Appendix M.5 – Foundations Report – CN Rail

Kennedy Road Environmental Assessment between Steeles Avenue and Major Mackenzie Drive

Preliminary Foundation Investigation and Design Report

Kennedy Road and Canadian National Rail Bridge
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
KENNEDY ROAD AND CANADIAN NATIONAL RAIL BRIDGE
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 Study 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 results of the foundation investigation carried out at Kennedy Road and the CN Rail bridge.

The purpose of the investigation is to evaluate the subsurface soil and groundwater conditions at Kennedy Road and the CN Rail bridge by means of a limited number of boreholes and, based on our interpretation of the data, to provide preliminary foundation engineering design recommendations for the structures.

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”*, attached to 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 Kennedy Road and CN Rail bridge is located approximately 0.4 km south of the Highway 407 and Kennedy Road interchange as shown on the Key Plan on Figure 1. The CN Rail bridge is approximately 11 m wide and approximately 22 m long. Kennedy Road consists of two lanes in each direction with a boulevard and sidewalk on each side of Kennedy Road.

The natural ground surface at the site slopes downward from south to north with ground surface elevations ranging from approximately Elevation 191 m to Elevation 187 m, being approximately Elevation 188 at the bridge location. The CN Rail bridge which crosses over Kennedy Road is at about Elevation 194 m. Commercial development is located south of the bridge and west of Kennedy Road. Residential development is located south of the bridge and east of Kennedy Road. Undeveloped lands are present north of the bridge that contain the Hydro One corridor and infrastructure which runs parallel to the CN rail.

3.0 INVESTIGATION PROCEDURES

The field work for the preliminary investigation was carried out on November 22 and 23, 2018 during which time two boreholes (designated as Boreholes CNR-101 and CNR-102) were advanced along existing Kennedy Road near the existing bridge abutments to a depth of 15.7 m. 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 CNR-102, 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 CNR-101 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 an 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

Borehole No.	Location (UTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
CNR-101	4,856,293.7	636,176.3	187.05	15.7
CNR-102	4,856,255.7	636,173.7	188.34	15.7

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 CN Rail bridge is located within the South Slope region.

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

[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.

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.

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.

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. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, underlying the pavement structure, the subsurface soil conditions consist of cohesive fill materials underlain by a till deposit ranging in gradation from silty sand to clayey silt and sand. a silty clay deposit was encountered within the till deposit in one of the boreholes. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Fill

Approximately 350 mm of asphalt was encountered at ground surface in both boreholes.

Non-cohesive and cohesive fill was encountered underlying the asphalt in both boreholes as noted in Table 2.

Table 2: Depth and Elevation to Surface and Base, Thickness and Type of Fill Layers

Borehole No.	Top of Layer		Bottom of Layer		Thickness (m)	Fill Type
	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)		
CNR-101	0.36	186.69	0.53	186.52	0.17	Gravelly Sand
	0.53	186.52	1.98	185.07	1.45	Sandy Silty Clay
CNR-102	0.35	187.99	0.73	187.61	0.38	Gravelly Sand
	0.73	187.61	1.36	186.98	0.63	Silty Clay
	1.36	186.98	2.13	186.21	0.77	Sandy Silty Clay

The non-cohesive fill is comprised of gravelly sand containing trace fines. One SPT “N” value of 14 blows per 0.3 m of penetration was measured within the non-cohesive fill, indicating a compact level of compactness.

The cohesive fill consists of sandy silty clay containing trace gravel and a black silty clay layer. The SPT “N” values measured within the cohesive fill range from 7 blows to 23 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

The results of a grain size distribution test carried out on a sample of sandy silty clay fill is shown on Figure B-1 in Appendix B. Atterberg limit testing was carried out on one sample of sandy silty clay fill and the results indicated a liquid limit of about 22 per cent, a plastic limit of about 13 per cent, and a plasticity index of about 9 per cent. These test results, which are plotted on a plasticity chart on Figure B-2 in Appendix B, indicate that the silty clay fill has low plasticity.

The in-situ moisture content measured on a sample from the non-cohesive fill is about 4 per cent. The water contents measured on two samples of the cohesive fill are about 13 per cent.

4.2.2 Silty Sand (Till) to Clayey Silt and Sand (Till)

A glacial till deposit was encountered underlying the fill in both boreholes and varies in composition from silty sand, some gravel to clayey silt and sand, trace to some gravel. The cohesive portion of the till deposit was encountered at around 1.80 m below ground surface in Borehole CNR-101 (Elevation 185.1 m) extending to 11.7 m below ground surface (Elevation 175.4 m).The non-cohesive component of the till was encountered in Boreholes CNR-101 and CNR-102 at depths of about 11.7 m and 2.1 m below ground surface (Elevation 175.4 m and Elevation 186.2 m), respectively, and extended to the borehole’s termination depths of about 15.7 m below ground surface (Elevation 171.4 m and Elevation 172.6 m). During drilling, the augers were grinding in Borehole CNR-101 between 13.4 m and 13.7 m below ground surface (Elevation 173.7 m). 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 cohesive till deposit range from 9 blows to 17 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. The SPT “N” values measured in the non-cohesive till deposit range from 12 blows to 78 blows per 0.3 m of penetration, indicating a compact to very dense level of compactness.

The results of grain size distribution testing carried out on four samples of the till deposit are shown on Figure B-3 in Appendix B. Atterberg limit testing was carried out on four samples of the till deposit. The results of the cohesive sample indicated a liquid limit of about 18 per cent, a plastic limit of about 12 per cent, and a plasticity index of about 6 per cent. These test results, which are plotted on a plasticity chart on Figure B-4 in Appendix B, indicate that the deposit can be classified as a clayey silt of low plasticity. The other three tested samples of the till deposit were non-plastic.

The water contents measured on samples of the non-cohesive till deposit range from about 8 per cent to 12 per cent. The water content measured on a sample of the cohesive till deposit is about 12 per cent.

4.2.3 Silty Clay

A deposit consisting of silty clay, some sand, and some gravel was encountered in Borehole CNR-102, within the till deposit. The silty clay was encountered at a depth of about 10.1 m below ground surface (Elevation 17.2 m) and extended to a depth of about 13.2 m below ground surface (Elevation 175.2 m)

The SPT “N” values in the deposit are 8 blows and 10 blows per 0.3 m of penetration, indicating a firm to stiff consistency.

The results of grain size distribution testing carried out on a sample of the silty clay is shown on Figure B-5 in Appendix B. Atterberg limit testing was carried out on one sample of the silty clay and the results indicate a liquid limit of about 41 per cent, a plastic limit of about 19 per cent, and a plasticity index of 22 per cent. These test results, which are plotted on a plasticity chart on Figure B-6 in Appendix B, indicate the silty clay has intermediate plasticity. The water contents measured on two samples of the silty clay are about 12 per cent and 16 per cent.

4.3 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist. 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 groundwater level in Borehole CNR-101 was measured at a depth of 9.2 m below ground surface in the open hole after the augers were pulled.

A monitoring well was installed in Borehole CNR-102 and sealed within the silty sand till deposit. Upon completion of drilling the water level in the monitoring well was measured to be at a depth of about 9.2 m below ground surface (Elevation 177.9 m). The recorded groundwater levels in the monitoring well is summarized in Table 3.

Table 3: 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
CNR-102	Silty Sand Till	187.05	7.7	180.6	November 29, 2018
			10.6	177.7	December 14, 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. Alan Mohammad, P.Eng., and was reviewed by 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 AND CANADIAN NATIONAL RAIL BRIDGE
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 replacement and widening of the existing Canadian National Rail bridge over Kennedy Road, 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 assessment of the bridge replacement and detours, for planning purposes.

Further investigations will be required during Detailed Design to obtain subsurface information specific to bridge replacement and temporary widened embankments 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 and embankments. 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 design drawings of the existing Kennedy Road and CN Rail bridge were provided by HDR and were reviewed in preparation of this report:

- Drawing titled, *"Canadian National Railways, Great Lakes Region, Toronto, Ontario, Kennedy Rd Subway"* Drawing Nos. 342-12.8-1.1 to 342-12.8-1.1, dated June 1, 1962.

Based on the available design drawings, the existing structure, constructed in 1963, is a single-span bridge approximately 11 m wide and 22 m long. The existing bridge foundations are noted to be supported on spread footings with the design underside of the footing at approximately Elevation 185 m.

It is understood that as part of the improvements to Kennedy Road, replacement of the existing bridge is anticipated to be required to accommodate the future widening of Kennedy Road. In addition, the bridge replacement will include construction of temporary bridge and rail detour track to either the north or south side of the existing rail alignment. The pavement surface along Kennedy Road at the CN bridge structure varies from between Elevation 187 m and 188 m resulting in approximately 6 m high fill embankments.

The subsurface conditions consist of about 2 m of fill overlying stiff to very stiff clayey silt and sand till or compact to very dense silty sand till. An interlayer of firm to stiff silty clay was also present at one location. The composition of the embankment fill is not known as no boreholes were drilled through the rail embankment.

Further, 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 63 and BH 64), advanced within the vicinity of the CN bridge, penetrated into the very dense "100 blow" soil below about Elevation 170 m. Further, these boreholes also indicate the presence of water-bearing sand and gravel layers within the till. It should be noted that our boreholes were terminated up to 2.6 m above Elevation 170 m and as such, they only penetrated the upper portion of the more competent (dense to very dense) silty sand till assumed to be present at depth.

6.2 Foundation Options

Both shallow and deep foundation options have been considered for support of the abutments for the proposed bridge abutments for both the temporary and permanent structures. Based on the subsurface conditions encountered in the boreholes advanced at the site, the following foundation recommendations should be considered for preliminary design:

Shallow Foundations

- **Strip or Spread Footings founded on the stiff to very stiff clayey silt and sand till and/or compact to dense silty sand till:** Although shallow footings are typically the most economical foundation option, considering the structural loads required to support the new or temporary bridge structures, shallow foundations may not be suitable for support of the bridge abutments at this site due to the relatively low capacities available in these materials. In addition, the fill extends to about 2 m depth below the Kennedy Road grade and must be sub-excavated and replaced with compacted granular fill prior to footing construction.

Deep Foundations

- **Driven steel H-piles founded in very dense ("100 blow") silty sand till assumed to be present below Elevation 170 m:** Steel H-Piles driven to "100 blow" glacial till would be considered feasible to support the proposed temporary or permanent abutments. The advantage of driven piles is that it is relatively straightforward design and construction and sub-excavation of the fill would not be required for pile-cap construction. The disadvantage of steel H-piles is that all glacial tills are known to contain cobbles and boulders and if present, the piles may 'hang-up' on these obstructions.

- **Drilled Shafts (Caissons) founded on/in the very dense silty sand till assumed to be present below Elevation 170 m:** Drilled shafts are not recommended for the support of the proposed structure due to the potential presence of water-bearing sand and gravel layers as well as the fact that the bases would be below the prevailing groundwater level. These conditions would require the use of temporary or permanent liners. In addition, due to the relatively deep depth required for the caissons to reach the competent till deposit, cleaning out any disturbed soils and inspection of the caisson base subgrade would be challenging, or potentially result in lower capacities.

Based on the above consideration, the following sections provide preliminary recommendations for shallow footings and driven steel H-piles for the CN bridge abutments for both the temporary and permanent structures.

6.3 Spread Footings

6.3.1 Footings on Native Soils

The temporary and permanent abutments may be supported on large spread footings placed below any fill and founded on the undisturbed stiff to very stiff and compact to dense glacial till deposits. All spread footings must be provided with a minimum of 1.4 m of soil cover for frost protection, in accordance with Section 3.2.4.3 of the *American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering (2018)*.

Such footings may be founded on the surface of the glacial till Elevation 185.1 m (vicinity of Borehole CNR-101) and Elevation 186.2 m (vicinity of Borehole CNR-102).

Spread footings of various widths founded on the properly prepared subgrade at or below the founding elevation given above may be designed based on allowable bearing capacities and allowable settlement pressures (for 25 mm of settlement) as provided in Table 4.

Table 4: Allowable Bearing Capacity and Settlement Pressure on Native Till

Footing Width	Allowable Bearing Capacity	Allowable Settlement Pressure
3 m	300 kPa	250 kPa
4 m	325 kPa	185 kPa
5 m*	350 kPa	150 kPa

*Similar to existing footing width.

The allowable bearing capacities are dependent on the footing width and founding elevation and as such, the values used in design should be reviewed if the footing width is greater or less than those specified above or if the founding elevation differs from that given above.

A coefficient of friction of 0.45 may be used in the assessment of sliding resistance between the cast-in-place concrete footing and the glacial till deposit in accordance with Section 5.4.2 of the *AREMA Manual*.

The allowable bearing capacities provided above are recommended under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination and eccentricity of the load should be taken into account, in accordance with Section 3.5 of the *AREMA Manual*.

The exposed subgrade of each footing excavation should be cleaned of loose / softened material and any standing water removed in accordance with OPSS 902 (*Excavating and Backfilling Structures*), prior to placing concrete. It is essential that the founding level for the footings be inspected by qualified geotechnical personnel immediately prior to placing concrete, to confirm the adequacy of the foundation conditions for the noted allowable bearing pressures. The founding soils will be susceptible to disturbance and degradation on exposure to water and construction traffic. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab having a minimum thickness of 100 mm and a minimum of 28-day compressive strength of 20 MPa be placed in the excavation within four hours of exposure of the founding level to protect the integrity of the subgrade. Where subexcavation is required due to loosening / softening of the subgrade or to remove the existing fill, the subexcavated 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*).

6.3.2 Footings on Granular Pad

Alternatively, the abutments for the proposed structures may be supported on spread footings “perched” on a compacted granular pad that extends from the underside of the footing to a depth equivalent to the width of the footing, providing that the compacted granular pad is constructed on engineered fill placed above the current grade of Kennedy Road and any existing fill is removed and replaced with similar granular fill.

All spread footings must be provided with a minimum of 1.4 m of soil cover for frost protection, in accordance with Section 3.2.4.3 of the *AREMA Manual*.

Spread footings of various widths founded on a compacted granular pad in accordance with the recommendations given above may be designed based on allowable bearing capacities and allowable settlement pressures (for 25 mm of settlement) as provided in Table 5.

Table 5: Allowable Bearing Capacity and Settlement Pressure on Granular Pad

Footing Width	Allowable Bearing Capacity	Allowable Settlement Pressure
3 m	500 kPa	270 kPa
4 m	550 kPa	230 kPa
5 m*	600 kPa	200 kPa

The allowable bearing capacities are dependent on the footing width and founding elevation and as such, the values used in design should be reviewed if the footing width is greater than that specified above or if the founding elevation differs from that given above.

A coefficient of friction of 0.65 may be used in the assessment of sliding resistance between the cast-in-place concrete footing and the compacted granular pad in accordance with Section 5.4.2 of the *AREMA Manual*.

The allowable bearing capacities provided above are recommended under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination and eccentricity of the load should be taken into account, in accordance with Section 3.5 of the *AREMA Manual.*

Any existing fill, organic soils and/or loose/soft soils present at the current ground surface should be sub-excavated and replaced with engineered fill. 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 1 Horizontal to 1 Vertical (1H:1V). The granular fill should be placed in accordance with OPSS.MUNI 501 (*Compacting*). The above recommendations consider that the engineered fill is not allowed to be left exposed (i.e. which can lead to degradation) and that the compacted granular pad is placed on the native soils after removal of any existing fill.

The allowable bearing capacity are recommended under the assumption that the granular pad and replacement fill are compacted to 95 per cent of Specified Modified Proctor Density (SMPD). It is essential that the subgrade of the granular pad and footings be inspected by qualified geotechnical personnel immediately prior to placing concrete, to confirm the adequacy of the foundation conditions for the noted bearing pressures.

6.4 Driven Steel H-Piles

As an alternative to shallow foundations, the proposed temporary and permanent bridge structures could be founded on steel H-piles (HP 310x110) driven to found 1 m into the “100 blow” till deposit. Based on the available borehole information, the piles should be driven to or below Elevation 169 m, corresponding to pile length in the order of 17 m from a depth of about 1.4 m below the existing road surface.

For HP 310x110 steel H-piles driven to or below the elevations given above, an allowable bearing capacity of 1050 kN per pile may be used for design, which is a combination of end-bearing resistance and frictional resistance.

The underside of the pile caps must be provided with a minimum of 1.4 m of soil cover for frost protection, in accordance with Section 3.2.4.3 of the *AREMA Manual*.

For the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the till deposits. The piles should be reinforced at the tip with driving shoes or flange plates for protection during driving in accordance with OPSD 3000.100 (*Steel H-Pile Driving Shoe*).

6.5 Lateral Earth Pressures for Design of Abutment Walls and Wingwalls

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. 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. 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.

6.5.1 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 6 may be used assuming the use of approved earth fill or OPSS.MUNI 1010 (*Aggregates*) Select Subgrade Material (SSM) for the general embankment fill:

Table 6: Coefficients of Static Lateral Earth Pressure for Restrained Wall

Fill Type	Unit Weight of Material	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Approved Earth Fill / SSM	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 7 may be used:

Table 7: Coefficients of Static Lateral Earth Pressure for 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.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.6 Approach Embankments (Permanent and Detour)

We understand that construction of temporary embankment widening to accommodate a detour track to either the north or south side of the existing rail alignment is required to allow for permanent bridge construction. Assuming that the grade of the track and the road will remain the same, the final and temporary widened embankments will be between 6 m and 7 m in height above the surrounding ground.

6.6.1 Subgrade Preparation and Embankment Construction

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

Fill for construction of the widened approach embankments (for the permanent case and for temporary widening) should consist of Granular ‘A’, Granular ‘B’ Type I or Type II meeting the specifications of OPSS.MUNI 1010 (Aggregates) or other pre-approved earth fill. 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 earth or granular fill and be properly benched and keyed into the existing embankment fill in accordance with Section 1.2.3.3.2, Figure 1-1-7, *Shear Key and Benching Detail*, of the *AREMA Manual*.

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.MUNI 511 (*Rip Rap, Rock Protection and Granular Sheeting*), and OPSS.MUNI 1004 (*Aggregates-Miscellaneous*) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.6.2 Global Stability

The *AREMA Manual* requires a Factor of Safety of 1.5 for the long-term/permanent stability of embankment slopes. The details of the side slopes inclination and height of the existing embankment and the composition of the embankment fill are currently unknown. In this regard, during Detailed Design a subsurface soil investigation should be carried out to obtain information on the embankment fill to confirm the global stability analysis. For new/widened earth fill side slopes formed at 2H:1V and properly keyed into the existing fill, and assuming that the embankment fill is reasonably competent (i.e. does not contain organics, etc.), a Factor of Safety of 1.5 is achieved against a global deep-seated failure surface.

6.6.3 Settlement

For new/widened approaches, fill will be required to be placed on and keyed into the existing slope. It is estimated that approximately up to 6 m to 7 m of fill may be required for the new approach embankments. The final width of the permanent approach embankment is not known at this time, but it is assumed that it will be similar to the existing embankment with less than 1 m of widening on each side. The width of the temporary embankment and which side of the embankment it will be constructed is also not known at this time but could be up to about 5 m assuming it will need to accommodate one rail track and that only limited property is available, as well as for the track alignment itself.

The estimated settlement of the subsoils under the additional fill is estimated to be up to 25 mm and is expected to occur during or shortly after construction and will be differential between the new and permanent/temporary widened sections of the embankment. The settlement of the subsoils under the existing embankment are expected to be less than 15 mm as a result of the new loading. Settlement of the properly placed earth fill itself is expected to be less than 25 mm and will occur during or shortly after construction.

These settlements should be considered and monitored as they may affect train traffic. The estimated settlements should be reassessed during the Detailed Design stage, once the proposed structure and embankment geometry is available.

6.7 Construction Considerations

6.7.1 Temporary Excavation

Temporary open cut excavations for shallow foundations and/or pile cap construction, including subexcavation of existing fill, will extend up to about 2 m below existing grade and will extend through the existing fill to the silty sand and clayey silt and sand till deposits. The excavations should be carried out in a manner which does not result in ground loss under the existing bridge footings. 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 the glacial till would be classified as Type 3 soil, according to the OHSA. Temporary excavations above the water table should be made with side slopes no steeper than 1 horizontal to 1 vertical (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.

6.7.2 Temporary Protection Systems

At this preliminary stage, it is anticipated that temporary protection systems will be required along Kennedy Road and adjacent to the existing footings, in order to facilitate the construction of the new structures. Temporary protection systems may also be required between the existing and temporary bridge foundations and/or the temporary and replacement bridge foundations.

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, CN rail tracks 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, braced shoring systems including 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.

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.

6.7.3 Groundwater and Surface Water Control

The groundwater level measured in the monitoring well installed in Borehole CNR-102, which was screened in the silty sand till deposit, was measured as high as 7.7 m depth below ground surface (Elevation 180.6 m), on November 29, 2018. Depending on the time of year of construction, perched groundwater conditions may be present within the fill materials above the glacial till deposit and/or within the CN rail embankment fill.

Considering the relatively low permeability of the fill and till soils, it is anticipated that water inflow from these layers for shallow excavations of up to about 2 m can be handled by pumping from properly filtered sump pumps placed at the base of the excavation and outside the foundation footprint in advance of the excavation reaching the 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.

6.7.4 Obstructions During Pile Driving

Glacially derived till soils in Ontario should be expected to contain cobbles and boulders, which could affect the installation of driven steel H-piles and/or installation of temporary protection system elements. 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 boulders).

6.7.5 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.

6.7.6 CN Track Monitoring

A comprehensive track settlement monitoring program should be developed in accordance with CN guidelines. The program will likely consist of a series of surface, in-ground shallow and in-ground deep monitoring points, as well as points on the structure, to measure the settlement of the tracks during construction. CN typically has settlement warning levels of 5 mm and critical levels of 10 mm, although these should be confirmed with CN prior to construction for this section of track and for the proposed construction activities. Discussion with CN should also be had concerning settlement of the detour track and special flagging requirements, although its likely that flagging will be required throughout the construction operations. Advance early discussions with CN will be beneficial.

6.7.7 Vibration Monitoring During Temporary Protection System or Pile Installation

Structures at/near the site include the existing bridge, the railway tracks, utilities and residential and commercial buildings. 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, 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. The PPV thresholds provided in Table 8 are generally considered applicable. The number and location of vibration instruments and frequency of readings should be determined during Detailed Design.

Table 8: PPV Thresholds

Structure Type	PPV Limit (mm/s)
Utilities*	10
Railway tracks	25
Residential Buildings/Private Structures	25
Commercial/Industrial Buildings	50
Existing Bridge Structure	100

*May depend on individual utility owners.

7.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION WORK DURING DETAILED DESIGN

Should the existing bridge be replaced, a separate temporary bridge structure be constructed, and the embankments be temporarily or permanently widened, additional boreholes will be required during the Detailed Design for the proposed works. The additional boreholes should be advanced within the footprint of the widened/new foundation elements to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report. Further, a minimum of one borehole on each side of the bridge should be advanced through the railway fill embankment to confirm the composition of the fill, which may be difficult due to the need for potential track closures/flagging on an active rail line. The detailed investigation should:

- Assess the type and depth of embankment and roadway fill present;
- Assess near surface soil deposits within the footprint of the proposed/widened embankments for stability and settlement analysis, where applicable;
- Advance the boreholes a minimum of 3 m into “100-blow” materials for termination criteria for driven piles;

- 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;
- Measure groundwater levels;
- 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 and temporary protection systems; and
- Determine the CN track settlement and vibration limits and provide details of the monitoring program.

8.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Alan Mohammad, P.Eng., and was reviewed by Ms. Sarah 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.

Alan Mohammad, P.Eng.	Sarah E. M. Poot, P.Eng.
Geotechnical Engineer	*Associate, Senior Geotechnical Engineer*

AM/SEMP/cr;mes

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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.

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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.

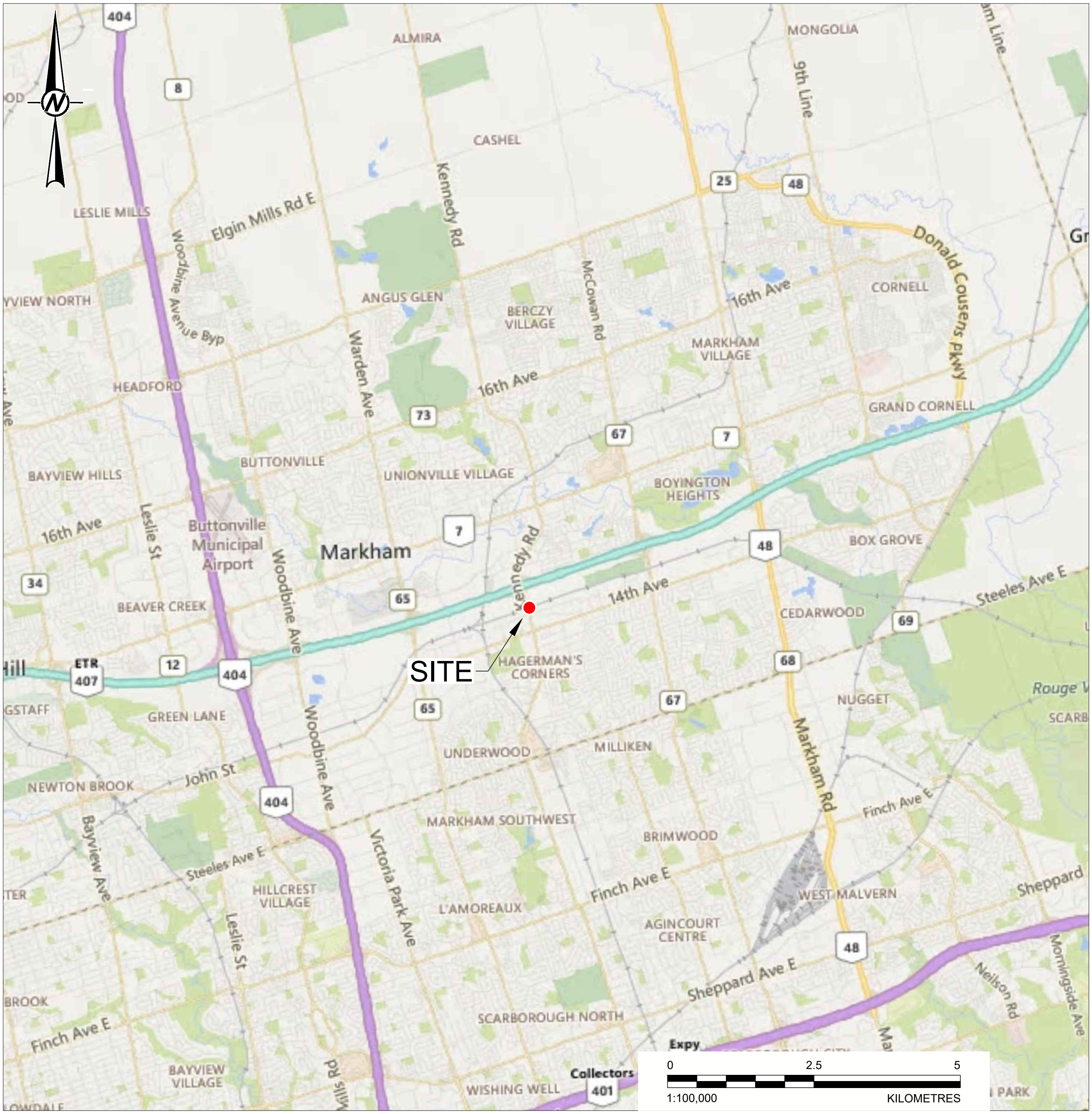
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client’s expense. In the event that actual contaminated soils, fills or groundwater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



CLIENT
HDR INC.

PROJECT
KENNEDY ROAD AND CANADIAN NATIONAL RAIL BRIDGE
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

TITLE
KEY PLAN

REFERENCE(S)

BASE IMAGERY - © 2019 DIGITALGLOBE IMAGE COURTESY OF USGS EARTHSTAR GEOGRAPHICS SIO © 2019 MICROSOFT CORPORATION

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

CONSULTANT

GOLDER

YYYY-MM-DD	2019-10-29
DESIGNED	
PREPARED	STB
REVIEWED	AMP
APPROVED	SEMP

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

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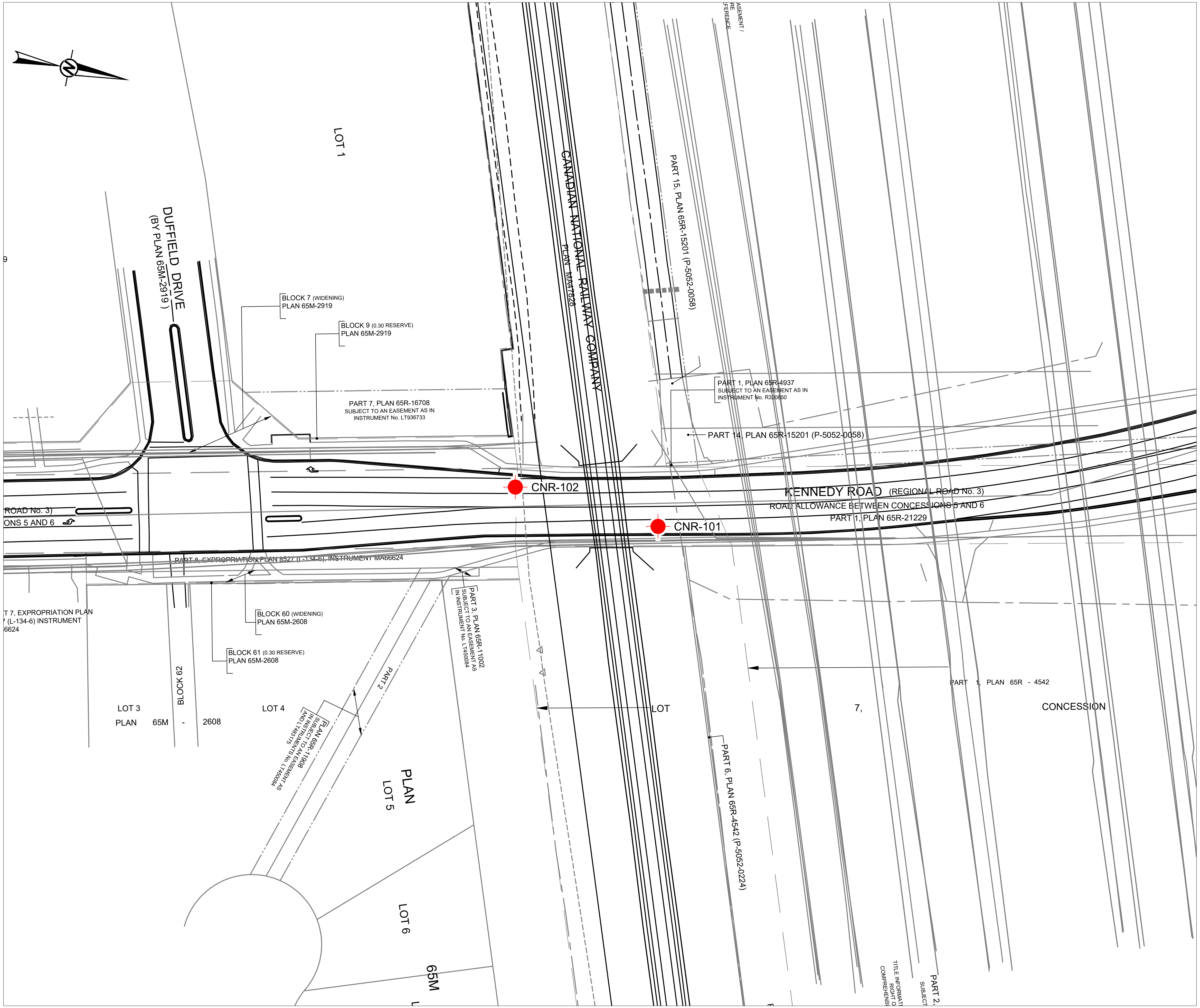

LEGEND

● 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 AND CANADIAN NATIONAL RAIL BRIDGE

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

TITLE

BOREHOLE LOCATION PLAN

CONSULTANT

GOLDER

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APPROVED	SEMP

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

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APPENDIX A

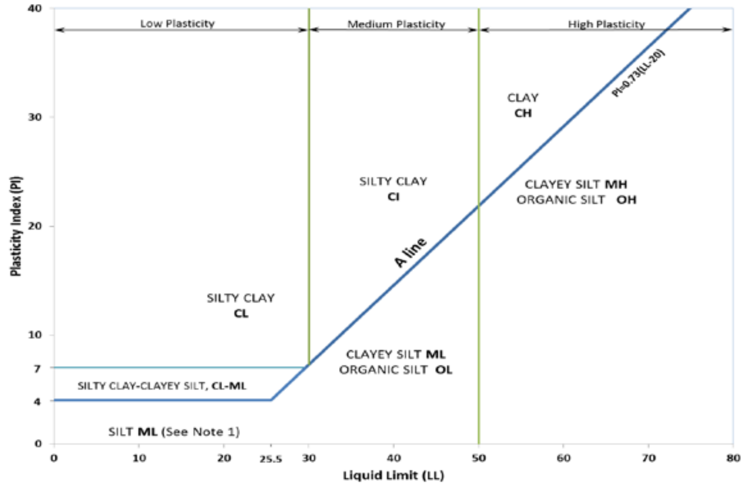
Record of Borehole Sheets

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					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

(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

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

PROJECT: 1664178 (2000)

RECORD OF BOREHOLE: CNR-101

SHEET 1 OF 2

LOCATION: N 4856293.71; E 636176.29

BORING DATE: November 23, 2018

DATUM: Geodetic

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

HAMMER TYPE: AUTOMATIC

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

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 METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	187.05					
0	ASPHALT (355 mm)	0.00					
	FILL - (SP) gravelly SAND, trace fines; brown, trace asphalt fragments; non-cohesive, moist	186.69 0.36	1A				
	FILL - (CL) sandy SILTY CLAY, trace gravel; brown, oxidation staining, solvent-like odour; cohesive, w~PL, stiff to very stiff	0.53	1B	SS	14		
			1C				
1			2	SS	16	MH	
	- No soil recovery from Sample 3		3	SS	23		
2	(CL-ML) CLAYEY SILT and SAND, trace to some gravel; brown to grey, (TILL); cohesive, w<PL to w>PL, stiff to very stiff	185.07 1.98					
			4	SS	13		
3			5	SS	17		
4							
	- Becoming grey at 4.6 m depth		6	SS	13		
5							
6			7	SS	10		
7							
8			8	SS	10		
9			9	SS	9	MH	∇ 23-Nov-18
10	*CONTINUED NEXT PAGE*						

DEPTH SCALE

1 : 50

GOLDER

LOGGED: YS

CHECKED: AMP

PROJECT: 1664178 (2000)

RECORD OF BOREHOLE: CNR-101

SHEET 2 OF 2

LOCATION: N 4856293.71; E 636176.29

BORING DATE: November 23, 2018

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	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) CLAYEY SILT and SAND, trace to some gravel; brown to grey, (TILL); cohesive, w<PL to w>PL, stiff to very stiff								
11				10	SS	14				
		(SM) SILTY SAND, trace to some gravel; grey, (TILL); non-cohesive, moist, dense to very dense	175.39 11.66							
12				11	SS	40				
13		- Augers grinding between 13.4 m and 13.7 m depth								
14				12	SS	60		○	MH NP	
15				13	SS	78				
		END OF BOREHOLE	171.35 15.70							
16		NOTES:								
		1. Borehole open upon completion of drilling.								
		2. Groundwater measured in open borehole at a depth of 9.2 m below ground surface (Elev. 177.9 m) upon completion of drilling.								
17		3. NP = Non-plastic								
18										
19										
20										

DEPTH SCALE

1 : 50

GOLDER

LOGGED: YS

CHECKED: AMP

PROJECT: 1664178 (2000)

LOCATION: N 4856255.69; E 636173.67

RECORD OF BOREHOLE: CNR-102

BORING DATE: November 22, 2018

SHEET 1 OF 2

DATUM: Geodetic

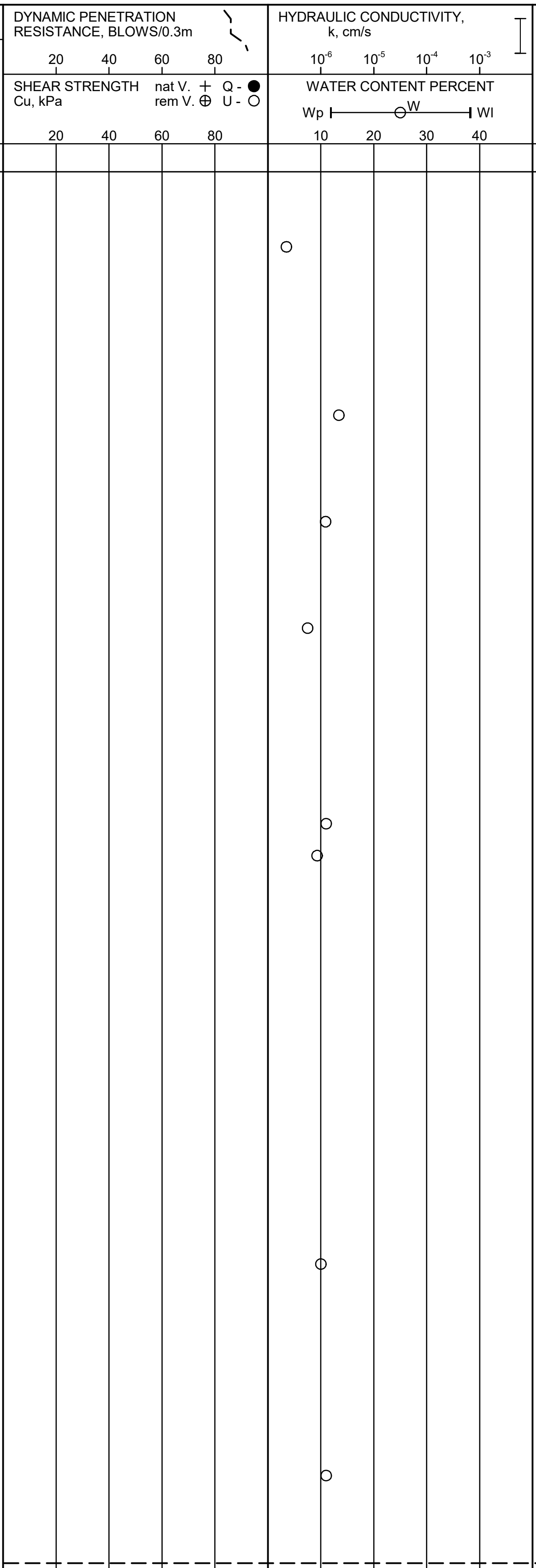
SPT/DCPT HAMMER: MASS, 64kg; DROP, 760mm

HAMMER TYPE: AUTOMATIC

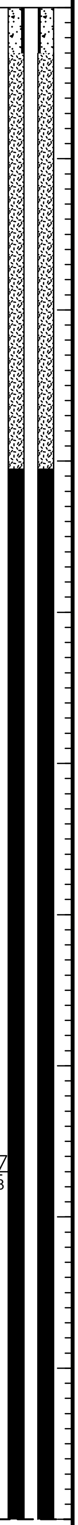
BORING METHOD: Power Auger
216 mm O.D. Hollow Stem Augers

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	188.34
ASPHALT (345 mm)	0.00
FILL - (SP) gravelly SAND; brown; non-cohesive, moist	187.99 0.35
FILL - (CL) SILTY CLAY, some sand; black; cohesive, w~PL	187.61 0.73
FILL - (CL) sandy SILTY CLAY, trace gravel; brown, oxidation staining; cohesive, w~PL, firm	186.98 1.36
(SM) SILTY SAND, some gravel; brown to grey, (TILL); non-cohesive, moist, compact to dense - Oxidation staining between 2.1 m and 3.3 m depth - No soil recovery from Sample 5 - Becoming grey at 6.1 m depth	186.21 2.13
CONTINUED NEXT PAGE	

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	AS	-	
2	AS	-	
3	SS	7	
4	SS	12	
5	SS	24	
6A 6B	SS	32	MH NP
7	SS	14	
8	SS	19	
9	SS	12	



PIEZOMETER OR STANDPIPE INSTALLATION



50 mm Diameter PVC Monitoring Well (Flushmount)

Cuttings

Bentonite

29-Nov-18

DEPTH SCALE METRES: 0 – 10

DEPTH SCALE 1 : 50

GOLDER

LOGGED: YS

CHECKED: AMP

PROJECT: 1664178 (2000)
LOCATION: N 4856255.69; E 636173.67

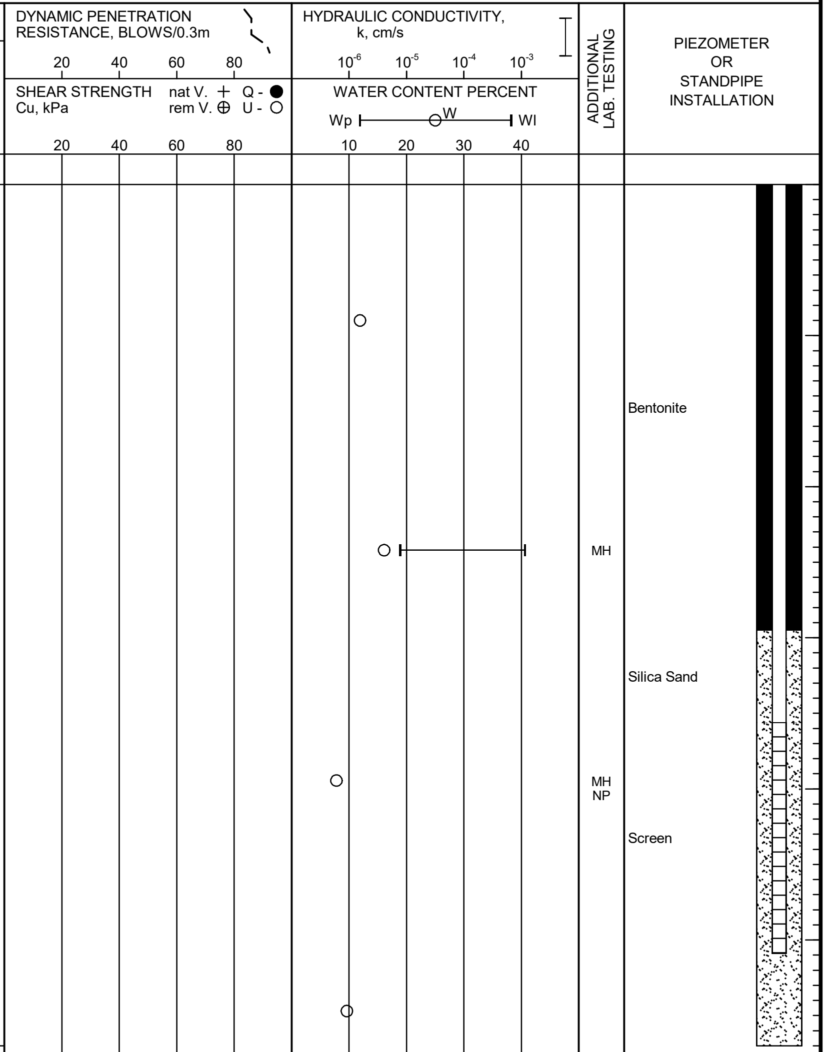
RECORD OF BOREHOLE: CNR-102

BORING DATE: November 22, 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	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
10	Power Auger 216 mm O.D. Hollow Stem Augers			178.21					
		(CI) SILTY CLAY, some sand, trace gravel; grey; cohesive ,w<PL, firm to stiff		10.13					
11					10	SS	10		Bentonite
12					11	SS	8	MH	
13				175.16					Silica Sand
		(SM) SILTY SAND, some gravel; grey, (TILL); non-cohesive, moist, compact to very dense		13.18					
14					12	SS	53	MH NP	Screen
15				172.64	13	SS	24		
		END OF BOREHOLE		15.70					



NOTES:

1. Water level measured in monitoring well as follows:

Date	Depth (m)	Elev. (m)
22-Nov-18	11.1	177.2
29-Nov-18	7.7	180.6
14-Dec-18	10.6	177.7

2. NP = Non-plastic

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: AMP

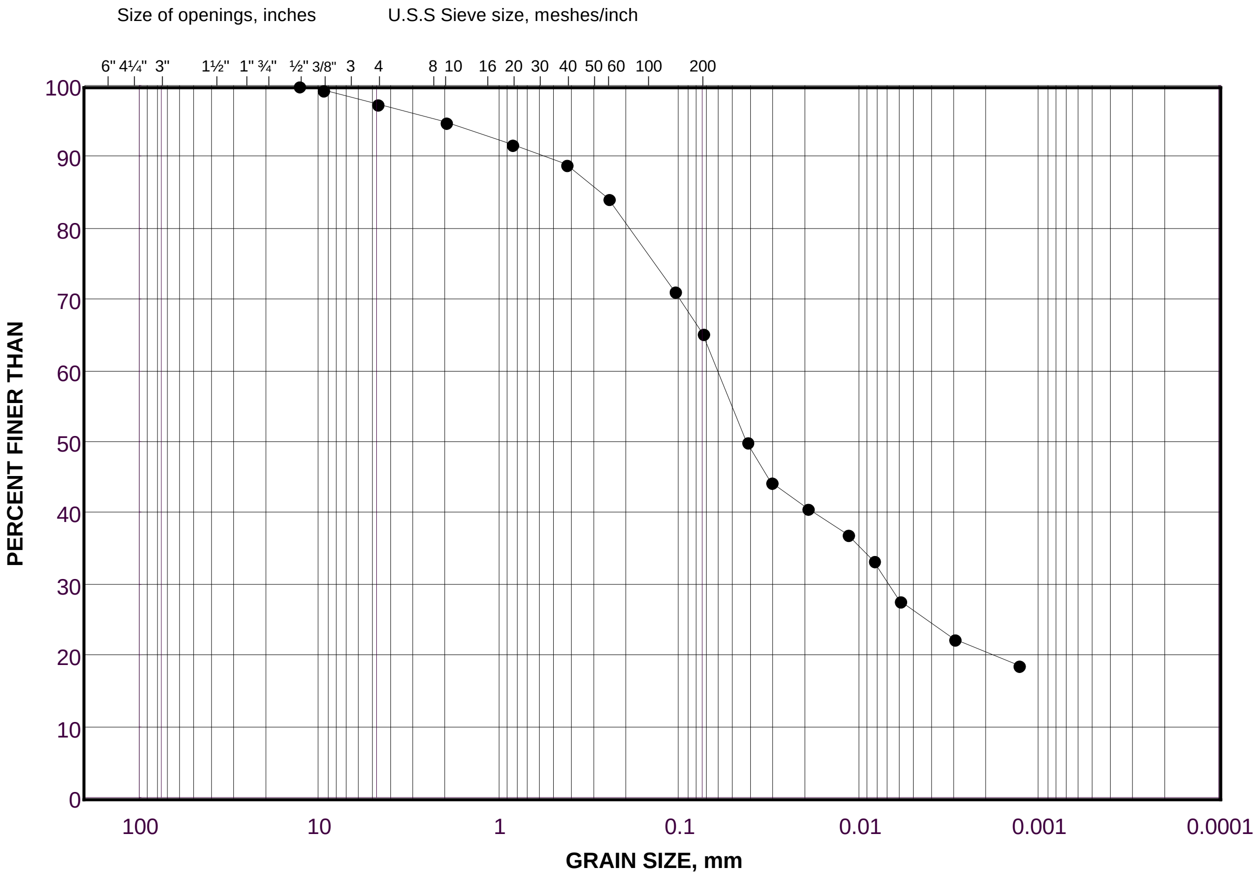
APPENDIX B

Geotechnical Laboratory Results

GRAIN SIZE DISTRIBUTION

FILL - (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)
●	CNR-101	2	185.9

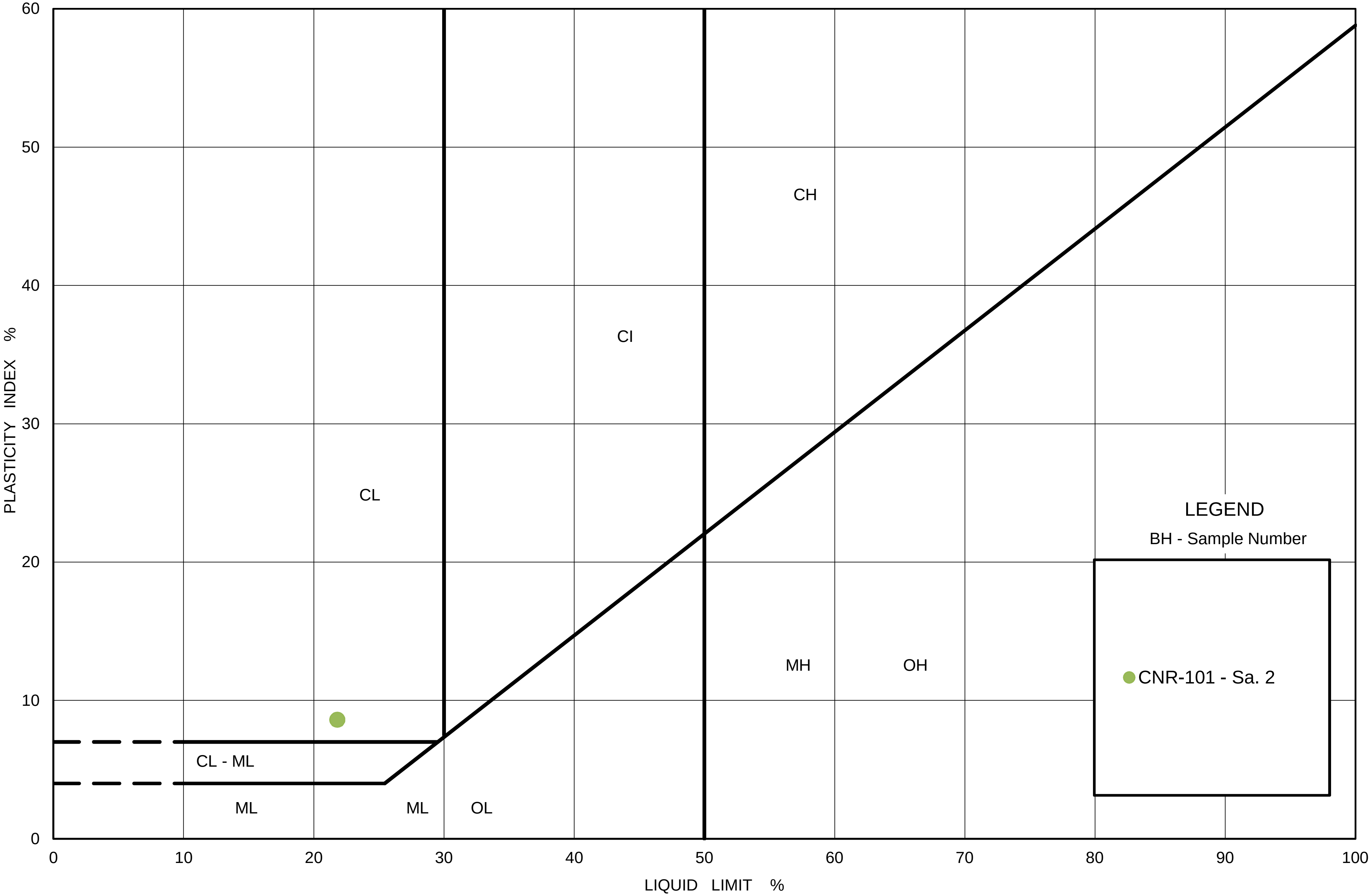
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 03-Feb-19

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



GOLDER

PLASTICITY CHART

FILL - (CL) sandy SILTY CLAY

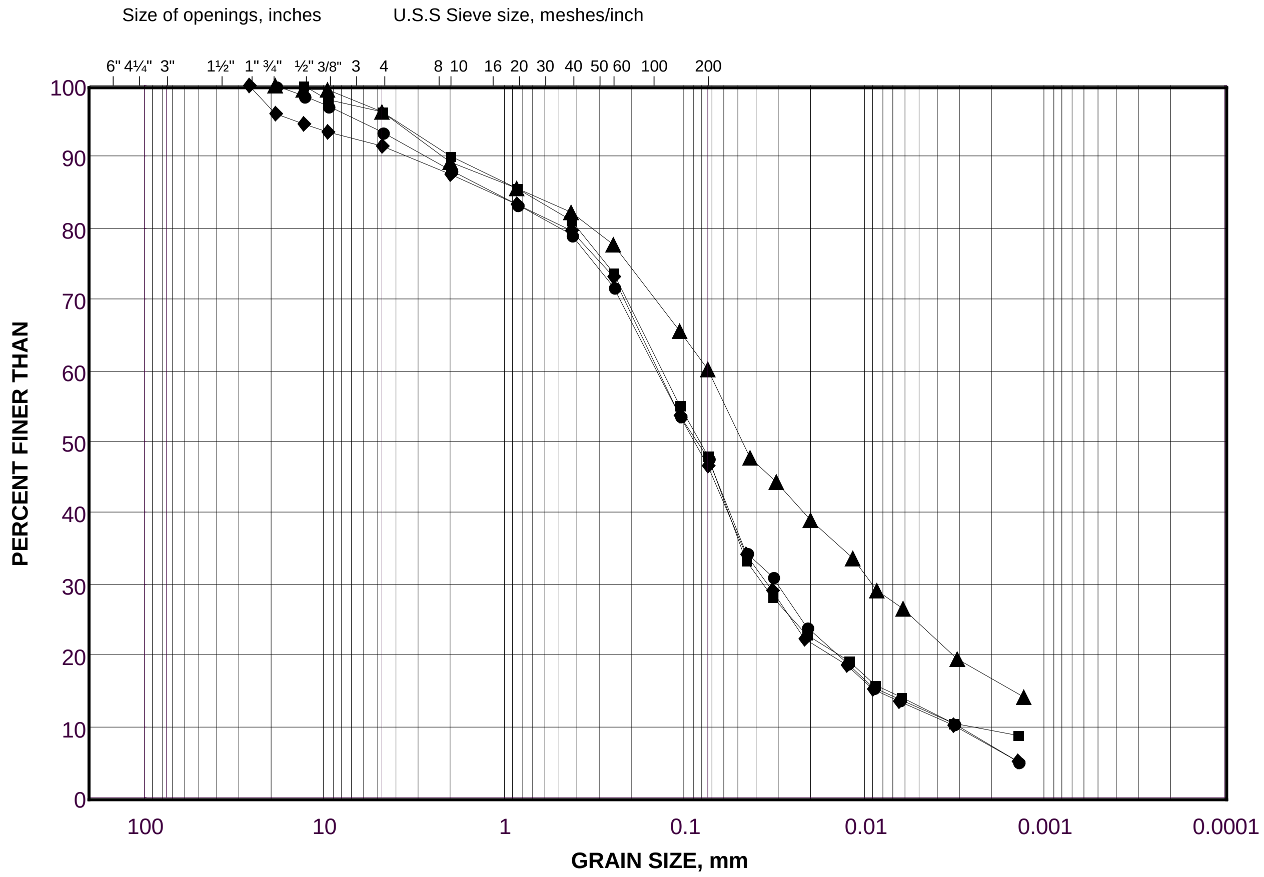
Figure No.: B-2

Project No.: 1664178 (2000)

Checked By: AMP

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

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)
●	CNR-102	12	174.4
■	CNR-101	12	173.1
◆	CNR-102	6A	183.7
▲	CNR-101	9	177.7

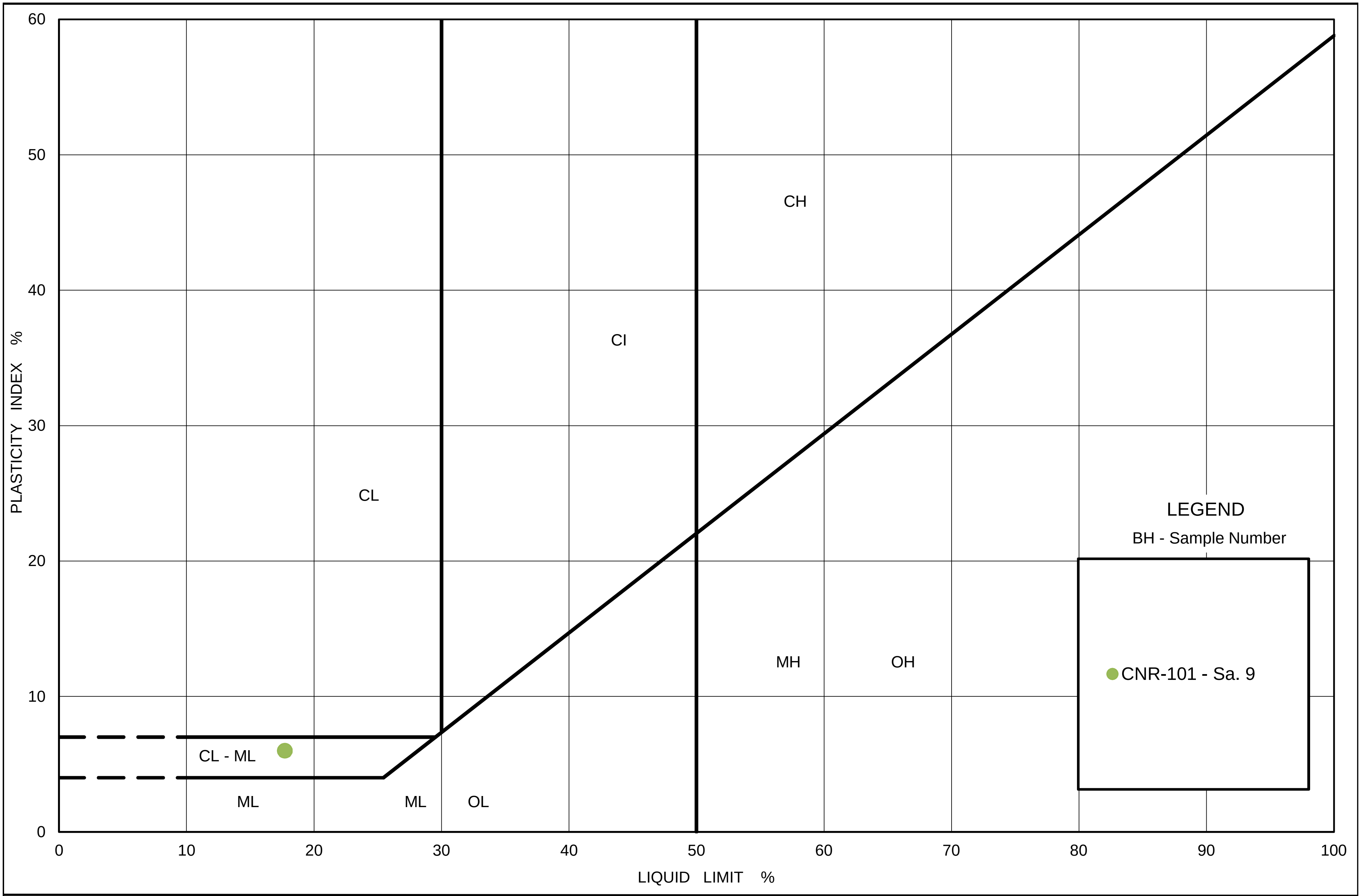
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 20-Feb-19

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



GOLDER

PLASTICITY CHART

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

Figure No.: B-4

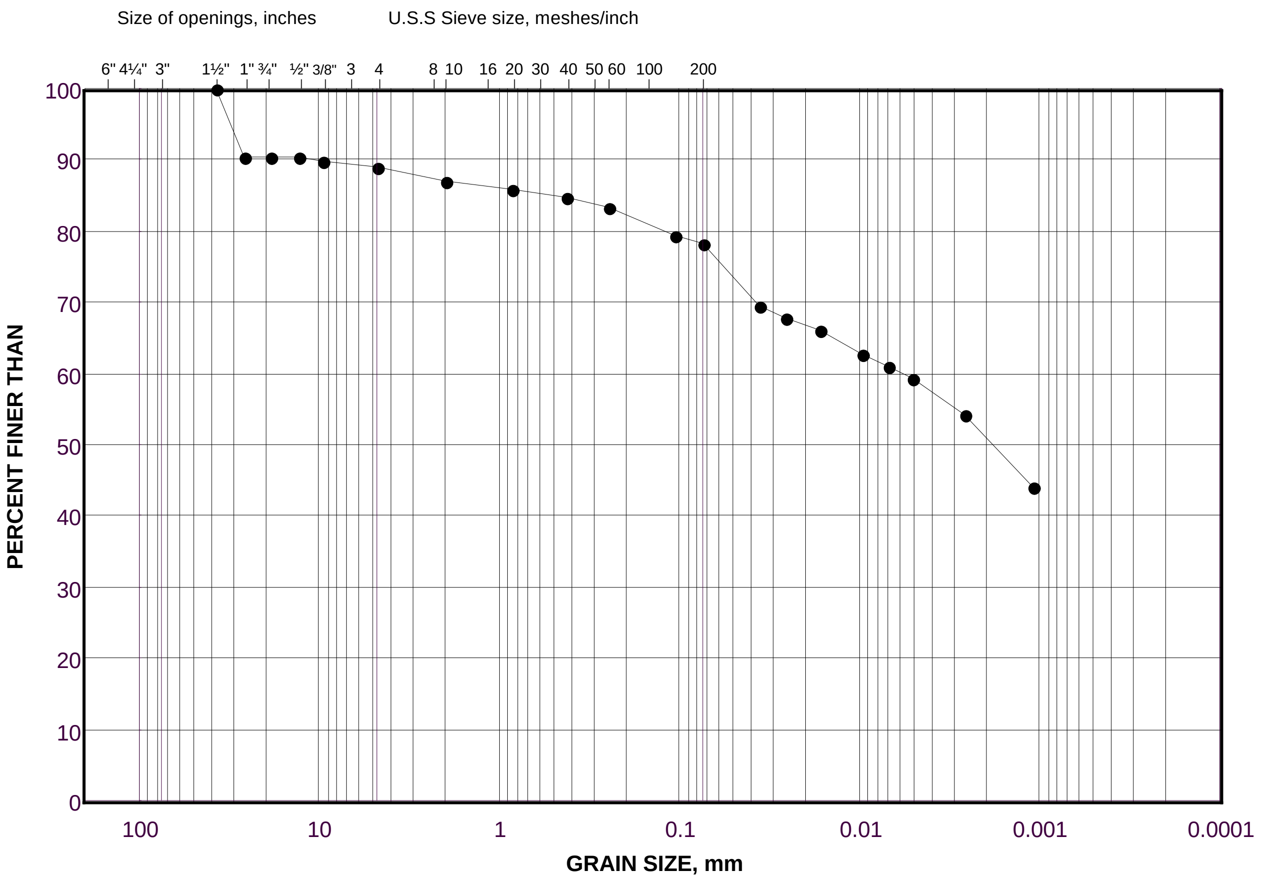
Project No.: 1664178 (2000)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

(CI) SILTY CLAY

FIGURE B-5



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	CNR-102	11	174.7

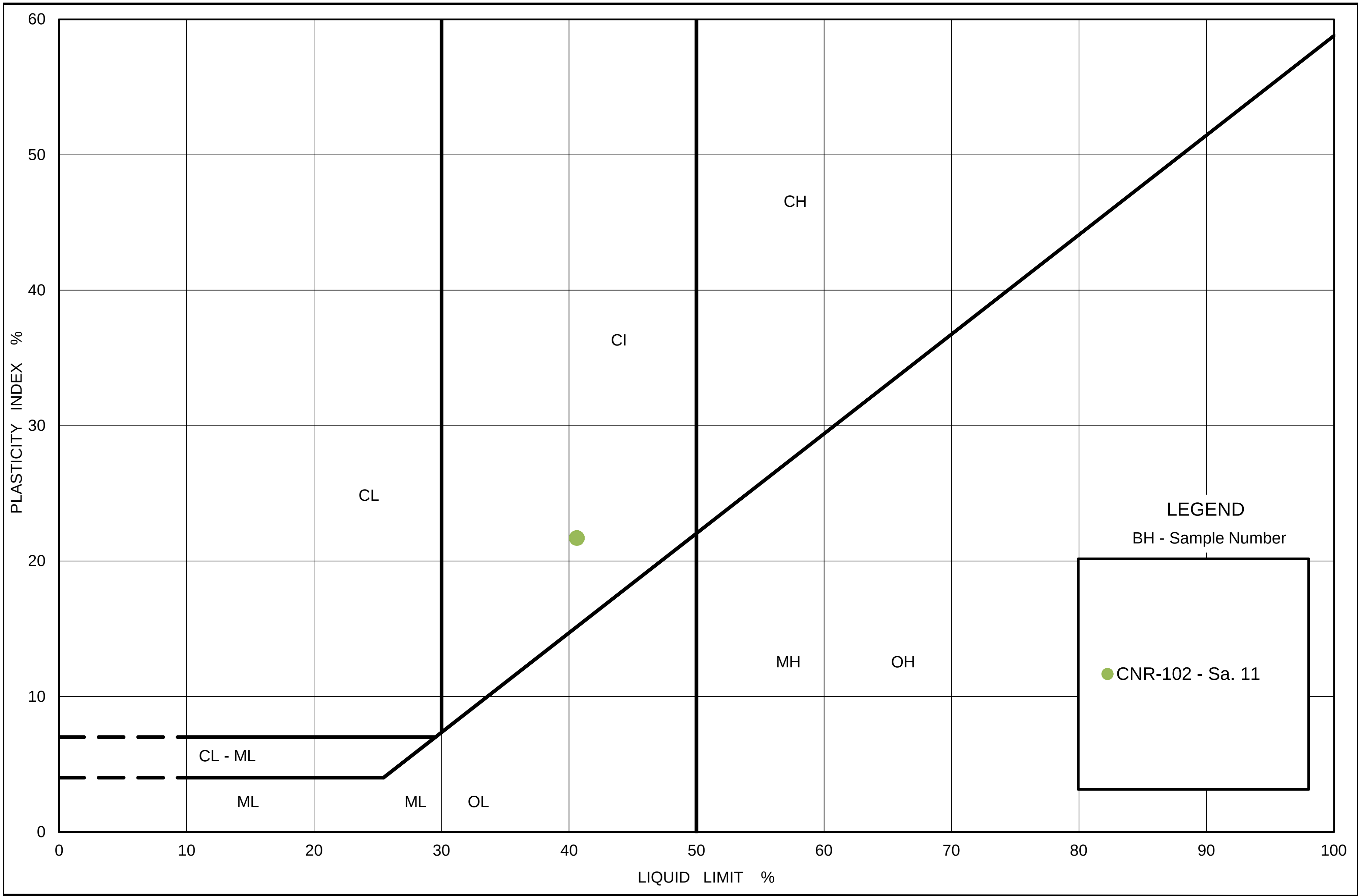
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 03-Feb-19

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



GOLDER

PLASTICITY CHART

(CI) SILTY CLAY

Figure No.: B-6

Project No.: 1664178 (2000)

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