Appendix M.6 – Foundations Report – 407ETR

Kennedy Road Environmental Assessment between Steeles Avenue and Major Mackenzie Drive

Preliminary Foundation Investigation and Design Report

Kennedy Road Overpass at 407 Express Toll Route, Class Environmental Assessment Study for Improvements to Kennedy Road from Steeles Avenue to Major Mackenzie Drive, Markham, Ontario

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
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CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by HDR Inc. (HDR) to provide foundation engineering services in support of the Class Environmental Assessment for the proposed improvements to Kennedy Road (Y.R. 3) from Steeles Avenue (Y.R. 95) to Major Mackenzie Drive (Y.R. 25), in the City of Markham, Regional Municipality of York (Region), Ontario. As part of this project, foundation investigations were carried out for multiple structures along Kennedy Road between Steeles Avenue and Major Mackenzie Drive, including the Canadian Nation (CN) Rail, 407 Express Toll Route, a tributary, and Rouge River, as well as the potential grade separations of the Go Rail crossing at Clayton Drive and the Go Rail crossing at Austin Drive. This report presents the factual results of the foundation investigation carried out at Kennedy Road and 407 Express Toll Route.

The purpose of the investigation was to evaluate the subsurface soil and groundwater conditions at the Kennedy Road Overpass at 407 Express Toll Route by means of a limited number of boreholes and, based on our interpretation of the data, provide preliminary foundations engineering recommendations on the geotechnical aspects of design of the project.

The investigation and reporting were carried out in general accordance with the scope of work provided in our "*Work Plan and Methodology*", of the Subconsultant Agreement between Golder and HDR dated November 9, 2017. The scope of work was developed based on the requirements of the Request for Proposal outlined in The Regional Municipality of York 's Request for Proposal (P-16-167) dated November 3, 2016 and associated addenda.

The factual data, interpretations and preliminary recommendations contained in this report pertain to a specific project as described in the report and are not applicable to any other project or site location. This report should be read in conjunction with *"Important Information and Limitations of This Report"* following the text of this report. The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2.0 SITE DESCRIPTION

The existing Kennedy Road Overpass at 407 Express Toll Route consists of a two-span integral abutment structure, approximately 107 m long and 30 m wide, carrying six lanes of traffic of Kennedy Road as shown on the Key Plan on Figure 1. The Kennedy Road grade at the crossing location varies from about Elevation 188 m to 189 m and the grade of 407 Express Toll Route is at about Elevation 180 m. The existing bridge abutments and piers are reportedly founded on driven steel H-Piles.

Commercial developments are located north of 407 Express Toll Route and agricultural land is located immediately south of 407 Express Toll Route with commercial and residential developments further south. The side slopes of the existing approach embankment north and south of 407 Express Toll Route appear to be at an inclination of 2 horizontal to 1 vertical (2H:1V) Based on observations of the embankment at the time of the subsurface investigation, the side slopes appear to be performing adequately with no visual evidence of surficial sloughing or slope instability.

3.0 INVESTIGATION PROCEDURES

The field work for the preliminary investigation was carried out on November 20 and 21, 2019, during which time two boreholes (designated as Boreholes ETR-1 and ETR-2) were advanced near the existing bridge abutments to a depth of 15.7 m below ground surface. The locations of the boreholes are shown on the Borehole Location Plan on Figure 2 and the borehole records are provided in Appendix A.

The investigation was carried out using a truck-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 216 mm outside diameter (O.D.) hollow-stem augers. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50-mm O.D. split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1]. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions. The results of in situ field tests (i.e., SPT "N" values) as presented on the borehole records and in sub-sections of Section 4.2 are uncorrected.

Groundwater conditions were noted during drilling and immediately following drilling operations. A monitoring well was installed in Borehole ETR-2, in accordance with Ontario Regulation 903 (as amended), to permit monitoring of the groundwater level at the borehole location. The monitoring well consists of a 50 mm diameter PVC pipe with a slotted screen sealed at depth within the borehole and is equipped with a flush-mount casing. Details of the monitoring well installation and water level readings are presented on the borehole record in Appendix A. Borehole ETR-1 was backfilled with bentonite and the ground surface was restored to as near to original condition as practical, using cold-patch asphalt.

Field work was observed by members of Golder's engineering and technical staff, who located the boreholes in the field, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and took custody of the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Whitby laboratory where the samples underwent further visual examination. Geotechnical laboratory testing (water content, grain size distribution, and Atterberg limits) was carried out on selected soil samples, to ASTM Standards.

The borehole locations and ground surface elevations were obtained using a mobile GPS unit (Trimble XH 3.5G), having accuracy of 0.1 m in the vertical and horizontal directions. The locations provided on the borehole records and shown on Figure 1 relative to UTM NAD 83 (Zone 17) northing and easting coordinates and the ground surface elevations are referenced to a geodetic datum, as detailed in Table 1.

Table 1: Borehole Coordinates, Ground Surface Elevation and Depth

Structure Location	Borehole No.	Location (UTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
		Northing (m)	Easting (m)		
South Abutment	ETR-1	4,856,584.0	636,055.4	188.50	15.7
North Abutment	ETR-2	4,856,684.6	635,999.6	187.90	15.7

[1] ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project length along Kennedy Road (between Steeles Avenue and Major Mackenzie Drive) is located within the South Slope (southern portion of the site) and the Peel Plain (northern portion of the site) Physiographic Regions, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2]. The Kennedy Road Overpass structure at 407 Express Toll Route is located within the Peel Plain region.

The Peel Plain physiographic region covers portions of the Regional Municipalities of York, Peel, and Halton. Shallow, localized deposits of loose silt and sand and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay.

The South Slope physiographic region covers portions of the Regional Municipalities of Peel, York and Durham. A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with occasional silt to sand zones and is mapped in this area as the Halton Till.

4.2 Subsurface Conditions

Subsurface soil and groundwater conditions as encountered in the boreholes are presented on the record of boreholes in Appendix A. Also included are the "*Method of Soil Classification and Symbols*", and "*Terms Used on the Record of Boreholes and Test Pits*" to assist in the interpretation of the borehole logs. The geotechnical laboratory results are presented in Appendix B.

The boundaries between the strata on the borehole records have been inferred from drilling observations and non-continuous sampling. Therefore, these boundaries typically represent transitions between soil types rather than exact planes of geological change. Further, the subsurface conditions will vary between and beyond the borehole locations and across the site and caution should be used when extrapolating subsurface conditions between the boreholes.

In general, the subsurface conditions consist of the Kennedy Road pavement structure underlain by embankment fill extending up to 7 m depth. The fill is underlain by a till deposit ranging in composition from consisting of clayey silt and sand to silty clay, which is further underlain by a deposit of silty clay. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Pavement Structure

4.2.1.1 Asphalt

An approximately 150 mm and 155 mm thick layer of asphalt pavement was encountered at ground surface in Boreholes ETR-1 and ETR-2, respectively.

[2] Chapman, L.J. and Putnam, D,F. 1984. The Physiography of Southern Ontario, Ontario Geological Survey, Special Volume 2, Third Edition. Accompanied by Map P. 2715, Scale 1:600,000.

4.2.1.2 Granular Fill

A 1.2 m and 0.7 m thick layer of granular road base fill was encountered underlying the asphalt in Boreholes ETR-1 and ETR-2, respectively. The granular fill varies in composition from sand to gravelly sand to sand and gravel and extended to depths of 1.4 m and 0.9 m below ground surface (Elevation 187.1 m and 187.0 m), respectively.

The SPT "N" values measured within the granular fill were 30 blows and 31 blows per 0.3 m of penetration, indicating a dense level of compaction. The in-situ moisture content measured on two samples of the granular fill are about 5 per cent.

4.2.2 Silt and Sand Fill

A 4.2 m and 6.2 m thick layer of silt and sand fill was encountered underlying the granular fill in Boreholes ETR-1 and ETR-2, at depths of 1.4 m and 0.9 m below ground surface and extended to depths of 5.6 m and 7.1 m below ground surface (Elevations 182.9 m and 180.8 m), respectively.

The SPT "N" values measured within the silt and sand fill range from 10 blows to 37 blows per 0.3 m of penetration, indicating a compact to very dense level of compactness.

The results of a grain size distribution test carried out on one sample of silt and sand fill is shown on Figure B-1 in Appendix B. Atterberg limit testing was carried out on two samples of silt and sand fill and the results indicate that the soil is non-plastic. The in-situ moisture content measured on samples of the fill range from about 7 per cent to 15 per cent.

4.2.3 Clayey Silt and Sand to Silty Clay (Till)

A 9.0 m and 7.5 m thick till deposit was encountered underlying the silt and sand fill in Boreholes ETR-1 and ETR-2, respectively. The till deposit varies in composition from clayey silt and sand to silty clay, trace sand, trace gravel. The till was encountered in Boreholes ETR-1 and ETR-2 at depths of 5.6 m and 7.1 m below ground surface and extended to a depth of 14.6 m below ground surface (Elevation 173.9 m and 173.3 m). During drilling, the augers were grinding in both boreholes at various depths within the till deposit. It can be inferred that boulders and/or cobbles are present at the depths where the augers were grinding. Previous experience in the region indicates that the glacial deposits contain cobbles and boulders that are not identified by conventional drilling, sampling, and laboratory testing methods.

The SPT "N" values measured within the till deposit range from 11 blows to 22 blows per 0.3 m of penetration with one SPT "N" value of 66 blows per 0.3 m, suggesting a stiff to hard consistency, but generally a stiff to very stiff consistency.

The results of grain size distribution tests carried out on two samples of the till deposit are shown on Figure B-2 in Appendix B. Atterberg limit testing was carried out on two samples of the till deposit and the results indicate liquid limits of about 17 per cent and 23 per cent, plastic limits of about 13 per cent, and plasticity indices of about 4 per cent and 10 per cent. These test results, which are plotted on a plasticity chart on Figure B-3 in Appendix B, indicate that the clayey silt has slight plasticity and the silty clay has low plasticity. The water contents measured on nine samples of the deposit range from about 8 per cent to 17 per cent.

4.2.4 Silty Clay

A deposit of silty clay, trace sand and gravel was encountered underlying the till deposit in both boreholes ETR-1 and ETR-2 at a depth of 14.6 m below ground surface. Boreholes ETR-1 and ETR-2 both terminated within the silty clay deposit at a depth of 15.7 m below ground surface (Elevation 172.8 m and 172.2 m).

The SPT "N" values measured within the silty clay deposit are 9 blows and 10 blows per 0.3 m of penetration, indicating a stiff consistency.

Atterberg limit testing was carried out on one sample of the silty clay deposit and the results indicate a liquid limit of 22 per cent, a plastic limit of 12 per cent, and a plasticity index of 10 per cent. These test results, which are plotted on a plasticity chart on Figure B-4 in Appendix B, indicate the silty clay has low plasticity. The water content measured on a sample of the silty clay deposit is about 28 per cent.

4.2.5 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist, with the exception of the samples from the lower silty clay deposit, which were wet (above the plastic limit). Details of the groundwater levels observed in the boreholes upon completion of drilling are summarized on the borehole records in Appendix A. Upon completion of drilling, the water level in Borehole ETR-1 was measured at a depth of 15.0 m below ground surface (Elevation 173.5 m)

A monitoring well was installed in Borehole ETR-2 and sealed within the till and silty clay deposits. The recorded groundwater level in the monitoring well is summarized in Table 2.

Table 2: Groundwater Depth and Elevation Reading

Borehole Number	Screened Stratigraphy	Ground Surface Elevation (m)	Water Level Depth (m)	Water Elevation (m)	Date of Monitoring Well Reading
ETR-2	Till / Silty Clay	188.50	10.8	177.1	November 30, 2018
			10.8	177.1	December 13, 2018

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

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Anastasia Poliacik, P.Eng., and was reviewed Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder.

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PART B

PRELIMNARY FOUNDATION DESIGN REPORT
KENNEDY ROAD OVERPASS AT 407 EXPRESS TOLL ROUTE
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

6.0 DISCUSSION AND ENGINEERING INVESTIGATION

This section of the report provides preliminary foundation design recommendations for the preliminary design of the proposed widening of the Kennedy Road Overpass at 407 Express Toll Route, associated with the proposed improvements to Kennedy Road in the City of Markham, Region of York, Ontario. The preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to allow for preliminary design of the widening of the Kennedy Road overpass at 407 Express Toll Route, for planning purposes.

Further investigations will be required during Detailed Design to obtain subsurface information specific to the widened foundation locations and to confirm that the subsurface conditions and the geotechnical parameters and resistance values provided in this preliminary design phase are appropriate for the Detailed Design of the foundations. All recommendations provided below are preliminary and should be reviewed and revised upon receiving updated design information during the Detailed Design phase of the project.

Where comments are made on construction, they are provided only to highlight aspects of construction that could affect the design of the project. Those requiring information on aspects of construction must make their own independent interpretation of the subsurface information provided as it affects their proposed construction methods, costs, equipment selection, scheduling and the like.

6.1 General

The following available information of the existing Kennedy Road overpass at 407 Express Toll Route was provided by HDR and was reviewed in preparation of this report:

- Tottem Sims Hubicki Associates drawings titled, *"Kennedy Road Over Hwy. 407, (Structure No. F03), General Arrangement",* Drawing Nos. 9401-17-165-23-0001 to 9401-17-165-23-0016, dated 1996;
- Existing ground surface profile of Kennedy Road crossing 407 Express Toll Route (AutoCAD drawing provided to Golder by HDR in December 2018).

Based on the available information, the existing Kennedy Road overpass at 407 Express Toll Route consists of a two-span integral abutment structure approximately 107 m long and 30 m wide, carrying six lanes of traffic of Kennedy Road over 407 Express Toll Route and was originally constructed in the late 1990's. The pavement surface along Kennedy Road at the site varies from about Elevation 188 m to 189 m and the pavement grade of 407 Express Toll Route at the site is at about Elevation 180 m, resulting in approach embankments up to about 9 m in height.

The existing bridge abutments are each supported on one row of vertical driven steel HP 310 x 110 piles within an upper 600 mm diameter CSP pipe sleeve and the existing bridge pier is founded on two rows of battered driven steel HP 310 x 110 piles. Additional pile details inferred from the available design drawing are summarized in Table 3.

Table 3: Existing Pile Design Details Inferred from Available Drawings

Location	Foundation Type	Underside of Pile Cap Elevation (m) E - W	Pile Cut-Off Elevation (m) E-W	Pile Length (m) E-W	Pile Tip Elevation (m)	Design ULS / SLS Loads Per Pile (kN)
South Abutment	Vertical Steel H-Piles within CSP Sleeve	184.9 – 186.1	185.5 – 186.6	23.5 – 24.7	162.0	1300 / 900
Pier	Battered Steel H-Piles	178.0	178.3	-	-	1300 / 900
North Abutment	Vertical Steel H-Piles with CSP Sleeve	183.8 – 185.0	184.4 – 185.6	22.4 – 23.6	162.0	1300 / 900

The subsurface conditions consist of about 5 m to 7 m of embankment fill overlying stiff to hard clayey silt and sand till and a deposit of stiff silty clay at depth, in which the boreholes terminated.

Based on review of previous borehole information available as part of this project in a report entitled "*Hydrogeological Assessment Report to Renew Permit to Take Water, 1500-mm Diameter Kennedy Road Watermain*" prepared by Coffey geotechnics Inc., dated March 25, 2014, two boreholes (identified as BH 59 and BH 60), advanced within the vicinity of the 407 Express Toll Route structure, penetrated into the very dense "100 blow" soil below about Elevation 165 m (south side) and 163 m (north side). Further, these boreholes also indicate the presence of water-bearing sand and gravel layers within the overburden. It should be noted that our boreholes were terminated up to 9 m above the "100-blow" till material and as such, they only penetrated the upper portion of the more competent (dense to very dense) till assumed to be present at depth.

6.2 Consequence and Site Understanding Classification

The structure passes over 407 Express Toll Route and has the potential to impact alternative transportation corridors. In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code and its Commentary* (CHBDC 2014), the structure and its foundation system is classified as having a "typical consequence level" associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of the 2014 CHBDC, the level of confidence for design is considered to be a "low degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ, from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Table 6.2 of the *CHBDC* (2014) have been used for design.

6.3 Foundation Options

To accommodate the proposed widening of Kennedy Road from Steeles Avenue to Major Mackenzie Drive, it is understood that there are currently two options being considered for the overpass at 407 Express Toll Route. The options are as follows:

- **Option 1:** Widening of the existing structure to accommodate the existing four lanes of traffic, the existing Active Transport (pedestrian traffic and bicycles) and two additional traffic lanes along Kennedy Road over 407 Express Toll Route.
- **Option 2:** Modification of the existing structure to accommodate the additional lane in the northbound and southbound direction along Kennedy Road and construction of a separate, new structure adjacent to the existing structure to accommodate the active transport (pedestrian traffic and bicycles).

Based on the existing structure geometry and the subsurface conditions at the site, the following foundation recommendations were considered for the preliminary design of new or widened bridge foundation elements:

Shallow Foundations

- **Option 1:** Considering the structural loads required to support the widened bridge structure, shallow foundations are not considered suitable for support of the widened bridge foundations (abutments and piers) at this site due to the presence of the relatively weak (stiff) clayey silt and sand till deposit below the fill. In addition, the existing structure is founded on driven steel H-piles and therefore if the new structure was supported by shallow foundations this may result in unacceptable differential settlement between the existing and new widened structure elements.
- **Option 2:** The required resistances for a separate, new Active Transport structure are expected to be lower than those for a structure carrying vehicular traffic. Depending on the load requirements, shallow foundations may be considered suitable for support of a separate, new, lightly loaded structure adjacent the existing structure.

Deep Foundations

- **Option 1:** Deep foundations are considered suitable to support the widened structure. The existing structure is supported on steel H piles founded at about Elevation 162 m and appears to be performing adequately. Driven steel H-piles are considered the most technically feasible option for support of the widened foundation elements. Drilled shafts (caissons) are also considered suitable deep foundation options, however, the caisson installation process may result in disturbance of the existing piles in terms of vibrations, soil loosening of the surrounding soils (and loss of shaft friction) due to hydrostatic pressures, etc.
- **Option 2:** Deep foundations are considered suitable to support a separate, new Active Transportation (AT) structure adjacent the existing structure; however, they may not be necessary as shallow foundations may provide sufficient geotechnical resistances.

The following sections provide preliminary recommendations for shallow and deep foundations for the two structure options discussed above.

6.4 Strip / Spread Footings

6.4.1 Founding Elevations

Detailed below in Table 4, for each abutment, are the recommended founding elevations for 3 m by 10 m strip footings on the stiff to hard clayey silt and sand till deposit.

Table 4: Founding Elevations Strip/Spread Footings

Foundation Unit	Reference Borehole	Founding Stratum	Highest Founding Elevation (m)
North Abutment	ETR-2	Stiff to hard clayey silt and sand till	182.3
South Abutment	ETR-1		180.2

The underside of footings should be founded at a minimum depth of 1.4 m below the lowest surrounding grade measured perpendicular to the outer edge of the underside of the footing to provide adequate protection against frost penetration, as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Depths for Southern Ontario*).

Consideration could also be given to sub-excavation of the fill material to the highest founding elevation given above and the abutment foundations could be "perched" on a compacted granular pad in the approach embankments above the 407 Express Toll Route grade. In this case, the compacted granular pad should have a minimum thickness of 2 m; any existing fill, organic soils and/or loose soils within the zone of influence below the compacted granular pad should be sub-excavated and replaced with engineered fill, or the pad thickened to found on the stiff to very stiff clayey silt and sand till deposit at the elevations given above for footings founded on these deposits. The pad should consist of OPSS.MUNI 1010 (*Aggregates*) Granular 'A' material extending at least 1 m beyond the edges of the footing(s), then outward and downward at 1H:1V. The granular fill should be placed in accordance with OPSS.MUNI 501 (*Compacting*). Consideration should be given to the possible interference between the existing pile cap and the proposed sub-excavation for engineered fill placement, as it may render this option not feasible.

6.4.2 Geotechnical Resistances

Spread/strip footings placed on the properly prepared subgrade should be designed based on the following factored ultimate geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) for footing widths between 2 m and 3 m provided in Table 5.

Table 5: Geotechnical Resistances for Shallow Foundations

Foundation Unit	Inferred Founding Material	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa)
Abutments	Clayey Silt and Sand Till	400	150
	Minimum 2 m thick compacted Granular Pad	650	300

The factored ultimate and factored serviceability geotechnical resistances are dependent on the footing width and founding elevation and as such, the geotechnical resistances should be reviewed if the footing width is greater than that specified above or if the founding elevation differs from that given above.

The factored ultimate geotechnical resistances provided are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footings, eccentricity and inclination of the load should be considered in accordance with Section 6.10.4 of the *Commentary to the CHBDC* (2014).

The footing subgrade should be inspected by qualified geotechnical personnel following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to check that all existing fill or other unsuitable material have been removed. Where sub-excavation is required, the sub-excavated area should be backfilled with granular material meeting OPSS.MUNI 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II that is placed and compacted in accordance with OPSS.MUNI 501 (*Compacting*), or the thickness of the footing increased to the full excavation depth.

The clayey silt and sand till subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a working slab of 100 mm thick, having a minimum 28-day compressive strength of 20 MPa be placed within four hours following inspection and approval of the subgrade, to protect the subgrade from softening.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the new concrete footings and the subgrade should be calculated in accordance with Section 6.10.5 of the 2014 CHBDC. For cast-in-place concrete footings constructed directly on the clayey silt and sand till, granular pad or working slab, the sliding resistance may be calculated based on the unfactored coefficient of friction, tan δ, which can be taken as follows:

- Cast-in-place footing or working slab to clayey silt and sand till: tan δ = 0.5
- Cast-in-place footing to granular pad (Granular 'A'): tan δ = 0.6
- Cast-in-place footing to concrete working slab: tan δ = 0.7

6.5 Driven Steel Piles

6.5.1 Pile Founding Elevation

The new or widened structure elements may be supported on end-bearing steel H-piles or steel pipe piles driven at least 1.5 m into the hard till having SPT "N" values of greater than "100-blows" per 0.3 m of penetration. As the piles will develop a majority of their resistance from side friction and since "100-blow" soil was not encountered in the boreholes drilled for this current investigation, based on the previous information, the pile tips should extend to at least Elevation 162 m; however, this elevation should be confirmed during Detailed Design with additional boreholes that extend a minimum of 3 m into "100-blow" soil.

The underside of the pile caps should be founded at a minimum depth of 1.4 m below the lowest surrounding grade including measured perpendicular to any sloping ground to provide adequate protection against frost penetration, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario).*

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 slightly 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 driving shoes or flange plates for protection during driving in accordance with OPSS.MUNI 903 (*Deep Foundations*) and OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*). In very dense / hard and / or bouldery soils, as may be encountered at this site, driving shoes such as Titus Standard “H” Bearing Pile Points may be preferred over flange plates. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001-100 Type II (*Steel Tube Pile Driving Shoe*).

6.5.2 Geotechnical Resistances

For HP 310x110 piles driven to the recommended tip elevation and based on current and previous subsurface information, the estimated factored ultimate axial geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) for design of the foundations is provided in Table 6. The factored ultimate axial geotechnical resistance provided below is for a pile with a tip Elevation of 172.0 m as the deepest borehole advanced during this investigation only extended to Elevation 172.2 m.

Table 6: Geotechnical Axial Resistances for Steel H-Piles

Founding Material	Design Pile Tip Elevation (m)	Factored Axial Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN)
Silty Clay	172	900	=[1]
“100-blow” till	162	1,400	=[1]

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

6.5.3 Pile Interference

In order to minimize the influence of the pile installation on the performance of the existing pile foundations, the designer must check that any new piles will not interfere with the existing piles along the full length of the piles, considering the existing pile batter, if any. Further, consideration should be given to a vibration monitoring program on the existing bridge abutments as discussed further in Section 6.8.6.

6.6 Lateral Earth Pressures for Design of Abutment Walls and Wing Walls

The lateral earth pressures acting on the abutment walls and any associated wingwalls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the height of the wall, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be considered in the design and should be addressed at Detailed Design.

The following recommendations are made concerning the design of the walls.

- Free-draining granular fill meeting the specifications of OPSS.MUNI 1010 (*Aggregates*) Granular ‘A’ or Granular ‘B’ Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill, as applicable. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.MUNI 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*) for wingwall / end walls as applicable.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with the *CHBDC* (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall, with limitations on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.MUNI 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.4 m behind the back of the wall in accordance with Figure C6.20(a) of the *Commentary to the CHBDC* (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the wall or footing, as applicable, in accordance with Figure C6.20(b) of the *Commentary to the CHBDC* (2014).

6.6.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- For a restrained wall, the pressures are based on the existing or proposed new embankment fill behind the granular backfill zone, and the following parameters (unfactored) in **Table 7** may be used assuming the use of earth fill or Select Subgrade Material for the general embankment fill:

Table 7: Coefficients of Static Lateral Earth Pressure for Restrained Walls

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Earth Fill / Select Subgrade Material	20 kN/m^3	0.47	0.31

- For an unrestrained wall, the pressures are based on the granular fill in the backfill zone, and the following parameters (unfactored) in **Table 8** may be used:

Table 8: Coefficients of Static Lateral Earth Pressure for Unrestrained Walls

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular ‘A’	22 kN/m^3	0.43	0.27
Granular ‘B’ Type II	21 kN/m^3	0.43	0.27

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the *Commentary to the CHBDC* (2014).
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.7 Approach Embankments

6.7.1 Subgrade Preparation and Embankment Construction

Prior to construction of the up to 9 m high widened approach embankments it is recommended that any loosened/softened fill and topsoil/organic soils be removed from the footprint of the widened approach embankments. After stripping of organics and fill, the exposed subgrade should be proof-rolled to identify any loose/softened areas requiring sub-excavation / replacement or additional compaction prior to fill placement.

Fill for construction of the approach embankments should consist of Granular 'A', Granular 'B' Type I or Type II meeting the specifications of OPSS.MUNI 1010 (*Aggregates*). The embankment fill should be placed and compacted in accordance with OPSS.MUNI 501 (*Compacting*) and OPSS.PROV 206 (*Grading*). Embankment side slopes should be constructed no steeper than 2 Horizontal to 1 Vertical (2H:1V) in granular fill and properly benched and keyed into the existing embankment fill in accordance with OPSD 208.010 (*Benching of Earth Slopes)*.

To control erosion of the side slopes, a minimum 2 m wide bench is recommended where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (*Slope Flattening*). To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (*Topsoil*) and OPSS.MUNI 804 (*Seed and Cover*) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS 511 (*Rip Rap, Rock Protection and Granular Sheeting*), and OPSS.MUNI 1004 (*Aggregates*) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.7.2 Global Stability

Limit equilibrium slope stability analyses were performed on the approach embankment side slopes using the commercially available program "Slide V.2018" published by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of embankment slopes under static conditions at the end of construction as per the *CHBDC* (2014). This FoS is considered adequate for the embankments at this site considering the design requirements provided that a suitable number of boreholes are completed during the Detailed Design stage to confirm the anticipated subsurface conditions as described herein. The stability analyses were performed to assess if the target minimum FoS was achieved for the design embankment height and geometries.

The existing 2H:1V side slopes have a Factor of Safety of about 1.4. Widening of no more than 5 m to accommodate a new pedestrian bridge on one side of the existing bridge, or up to about 2 m on each side to accommodate a widened structure, will result in a FoS of 1.5 for global stability only if the widened embankment is constructed using granular material and be properly keyed into the existing earth fill. Embankments constructed of "earth fill" meeting the requirements of OPSS.MUNI 212 (*Borrow*); would potentially require flatter side slopes in order to achieve the required FoS.

6.7.3 Settlement

For the up to 9 m high approach embankments with up to 5 m of widening, the estimated settlement of the foundation soils under the additional fill is estimated to be in the range of 15 mm to 30 mm and is expected to occur during construction. Further, this settlement will be differential across the width of the embankment being highest under the widened portion along the slope. The estimated settlements should be reassessed during the Detailed Design stage, once the proposed structure and embankment geometry is available, and in particular in regard to imposed settlement on the already existing embankment and existing piles.

6.8 Construction Considerations

6.8.1 Temporary Excavations

Temporary excavations for shallow foundations and/or pile cap construction will extend through the existing fill and into the stiff to hard clayey silt and sand till deposit. Excavation works must be carried out in accordance with the guidelines outlined in the *Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects*. The existing fill and till materials would be classified as Type 3 soil, according to *OHSA* criteria. Temporary excavations above the water table should be made with side slopes no steeper than 1H:1V. During wet periods of the year some local flattening of slopes may be required. Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the height of the open cut excavation.

6.8.2 Temporary Protection Systems

At this preliminary stage, it is anticipated that temporary protection systems will be required along Kennedy Road in order to facilitate the construction of the widened/new structure. The temporary excavation support systems should be designed and constructed in accordance with OPSS.MUNI 539 (*Temporary Protection Systems*). The lateral movement of the temporary protection systems should meet Performance Level 2 as specified in OPSS.MUNI 539, provided that the existing structures and any adjacent utilities can tolerate this magnitude of deformation. Although the selection and design of the protection systems will be the responsibility of the Contractor, for conceptual purposes, a driven, interlocking sheet pile system or soldier pile and timber lagging system would be suitable for the temporary excavation support at this structure site, based on the anticipated subsurface soil and groundwater conditions. Parameters for lateral earth pressure coefficients should be provided at the Detailed Design stage.

6.8.3 Groundwater and Surface Water Control

The groundwater level measured in the monitoring well installed in Borehole ETR-2, which was screened in the clayey silt and sand till deposit and the lower silty clay deposit, was measured at a depth of 10.8 m below ground surface (Elevation 177.1 m). Depending on the time of year of construction, perched groundwater conditions may also be present within the compact to dense silt and sand fill materials above the clayey silt and sand till deposit.

Considering the relatively low permeability of the stiff to hard clayey silt and sand till, it is anticipated that water inflow from the till and also from the silt and sand fill can be handled by pumping from properly filtered sump pumps placed at the base of the excavation and outside the foundation footprint, installed prior to reaching the excavation base. Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations to the extent practicable.

6.8.4 Obstructions During Pile Driving

The glacially derived till soils at the site should be expected to contain cobbles and boulders, which could affect the installation of driven steel H-piles and/or temporary protection systems. It is recommended that driving shoes be used to facilitate pile driving into/through the very dense to hard till deposits to minimize damage to pile tips. The geotechnical investigation at Detailed Design should note on the borehole records any observation of grinding of the augers (i.e. an indication of the presence of a cobble or boulder at depth).

6.8.5 Subgrade Protection

The native soils that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a minimum 100 mm thick concrete working slab having a minimum 28-day compressive strength of 20 MPa be placed on the subgrade of foundation excavations within four hours after preparation, inspection and approval of the subgrade.

6.8.6 Vibration Monitoring During Temporary Protection System or Pile Installation

Structures near the site include the existing bridge and commercial properties (approximately 150 m from the site). A maximum partial peak velocity (PPV) of 100 mm/s is generally considered acceptable 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) will reach this threshold level, however, since the pile driving will be immediately adjacent to the existing piles, short term peaks in PPV may be measured and the pile driving energy may need to be reduced in order to avoid damage to the existing piles/bridge. It is considered prudent that pre- and post-construction condition surveys and vibration monitoring at the nearby structures be considered to defend against potential damage claims associated with vibration-inducing activities at the site. A PPV threshold of 25 mm/s is generally considered applicable for residential buildings and 50 mm/s applicable for steel/concrete commercial buildings.

7.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION WORK DURING DETAILED DESIGN

Should the existing structure be widened, or a separate Active Transportation structure be proposed, additional boreholes will be required during the Detailed Design. The additional boreholes should be advanced within the footprint of the widened/new foundation elements and widened approach embankments to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report. The detailed investigation should:

- Assess the type and depth of fill present;
- Assess near surface soil deposits within the footprint of the proposed embankments for settlement analysis, where applicable;
- Advance the boreholes a minimum of 3 m into “100-blow” material;
- Test parameters used to assess the corrosive potential of the soil to concrete and buried steel;
- Evaluate the seismic Site Class and seismic hazard values;
- Confirm groundwater elevations in the till materials and the piezometric levels in any hydraulically significant deposits that may be encountered; and,
- Record the occurrence of grinding of the augers during advancement of the boreholes to assess the presence of such obstructions as they may affect excavations and the installation of driven steel H-piles.

8.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Anastasia Poliacik, P.Eng. and Ms. Sarah Poot, P.Eng., a senior geotechnical engineer and Associate of Golder conducted a technical review of this report.

This Report was authored under a Subconsultant Agreement between HDR and Golder for the Regional Municipality of York's ("Owner") projects. The Report is provided to HDR and Regional Municipality of York for their use, utilizing their judgment, in fulfilling a portion of HDR's particular scope of work. No other party may rely upon this report, or any portion thereof, without Golder's express written consent and any reliance of the reports by others will be at that user's sole risk and liability, notwithstanding that they may have received this Report through an appropriate user. In addition, Golder shall not be liable for any use of the Report for any purpose other than that for which the same was originally prepared or provided by Golder, or any improper use of this Report, or to any party other than HDR.

Golder Associates Ltd.

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AMP/SEMP/cr;mes

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Canadian Standards Association 2006. Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S06-06 and Commentary.

Ontario Provincial Standard Drawings (OPSD)

OPSD 202.010 Slope Flattening

OPSD 208.010 Benching of Earth Slopes

OPSD 3000.100 Foundation, Piles, Steel H-Pile Driving Shoe

OPSD 3001.100 Steel Tube Pile Driving Shoe

OPSD 3090.101 Foundation, Frost Penetration Depths for Southern Ontario

OPSD 3121.150 Walls, Retaining, Backfill, Minimum Granular Requirement

OPSD 3190.100 Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specifications (OPSS)

OPSS.MUNI 206 Construction Specification for Grading
OPSS.MUNI 212 Construction Specification for Borrow
OPSS.MUNI 501 Construction Specification for Compacting
OPSS.MUNI 511 Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.MUNI 539 Construction Specification for Temporary Protection Systems
OPSS.802 Construction Specification for Topsoil
OPSS.MUNI 804 Construction Specification for Seed and Cover
OPSS.902 Construction Specification for Excavating and Backfilling - Structures
OPSS.MUNI 903 Construction Specification for Deep Foundations
OPSS.MUNI 1004 Material Specification for Aggregates - Miscellaneous
OPSS.MUNI 1010 Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

ASTM International:

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

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client can not rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder can not be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Ground Water Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

APPENDIX A

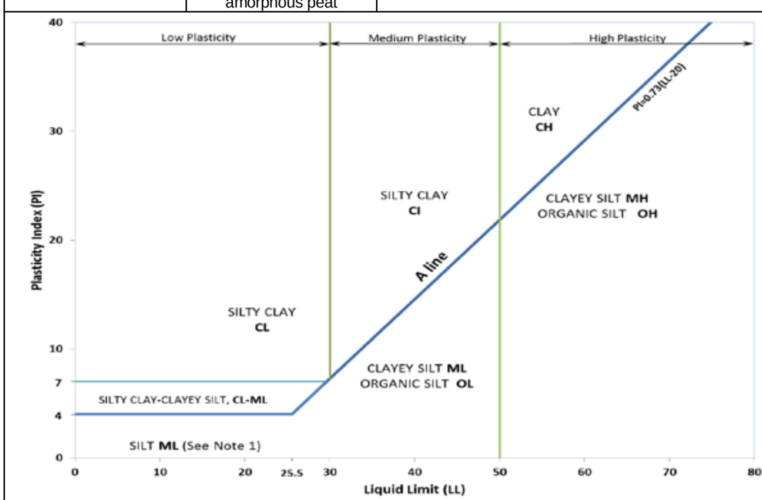
Record of Borehole Sheets

METHOD OF SOIL CLASSIFICATION

The Golder Associates Ltd. Soil Classification System is based on the Unified Soil Classification System (USCS)

Organic or Inorganic	Soil Group	Type of Soil		Gradation or Plasticity	$Cu = \frac{D_{60}}{D_{10}}$	$Cc = \frac{(D_{30})^2}{D_{10} x D_{60}}$	Organic Content	USCS Group Symbol	Group Name
INORGANIC (Organic Content ≤30% by mass)	COARSE-GRAINED SOILS (>50% by mass is larger than 0.075 mm)	GRAVELS (>50% by mass of coarse fraction is larger than 4.75 mm)	Gravels with ≤12% fines (by mass)	Poorly Graded	<4	≤1 or ≥3	≤30%	GP	GRAVEL
				Well Graded	≥4	1 to 3		GW	GRAVEL
			Gravels with >12% fines (by mass)	Below A Line	n/a			GM	SILTY GRAVEL
				Above A Line	n/a			GC	CLAYEY GRAVEL
		SANDS (≥50% by mass of coarse fraction is smaller than 4.75 mm)	Sands with ≤12% fines (by mass)	Poorly Graded	<6	≤1 or ≥3		SP	SAND
				Well Graded	≥6	1 to 3		SW	SAND
			Sands with >12% fines (by mass)	Below A Line	n/a			SM	SILTY SAND
				Above A Line	n/a			SC	CLAYEY SAND

Organic or Inorganic	Soil Group	Type of Soil	Laboratory Tests	Field Indicators: Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)	Organic Content	USCS Group Symbol	Primary Name
INORGANIC (Organic Content ≤30% by mass)	FINE-GRAINED SOILS (≥50% by mass is smaller than 0.075 mm)	SILTS (Non-Plastic or PI and LL plot below A-Line on Plasticity Chart below)	Liquid Limit <50	Rapid	None	None	>6 mm	N/A (can't roll 3 mm thread)	<5%	ML	SILT
				Slow	None to Low	Dull	3mm to 6 mm	None to low	<5%	ML	CLAYEY SILT
				Slow to very slow	Low to medium	Dull to slight	3mm to 6 mm	Low	5% to 30%	OL	ORGANIC SILT
			Liquid Limit ≥50	Slow to very slow	Low to medium	Slight	3mm to 6 mm	Low to medium	<5%	MH	CLAYEY SILT
				None	Medium to high	Dull to slight	1 mm to 3 mm	Medium to high	5% to 30%	OH	ORGANIC SILT
		CLAYS (PI and LL plot above A-Line on Plasticity Chart below)	Liquid Limit <30	None	Low to medium	Slight to shiny	~ 3 mm	Low to medium	0% to 30% (see Note 2)	CL	SILTY CLAY
			Liquid Limit 30 to 50	None	Medium to high	Slight to shiny	1 mm to 3 mm	Medium		CI	SILTY CLAY
			Liquid Limit ≥50	None	High	Shiny	<1 mm	High		CH	CLAY
HIGHLY ORGANIC SOILS (Organic Content >30% by mass)		Peat and mineral soil mixtures							30% to 75%	PT	SILTY PEAT, SANDY PEAT
		Predominantly peat, may contain some mineral soil, fibrous or amorphous peat							75% to 100%		PEAT



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.

Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.

For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.

For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.

A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

Soil Constituent	Particle Size Description	Millimetres	Inches (US Std. Sieve Size)
BOULDERS	Not Applicable	>300	>12
COBBLES	Not Applicable	75 to 300	3 to 12
GRAVEL	Coarse Fine	19 to 75 4.75 to 19	0.75 to 3 (4) to 0.75
SAND	Coarse Medium Fine	2.00 to 4.75 0.425 to 2.00 0.075 to 0.425	(10) to (4) (40) to (10) (200) to (40)
SILT/CLAY	Classified by plasticity	<0.075	< (200)

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

Percentage by Mass	Modifier
>35	Use 'and' to combine major constituents (*i.e.,* SAND and GRAVEL)
> 12 to 35	Primary soil name prefixed with "gravelly, sandy, SILTY, CLAYEY" as applicable
> 5 to 12	some
≤ 5	trace

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.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

Cone Penetration Test (CPT)
An electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (q_t), porewater pressure (u) and sleeve frictions are recorded electronically at 25 mm penetration intervals.

Dynamic Cone Penetration Resistance (DCPT); 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

SAMPLES

AS	Auger sample
BS	Block sample
CS	Chunk sample
DD	Diamond Drilling
DO or DP	Seamless open ended, driven or pushed tube sampler – note size
DS	Denison type sample
GS	Grab Sample
MC	Modified California Samples
MS	Modified Shelby (for frozen soil)
RC	Rock core
SC	Soil core
SS	Split spoon sampler – note size
ST	Slotted tube
TO	Thin-walled, open – note size (Shelby tube)
TP	Thin-walled, piston – note size (Shelby tube)
WS	Wash sample

SOIL TESTS

w	water content
PL , w_p	plastic limit
LL , 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, Gs)
DS	direct shear test
GS	specific gravity
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 (FV)	field vane (LV-laboratory vane test)
γ	unit weight

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

Term	SPT 'N' (blows/0.3m)[1]
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	>50

1. SPT 'N' in accordance with ASTM D1586, uncorrected for the effects of overburden pressure.
2. Definition of compactness terms are based on SPT 'N' ranges as provided in Terzaghi, Peck and Mesri (1996). Many factors affect the recorded SPT 'N' value, including hammer efficiency (which may be greater than 60% in automatic trip hammers), overburden pressure, groundwater conditions, and grainsize. As such, the recorded SPT 'N' value(s) should be considered only an approximate guide to the soil compactness. These factors need to be considered when evaluating the results, and the stated compactness terms should not be relied upon for design or construction.

Field Moisture Condition

Term	Description
Dry	Soil flows freely through fingers.
Moist	Soils are darker than in the dry condition and may feel cool.
Wet	As moist, but with free water forming on hands when handled.

COHESIVE SOILS

Consistency

Term	Undrained Shear Strength (kPa)	SPT 'N'[1,2] (blows/0.3m)
Very Soft	<12	0 to 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

1. SPT 'N' in accordance with ASTM D1586, uncorrected for overburden pressure effects; approximate only.
2. SPT 'N' values should be considered ONLY an approximate guide to consistency; for sensitive clays (e.g., Champlain Sea clays), the N-value approximation for consistency terms does NOT apply. Rely on direct measurement of undrained shear strength or other manual observations.

Water Content

Term	Description
w < PL	Material is estimated to be drier than the Plastic Limit.
w ~ PL	Material is estimated to be close to the Plastic Limit.
w > PL	Material is estimated to be wetter than the Plastic Limit.

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$ - u)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

PROJECT: 1664178 (2000)

LOCATION: N 4856583.95; E 636055.42

RECORD OF BOREHOLE: ETR-1

SHEET 1 OF 2

BORING DATE: November 21, 2018

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

BORING METHOD: Power Auger — 216 mm O.D. Hollow Stem Augers

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
	GROUND SURFACE	188.50				
0	ASPHALT (150 mm)	0.00				
	FILL - (SP) SAND and GRAVEL, some fines, trace silt pockets; brown; non-cohesive, moist, dense	0.15	1	SS	30	
1			2	SS	31	
	FILL - (ML) SILT and SAND, trace to some gravel, trace to some plastic fines; brown; non-cohesive, moist, compact to dense	187.13 / 1.37	3	SS	23	
2	- Trace organics in sample 3					
	- No soil recovery from sample 4		4	SS	22	
3			5	SS	10	
4						
5			6	SS	37	MH NP
	- Augers grinding from 5.2 m to 5.5 m depth					
	(CL-ML/CL) CLAYEY SILT and SAND to SILTY CLAY, some sand, trace to some gravel; brown to grey, (TILL); cohesive, w<PL to w~PL, stiff to hard	182.94 / 5.56				
6			7	SS	20	
7						
8			8	SS	22	
9			9	SS	14	MH
10						

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp —○W— Wl : 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: YS

CHECKED: AMP

PROJECT: 1664178 (2000)

LOCATION: N 4856583.95; E 636055.42

RECORD OF BOREHOLE: ETR-1

BORING DATE: November 21, 2018

DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT / REMARKS	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
10	Power Auger; 216 mm O.D. Hollow Stem Augers	(CL-ML/CL) CLAYEY SILT and SAND to SILTY CLAY, some sand, trace to some gravel; brown to grey, (TILL); cohesive, w<PL to w~PL, stiff to hard						
11				10	SS	19		
12		- Augers grinding from 11.9 m to 12.2 m depth		11	SS	66		
13		- Augers grinding from 12.8 m to 13.4 m depth						
14		- Becoming grey at 13.7 m depth		12	SS	14		
		(CL) SILTY CLAY, trace sand, trace gravel; grey; cohesive, w>PL, stiff	173.87 / 14.63					
15								21-Nov-2018
				13	SS	10		
		END OF BOREHOLE	172.80 / 15.70					

NOTES:

1. Borehole open upon completion of drilling.

2. Groundwater measured in open borehole at a depth of 15.0 m below ground surface (Elev. 173.5 m) upon completion of drilling.

3. NP = Non-plastic

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ●; rem V. ⊕ U - ○; 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp —○W— Wl; 10 20 30 40
ADDITIONAL LAB. TESTING

DEPTH SCALE 1 : 50

GOLDER

LOGGED: YS

CHECKED: AMP

RECORD OF BOREHOLE: ETR-2

PROJECT: 1664178 (2000)
LOCATION: N 4856684.60; E 635999.62
BORING DATE: November 20, 2018
SHEET 1 OF 2
DATUM: Geodetic

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm
HAMMER TYPE: AUTOMATIC

BORING METHOD: Power Auger — 216 mm O.D. Hollow Stem Augers

DEPTH (m)	DESCRIPTION	ELEV. / DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT (%)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	187.90						
0	ASPHALT (155 mm)	0.00						50 mm Diameter PVC Monitoring Well (Flushmount)
	FILL - (SP) gravelly SAND; brown; non-cohesive, dry	0.16	1	AS	-			
	FILL - (SP) SAND, some gravel, some fines; brown; non-cohesive, dry	187.31 / 0.59	2	AS	-			
1	FILL - (ML) SILT and SAND, trace to some gravel, trace to some plastic fines; brown; cohesive, w<PL to w~PL, compact	187.03 / 0.87						
			3	SS	16			Cuttings
2–3			4	SS	22			
3			5	SS	16	~15	NP	
4–5			6	SS	27			Bentonite
6			7	SS	11			
	- Augers grinding at 6.7 m depth							
7	(CL-ML/CL) CLAYEY SILT and SAND to SILTY CLAY, some sand, some gravel; brown, (TILL); cohesive, w~PL to w>PL, very stiff	180.81 / 7.09						
8			8	SS	15	~9	MH	
	- Augers grinding between 8.2 m and 8.5 m depth							
9			9	SS	19			
10	*CONTINUED NEXT PAGE*							

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80
SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80
HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}
WATER CONTENT PERCENT: Wp —○W— Wl : 10 20 30 40

DEPTH SCALE 1 : 50

GOLDER

LOGGED: YS
CHECKED: AMP

GTA-BHS 001 S:\CLIENTS\REGION_OF_YORK\KENNEDY_ROAD\02_DATA\GINT\KENNEDY_ROAD.GPJ GAL-MIS.GDT 3/21/19

PROJECT: 1664178 (2000)

LOCATION: N 4856684.60; E 635999.62

RECORD OF BOREHOLE: ETR-2

DATUM: Geodetic

BORING DATE: November 20, 2018

SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. / DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - ○ (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp —○W— Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- *CONTINUED FROM PREVIOUS PAGE* ---									
10	Power Auger; 216 mm O.D. Hollow Stem Augers	(CL-ML/CL) CLAYEY SILT and SAND to SILTY CLAY, some sand, some gravel; brown, (TILL); cohesive, w~PL to w>PL, very stiff									Bentonite
					10	SS	21				13-Dec-18
11		- Shale fragments at 11.0 m depth									
12											Silica Sand
					11	SS	19				
13											
		- Becoming grey at 13.7 m depth									Screen
14					12	SS	11				
				173.27 / 14.63							
15		(CL) SILTY CLAY, trace sand, trace gravel; grey; cohesive, w>PL, stiff									
					13	SS	9				
		END OF BOREHOLE		172.20 / 15.70							

NOTES:

1. Water level measured in monitoring well as follows:

Date	Depth (m)	Elev. (m)
30-Nov-18	10.8	177.1
13-Dec-18	10.8	177.1

2. NP = Non-plastic

16 17 18 19 20

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DEPTH SCALE

1 : 50

GOLDER

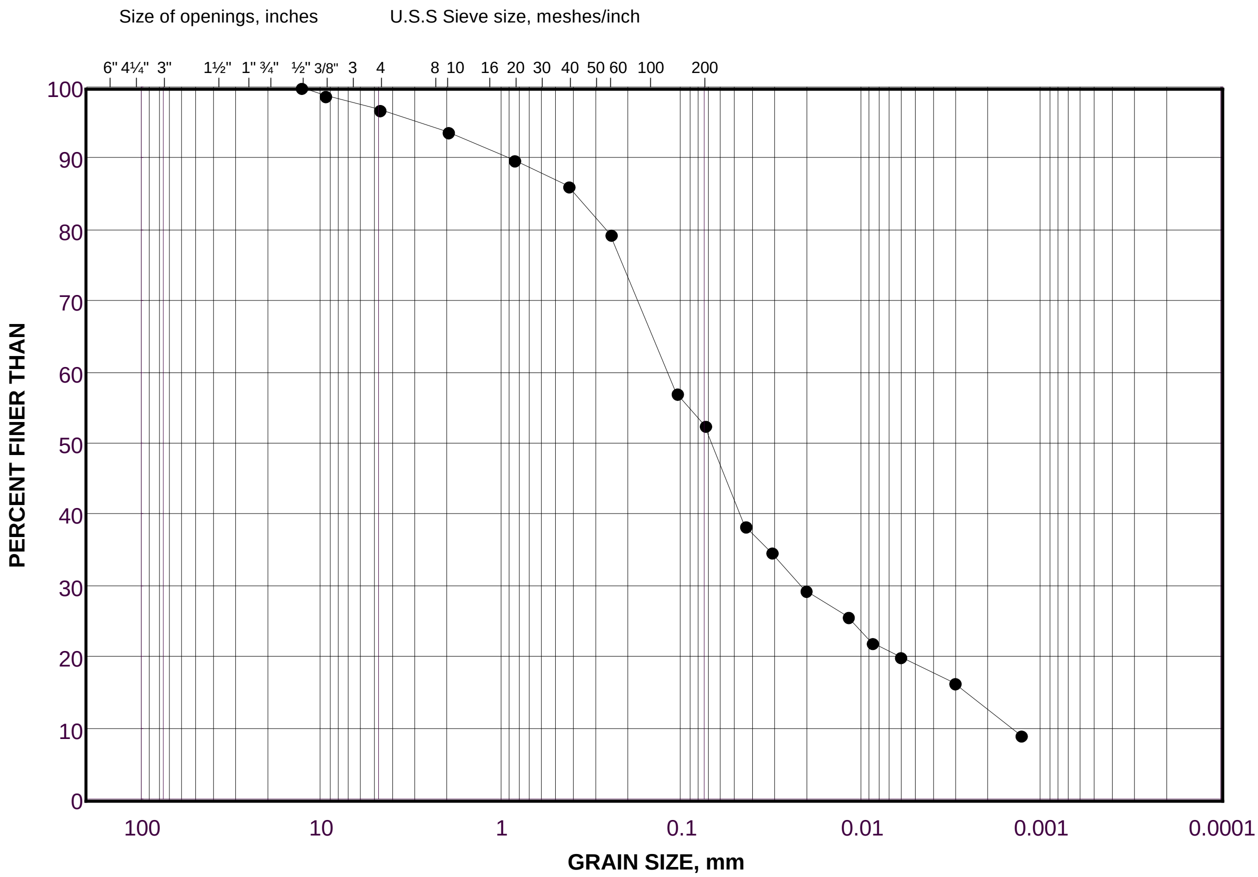
LOGGED: YS

CHECKED: AMP

APPENDIX B

Geotechnical Laboratory Results

GRAIN SIZE DISTRIBUTION FILL- (ML) SILT and SAND	FIGURE B-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	ETR-1	6	183.7

Project Number: 1664178 (2000)

Checked By: AMP

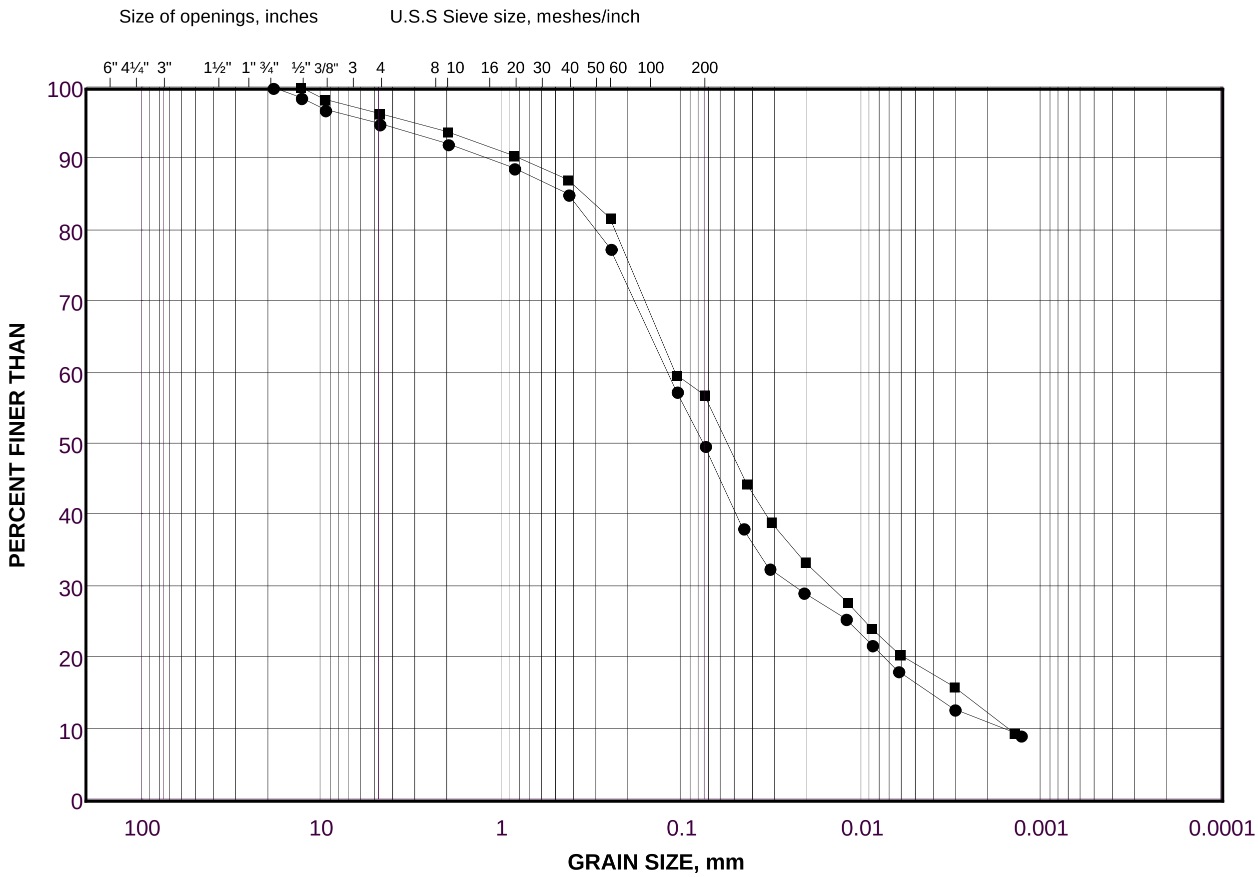
Golder Associates

Date: 25-Jan-19

GRAIN SIZE DISTRIBUTION

(CL-ML) CLAYEY SILT and SAND (TILL)

FIGURE B-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	ETR-2	8	180.1
■	ETR-1	9	179.1

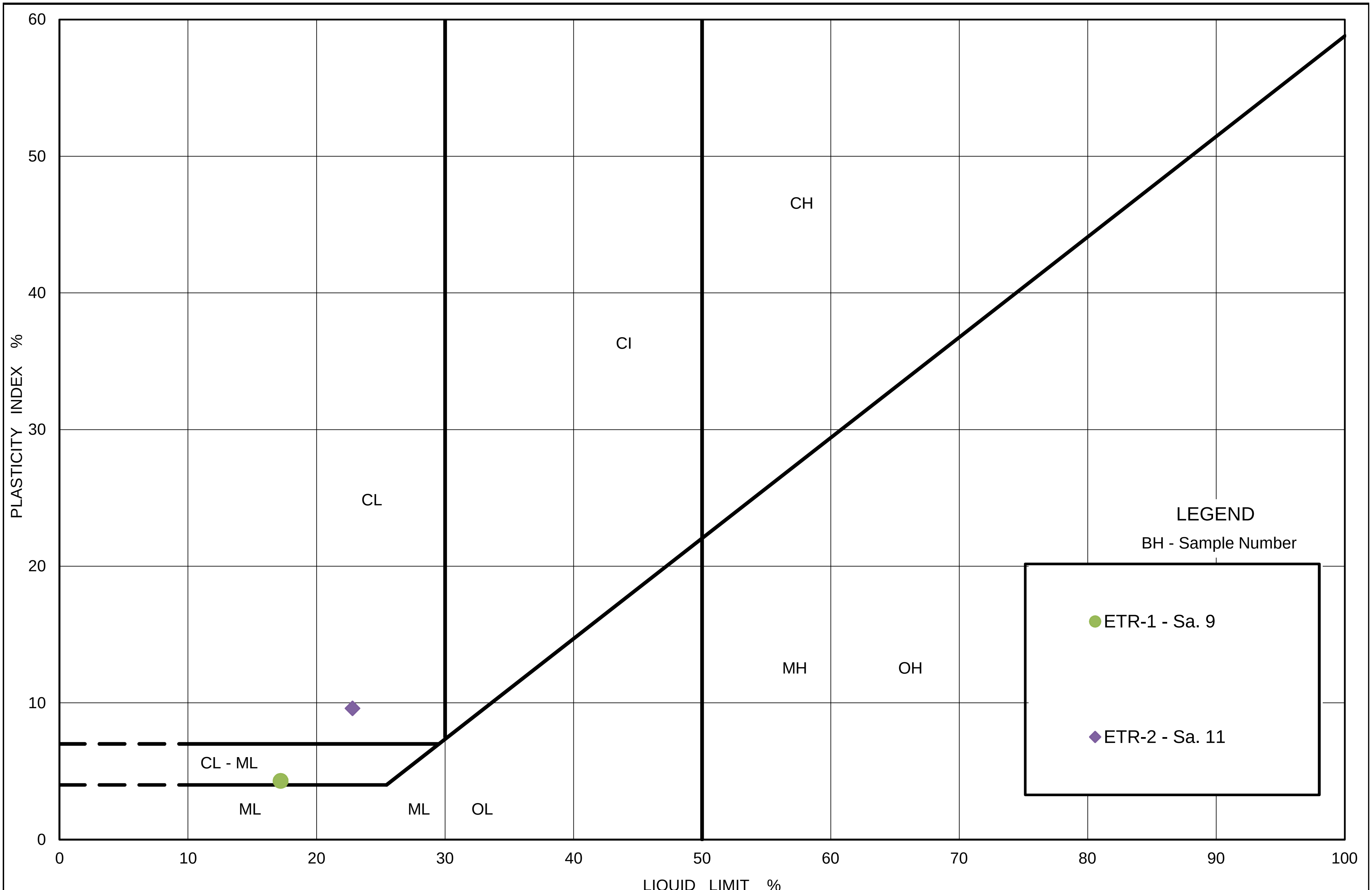
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 28-Jan-19

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ASTM D4318)



GOLDER

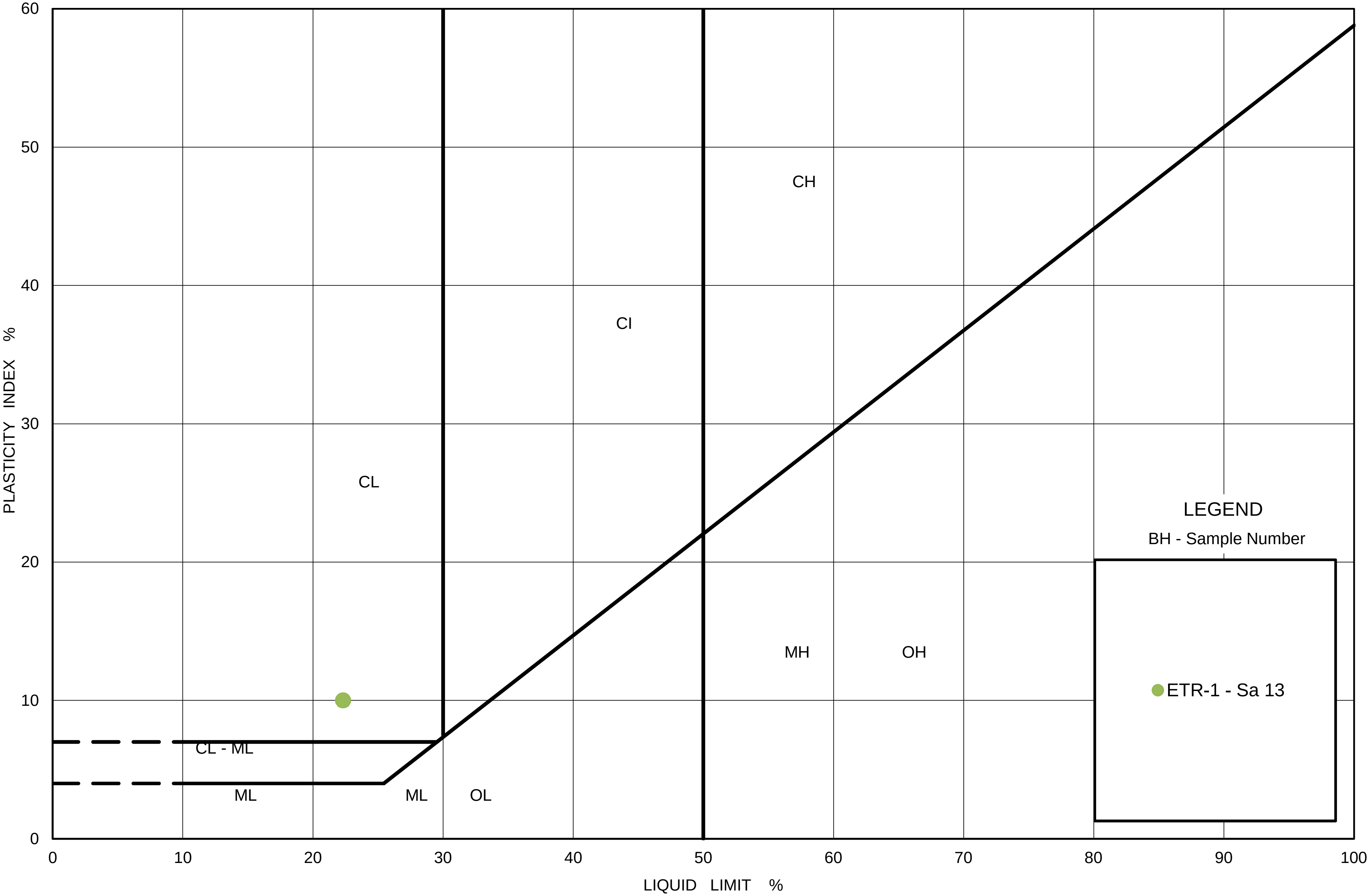
PLASTICITY CHART
(CL-ML) CLAYEY SILT and SAND (TILL)
to (CL) SILTY CLAY (TILL)

Figure No.: B-3

Project No.: 1664178 (2000)

Checked By: AMP

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ASTM D4318)



GOLDER

PLASTICITY CHART

(CL) SILTY CLAY

Figure No.: B-4

Project No.: 1664178 (2000)

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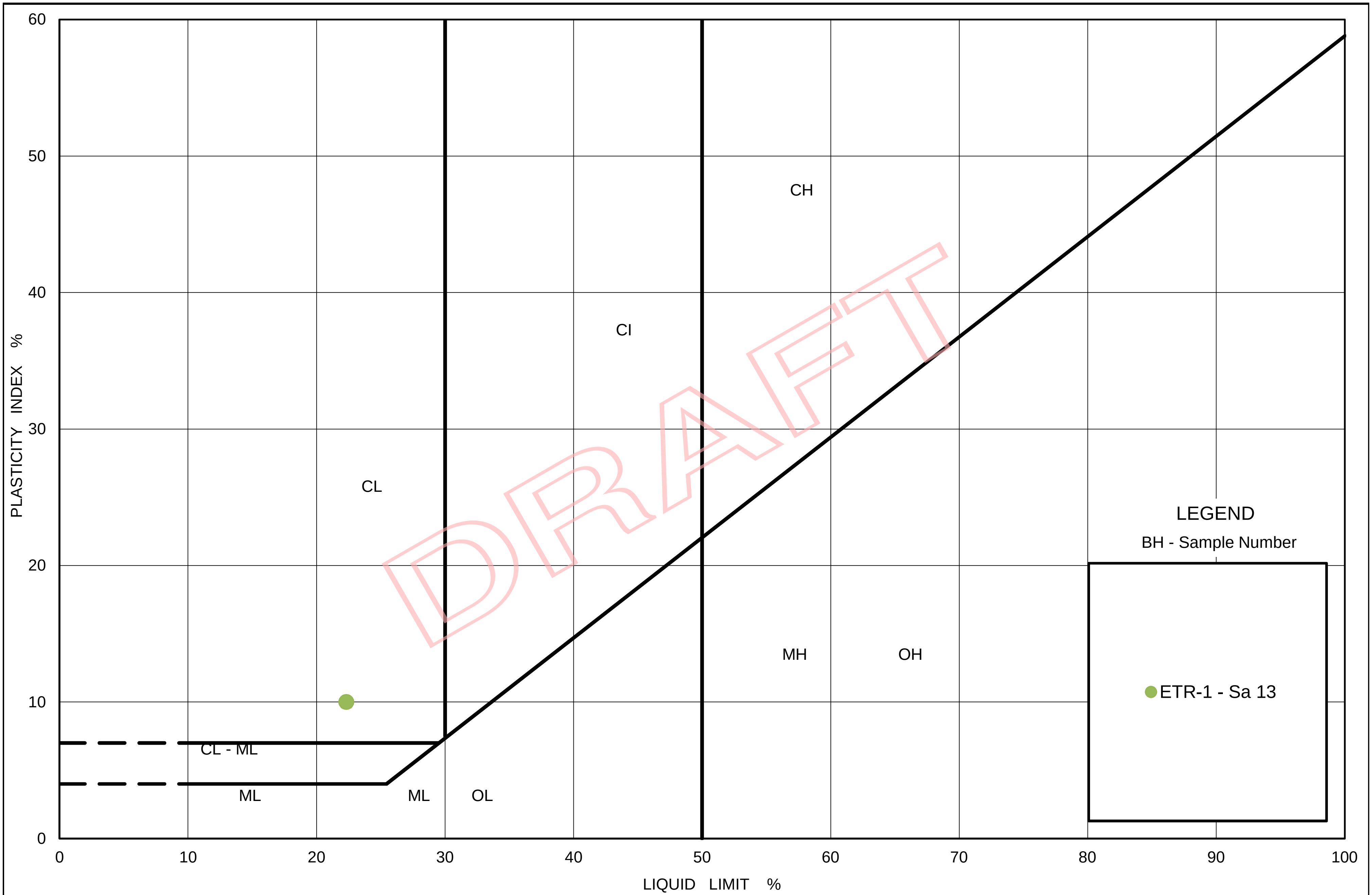
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ASTM D4318)



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PLASTICITY CHART

(CL) SILTY CLAY

Figure No.: B-4

Project No.: 1664178 (2000)

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