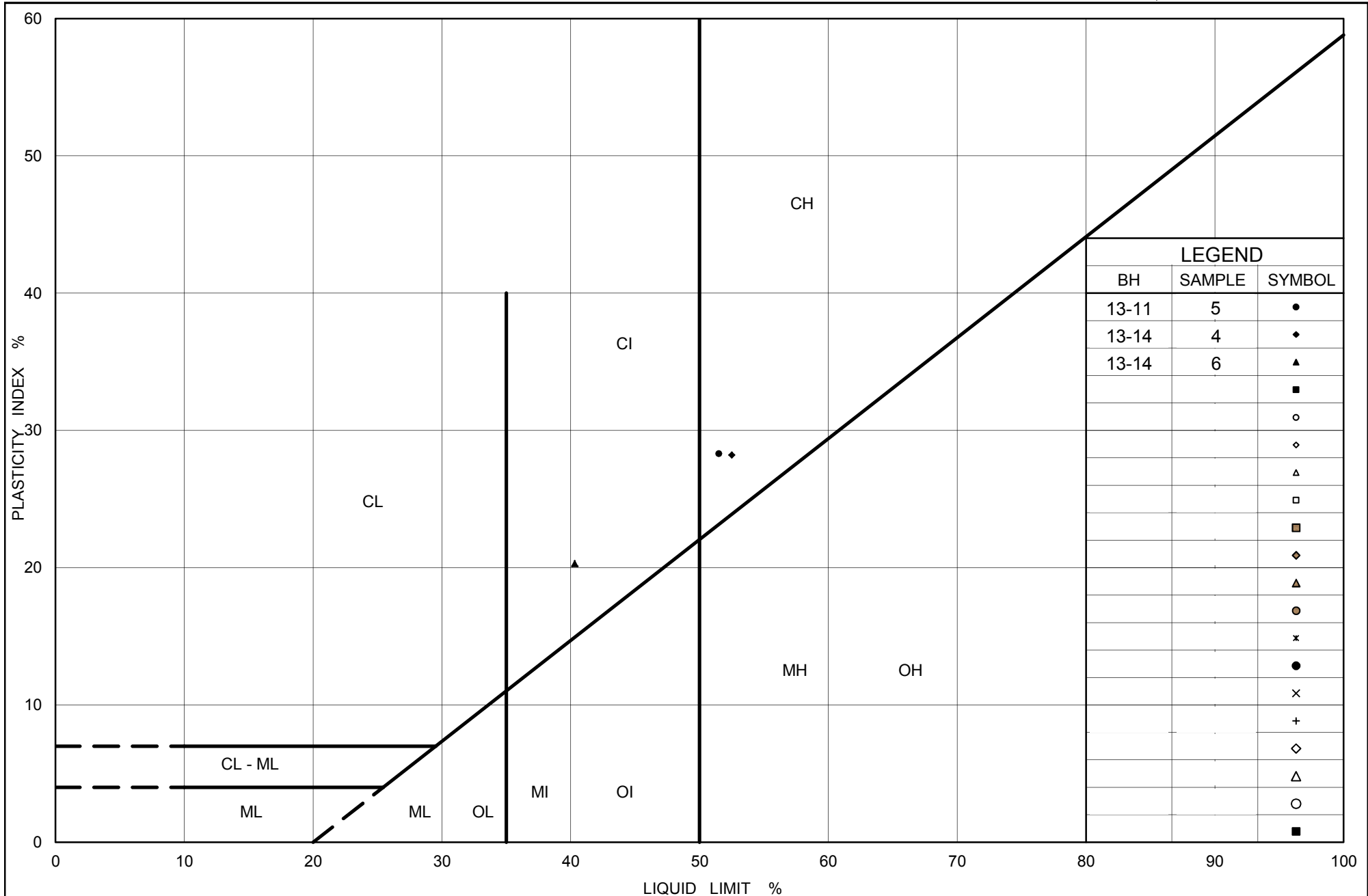
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-14	4	173.2
■	13-11	5	172.3
◆	13-13	6	170.9

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



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Ontario

PLASTICITY CHART SILTY CLAY to CLAY

Figure No. B4

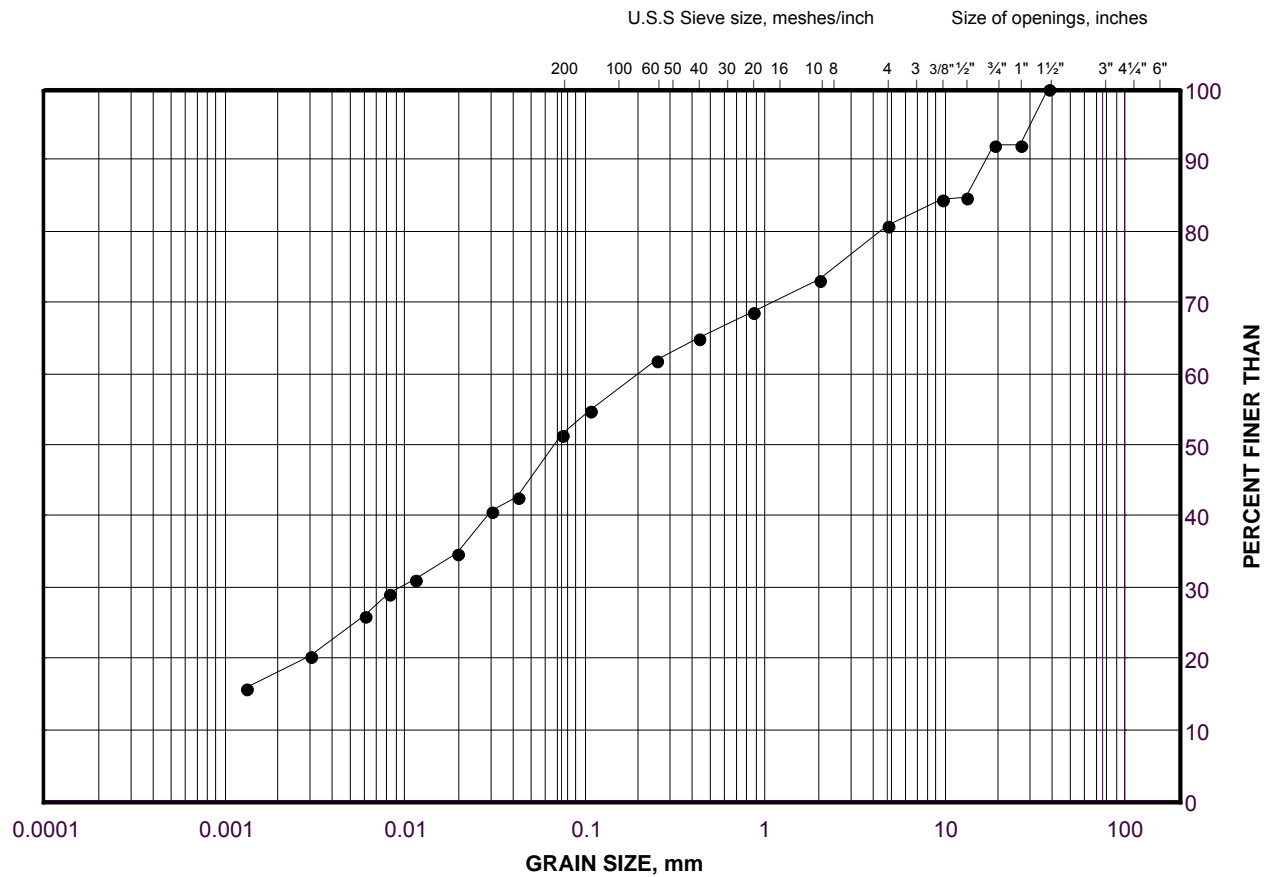
Project No. 12-1111-0088

Checked By: MM

GRAIN SIZE DISTRIBUTION

Sandy CLAYEY SILT Till

FIGURE B5



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

LEGEND

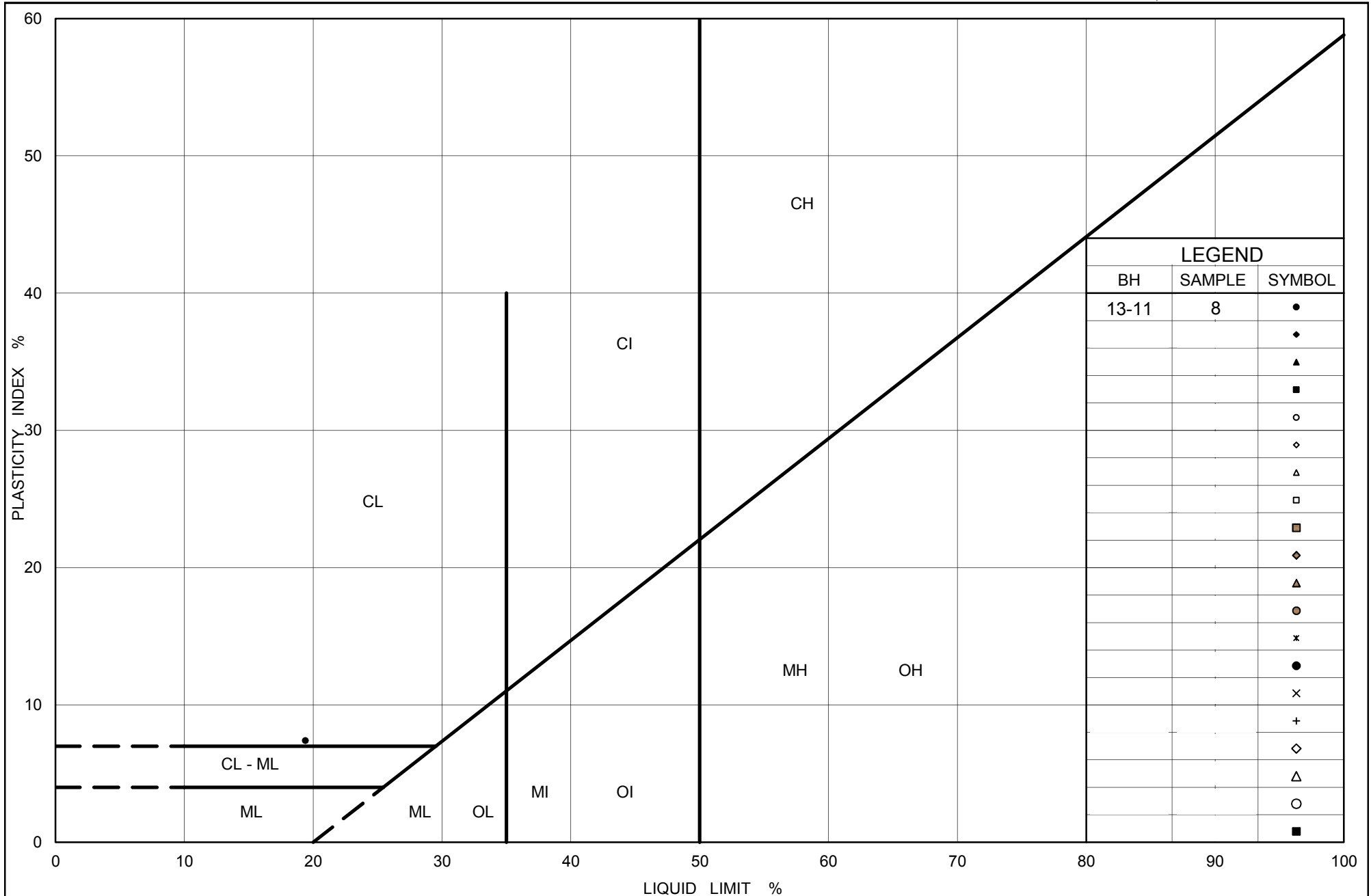
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	13-11	8	169.3

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
Transportation

Ontario

PLASTICITY CHART CLAYEY SILT TILL

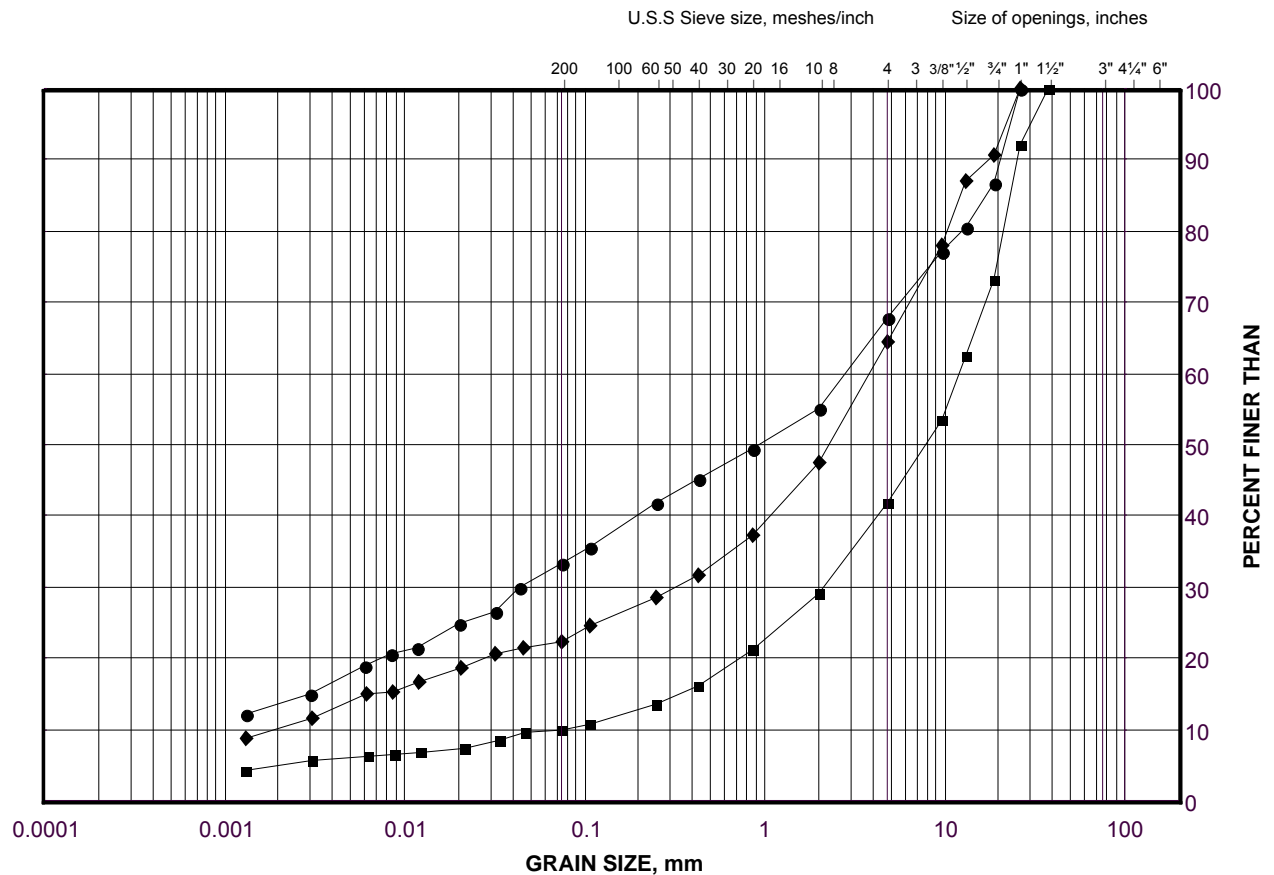
Figure No. B6

Project No. 12-1111-0088

Checked By: MM

SÁND and GRAVEL Till

FIGURE B7



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

LEGEND

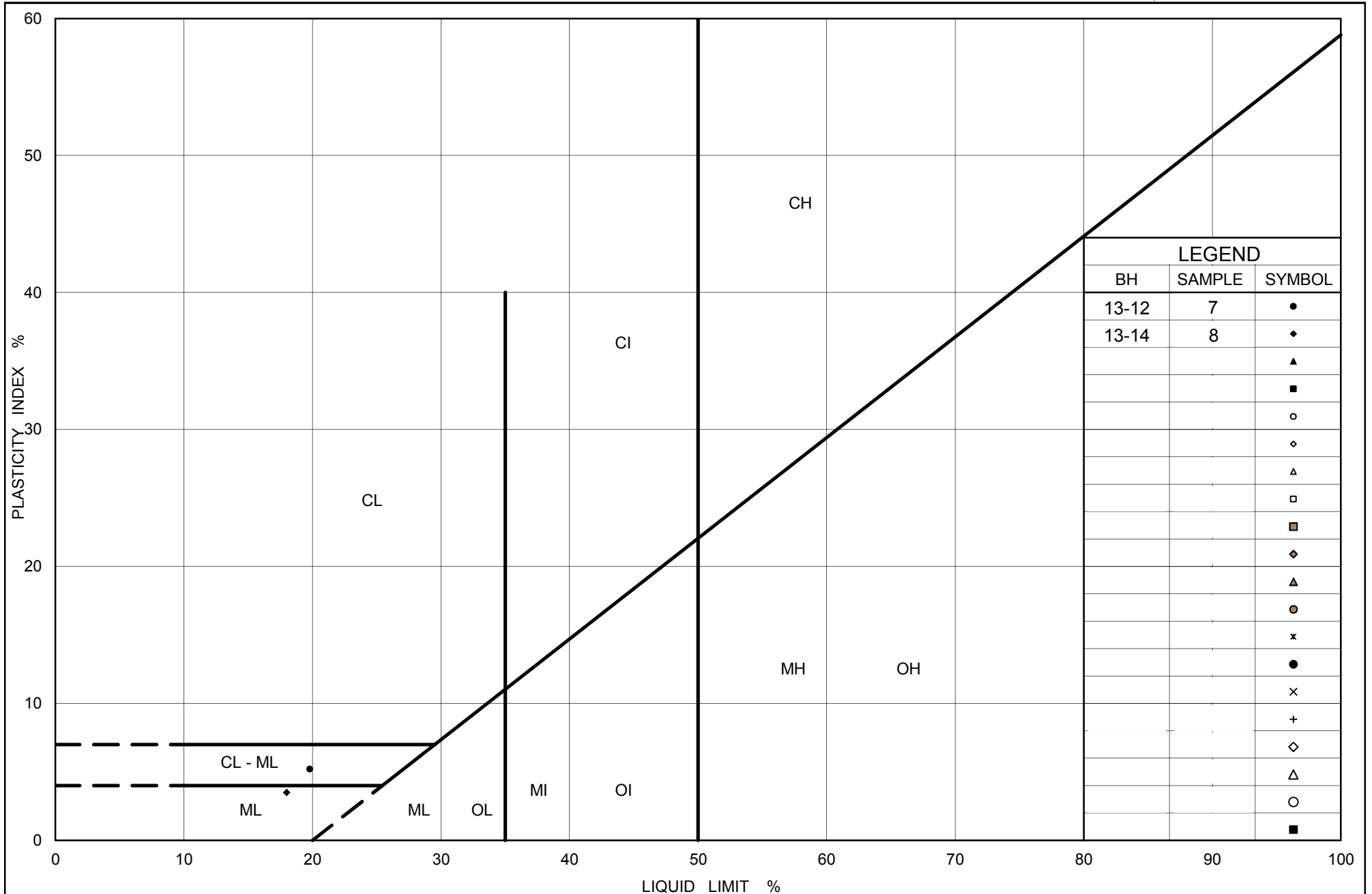
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-12	7	169.2
■	13-13	7	169.4
◆	13-14	8	167.9

Project Number: 12-1111-0088

Checked By: MM

Golder Associates

Date: 08-Oct-13



Ministry of
Transportation

Ontario

PLASTICITY CHART

Sand and Gravel Till

Figure No. B8

Project No. 12-1111-0088

Checked By: MM



APPENDIX C

**Records of Boreholes from 1968 Investigation, Department of
Highways Ontario (GEOCRES No. 30L14-031)**

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 2

FOUNDATION SECTION

JOB 68-F-31

LOCATION Sta. 32 + 24 @ Prop. Revn. Townline Rd. o/s 18' Rt.

VE

W.D. 167-64-01

BORING DATE May 1, 1968

ORIGINATED BY

CM

DATUM Geodetic

BOREHOLE TYPE Diamond Drill NX Casing

CHECKED BY

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					PLASTIC LIMIT			WATER CONTENT %	DENSITY	REMARKS
DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT		20	40	60	80	100	15	30	45			
567.5	Ground Level															
0.0	Silty clay with trace sand & gravel.	1	TW	PM											126	
	Stiff to very stiff															
560.5	Reddish Brown	2	TW	PM	560										126	
7.0	Clayey silt with sand & gravel (glacial till)	3	SS	25												
	occ. sand seams	4	SS	33												
552.5	Very stiff to hard.															
15.0	with Boulders	5	AXT	5%	550											
547.5				RC Rec												
20.0	Interbedded shale & dolomite bedrock with	6	AXT	100%												
543.0	gypsum inclusions.			RC Rec												
24.5	End of Borehole				540											

0
15 5 1/2 strain at failure
10

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO 3

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 68-F-31

LOCATION Sta. 31 + 78 @ Prop. Revn. Townline Rd. o/s 16' Lt.

ORIGINATED BY VK

167-64-01

BORING DATE May 2, 1968

COMPILED BY CM

DATUM Geodetic

BOREHOLE TYPE Diamond Drill - NX Casing

CHECKED BY

SOIL PROFILE		SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE					WATER CONTENT %	PLASTIC LIMIT %	REMARKS
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE		20	40	60	80	100			
566.9	Ground Level				500	1000	1500	2000	2500	15	30	45
0.0	Silty clay with trace sand & gravel. occ. gypsum pockets. Stiff to very stiff.	1	SS	16								
559.9	Reddish Brown	2	TW	P	560							128
7.0	Clayey silt with some sand & gravel (gl. till)	3	SS	2L								
	Very stiff to hard.	4	SS	100/5"								
		5	SS	187	550							
546.9	20.0 Dolomite bedrock with gypsum inclusions.	6	AXT RC	100% Rec								
541.9												
25.0	End of Borehole			540								

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 4

FOUNDATION SECTION

JOB 68-F-31

LOCATION Sta. 31+08 @ Prop. Revn. Townline Rd. o/s 16' Lt.

ORIGINATED BY V PBS

W P 167-64-01

BORING DATE April 15, 1968

COMPILED BY TC

DATUM Geodetic (raft)

BOREHOLE TYPE Diamond Drill BX Casing

CHECKED BY

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		LIQUID LIMIT		PLASTIC LIMIT		WATER CONTENT		REMARKS
ELEV	DEPTH	NUMBER	TYPE	BLOWS / FT	ELEV SCALE	20	40	60	80	100	WATER CONTENT %	
561.3	Water Level											Gr. Sa. Si. Cl
0.0					560							
557.3	Creek Bottom											
4.0	Clayey silt with sand and gravel (glacial till)	1	SS	41								
	Very stiff to hard.	2	SS	21	550							
	Reddish Brown to Grey	3	SS	60								21 40 (39)
544.9		4	SS	107/10 1/2								
16.5	Interbedded shale & dolomite bedrock. Occ. seams of gypsum.	5	RC	30% Rec	540							
536.8		6	RC	74% Rec								
21.5	End of Borehole											
					530							

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 5

FOUNDATION SECTION

JOB 68-F-31

LOCATION Sta. 31+30 @ Prop. Revn. Townline Rd.

ORIGINATED BY WK

W.P. 167-64-01

BORING DATE May 31, 1968

COMPILED BY CM

DATUM Geodetic

BOREHOLE TYPE Diamond Drill NX Casing

CHECKED BY

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT — W PLASTIC LIMIT — WP WATER CONTENT — W			REMARKS
ELEV DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		20	40	60	80	100	WP	PL	WC	
572.5	Ground Level					500	1000	1500	2000	2500	15	30	45	
0.0	Silty clay to clay with organic inclusions Stiff.		1	SS	13									5.5% org.
565.0	(Fill Material)		2	SS	14									
7.5	Silty clay with trace sand, occ. gravel. Very stiff		3	2"TW	PM									128
557.5	Reddish Brown		4	2"TW	PM									124
15.0	Clayey silt with some sand & gravel (Glacial Till) occ. boulders		5	SS	68									
			6	SS	177/11"									
			7	SS	176									
547.2	Hard		8	SS	100/4"									
545.2	Interbedded shale & dolomite bedrock		9	BC	50% Rec									
27.3	End of Borehole													

15 — 5 % Strain at failure

10

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 6

FOUNDATION SECTION

JOB 68-F-31 LOCATION Sta. 31+54 @ Prop Revn. Townline Rd. o/s 18' Rt. ORIGINATED BY VK
 P 167-64-01 BORING DATE May 3, 1968 SAMPLED BY CM
 DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX Casing CHECKED BY

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					PLASTIC LIMIT (LL) & WATER CONTENT (W)			REMARKS				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		20	40	60	80	100	500	1000	1500		2000	2500	15	30
573.5	Ground Level																	
0.0	Silty clay to clay with organic inclusions (Fill Material)		1	SS	14													
566.0	Stiff		2	SS	16													
7.5	Silty clay with trace sand & gravel. Very stiff.		3	TW	P													
			4	TW	P													
558.0	Reddish Brown		5	SS	100/4"													
15.5	Clayey silt with sand & gravel (Glacial Till) occ. gypsum pockets.		6	SS	128													
	Hard or Very dense.																	
547.5			7	SS	178													
26.0	Interbedded shale & dolomite bedrock with gypsum inclusions.		8	AXT	100% RC Rec													
542.5																		
31.0	End of Borehole																	

3610

132

125

0

15 5 10 % Strain at failure

0
15 5 % Strain at failure
10

1. NAME OF SECTION

MATERIALS & TESTING DIVISION

LOCATION Sta. 29+78 @ Prop.Revn. Townline Rd. o/s 16' Lt.

OR DONATED BY VX

FORING DATE May 7, 1968

REF ID: A61111

BORE HOLE TYPE Diamond Drill NX Casing

CHECKED BY

SOIL PROFILE		SAMPLES		ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LONG LIMIT --- WL PLASTIC LIMIT --- PL WATER CONTENT --- W	DENSITY	REMARKS
DEPTH	DESCRIPTION	NUMBER	TYPE		20 40 60 80 100			
575.2	Ground Level				500 1000 1500 2000 2500	WATER CONTENT % 15 30 45		Gr.Sa.St.Cl
575.2	Silty clay to clay with organic inclusions. Firm to stiff. (Fill material)	1	SS 12	570	x s=11		120	
		2	TW P		+ s=4		115	564.7 V
		3	TW P				116	
		4	TW P		+ s=5		113	
559.2		5	TW P	560	s=7 s=4			
17.0	Clayey silt with sand & gravel (glacial till).	6	SS 12					11 37 39 13
551.9	Very stiff to hard.	7	SS 100%					
23.3	Interbedded shale & dolomite bedrock with gypsiferous seams.	8	AXT 100% RC Rec	550				
548.2								
27.0	End of Borehole			540	15 0 5 % strain at failure 10			

1. CONDUCTOR'S SECTION

COMPLETED BY VK

BOOKING DATE May 6, 1968

100-443887-100

BOREHOLE TYPE Diamond Drill - NX Casing

ACKNOWLEDGMENTS

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	PLASTIC LIMIT WATER CONTENT	Gr. Sa. Si. Cl.
ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE				
575.7	Ground Level				20 40 60 80 100		
0.0	Silty clay with organic inclusions. Firm to stiff.	1	SS	13			
		2	SS	5			
		3	SS	6			
	(Fill material)	4	SS	7			
		5	TW	P			
558.2							
17.5	Clayey silt with sand & gravel. (Glacial Fill)	6	SS	12			
		7	SS	100			
549.7	Very stiff to hard.						
26.0	Interbedded shale & dolomite bedrock with gypsiferous seams.	8	AXT RC	100 Rec.			
544.7							
31.0	End of Borehole						

IN REPLY TO

JOB	68-F-31	LOCATION	Sta. 23+48 Prop. Rev. Townline Rd. o/s 16' Lt.	ORIGINATED BY	VK
W P	167-4-01	BORING DATE	May 8, 1968	COMPLETED BY	VK
EQUIP	Geodetic	BORE HOLE TYPE	Diamond Drill - NX Casing	CHECKED BY	

SOIL PROFILE		SAMPLES		DYNAMIC PENETRATION RESISTANCE		WATER CONTENT		REMARKS
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE	WATER CONTENT %	WATER CONTENT %	
572.3	Ground Level							
0.2	Silty clay with organic inclusions. Very stiff.	1	SS	18	570			
565.8	(Fill material)	2	SS	28				562.1
6.5	Clayey silt with sand & gravel. (Glacial Till)	3	SS	29				10 27 47-56
	Hard.	4	SS	37	560			7 32 45 16
553.3		5	SS	100/50				
19.0	Interbedded shale & dolomite bedrock with gypsum inclusions.	6	AXT RC	100% Ret.				
549.8								
22.5	End of Borehole							

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

68-E-31

W F 167-64-01

DATUM Geodetic

RECORD OF BOREHOLE NO. 12

LOCATION Sta. 28+94 @ Prop. Rev. Townline Rd. o/s 18th Rt.

BORING DATE May 9, 1968

BOREHOLE TYPE Diamond Drill - NX Casing

FOUNDATION SECTION

ORIGINATED BY VR

COMPILED BY _____ VK

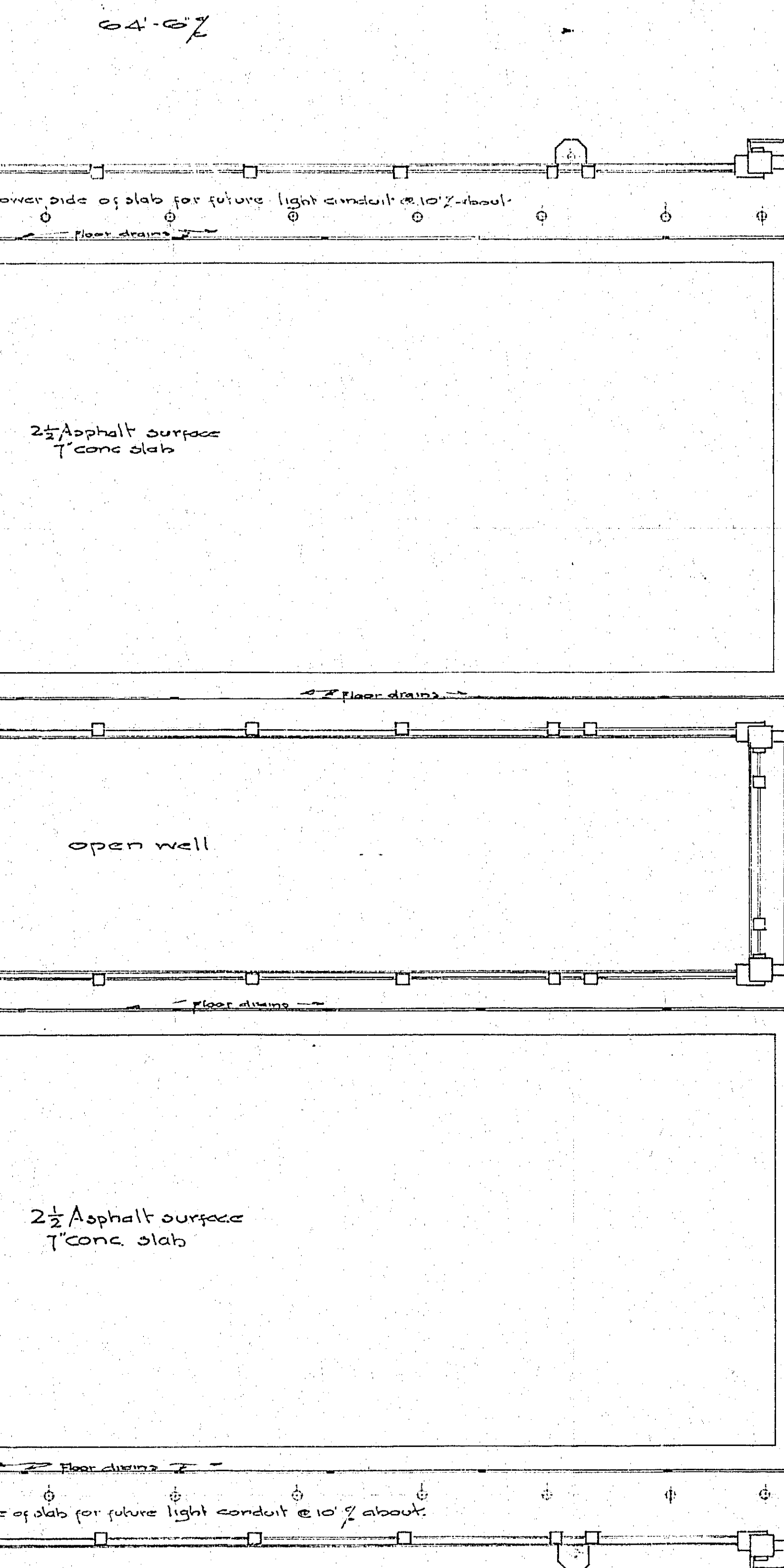
CHECKED BY

SOIL PROFILE		SAMPLES			ELEV SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— % PLASTIC LIMIT ——— % WATER CONTENT ——— %	BULK DENSITY PCF	REMARKS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		BLOWS / FOOT	SHEAR STRENGTH P.S.F. x Lab Vane o Unconfined		
571.6	Ground Level					20 40 60 80 100 500 1000 1500 2000 2500	15 30 45		
0.0	Silty clay with trace of sand & organics. Firm to very stiff. (Fill material)	X	1	TW	P	x=1 o		119	
564.1		X	2	SS	24				
7.5	Clayey silt with some sand & gravel. (Glacial till) Very stiff to hard.	O	3	SS	22				
		O	4	SS	91				
		O	5	SS	114				
553.1									
18.5	Dolomite Bedrock with gypsiferous seams.		6	AXT RC	60% Rec				
548.1									
23.5	End of Borehole					0 15 — 5 % strain at failure 10			

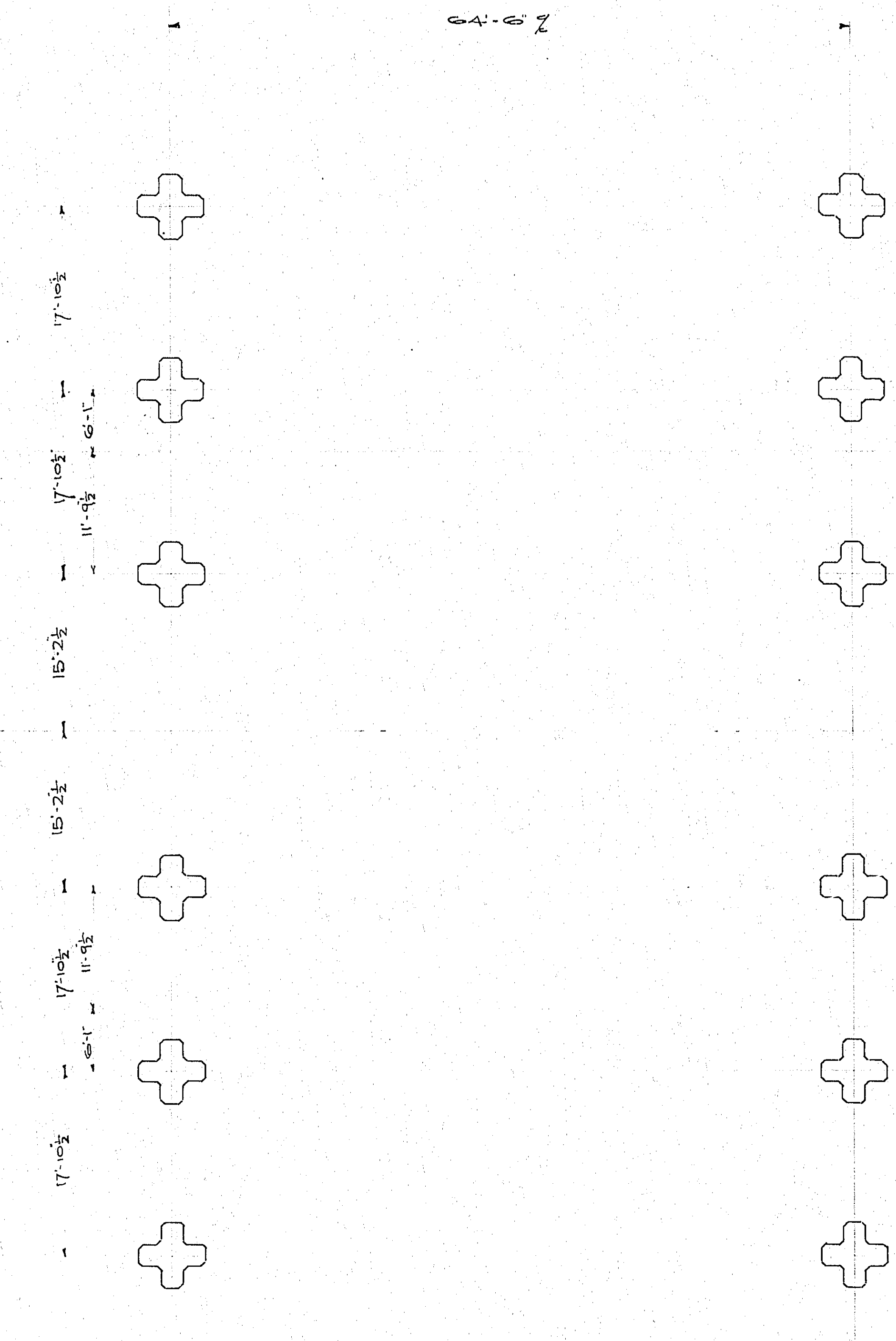


APPENDIX D

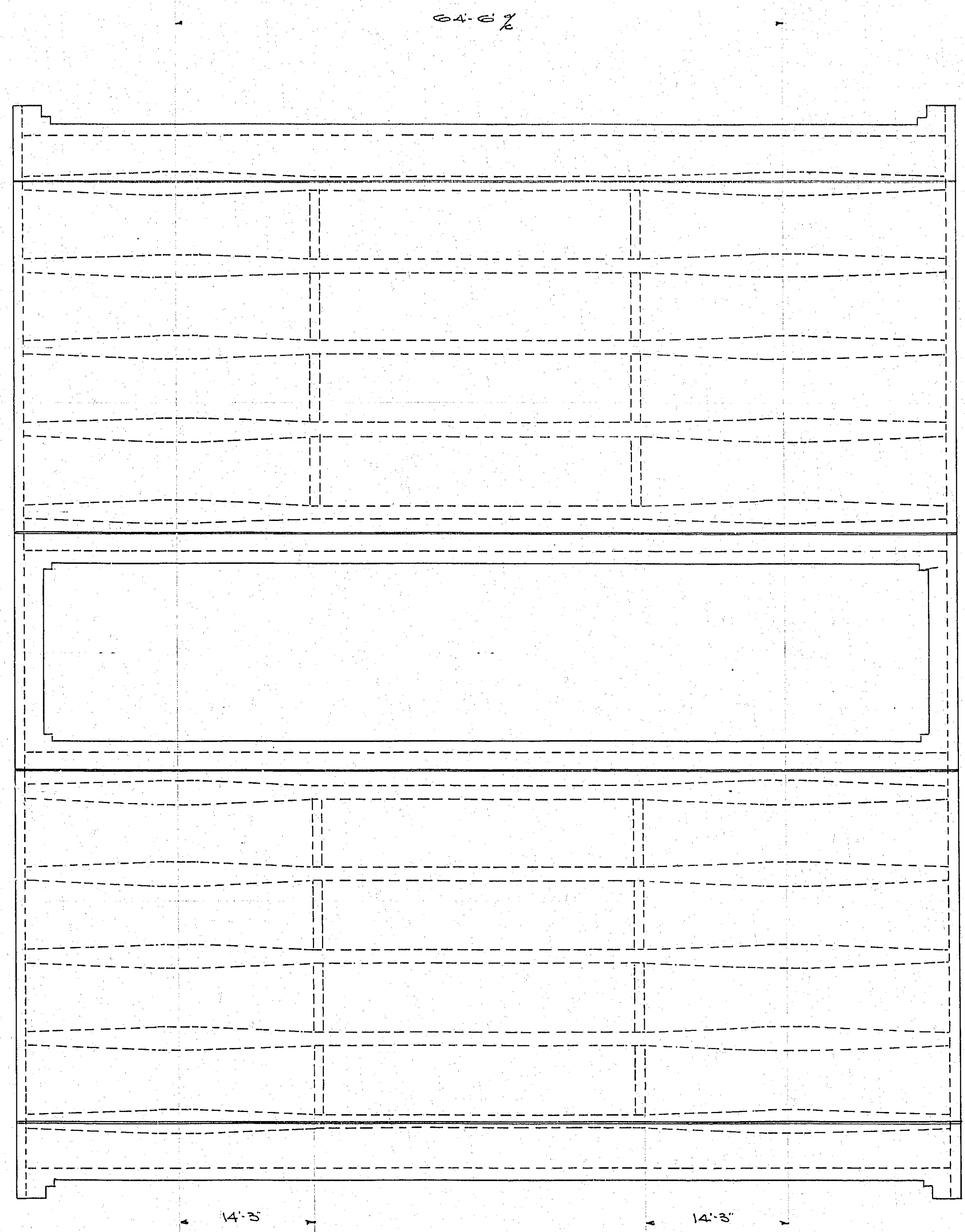
Existing Foundation Design Drawings



DECK PLAN

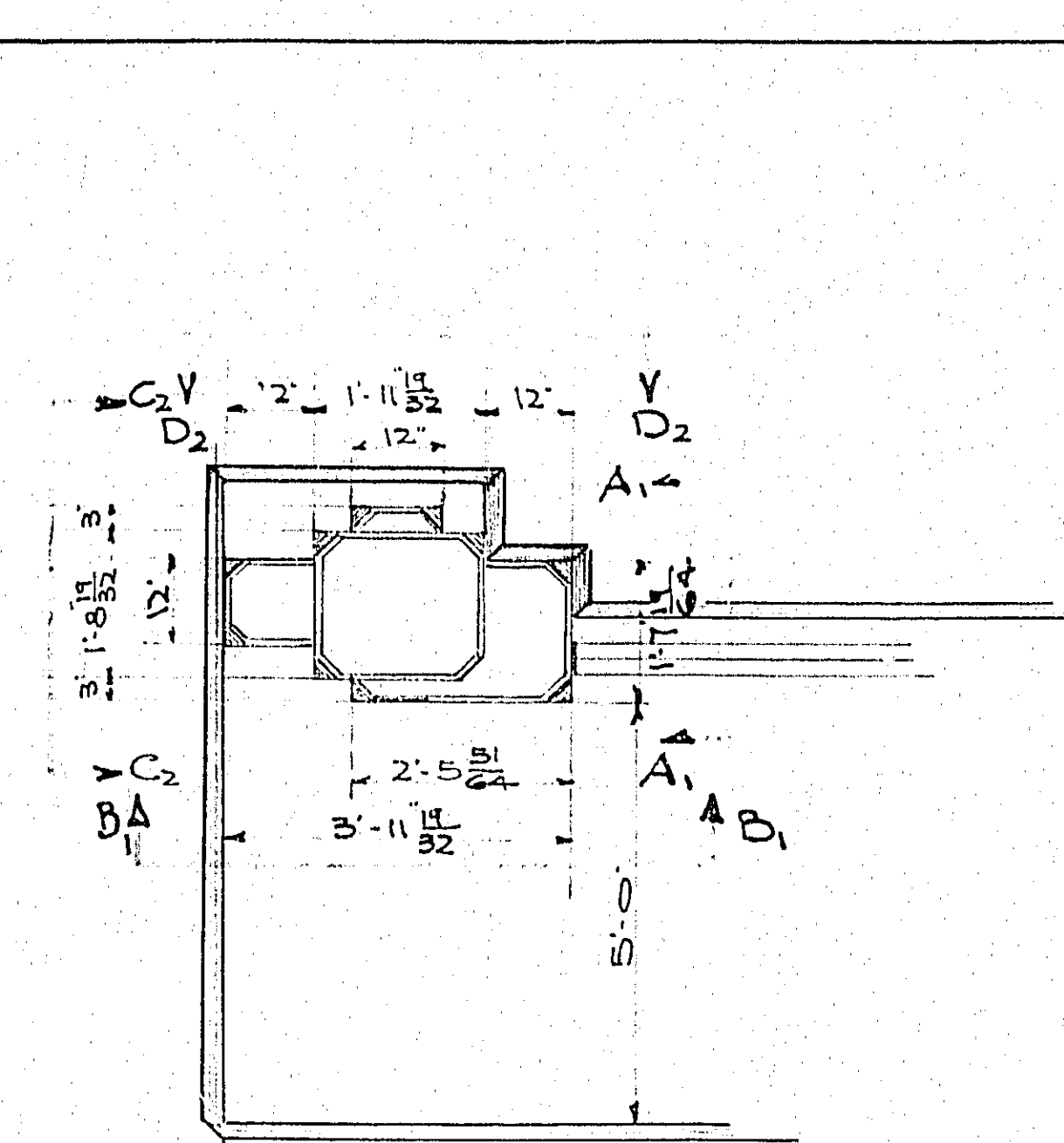


FOUNDATION PLAN

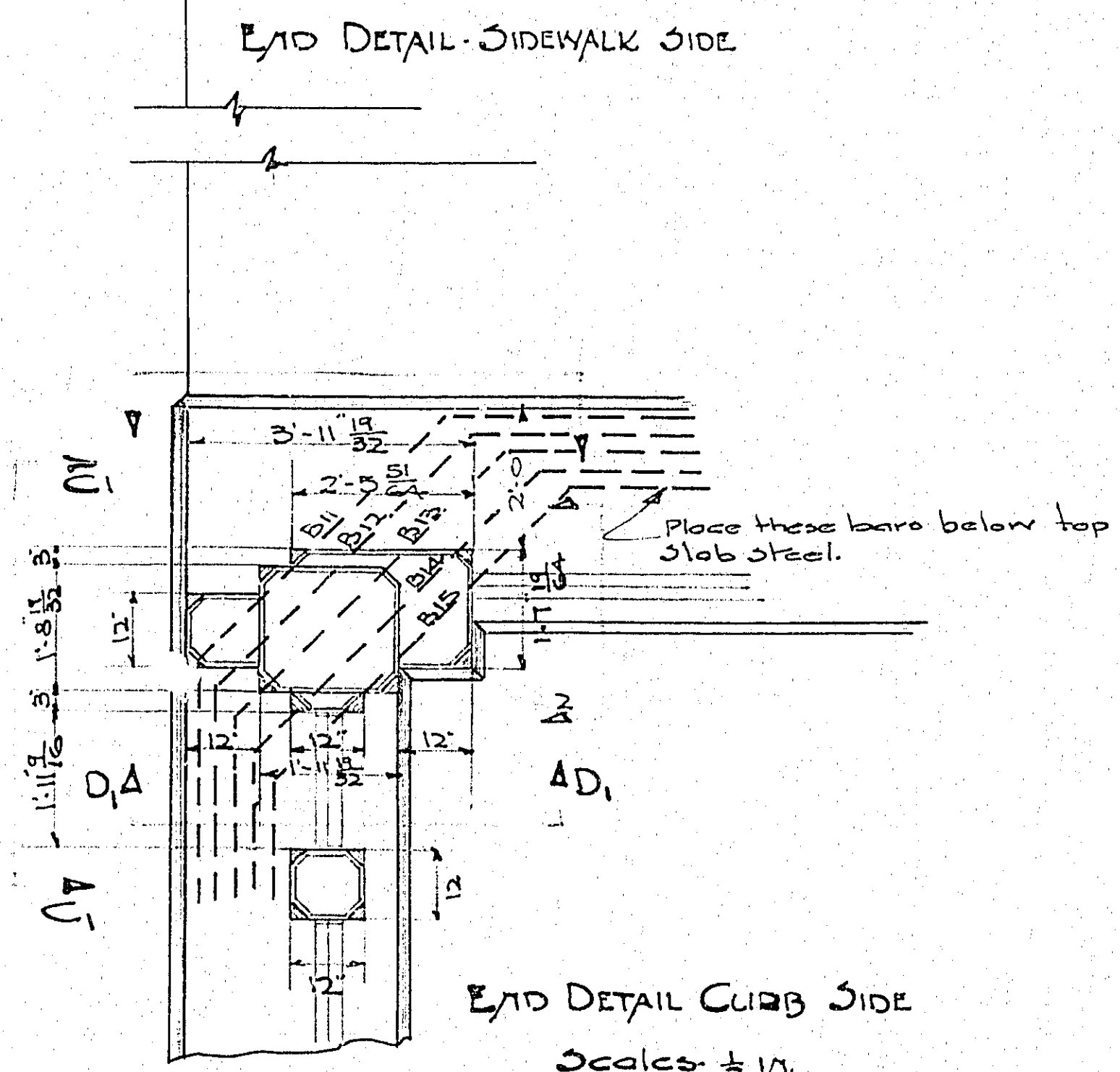


DECK FRAMING PLAN

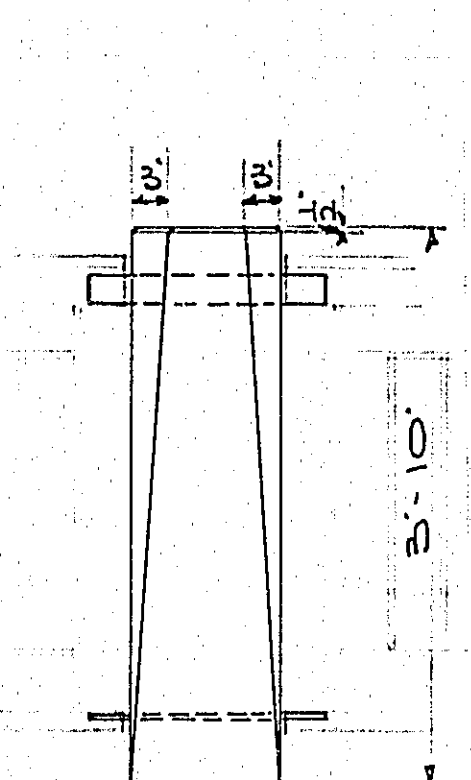
REVISIONS		DATE		BY	DESCRIPTION
DEPARTMENT OF HIGHWAYS - ONTARIO BRIDGE OFFICE					
LYONS CREEK BRIDGE BLACK CREEK BRIDGE No. 9					
THE QUEEN ELIZABETH WAY DIVISION 110.4 COUNTY OF WELLAND - TOWNSHIP OF BERTIE - LOT 16 - CON. 18 N.R.					
PLANS DECK - FOUNDATION - FRAMING					
BRIDGE ENGINEER			CHIEF ENGINEER		
Design	Check		Leading	Contract	
Drawing			4-20	39-140	
Steel			Toronto	Drawing No	Sheet
Tracing			Oct. 21, 1940	0-2707	1.



ELEVATION CLOSURE BETWEEN BRIDGES



HALF INTERIOR ELEVATION.



ELEVATION H.R. Post
TYPICAL

PLAN 12-12 POST

REVISIONS				
DATE	BY	DESCRIPTION		
DEPARTMENT OF HIGHWAYS - ONTARIO. BRIDGE OFFICE				
LYONS CREEK BRIDGE BLACK CREEK BRIDGE, NO. 9. OR THE QUEEN ELIZABETH WAY DIVISION NO. 4. COUNTY OF WELLAND - TOWNSHIP OF BERTIE - Lot 6 Con IX / 1 R.				
DETAILS ABUTMENT - END CLOSURE & POSTS - ELEVATIONS.				
----- BRIDGE ENGINEER		----- CHIEF ENGINEER		
Design	check	Loadings	Contract	
Drawing	"	H-20	39-140	
Steel	"	Toronto	Drawing No	
Tracing	"	Oct 21 1940	D-2707	Sheet
				3-

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Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

solutions@golder.com
www.golder.com

Golder Associates Ltd.
6925 Century Avenue, Suite #100
Mississauga, Ontario, L5N 7K2
Canada
T: +1 (905) 567 4444

