Appendix M.7 – Foundations Report – Tributary Culvert

Kennedy Road Environmental Assessment between Steeles Avenue and Major Mackenzie Drive

Preliminary Foundation Investigation and Design Report

Kennedy Road - Rouge River Tributary Crossing
Class Environmental Assessment Study for Improvements to Kennedy Road from Steeles Avenue to Major Mackenzie Drive, Markham, Ontario

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PART A

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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, in the Regional Municipality of York, Ontario. As part of this project, a foundation investigation was carried out for multiple structures along Kennedy Road between Steeles Avenue and Major Mackenzie Drive, including the Canadian National (CN) Rail bridge, 407 Express Toll Route bridge, a tributary culvert, and Rouge River bridge, 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 the Rouge River Tributary Culvert on Kennedy Road.

The purpose of the investigation was to evaluate the subsurface soil and groundwater conditions at the Tributary Culvert by means of a limited number of boreholes and, based on our interpretation of the data, to provide preliminary foundation 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 culvert structure at the Rouge River tributary is located about 190 m southwest of Unionville Gate and consists of twin 900 mm by 1430 mm corrugated steel pipe (CSP) arch culverts crossing beneath Kennedy Road in the City of Markham, in the Regional Municipality of York, Ontario, as shown on the Key Plan on Figure 1. At the tributary culvert location, Kennedy Road consists of six lanes of traffic with four northbound lanes and two southbound lanes. Wide boulevards are present along with sidewalks on both sides of Kennedy Road. Commercial developments surround the site. The grade of Kennedy Road in the vicinity of the culvert is about Elevation 175 m and the surrounding lands are general flat. Based on information provided by HDR, the existing culvert inverts are at about Elevation 173 m.

3.0 INVESTIGATION PROCEDURES

The field work for the preliminary investigation was carried out on November 27 and December 7, 2018, during which time two boreholes (designated as Boreholes TC-1 and TC-2) were advanced near the existing Rouge River tributary culvert to depths between 14.9 m and 15.2 m below the existing 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 Mobile B60 drill rig, supplied and operated by Landshark Drilling of Brantford, Ontario. The boreholes were advanced through the overburden using 216 mm outer 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). 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 the in-situ tests (i.e., SPT "N" values) as presented on the borehole records and throughout this report are uncorrected.

Groundwater conditions were noted during drilling and immediately following drilling operations. A monitoring well was installed in Borehole TC-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. The remaining borehole 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 testing) was carried out on selected soil samples, to ASTM Standards.

The borehole location and ground surface elevation of TC-2 was 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 2 are 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. The borehole location and ground surface elevation at TC-1 was not surveyed and as such, the ground surface elevation was determined using a topographic plan drawing received and dated March 30, 2018 from HDR.

Table 1: Borehole Coordinates, Ground Surface Elevation and Depth

Borehole No.	**Location (UTM NAD 83)**		**Ground Surface Elevation (m)**	**Borehole Depth (m)**
	Northing (m)	Easting (m)		
TC-1	4857095.28	636094.93	175.0	15.2
TC-2	4857126.44	636095.61	174.9	14.9

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). The Kennedy Road and Rouge River tributary culvert 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 boreholes records in Appendix A. Also included are the "*Method of Soil Classification*", "*Terms Used on the Record of Boreholes and Test Pits*" and "*List of* Symbols" 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 generally consist of asphalt and road base granular fill underlain by a deposit of which is subsequently underlain by a till deposit that varies in composition from silty sand to clayey silt and sand. Interlayers of silt and sand and gravelly sand were encountered within the till deposit in Borehole TC-1. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt/Fill

A 205 mm and 180 mm thick layer of asphalt was encountered at ground surface in Boreholes TC-1 and TC-2 at Elevation of 175.0 m and 174.9 m, respectively.

Non-cohesive gravelly sand fill was encountered underlying the asphalt in both boreholes. The fill extends to depths of 1.5 m and 0.9 m below ground surface (Elevation 173.5 m and 174.0 m) in Boreholes TC-1 and TC-2, respectively.

The SPT "N" values measured within the fill range between 18 blows and 47 blows per 0.3 m of penetration, indicating a compact to dense level of compaction. In situ moisture contents measured in the fill range between 3 per cent and 6 per cent.

4.2.2 Silty Clay

A 5.7 m to 6.3 m thick deposit of silty clay containing some sand to sandy was encountered underlying the non-cohesive fill at depths of 1.5 m and 0.9 m below ground surface and extended to a depth of 7.2 m below ground surface (Elevation 167.8 m and 167.7 m) in Boreholes TC-1 and TC-2, respectively.

The SPT "N" values measured within the silty clay deposit range from 19 blows to 1 blow per 0.3 m of penetration, generally decrease with depth, and indicating a very stiff to very soft consistency.

The results of grain size distribution testing carried out on one sample of the silty clay deposit are shown on Figure B-1 in Appendix B. Atterberg limits testing was carried out on one sample of silty clay deposit and measured a liquid limit of about 20 per cent, a plastic limit of about 12 per cent, and a corresponding plasticity index of about 8 per cent. These test results, plotted on a plasticity chart on Figure B-2 in Appendix B, indicate that the silty clay has low plasticity.

The natural water contents measured on samples of the silty clay deposit range from about 14 per cent to 30 per cent.

4.2.3 Silt and Sand

In Borehole TC-1, underlying the silty clay deposit, a 1.5 m thick layer of silt and sand was encountered at a depth of about 7.2 m below ground surface and extended to a depth of about 8.7 m (Elevation 166.3 m).

One SPT "N" value measured within the silt and sand deposit is 7 blows per 0.3 m of penetration, indicating a loose level of compaction.

The results of grain size distribution testing carried out on one sample of the silt and sand deposit are shown on Figure B-3 in Appendix B. Atterberg limits testing was carried out on one sample of the silt and sand deposit and returned a non-plastic result. The natural water content measured on one sample of the deposit is about 18 per cent.

4.2.4 Gravelly Silty Sand to Clayey Silt and Sand (Till)

A 4.6 m and 7.8 m thick till deposit varying in composition from gravelly sand silt to clayey silt and sand was encountered underlying the silt and sand deposit in Borehole TC-1 at a depth of 8.7 m below ground surface and underlying the silty clay deposit in Borehole TC-2 at a depth of 7.2 m below ground surface (Elevation 167.7 m). In Borehole TC-1, the till deposit was fully penetrated to a depth of 13.3 m below ground surface (Elevation 161.7 m) while Borehole TC-2 was terminated within the till deposit at a depth of 14.9 m below ground surface (Elevation 159.9).

The SPT "N" values measured within the cohesive till range between 10 blows and 38 blows per 0.3 m of penetration, indicating a stiff to hard consistency. The SPT "N" values measured within the non-cohesive till range between 21 blows and 86 blows per 0.3 m of penetration with one "N" value of 100 blows per 0.15 m of penetration, indicating a compact to very dense level of compaction.

The results of grain size distribution testing carried out on three samples of the till deposit are shown on Figure B-4 in Appendix B. Atterberg limits testing was carried out on two samples of the till deposit. The results of one cohesive sample indicated a liquid limit of about 16 per cent, a plastic limit of about 11 per cent, and a corresponding plasticity index of about 5 per cent, indicating the clayey silt has slight plasticity. The other tested sample returned a non-plastic result.

The natural water contents measured on samples of the till deposit range from about 7 per cent to 21 per cent.

4.2.5 Gravelly Sand

In Borehole TC-1, underlying the till deposit, a 2.1 m thick layer of gravelly sand was encountered at a depth of 13.3 m below ground surface and extended to the borehole termination depth of 15.2 m below ground surface (Elevation 159.8 m).

One SPT "N" value measured within the gravelly sand deposit is 48 blows per 0.3 m of penetration, indicating a dense level of compaction.

The results of grain size distribution testing carried out on one sample of the gravelly sand deposit are shown on Figure B-6 in Appendix B. The natural water content measured on one sample of the deposit is about 9 per cent.

4.2.6 Groundwater Conditions

Details of the water levels observed in the boreholes upon completion of drilling are summarized on the borehole records in Appendix A. The overburden samples obtained from the boreholes were generally moist, with the exception of a sample from the silt and sand in Borehole TC-1, which was wet. Groundwater level observations upon completion of drilling measured water in the open portion of Borehole TC-1 and was at a depth of 5.5 m below ground surface.

In Borehole TC-1, ‘heaving’ or ‘blowing sands’ were encountered inside the augers while advancing the augers through the gravelly sand deposit between 13.3 m and 15.2 m below ground surface. Therefore, the gravelly sand deposit is under hydrostatic pressure.

A monitoring well was installed in Borehole TC-2 and sealed within the sandy silty clay and gravelly silty sand till deposits. The recorded groundwater levels are summarized in Table 2.

Table 2: Depth and Elevation of Measured Groundwater Level

Borehole Number	Screened Stratigraphy	Depth(m)	Elevation (m)	Date of Monitoring Well Reading
TC-2	Sandy Silty Clay and Gravelly Silty Sand Till	3.5	171.4	November 27, 2018
		5.7	169.2	November 29, 2018
		4.6	170.3	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 Mr Michael Bentley, M.A.Sc., and was reviewed Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
KENNEDY ROAD – ROUGE RIVER TRIBUTARY CROSSING
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the preliminary design of the widening of the existing Rouge River Tributary Culvert associated with the proposed improvements to Kennedy Road in the City of Markham, Regional Municipality 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 culvert and foundation alternatives, for planning purposes.

Further investigations will be required during Detailed Design to obtain subsurface information specific to the culvert extensions or replacement 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 existing culvert at the Rouge River tributary consists of twin 900 mm by 1430 mm corrugated steel pipe (CSP) arch culverts crossing beneath Kennedy Road. Based on information provided by HDR, the existing culvert invert is assumed to be at about Elevation 173 m, and water flows from the southeast to northwest. The pavement surface of Kennedy Road is at about Elevation 175 m, with a resultant about 1 m of soil cover over the culvert.

It is understood that as part of the improvements to Kennedy Road, replacement and/or extension of the existing culvert is anticipated to be required to accommodate the future widening of Kennedy Road. The proposed road widening method (i.e., asymmetrical or symmetrical) is currently not known. Based on the existing road and culvert geometry and the subsurface conditions at the site, the following culvert types and foundation types are considered for preliminary design.

- Provided the existing pipes have sufficient hydraulic capacity and are in good condition, the preferred culvert option is to extend the existing CSP arch culverts with matching pipes to accommodate the widening.
- If consideration is given to full replacement of the structure, CSP pipe(s), box culvert(s) or an open footing culvert with wingwalls are considered feasible culvert options supported on shallow foundations.
- It is not recommended to extend the existing pipe culverts with box culverts or open footing culverts due to the difficulties with connecting the pipes to the concrete elements.
- Deep foundations are not considered practical or necessary from a foundation perspective as this may create unacceptable differential settlement between the existing and new widened structure elements and are also not economical for a culvert of this size.

The following sections provide preliminary recommendations for the shallow or deep foundation options for extension/replacement of the Rouge River tributary culverts to support the proposed widening along Kennedy Road.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code and its Commentary* (CHBDC 2014), the 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 “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 Geotechnical Resistance

6.3.1 Open Footing Culvert

The underside of strip footings for open footing culverts or wing walls should be founded at a minimum depth of 1.4 m below the lowest surrounding grade to provide adequate protection against frost penetration, as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Depths for Southern Ontario*). In addition, the footings should extend below any existing fill, surficial organic materials, or loose/soft soils, where present. The fill extends to approximately Elevation 173.5 m and Elevation 174.0 m, respectively in Boreholes TC-1 and TC-2, respectively; however, in order to provide the necessary frost protection at the culvert ends, the footing will need to extend to 1.4 m below the invert or approximately Elevation 171.5 m, which is up to 3.5 m below the road grade.

Strip footings placed on the exposed stiff silty clay subgrade should be designed based on the 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 3.

Table 3: Geotechnical Resistances for Open Footing Culverts

Highest Founding Elevation (m)	Founding Material	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
171.5	Stiff Silty Clay	125	75

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 or less than that specified above or if the founding elevation differs from that given above.

6.3.2 Box Culverts

It is not necessary to found box or pipe culverts below the depth for frost protection, as the structures are tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur. Box or pipe culverts should, however, be founded below any existing fill / softened soils and surficial organic materials, at or below the founding level recommended in Table 4, following sub-excavation of unstable materials as may be required and backfilling with Ontario Standard Special Provision (OPSS).MUNI 1010 (*Aggregates*) Granular ‘A’ or ‘B’ Type II, based on a box culvert base slab thickness of 250 mm.

Box culverts having an assumed width of 3 m or 6 m, placed on the properly prepared subgrade with the appropriate bedding founded at or below the design elevations in Table 4, should be designed based on the factored ultimate geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) provided in Table 4:

Table 4: Geotechnical Resistances for Box Culverts

Box Culvert Width (m)	Highest Founding Elevation (m)	Founding Material	Factored Ultimate Geotechnical Resistance (kPa)	Factored Serviceability Geotechnical Resistance (kPa) (for 25 mm of Settlement)
3.0	172.5	Stiff Silty Clay	125	75
6.0	172.5	Stiff Silty Clay	175	50

6.3.3 Pipe Culverts

It is not necessary to find a pipe culvert at the standard depth for frost protection purposes, as such a culvert is tolerant to small magnitudes of movement related to freeze-thaw cycles, should these occur. The existing stiff silty clay is considered a suitable bearing surface for pipe culverts provided that they are founded below any existing fill / softened soils and surficial organic materials at or below Elevation 173.5 m, and have the appropriate bedding as discussed below.

6.3.4 General

The geotechnical resistances provided above are based on loading applied perpendicular to the surface of the culvert base slab. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *CHBDC* (2014).

The footing/box/pipe subgrade should be inspected by qualified geotechnical personnel to ensure that all existing fill / softened soils or other unsuitable material have been removed, in accordance with OPSS 902 (*Excavating and Backfilling-Structures*) and OPSS 422 (*Precast Reinforced Concrete Box Culverts in Open Cut*), as applicable. Following inspection, any sub-excavated area should be backfilled with granular material meeting Granular ‘A’ or Granular ‘B’ Type II material that is placed and compacted in accordance with OPSS.MUNI 501 (*Compacting*).

The silty clay subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick, 20 MPa concrete working slab be placed within four hours following inspection and approval of the subgrade, to protect the subgrade from softening.

6.3.5 Settlement

It is understood that if the existing culverts are to be extended the ditch grade will be raised, by about 2 m, to match the widened road grade at the inlet and / or outlet of the culvert, depending on whether a symmetrical or asymmetrical widening is selected. The settlement of the foundation soils under approximately 2 m of additional fill placed to extend the existing slope is estimated to be less than 25 mm. This settlement will be differential between the existing and new culvert extensions. For these predicted settlements, settlement mitigation measures are not required; however, this should be re-assessed at the Detailed Design stage once the final grading plans have been determined.

6.3.6 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces / sliding resistance between the base slab of the new box culvert extensions and the subgrade should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). For the interface between pre-cast concrete box culvert sections and Granular ‘A’ levelling course, a coefficient of friction, tan δ, (unfactored) of 0.45 may be used in the design. For cast-in-place strip footings placed directly on the silty clay subgrade, a tan δ, (unfactored) of 0.30 may be used in the design.

6.3.7 Culvert Bedding, Backfill and Erosion Protection

Open footings are founded directly on the subgrade, so bedding is not required. Box culvert replacements and extensions should be provided with at least 150 mm of Granular ‘A’ material for bedding purposes. Alternatively a 100 mm thick, 20 MPa concrete working slab, and 75 mm thick Granular ‘A’ levelling course (or OPSS.MUNI 1002 (*Aggregates - Concrete*) concrete fine aggregate) should be placed on top of the concrete working slab to provide a “levelling course” for the pre-cast box culvert segments. Pipe culvert replacement / extension should be provided with at least 300 mm of Granular ‘A’ material for bedding purposes in accordance with OPSD 802.020 *(Flexible Pipe Arch Embedment and Backfill – Earth Excavation)*.

Backfill and cover for the culverts should be completed in accordance with OPSD 803.010 *(Backfill and Cover for Concrete Culverts)* or OPSD 802.020 *(Flexible Pipe Arch Embedment and Backfill – Earth Excavation).* Backfill to culvert walls should consist of Granular ‘A’ or Granular ‘B’ Type II fill.

The backfill and bedding should be placed and compacted in accordance with OPSS.MUNI 501 *(Compacting).* The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 400 mm. The culvert replacements or extensions should be designed for the full overburden and hydrostatic pressures and live load, assuming that the embankment fill above and/or surrounding the culverts has a unit weight of 22 kN/m^3 for Granular ‘A’, and 21 kN/m^3 for Granular ‘B’ Type II or select earth fill.

To prevent surface water from flowing either beneath the culverts (potentially causing undermining and scouring) or around the culverts (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal should be provided at the upstream end of open footing culverts. Clay seals should also be placed adjacent to the culvert inlet opening. The clay material should meet the requirements of OPSS.MUNI 1205 *(Material Specification for Clay Seal)*. The clay seal should have a thickness of 1 m, and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high-water level including treatment of the adjacent side slopes. Alternatively, a clay blanket may be constructed, extending upstream to a distance equal to three times the culvert height, and extending along the adjacent side slopes to a height of two times the culvert height or the high-water level, whichever is higher.

If the water flow velocities are sufficiently high under the base or design storm condition(s), a provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlets and outlets, including in front of any wing walls/retaining walls adjacent to the water channel. The requirements for and design of erosion protection measures for the culvert inlets should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlets should be consistent with the standard Treatment Type A presented in OPSD 810.010 *(Rip-Rap Treatment for Sewer and Culvert Outlets),* with rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket if adopted, including the side slopes and embankment fill slope adjacent to the culverts.

6.4 Lateral Earth pressures for Design of Culvert Walls and Wing Walls

The lateral earth pressures acting on the culvert walls and headwalls / wingwalls, if applicable, will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account 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 803.010 (*Backfill and Cover for Concrete Culverts*) for box culverts, OPSD 803.031 (*Frost Treatment Pipe Culverts, Frost Penetration Line Between Top of Pipe and Bedding Grade*) for a pipe culvert and 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 *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).

6.4.1 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) may be used assuming the use of earth fill or SSM for the general embankment fill in Table 5 below:

Table 5:Coefficients of Static Lateral Earth Pressure for a Restrained Wall

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) may be used in Table 6 below:

Table 6: Coefficients of Static Lateral Earth Pressure for an Unrestrained Wall

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.

6.5 Widened Embankments

The stability of the approximately 2 m high embankment slopes should be assessed during the Detailed Design phase once the configuration (based on structure layout and property restrictions) is available. The *CHBDC* (2014) requires a Factor of Safety of 1.33 (short-term/temporary) and 1.54 (long-term/permanent) for the stability of embankment slopes.

Widened embankments comprised of granular material or OPSS.MUNI 1010 (*Aggregates*) for Select Subgrade Material (SSM) and constructed at side slopes inclined at 2 horizontal to 1 vertical (2H:1V) will achieve the required minimum Factors of Safety. Alternatively, the widened embankments may be constructed of earth fill meeting the requirements of OPSS.MUNI 212 (*Borrow*); however, the side slopes must be constructed at 3H:1V or flatter in order to achieve the long-term Factor of Safety.

6.6 Construction Considerations

The following subsections identify future construction considerations that should be considered at this stage as they may impact the planning and design.

6.6.1 Temporary Open Cut Excavations

Temporary open cut excavations for the culvert extensions/replacement will be made through the existing non-cohesive fill and into the cohesive silty clay 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 would be classified as Type 3 soil and the cohesive silty clay would be classified as Type 3 soil above Elevation of 170 m and Type 4 soil below this elevation, according to the OHSA. Temporary excavations in Type 3 soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) from the trench bottom. In Type 4 soils, side slopes should be formed at no steeper than 3H:1V.

6.6.2 Groundwater and Surface Water Control

The groundwater level measured in the monitoring well installed in Borehole TC-2, screened in the silty clay/silty sand till deposits, ranged between depths of 5.7 m and 4.6 m bgs (Elevations 169.2 m and 170.3) across monitoring events. Foundation excavations (culverts and shallow footings) are anticipated to extend to as low as Elevation 171.5 m for an open footing culvert, about 1.5 m above the measured groundwater level. Therefore, active dewatering is not likely to be required.

Considering the relatively low permeability of the silty clay soils at the site, it is anticipated that water inflow from these layers can be handled by pumping from properly filtered sump pumps placed just below the base of the excavation prior to digging. Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation. All surface water should be directed away from the excavations to prevent ponding of water resulting in disturbance and weakening of the subgrade or granular backfill / bedding material.

Control of the surface water will be necessary for the construction of the culvert extensions/replacement to allow excavation and foundation construction to be carried out in dry conditions. Depending on the tributary flows at the time of construction, the surface water being conveyed by the existing culverts could bypass the culvert construction area by means of a temporary pipe, one of the existing pipes, or be diverted by pumping from behind a temporary barrier (cofferdam).

6.6.3 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 culvert.

These 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.

The selection and design of the temporary protection system will be the responsibility of the contractor. Parameters for lateral earth pressure coefficients should be provided at the Detailed Design stage.

It must be emphasized that if a trench liner box (certified by an engineer) is used instead of shoring, it provides protection for construction personnel only and does not provide any lateral support for adjacent excavation walls, underground services or existing structures. It is imperative that underground services and existing structures adjacent to the trench excavations be accurately located prior to construction and adequate support provided where required as noted above.

Further, were trench boxes are utilized, the unsupported soils on the trench sides will relax, filling the void between the trench walls and trench box. This may lead to loss of ground below the adjacent pavement and potentially undermine and reduce the stability of the pavement structure adjacent to the open traffic lanes. To minimize this effect, the gap between the trench walls and trench box should be minimized during the excavation and trench box installation.

Excavated materials should be placed away from the edge of the excavation a distance equal to the depth of the excavation or greater. In addition, stockpiling of the material should be prohibited adjacent to the excavation to minimize surcharge loading near the excavation crest.

6.6.4 Obstructions

The glacial till soils at the site should be expected to contain cobbles and boulders, which could affect the installation of protection systems. Further observation is recommended in the next stage of investigation in support of the Detail Design.

6.6.5 Subgrade Protection

The native soils that will be exposed at the shallow 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 20 MPa concrete working slab be placed on the subgrade of foundation excavations within four hours after preparation, inspection and approval of the subgrade.

7.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION WORK DURING DETAILED DESIGN

An additional borehole investigation is recommended during the Detailed Design if the Rouge River tributary culverts are to be extended/replaced to accommodate the widening of Kennedy Road. The additional boreholes should be advanced within the footprint of the new / widened foundation elements to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report as follows:

- Assess the type and depth of fill present;
- Assess near surface soil deposits within the footprint of the widened embankments, to allow analysis of settlement, where applicable;
- Test parameters used to assess the corrosive potential of the soil to concrete and buried steel;
- Observe the presence of cobbles and/or boulders within the soil deposits to assess the presence of such obstructions as they may affect excavations and the installation of temporary protection systems

8.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Michael Bentley, M.A.Sc., and reviewed by Ms. Sarah E. M. Poot, P.Eng. a senior geotechnical engineer and Associate of Golder.

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.

Michael Bentley, M.A.Sc.	Sarah E. M. Poot, P.Eng.
Geotechnical Engineering Group	*Associate, Senior Geotechnical Engineer*

MJB/SEMP/cr;ljv;mes

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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.

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

ASTM International

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

Ontario Water Resources Act:

Ontario Regulation 903	Wells (as amended)

Ontario Provincial Standard Drawings (OPSD)

OPSD 802.020	Flexible Pipe Arch, Embankment and Backfill, Earth Excavation
OPSD 803.010	Backfill and Cover for Concrete Culverts with Spans Less Than or Equal to 3.0m
OPSD 803.031	Frost Treatment – Pipe Culverts, Frost Penetration Line Between Top of Pipe and Bedding Grade
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
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 212	Construction Specification for Borrow
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS.MUNI 501	Construction Specification for Compacting
OPSS.MUNI 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.MUNI 1010	Material Specification for Aggregate – Base, Subbase, Select Subgrade, and Backfill Material
OPSS.MUNI 1205	Material Specification for Clay Seal

Ontario Occupational Health and Safety Act

Ontario Regulation 213/91 Construction Projects (as amended)

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.

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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.

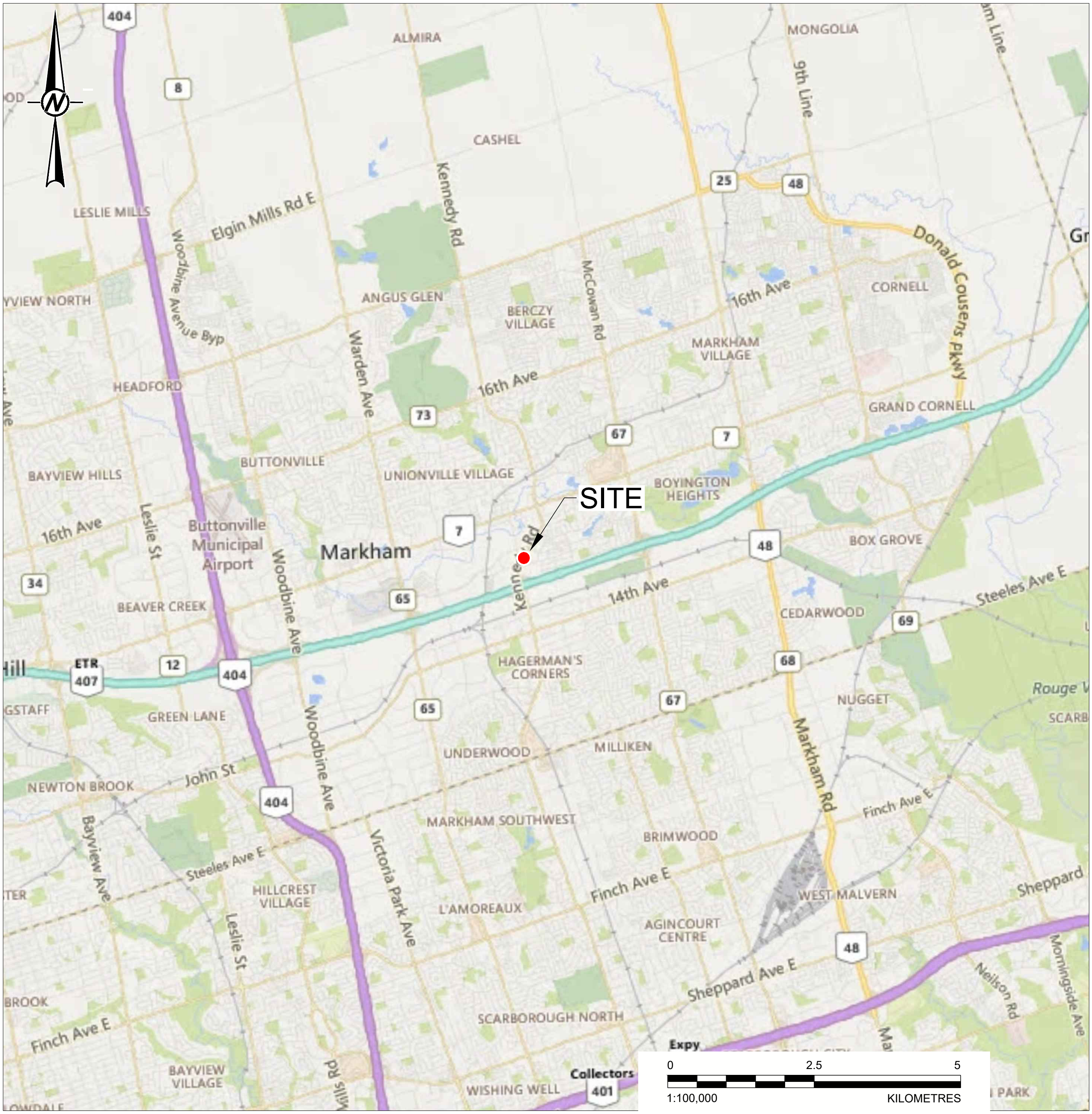
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater 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.



CLIENT
HDR INC.

PROJECT
KENNEDY ROAD AT TRIBUTARY CROSSING
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

TITLE
KEY PLAN

REFERENCE(S)
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PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17N

CONSULTANT
GOLDER

YYYY-MM-DD	2019-03-21
DESIGNED	
PREPARED	STB
REVIEWED	AMP
APPROVED	SEMP

PROJECT NO.	CONTROL	REV.	FIGURE
1664178 (2000)	0006	A	1

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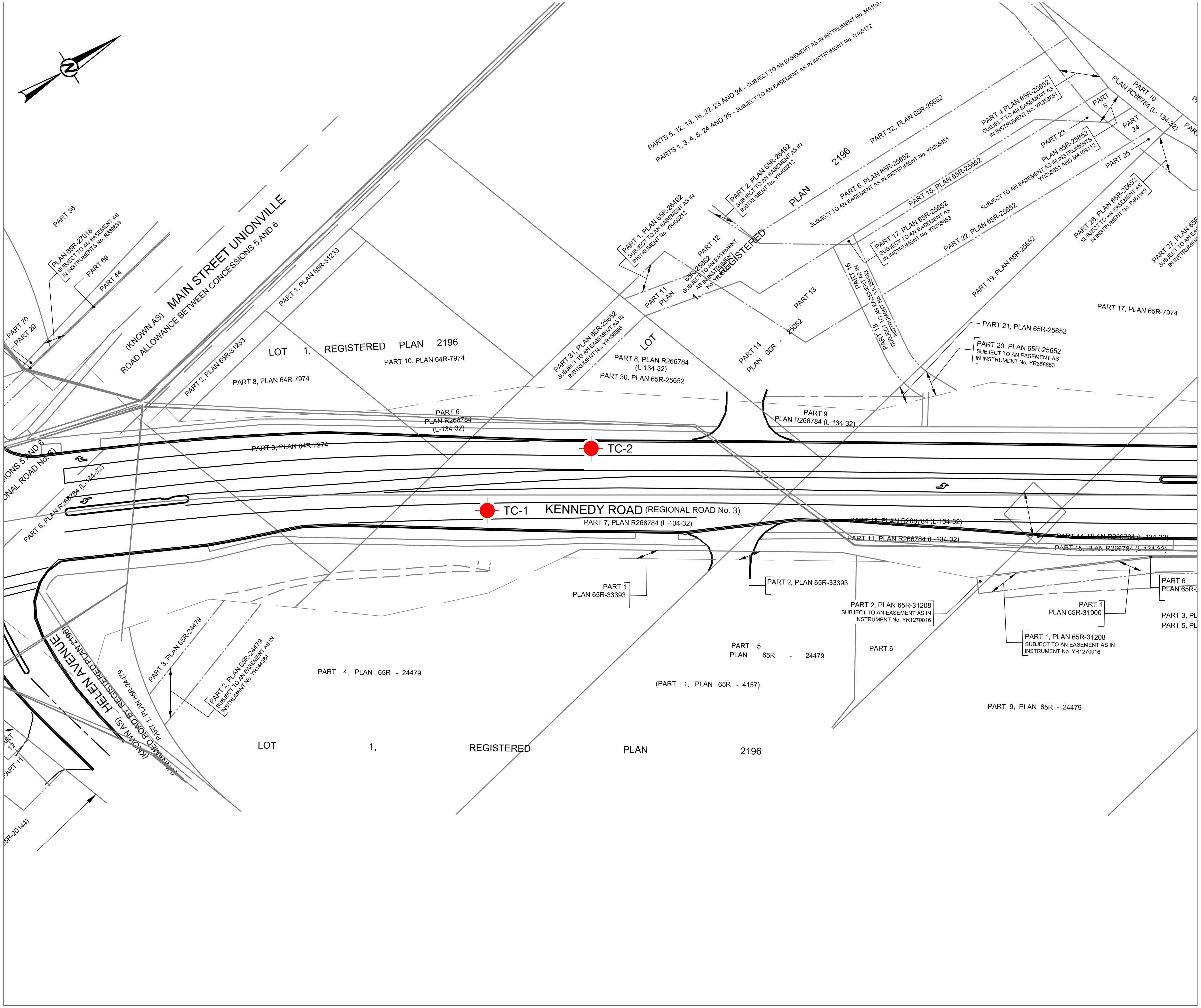

LEGEND

● BOREHOLE LOCATION

0 25 50

1:1,000 METRES

REFERENCE(S)

BASE DATA PROVIDED BY HDR CORPORATION, THAM SURVEYING LIMITED ONTARIO LAND SURVEYORS, OBTAINED DECEMBER, 2018.

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PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83 COORDINATE SYSTEM: UTM ZONE 17N

CLIENT

HDR INC.

PROJECT

KENNEDY ROAD AT TRIBUTARY CROSSING
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

TITLE

BOREHOLE LOCATION PLAN

CONSULTANT

GOLDER

YYYY-MM-DD	2019-03-21
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PREPARED	STB
REVIEWED	AMP
APPROVED	SEMP

PROJECT NO.	CONTROL	REV.	FIGURE
1664178 (2000)	0006	A	2

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APPENDIX A

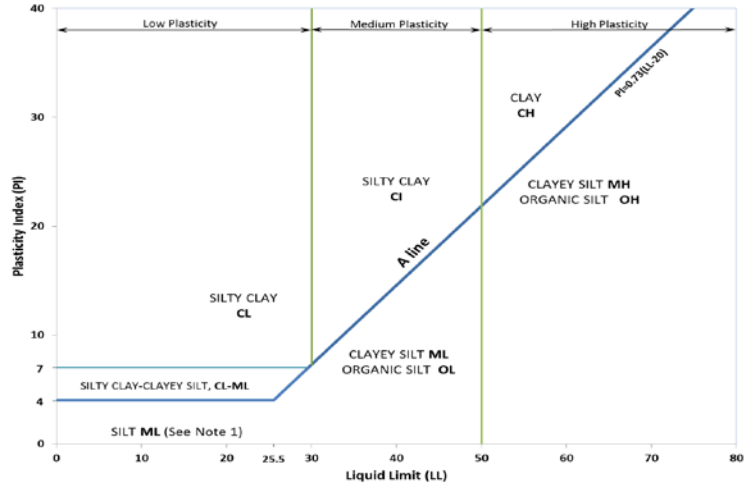
Records of Borehole Sheets

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					Organic Content	USCS Group Symbol	Primary Name
				Dilatancy	Dry Strength	Shine Test	Thread Diameter	Toughness (of 3 mm thread)			
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 ($\phi = 0$ analysis)
p	mean total stress ($\sigma_1 + \sigma_3$)/2
p′	mean effective stress ($\sigma'_1 + \sigma'_3$)/2
q	($\sigma_1 - \sigma_3$)/2 or ($\sigma'_1 - \sigma'_3$)/2
q_u	compressive strength ($\sigma_1 - \sigma_3$)
S_t	sensitivity

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

PROJECT: 1664178 (2000)

LOCATION: N 4857095.28; E 636094.93

RECORD OF BOREHOLE: TC-1

BORING DATE: December 7, 2018

SHEET 1 OF 2

DATUM: Geodetic

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

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	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
		GROUND SURFACE	175.00							
0	Power Auger 216 mm O.D. Hollow Stem Augers	ASPHALT (205 mm)	0.00 174.80							
		FILL - (SP) gravelly SAND; brown; non-cohesive, moist, compact to dense	0.20	1	SS	47				
1				2	SS	18				
		(CL) SILTY CLAY, trace sand to sandy, trace gravel; brown, oxidation staining; cohesive, w<PL, very stiff to very soft	173.53 1.47	3	SS	14				
2				4	SS	19				
3		- Oxidation staining from 2.9 m to 4.1 m depth		5	SS	12				
4										
5				6	SS	2				∇ 7-Dec-18
6				7	SS	2				
7		(ML) SILT and SAND, trace gravel; grey; non-cohesive, wet, loose	167.84 7.16							
8				8	SS	7		○	MH	
		(CL-ML) CLAYEY SILT and SAND, some gravel; grey, (TILL); cohesive, w<PL, stiff to hard	166.31 8.69							
9				9	SS	10				
10		*CONTINUED NEXT PAGE*								

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JS

CHECKED: AMP

PROJECT: 1664178 (2000)
LOCATION: N 4857095.28; E 636094.93

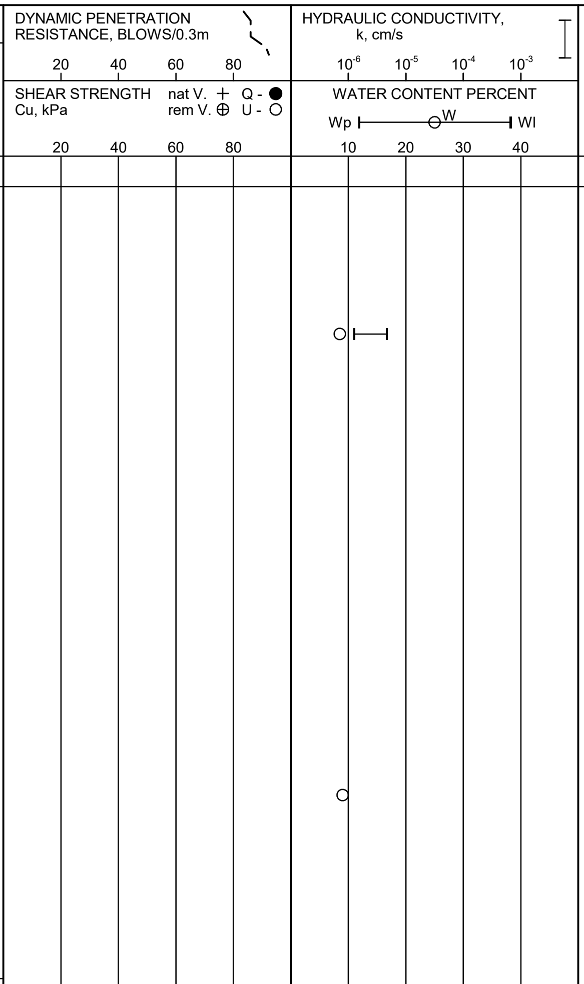
RECORD OF BOREHOLE: TC-1

BORING DATE: December 7, 2018

SHEET 2 OF 2
DATUM: Geodetic

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

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---						
10	Power Auger 216 mm O.D. Hollow Stem Augers	(CL-ML) CLAYEY SILT and SAND, some gravel; grey, (TILL); cohesive, w<PL, stiff to hard						
11				10	SS	38	MH	
12								
				11	SS	24		
13								
		(SP) gravelly SAND; grey; non-cohesive, wet, dense	161.74 13.26					
14				12	SS	48	MH	
15								
		END OF BOREHOLE	159.76 15.24					
16								
17								
18								
19								
20								



NOTES:

1. Groundwater measured in open portion of borehole at a depth of 5.5 m below ground surface upon completion of drilling.

2. Borehole caved to a depth of 7.8 m below ground surface upon completion of drilling.

3. Heaving sand encountered in augers at a depth of 15.2 m below ground surface during drilling.

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JS
CHECKED: AMP

PROJECT: 1664178 (2000)

LOCATION: N 4857126.44; E 636095.61

RECORD OF BOREHOLE: TC-2

BORING DATE: November 27, 2018

SHEET 1 OF 2

DATUM: Geodetic

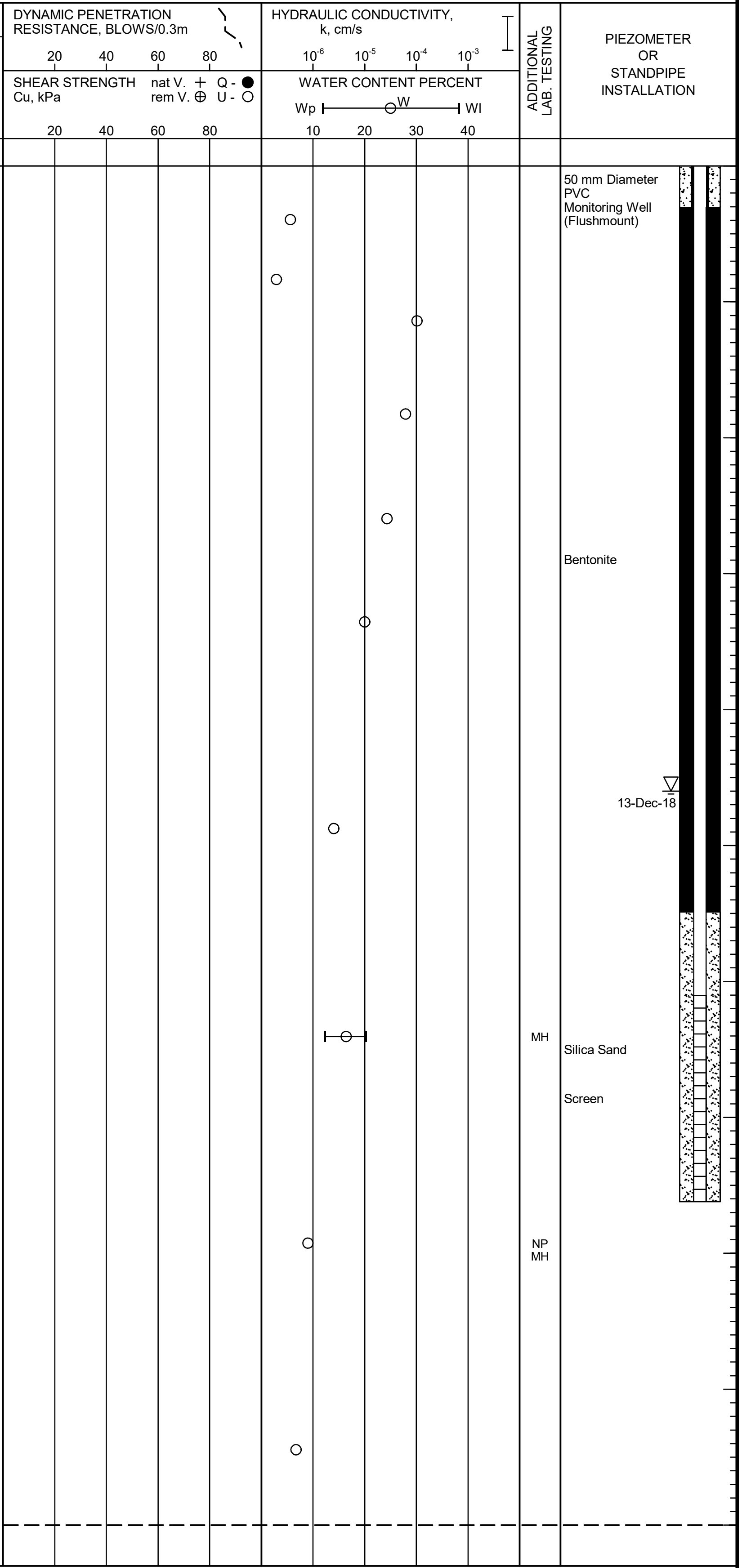
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

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

DESCRIPTION	ELEV. DEPTH (m)
GROUND SURFACE	174.88
ASPHALT (180 mm)	0.00 174.70
FILL - (SP) gravelly SAND, trace fines; brown; non-cohesive, moist, compact to dense	0.18
(CL) SILTY CLAY, trace sand to sandy, trace gravel; brown to grey, oxidation staining; cohesive, w>PL, stiff to very soft - Oxidation staining to 4.1 m depth - Becoming grey at 4.1 m depth	173.97 0.91
(SM) gravelly SILTY SAND; grey, (TILL); non-cohesive, moist, compact to very dense	167.72 7.16
CONTINUED NEXT PAGE	

NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	34	
2A			
2B	SS	11	
3	SS	10	
4	SS	14	
5	SS	9	
6	SS	5	
7	SS	1	MH
8	SS	24	NP MH
9	SS	46	



DEPTH SCALE

1 : 50

GOLDER

LOGGED: JS

CHECKED: AMP

PROJECT: 1664178 (2000)
LOCATION: N 4857126.44; E 636095.61

RECORD OF BOREHOLE: TC-2

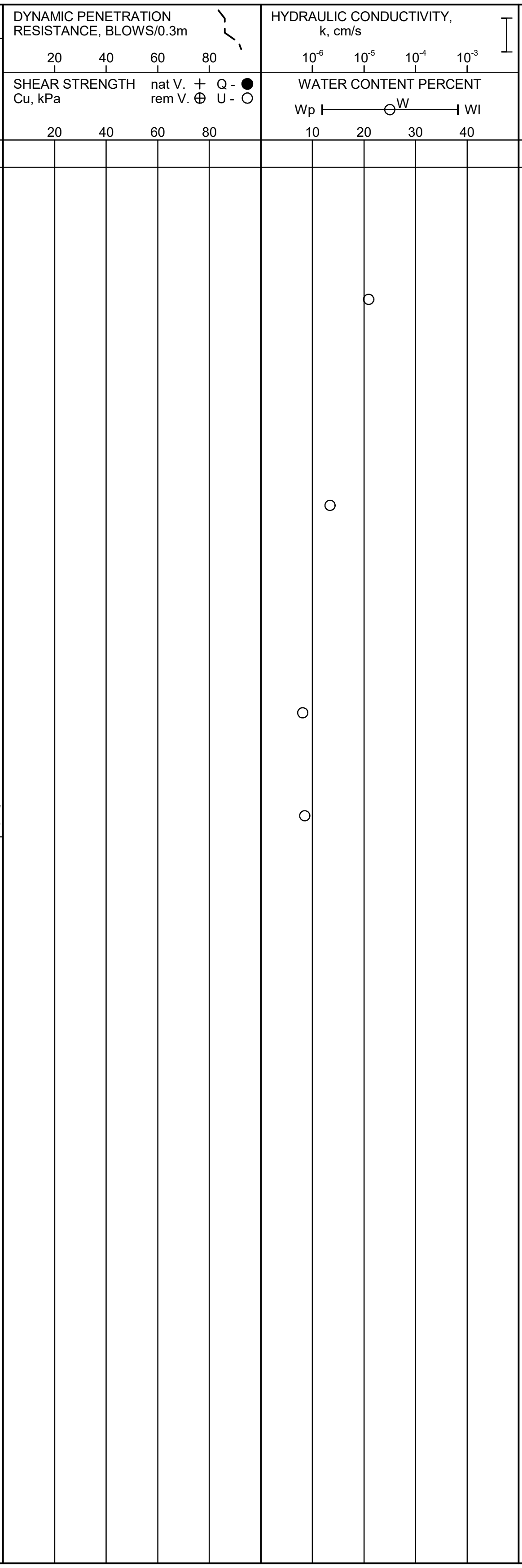
BORING DATE: November 27, 2018

SHEET 2 OF 2
DATUM: Geodetic

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

--- *CONTINUED FROM PREVIOUS PAGE* ---

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
10	Power Auger 216 mm O.D. Hollow Stem Augers	(SM) gravelly SILTY SAND; grey, (TILL); non-cohesive, moist, compact to very dense						
11				10	SS	23		
12				11	SS	21		
13								
14				12	SS	86		
			159.94 14.94	13	SS	100/0.15	MH	
15		END OF BOREHOLE						



NOTES:

1. Borehole caved to a depth of 12.8 m below ground surface upon completion of drilling.

2. Water level measured in monitoring well as follows:

Date	Depth (m)	Elev. (m)
27-Nov-18	3.5	171.4
29-Nov-18	5.7	169.2
13-Dec-18	4.6	170.3

3. NP = Non-plastic

DEPTH SCALE
1 : 50

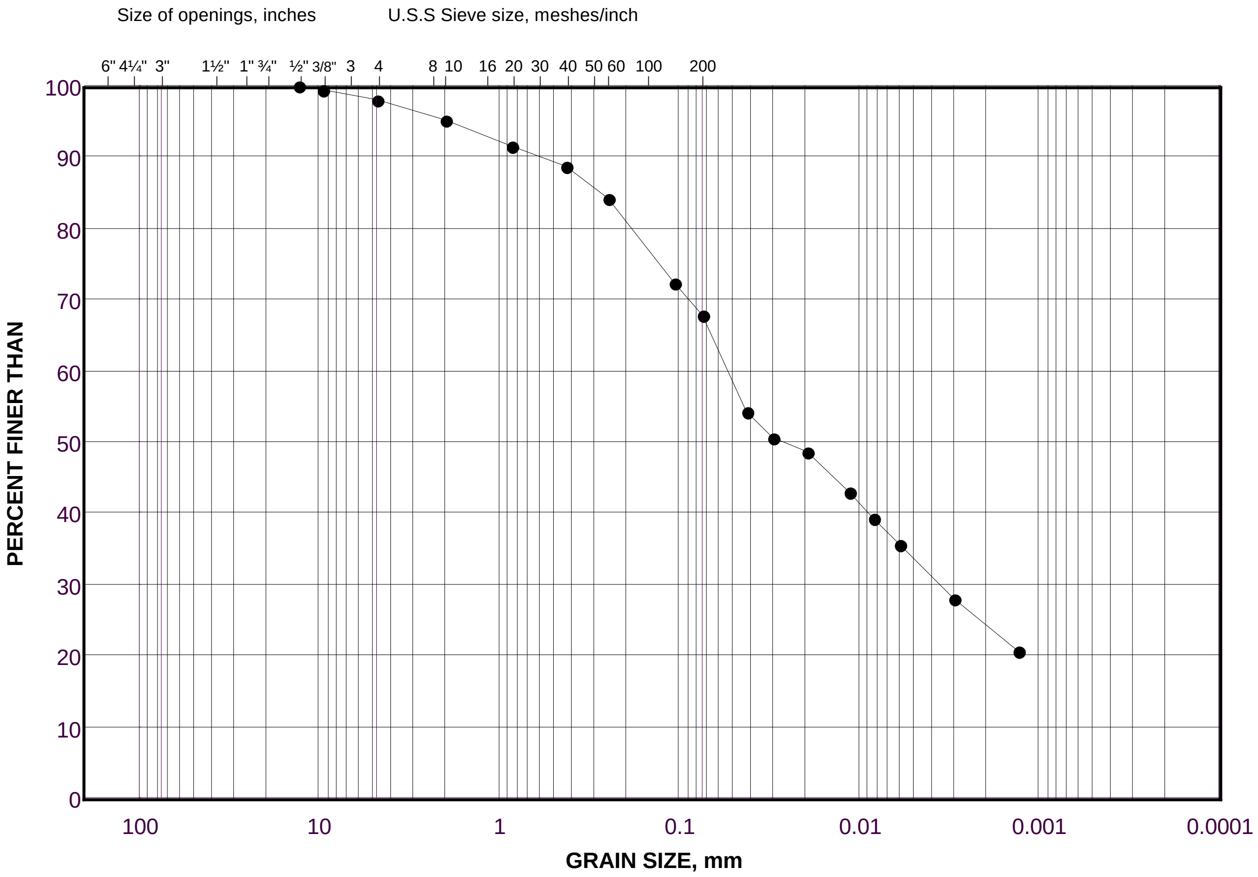
GOLDER

LOGGED: JS
CHECKED: AMP

APPENDIX B

Geotechnical Laboratory Results

GRAIN SIZE DISTRIBUTION (CL) Sandy SILTY CLAY	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)
●	TC-2	7	168.5

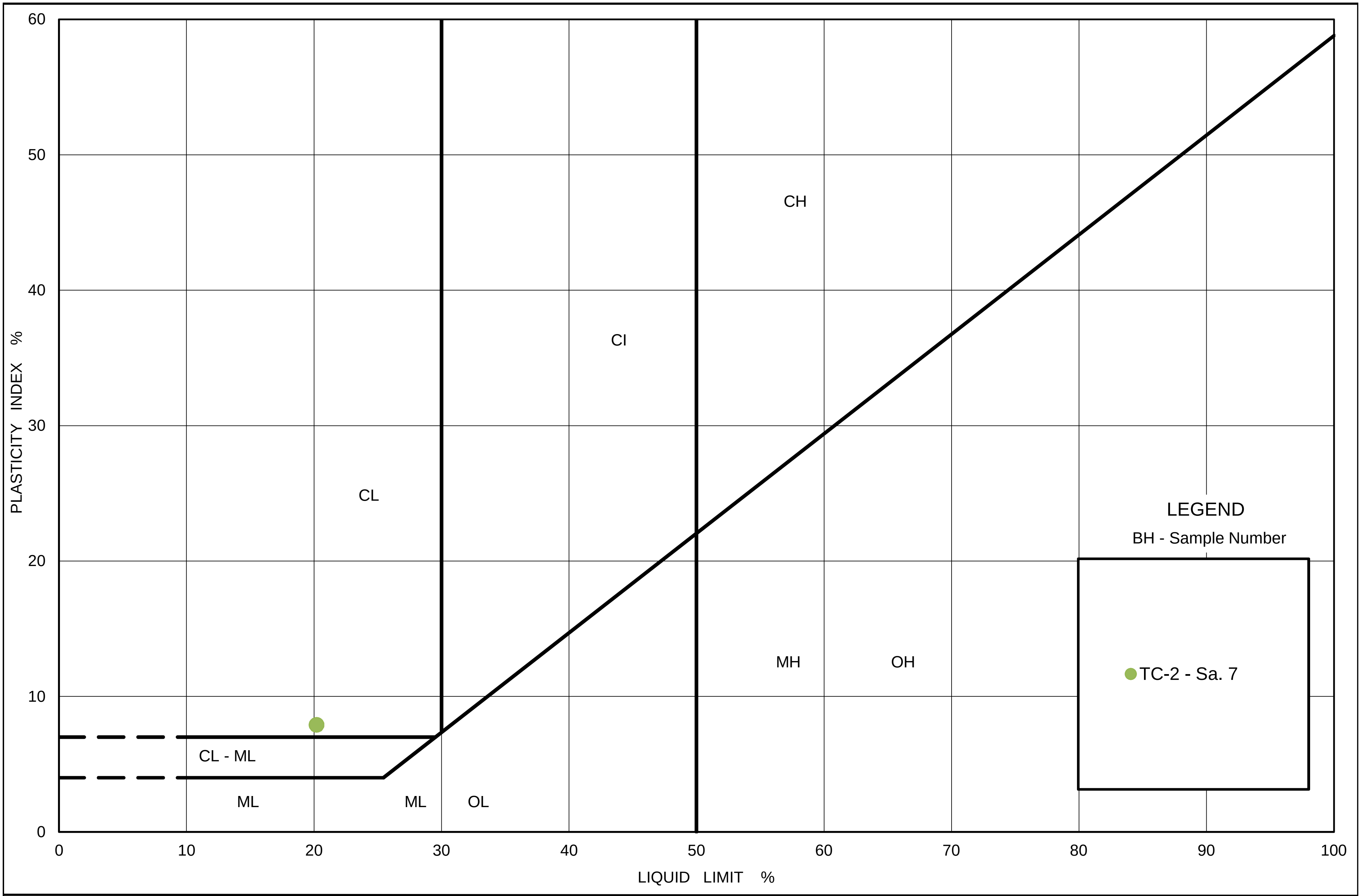
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 01-Feb-19

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



GOLDER

PLASTICITY CHART

(CL) Sandy SILTY CLAY

Figure No.: B-2

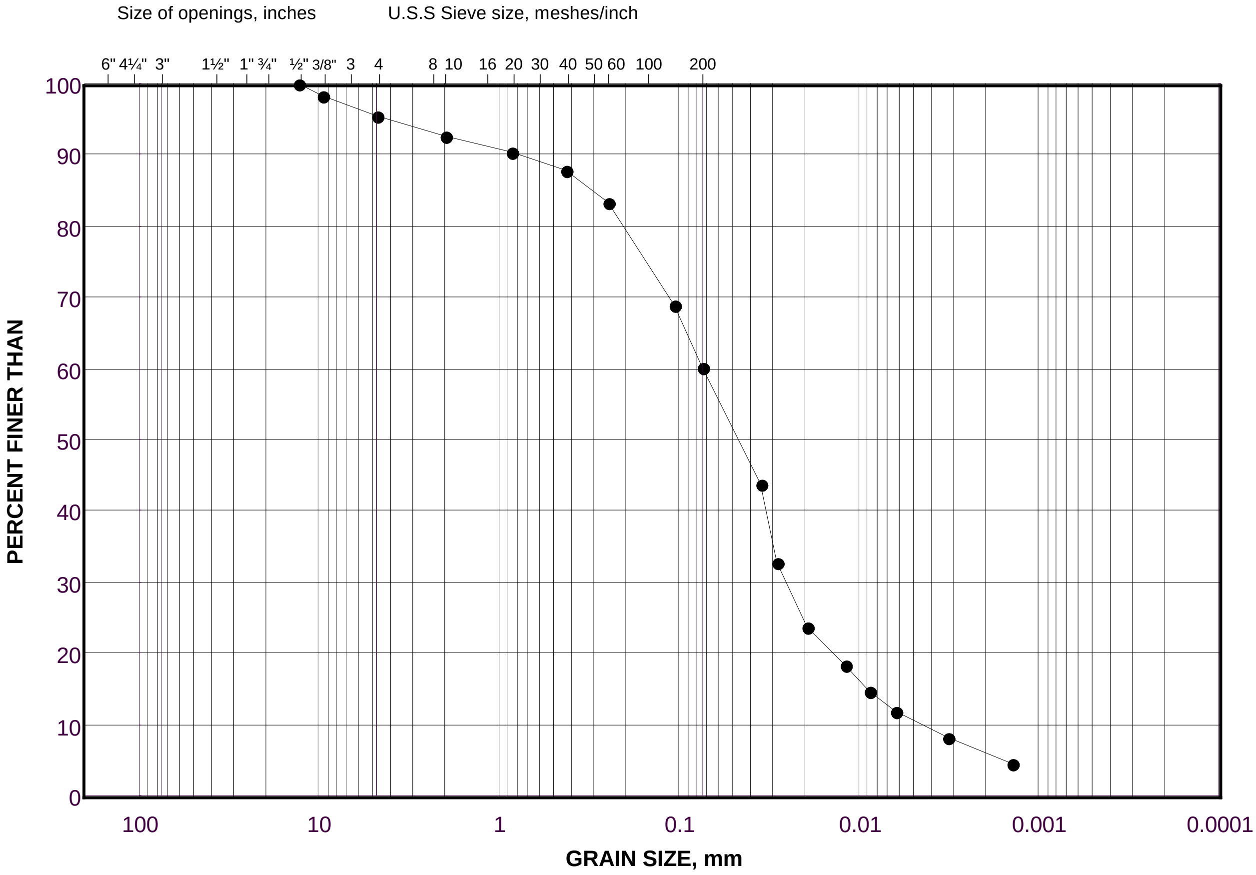
Project No.: 1664178 (2000)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

(ML) SILT and SAND

FIGURE B-3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	TC-1	8	167.1

Project Number: 1664178 (2000)

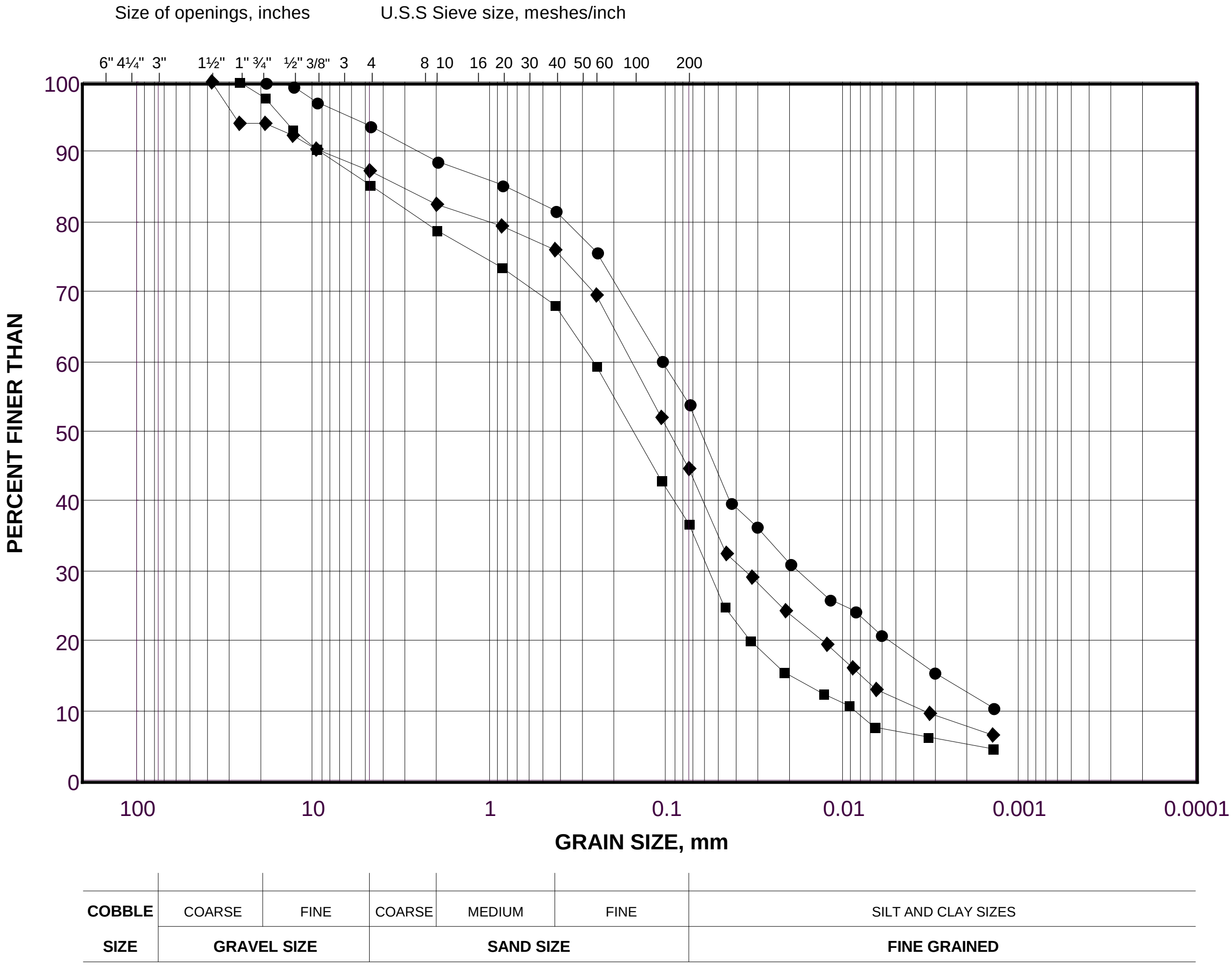
Checked By: AMP

Golder Associates

Date: 01-Feb-19

GRAIN SIZE DISTRIBUTION
(SM) Gravelly SILTY SAND (TILL) to (CL-ML) CLAYEY SILT and SAND (TILL)

FIGURE B-4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	TC-1	10	164.2
■	TC-2	13	160.1
◆	TC-2	8	167.0

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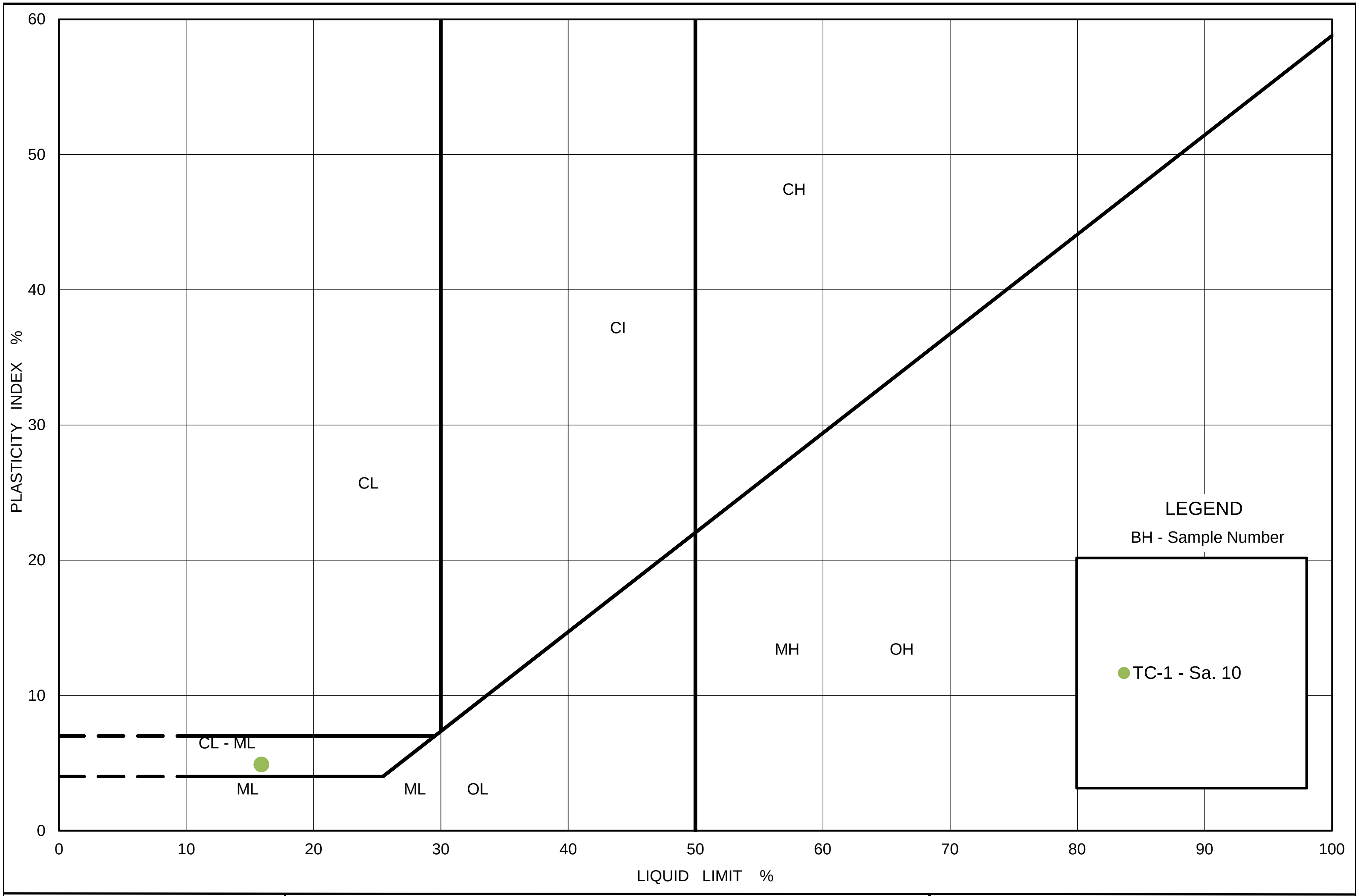
Date: 01-Feb-19

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



PLASTICITY CHART

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

Figure No.: B-5

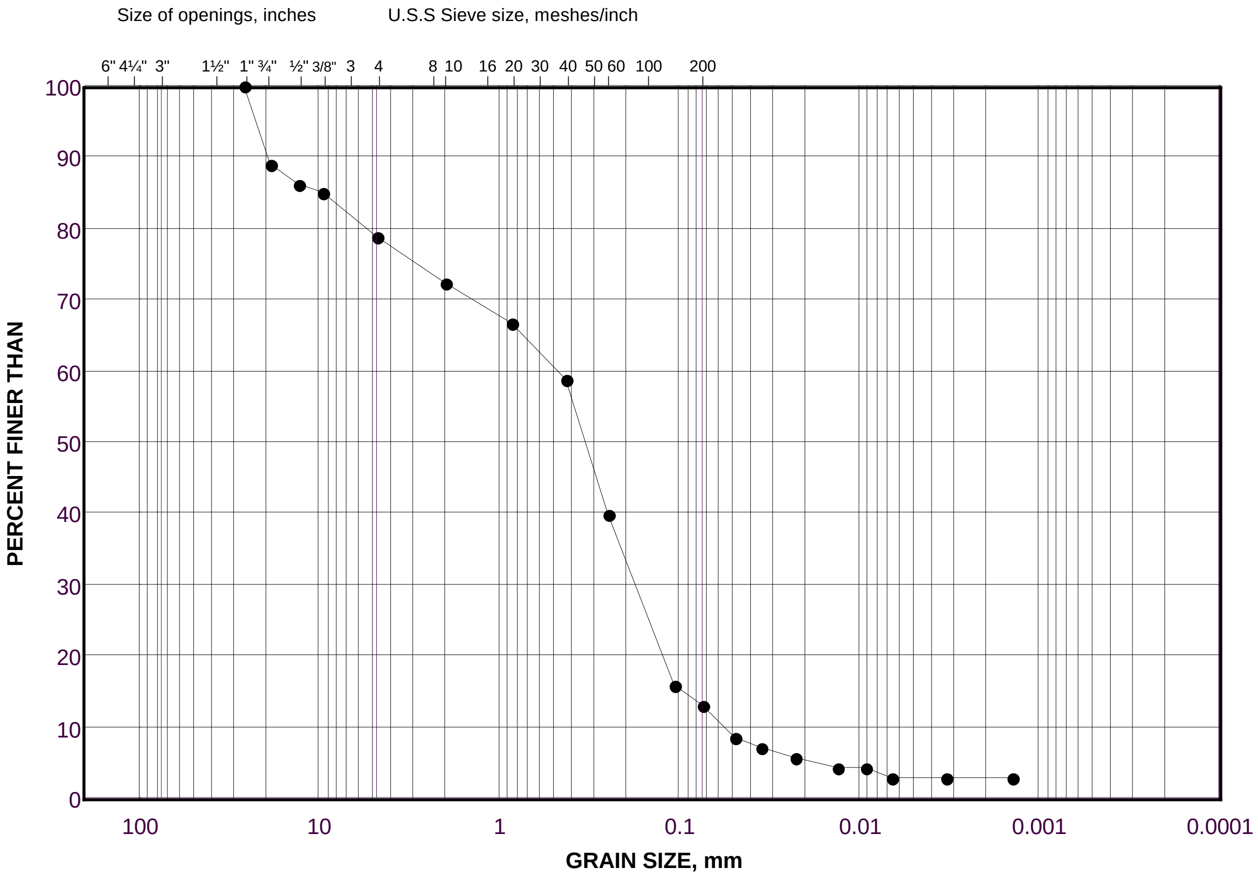
Project No.: 1664178 (2000)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

(SP) Gravelly SAND

FIGURE B-6



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	TC-1	12	161.0

Project Number: 1664178 (2000)

Checked By: AMP

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Date: 01-Feb-19

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