Appendix M.9 – Foundations Report – Stouffville GO Rail Crossing at Austin Drive

Kennedy Road Environmental Assessment between Steeles Avenue and Major Mackenzie Drive

Preliminary Foundation Investigation and Design Report

Kennedy Road and Go Transit Rail Crossing at Austin Drive
Class Environmental Assessment Study for Improvements to Kennedy Road from Steeles Avenue to Major Mackenzie Drive, Markham, Ontario

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July 3, 2020

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT KENNEDY ROAD AND GO TRANSIT RAIL CROSSING AT AUSTIN DRIVE CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by HDR Inc. (HDR) to provide foundation engineering services in support of the Class Environmental Assessment for the proposed improvements to Kennedy Road (Y.R. 3) from Steeles Avenue (Y.R. 95) to Major Mackenzie Drive (Y.R. 25), in the City of Markham, 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 Metrolinx GO Rail crossing at Austin Drive.

The purpose of the investigation was to establish the subsurface soil and groundwater conditions at the GO Rail grade separation 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

An existing GO Rail at-grade crossing is present on Kennedy Road approximately 90 m north of Austin Drive, as shown on Figure 1. For this project, this rail line is referred to as the GO Rail at Austin Drive. Kennedy Road consists of two lanes in each direction with a boulevard and sidewalk on each side of Kennedy Road. Residential developments are located northwest, northeast, and southeast of the crossing and landscaped park lands are located southwest of the crossing.

The grade of Kennedy Road in the vicinity of the GO Rail crossing at Austin Drive is at about Elevation 175.6 m and the surrounding lands are generally flat.

3.0 INVESTIGATION PROCEDURES

The field work for the preliminary investigation was carried out on November 20, 22, and 23, 2018 during which time two boreholes (designated as Boreholes CNR-201 and CNR-202) were advanced near the structure location to a depth of 15.9 m, as shown on Figure 1. A separate borehole (designated as Borehole CNR-202B) was advanced 3 m south of Borehole CNR-202 for well installation purposes. 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 CME 60 drill rig, supplied and operated by Landshark Drilling of Brantford, 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]. At Borehole CNR-202B, split spoon sampling was only carried out between 6.2 m and 8.1 m depth only. 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-202B, in accordance with Ontario Regular 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 boreholes were backfilled with bentonite and the ground surface was restored to near original condition as practical using cold-patch asphalt, as applicable.

Field work was observed by members of Golder's engineering and technical staff, who located the boreholes, 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 locations and ground surface elevations were obtained using a GPS (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 Figures 1 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.

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-201	4,858,566.68	636,272.59	175.7	15.9
CNR-202	4,858,523.09	636,263.23	175.6	15.9
CNR-202B	4,858,519.95	636,262.96	175.6	8.1

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,

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

as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)[2]. The GO Rail crossing at Austin Drive 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 generally 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

The subsurface soil and groundwater conditions encountered in the boreholes, as well as the results of the field and laboratory testing are shown on the attached Record of Borehole sheets in Appendix A. Golder's *"Methods of Soil Classification*", "*Abbreviations and Terms Used on Records of Boreholes and Test Pits"* and *"List of Symbols"* are attached to assist in the interpretation of the borehole records. 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 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, the subsurface conditions generally consist of asphalt/topsoil and fill underlain by alternating native deposits of silty clay, silty sand, a layered silty clay and sandy silt, sand and silt, and sand. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Topsoil/Asphalt

A 100 mm thick asphalt layer was encountered in Borehole CNR-201 at Elevation of 175.7 m. A 50 mm thick topsoil layer was encountered in Borehole CNR-202 at Elevation of 175.6 m.

4.2.2 Fill

Fill was encountered in both boreholes below the topsoil or asphalt and consisted of layers of cohesive clayey silt and non-cohesive sand to gravelly sand. The depth, elevation, thickness, and type of fill is presented below in Table 2.

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[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.

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-201	0.10	175.64	1.45	174.29	1.35	Gravelly Sand
CNR-202	0.05	175.55	1.45	174.15	1.40	Sandy Clayey Silt
	1.45	174.15	2.21	173.39	0.76	Sand

The SPT "N" values measured within the non-cohesive fill layers range from 9 to 28 blows per 0.3 m of penetration indicating a loose to compact level of compaction. Two SPT "N" values measured within the cohesive fill layer are 10 and 14 blows per 0.3 m of penetration indicating a stiff consistency.

A grain size distribution test was carried out on one sample of the gravelly sand fill and the results are shown on Figure B-1 in Appendix B. A single water content of about 10 per cent was measured in the non-cohesive fill.

4.2.3 Silty Clay

A cohesive deposit of silty clay was encountered underlying the fill in Boreholes CNR-201 and CNR-202, at depths of 1.5 m and 2.2 m below ground surface (Elevation 174.3 m and 173.4 m), respectively. The deposit extended to depths of 4.1 m and 5.6 m below ground surface (Elevation 171.6 m and 170.0 m) in Boreholes CNR-201 and CNR-202, respectively. The SPT "N" values measured within the silty clay deposit range from 5 to 19 blows per 0.3 m of penetration, indicating a firm to very stiff consistency.

Grain size distribution testing was carried out on two samples of the silty clay deposit and the results are shown on Figure B-2 in Appendix B. Atterberg limit testing was carried out on two samples of the silty clay deposit and the results indicate liquid limits of about 38 and 42 per cent, plastic limits of about 18 and 19 per cent, and plasticity indices of about 20 and 23 per cent. These test results, which are plotted on a plasticity chart on Figure B-3 in Appendix B, indicate the deposit is a silty clay has intermediate plasticity. The natural water contents measured on two samples of the silty clay deposit are about 17 and 23 per cent.

4.2.4 Silty Sand

A deposit of silty fine sand to gravelly silty sand was encountered below the silty clay deposit in all boreholes. The surface of the deposit was encountered at depths of 4.1 m to 5.6 m below ground surface (Elevation 171.6 m and 170.0 m) in Boreholes CNR-201 and CNR-202, respectively. The deposit extended to depths of 8.2 m and 8.6 m below ground surface (Elevation 167.5 m and 167.0 m) in Boreholes CNR-201 and CNR-202, respectively. During drilling, the augers were grinding at depths of 7.0 m, 7.3 m, and 8.2 m below ground surface in both boreholes. Further, cobble fragments were also observed in the recovered samples within the deposit. It can be inferred that boulders and/or cobbles are present at the depths where the augers were grinding. Previous experience in the region indicates that the glacial deposits contain cobbles and boulders that are not identified by conventional drilling, sampling, and laboratory testing methods.

One SPT 'N' value measured within the upper silty fine sand portion of the deposit in Borehole CNR-201 is 8 blows per 0.3 m of penetrations, indicating a loose density. The SPT 'N' values measured within the gravelly silty sand portion of the deposit range from 27 to 181 blows per 0.3 m of penetration, with one measurement of 183 blows per 0.28 m of penetration in Borehole CNR-202B, indicating a compact to very dense level of compaction.

Grain size distribution tests were carried out on two samples of the silty sand to gravelly silty sand deposit and the results are shown on Figure B-4 in Appendix B. Two natural water contents measured on samples of the deposit are about 7 and 11 per cent.

4.2.5 Layered Silty Clay and Sandy Silt

A cohesive deposit comprised of layers of silty clay and sandy silt was encountered below the gravelly silty sand deposit at depths of 8.2 m and 8.6 m below ground surface (Elevation 167.5 m and 167.0 m) in Boreholes CNR-201 and CNR-202, respectively. The deposit extended to a depth of 14.8 m below ground surface in these two boreholes (Elevation 161.0 m and 160.8 m). The SPT ‘N’ values measured within the layered deposit range from 20 to 47 blows per 0.3 m of penetration indicating a very stiff to hard consistency.

Grain size distribution tests were carried out on one sample of the silty clay portion of the layered deposit and the results are shown on Figure B-5 in Appendix B. Atterberg limit testing was carried out on three samples of the silty clay portion of the layered deposit and measured liquid limits ranging from about 36 to 43 per cent, plastic limits ranging from about 18 to 19 per cent, and plasticity indices ranging from about 18 to 24 per cent. The test results are plotted on a plasticity chart on Figure B-6 in Appendix B and indicate the silty clay portion of the deposit has intermediate plasticity. The natural water contents measured within the deposit range from about 22 to 28 percent.

4.2.6 Sand to Sand and Silt

A lower deposit of sand to sand and silt was encountered below the layered deposit in both boreholes at a depth of 14.8 m below ground surface (Elevations 161.0 m and 160.8 m). The deposit extended to the borehole termination depth of 15.9 m (Elevations 159.9 m and 159.8 m).

Two SPT ‘N’ values measured within the sand to sand and silt deposit were 12 and 37 blows per 0.3 m of penetration indicating a compact to dense level of compaction.

A grain size distribution test was carried out on one sample of the sand deposit and the result is shown on Figure B-7 in Appendix B. One natural water content measured within the sand and silt deposit is about 14 percent.

4.2.7 Groundwater Conditions

The overburden samples obtained from the boreholes were generally wet. Details of the groundwater levels observed in the boreholes upon completion of drilling are summarized on the borehole records. Upon completion of drilling, the groundwater levels in Boreholes CNR-201 and CNR-202 were measured at depths of 10.4 m and 4.3 m below ground surface, respectively. These water level observations will have been influenced by drilling methods, drilling rate and local soil permeability characteristics and, therefore, will not reflect and should not be interpreted as stabilized groundwater levels.

A monitoring well was installed in Borehole CNR-202B and the screen was sealed within the gravelly silty sand deposit and the recorded groundwater levels are summarized in Table 3 below.

Table 3: Depth and Elevation of Measured Groundwater Level

Borehole	Screened Stratigraphy	Depth (m)	Elevation (m)	Date of Measurement
CNR-202B	Gravelly Silty Sand	5.3	170.3	November 23, 2018
		5.6	170.0	November 30, 2018
		5.0	170.6	December 13, 2018

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

5.0 CLOSURE

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

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PART B

PRELIMNARY FOUNDATION DESIGN REPORT
KENNEDY ROAD AND GO TRANSIT RAIL CROSSING AT AUSTIN DRIVE
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

6.0 DISCUSSION AND ENGINEERING INVESTIGATION

This section of the report provides foundation design recommendations for the preliminary design of the Kennedy Road grade separation at the GO Rail crossing located about 90 m north of Austin Drive and about 500 m north of Highway 7, 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 for the structure and to allow for preliminary assessment of permanent slopes and retaining walls, for planning purposes.

Further investigations will be required during Detailed Design to obtain subsurface information at the proposed grade separation 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 structure(s). 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 ground surface elevations at the borehole locations are approximately 175.7 m and 175.6 m. Borehole CNR-201 was advanced north of the railway tracks on the east side of Kennedy Road, and Borehole CNR-202 was advanced south of the railway tracks on the west side of Kennedy Road. Borehole CNR-202B was advanced in the vicinity of Borehole CNR-202 to install a standpipe piezometer.

It is understood that there is currently no preferred option for the proposed rail grade separation and that an underpass (carrying Kennedy Road under the existing GO rail line), an overpass (carrying Kennedy Road over the existing GO rail line) and a hybrid option (lowering the road grade and raising the railway grade) are being considered.

It is assumed that for the underpass alternative at the rail-road grade separation structure (i.e., the lowest point of the depressed corridor) Kennedy Road would be at a depth of about between about 8 m below existing ground surface (i.e., at about Elevation 167.6 m). In the case of an overpass alternative, the bridge structure (and associated approach embankments) would be about 10 m higher than the existing ground surface (i.e., at about Elevation 185.6 m). For the hybrid alternative it is assumed that the road grade would be lowered to about Elevation 171.6 m and the rail grade would be raised to about 179.6 m. For all alternatives, it is assumed that the grade separation structure would be a single span bridge carrying the railway over the road or road over the railway.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary, the proposed grade separation structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors hence having a “typical consequence level” associated with exceeding limits states design. In addition, given the limited investigation carried out at each proposed foundation element, in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a “low degree of site and prediction model understanding." Accordingly, for an overpass structure configuration, the appropriate corresponding ULS and SLS consequence factor, Ψ, geotechnical resistance factors, ϕgu and ϕgs, and embankment settlement factor, ϕgs, from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.9 below.

If an underpass or hybrid configuration is selected the design should be in accordance with the latest version of the *American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering (2018)* as described in the following sections.

6.3 Grade Separation Constructability

Excavations for a depressed corridor, or for shallow spread footings will encounter between about 1.5 m to 2.2 m of variable fill materials overlying a deposit of firm to very stiff silty clay, underlain by a deposit of silty sand to gravely silty sand at depths between about 4.1 m and 5.6 m below ground surface.

Measured groundwater elevations in the monitoring well installed in Borehole CNR-202B which was screened within the gravelly silty sand deposit, at about 5.5 to 7.6 m below ground surface, indicated a groundwater surface at about Elevation 170.6 m, about 3 m above the assumed grade of Kennedy Road for the underpass alternative and about 1 m below the assumed grade for the hybrid alternative. Accordingly, the most significant issue for this site if the underpass alternative is selected for Detailed Design will be short-term and long-term management of groundwater pressures and flow rates to permit construction of foundations on undisturbed conditions and control potential hydraulic uplift pressures on the underside of pavements. During construction, groundwater levels will need to be managed to allow for construction. For long-term conditions management of groundwater pressure and flow will also be required.

In this case, the silty sand to gravelly silty sand soils appear to be of relatively moderate to high permeability, on the order of about $1x10^{-2}$ to $2x10^{-4}$ cm/s. Groundwater levels, coupled with the permeability of the soils found between the depths of about 4 and 8 m may render the use of passive groundwater control systems impractical. Pending additional subsurface explorations, hydrogeologic evaluations and permitting studies, it may be feasible to design the underpass using groundwater cut-off walls, especially since the site appears to be underlain by a relatively thick and low permeability silty clay layer between the depths of about 8 and 15 m.

Supplementary internal drainage and pumping system would also likely be required. In this case, the structure and permanent retaining walls will be required to be designed for the full hydrostatic pressure. In accordance with AREMA (2018), a Factor of Safety of 2 is required for uplift resistance. Therefore, in order to satisfy this requirement, raft structures (i.e., for an underpass base slab) deeper than about 5 m below current ground surface will require a concrete base slab having a thickness that counteracts the water pressures sufficiently to increase the Factor of Safety against basal instability to 2. Depending on the final width of the roadway, it may also be possible to structurally tie the base slab to permanent secant pile walls and resist hydraulic uplift with a structural base slab and the frictional uplift resistance of the retaining walls. Consideration could also be given to installing anchors to "tie-down" the base slab and resist the uplift pressures. Permanent anchors, however, require specialized corrosion protection systems, particularly when installed beneath roadways that are subjected to de-icing salts. At this site, a full-depth groundwater cut-off wall surrounding the roadway where it is lower than the groundwater levels with an internal passive groundwater pressure relief and pumping system may be the most practical alternative for an underpass design.

For the underpass and hybrid alternatives, retaining walls will be required along the east and west sides of Kennedy Road. The vertical walls of the depressed corridor may be supported with a continuous secant pile (caisson) wall, soldier pile and lagging walls with a concrete panel facing or, a conventional cantilever retaining wall or Retained

Soil System (RSS) wall may be constructed; however, if a permanent groundwater management system is not incorporated into the design for the underpass alternative the retaining walls will need to be designed to be waterproof and should be connected to the base slab and designed for uplift resistance. Where space and property permit, and if a permanent groundwater management system is incorporated into the design, permanent cut slopes may be constructed at no steeper than 2H:1V.

Alternatively, the grade separation could be designed as an overhead structure to carry Kennedy Road over the GO rail line and, from a foundations perspective, an advantage of the overhead structure is long-term groundwater management or design for uplift resistance would not be required. Where space permits the approach embankment north and south of the rail line can be constructed with side slopes at 2H:1V or flatter. Alternatively, RSS or cast-in-place concrete walls could be constructed to retain the approach embankment.

6.4 Foundation Options

Both shallow and deep foundations options have been considered for support of the abutments for the proposed rail-road grade separation structure at Kennedy Road. Based on the subsurface conditions encountered in the boreholes advanced at the site, the following foundations recommendations were considered for preliminary design of the grade separation structure:

- **Spread or strip footings founded on native soil deposits:** Based on the limited number of boreholes advanced at this site shallow spread footings founded on the upper silty clay deposit are not considered capable of providing sufficient resistance to support spread footings. Strip/spread footings founded on the gravelly silty sand (for a hybrid alternative) or the lower varved silty clay and sandy silt deposit (for an underpass alternative) are considered feasible for the support of the grade separation structure. During construction, groundwater levels will be required to be below the proposed underside of the footings in order to minimize disturbance to the excavation base.

- **Spread or strip footings “perched” on a compacted granular pad in the approach embankments:** For an overpass configuration, shallow footings “perched” within the proposed Kennedy Road approach embankments are feasible for support of the proposed structure and could minimize the depth of excavation below the existing grade; however, the anticipated time dependant settlement associated with the approach embankment construction would also affect the settlement of the perched strip/spread footings, and the requirement for settlement mitigation measures may make this alternative impractical.

- **Steel H-piles or pipe piles founded within the compact to dense sand to sand and silt deposits:** Driven steel H-piles or steel pipe (tube) piles could also be considered for support of the proposed bridge abutments. In this case, the piles would their resistance from both side friction and end-bearing. Design tip elevations will vary depending on the grade separation configuration selected for design (either underpass, overpass or hybrid). As inferred from grinding of the augers during borehole advancement and given that the site soils are glacially derived, the presence of cobbles and boulders within the native soil deposits should be anticipated which could affect deep foundation installation. If driven piles are considered necessary for structural reasons (e.g., integral abutments), it may be prudent or necessary to extend boreholes deeper than those completed during the preliminary explorations to determine whether conditions conducive to higher pile end bearing stresses are found within reasonable and practical depths.

- **Drilled shafts (caissons) founded within the very stiff to hard silty clay and sandy silt or compact to dense sand to sand and silt deposits:** Drilled shafts are considered feasible for support of the abutments for the proposed grade separation. In general, temporary liners filled with water, or controlled-density drilling fluids, as well as tremie concrete may be required during caisson installation to control the ground and groundwater within these water-bearing zones. At this site the caissons will develop their resistance from side friction and end bearing, depending on the loading conditions and tip elevations. Design tip elevations will also need to vary depending on the grade separation configuration selected for design (either underpass, overpass or hybrid) to provide a minimum caisson length below the underside of pile cap as discussed in Sections 6.5 and 6.6. As inferred from grinding of the augers during borehole advancement and given that the site soils are glacially derived, the presence of cobbles and boulders within the native soil deposits should be anticipated which could affect deep foundation installation. As for driven piles, during subsequent explorations for final design it may be prudent or necessary for deeper boreholes.

From a foundations perspective an overpass structure to carry Kennedy Road over the GO rail line is preferred as compared to an underpass structure as this alternative minimises the requirement for deep excavations and permanent groundwater control, or construction of a water-tight, below-grade structure. Preliminary recommendations for the design of the foundations for an underpass alternative are provided in Section 6.5 and for an overpass are provided in Section 6.6. Recommendations for design of a hybrid alternative will consist of a combination of the recommendations for the underpass and overpass alternative as noted in the following sections.

6.5 Foundation Recommendations – Underpass

6.5.1 Shallow Foundations

The retaining walls and the undercrossing or hybrid bridge structures may be founded on conventional spread/strip foundations, depending on settlement tolerances and constructability considerations. Provided appropriate groundwater control is implemented during construction and protection against freezing is provided, spread foundations for the bridge abutments may be founded on the dense to very dense gravelly silty sand or very stiff to hard clayey silt and sandy silt deposits.

All spread footings should be provided with a minimum of 1.4 m of soil cover for frost protection, as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Foundation Frost Depths for Southern Ontario*) and in accordance with Section 3.2.4.3 of the *American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering*. In addition, the footings should extend below any existing fill and the near surface firm to stiff silty clay deposit. In Borehole CNR-202, the firm to stiff silty clay deposits extends to a depth of about 5.6 m below ground surface (Elevation 170.0 m).

For an underpass finished road grade at Elevation 167.6 m, the spread footings would have to therefore extend 1.4 m below the finished road grade to provide for adequate soil cover; this corresponds to a founding level of Elevation 166.2 m.

For a hybrid alternative with finished road grade at about Elevation 171.6 m, the spread footings would have to therefore extend 1.4 m below the finished road grade to provide for adequate soil cover; this corresponds to a founding level of Elevation 170.2 m.

Spread footings placed on properly prepared subgrade, at or below the maximum founding elevations given above should be designed based on an allowable bearing capacity (in accordance with the *AREMA Manual for Railway Engineering*) defined using working stress methods rather than limit states design methodologies currently used in Ontario. The allowable bearing capacity, similar to the Serviceability Limit State (SLS) condition, for footings with a width of about 3 m is 200 kPa. The allowable bearing capacity is dependent on the footing width and founding elevation and as such, the values used in design should be reviewed if the footing width is different than that specified above or if the founding elevation differs from that given above.

The allowable bearing capacity provided above assume 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 considered, in accordance with Section 3.5 of the *AREMA Manual for Railway Engineering*.

The exposed base 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 bearing pressures. The founding soils 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/loosening before footing construction. Where sub-excavation is required, the sub-excavated area should be backfilled with granular material meeting OPSS.MUNI 1010 (Aggregates) Granular ‘A’ or Granular ‘B’ Type II that is placed and compacted in accordance with OPSS.MUNI 501 (Compacting).

A coefficient of friction of 0.40 may be used in the assessment of sliding resistance between the cast-in-place concrete footing and the native soils (in accordance with Section 5.4.2 of the *AREMA Manual for Railway Engineering*).

6.5.2 Deep Foundations – Driven Piles

As an alternative to conventional shallow foundations, the retaining walls and track bridge structure could be supported by deep foundations. Assuming that stiff to hard silty or compact to dense silty sand soils extend well below the bottom of the existing borehole depths, HP 310x110 or heavier section piles driven to the tip elevation of about 151 m (for an underpass configuration) or 155m (for a hybrid alternative) may be designed for preliminary purposes using an allowable shaft bearing capacity of 800 kN. Assuming an underside of pile cap elevation at a minimum 1.4 m below finished grade to provide adequate protection from frost effects, the driven piles will be about 15 m long. The boreholes advanced as part of the current investigation did not penetrate to the tip elevation given above and additional information (i.e. deeper boreholes) will be required at the preliminary and/or Detailed Design stage to confirm the type and density/consistency of the soils to below the design pile tip elevation. Higher capacities may be able to be achieved if driven piles extended deeper into the hard/dense native soil deposits (following confirmation of the presence of these deposits at a greater depth).

The underside of the pile caps should be founded at a minimum depth of 1.4 m below the lowest surrounding grade including measured perpendicular to any sloping ground to provide adequate protection against frost penetration, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario)* and in accordance with Section 3.3.1 of the *American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering*.

For the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the native soil deposits. The piles should be reinforced at the tip with specialized driving shoes for protection during driving in accordance with OPSS 903 (*Deep Foundations*) and OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*). In these soils, welded flange plates used for tip reinforcement may not provide enough protection. In very dense / hard and / or boulder soils, as may be encountered at this site, driving shoes such as

Titus Standard “H” Bearing Pile Points are preferred over flange plates. Heavier pile sections may also be preferable to reduce the potential for damage in hard driving conditions.

6.5.3 Deep Foundations – Drilled Shafts

As an alternative to conventional shallow foundations or driven piles, the underpass or hybrid structures could be founded on approximately 8 m long drilled shaft (caisson) foundations. Based on the available subsurface information, drilled shafts installed to the tip elevation of about 158 to 159 m (for an underpass alternative) or 162 m for a hybrid alternative may be designed using the allowable bearing resistance given in Table 4 below.

Table 4:Preliminary Allowable Design Bearing Resistance for Drill Shafts

Nominal Caisson Diameter	Founding Stratum	Preliminary Allowable Design Bearing Resistance (kN)[1]
0.9 m	Compact to dense sand or very stiff to hard silty clay and sandy silt	1,300
1.2 m		2,000
1.5 m		2,500

1. Based on a factor of safety of 2.5 and assuming drilled shafts are constructed below groundwater level (AREMA).

The above allowable bearing resistance is based on the caissons installed from a depth of 1.4 m below the anticipated road grade elevation and assuming that similar conditions identified at the bottom of the boreholes completed for this report extend well below the explored depths. Greater resistances may potentially be achieved for deeper caissons pending additional exploration and testing. Deeper boreholes will be required for Detailed Design of any deep foundation alternative.

In accordance with Section 24.3.4.2 of the *AREMA Manual for Railway Engineering* a factor of 0.67 must be applied to the above allowable resistances for drilled shafts that have a centre-to-centre spacing of three times the diameter of the caisson. For caissons that have a centre-to-centre spacing of eight times the diameter apart the reduction factor is 1.0 (i.e. no reduction for the group effects is required).

For the installation of drilled shafts, consideration must be given to the potential presence of cobbles and boulders within the native soil deposits. Appropriate construction equipment and techniques must be selected to penetrate the anticipated cobbles and boulders. Given the presence of saturated cohesionless soil deposits, particularly the gravelly silty sand layer, temporary steel liners are required to stabilize the sides and base of the augered holes. The relatively high groundwater pressures in the non-cohesive soils may cause difficulties during caisson installation. A sufficient head of drilling slurry may need to be maintained within the caisson liner and concrete will need to be placed using tremie methods.

The performance of drilled shafts will depend upon the final cleaning and verification of the subgrade quality at the base of the drilled shaft. Each drilled shaft excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The inspection of the base of the drilled shafts can be accomplished by means of observing the base cleaning processes by qualified personnel, probing, using appropriate steel bar on a wireline and Shaft Inspection Devices (SID). Should the inspection indicate that loosened/unacceptable soil is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected. A Foundation Engineer must confirm that the conditions encountered are consistent with the information obtained from the boreholes and that the drilled shaft holes and base have been properly prepared. Concrete must be placed using tremie methods immediately following cleaning and inspection of the base. Concrete placement should also be observed by qualified foundation engineering staff to observe mixed cuttings and concrete that rise to the surface as the high-slump concrete displaces these materials to the top of the column.

6.6 Foundation Recommendations – Overpass Structure

As an alternative to undercrossing, an overpass structure could be considered for the proposed Kennedy Road / GO Rail crossing at Austin Drive grade separation. The overpass structure alternative can reduce the total bulk excavation as compared to the undercrossing alternative and would be a better alternate to short-term and long-term management of groundwater levels.

The overhead structure abutments can be supported on deep foundations, as the near surface soils consist of firm to stiff silty clay and are not considered capable of supporting the overpass structure.

6.6.1 Deep Foundations – Driven Piles

Based on the available subsurface information, HP 310x110 piles driven to the tip elevation of about 159.2 m may be designed using a factored axial resistance at ULS of 900 kN. The factored resistance at SLS for 25 mm of settlement will be greater than the factored resistance at ULS for this type and length of pile. Assuming an underside of pile cap elevation of 174.2 m (minimum 1.4 m below finished grade to provide adequate protection from frost effects), the driven piles will be about 15 m long. It should be noted that the resistances given above are based on an assumption that the compact to dense sand deposit (present at the bottom of both boreholes advanced at this site) extends to below the tip elevation provided. The boreholes advanced as part of the current investigation did not penetrate to the tip elevation given above and additional information (i.e. deeper boreholes) will be required at the preliminary and/or Detailed Design stage to confirm the type and density/consistency of the soils to below the design pile tip elevation. Higher capacities may be able to be achieved if driven piles extended deeper into the hard/dense soil deposits (following confirmation of the presence of these deposits at a greater depth).

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

For the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the native soil deposits. The piles should be reinforced at the tip with specialized driving shoes for protection during driving in accordance with OPSS 903 (*Deep Foundations*) and OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*). In these soils, welded flange plates used for tip reinforcement may not provide enough protection. In very dense / hard and / or boulder soils, as may be encountered at this site, driving shoes such as Titus Standard “H” Bearing Pile Points are preferred over flange plates. Heavier pile sections may also be preferable to reduce the potential for damage in hard driving conditions.

6.6.2 Deep Foundations – Drilled shafts

As an alternative to driven piles, the overpass structure could be founded on approximately 19 drilled shaft (caisson) foundations. Based on the available subsurface information, drilled shafts installed to the tip elevation of about 164.2 m may be designed for preliminary purposes using the factored axial resistance at ULS and factored resistance at SLS (for 25 mm of settlement) given in Table 5 below.

Table 5: Overpass - Factored Geotechnical Resistance for Drilled Shafts

Caisson Diameter	Founding Stratum	Factored Geotechnical Resistance at ULS (kN)	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement (kN)
0.9 m	Very stiff to hard silty clay and sandy silt	1,300	--[1]
1.2 m		2,000	--[1]
1.5 m		2,900	--[1]

1. The factored serviceability geotechnical resistance (for 25 mm of settlement) is greater than the factored ultimate geotechnical resistance therefore the factored ultimate geotechnical resistance will govern the design.

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

For the installation of drilled shafts, consideration must be given to the potential presence of cobbles and boulders within the native soil deposits. Appropriate construction equipment and techniques must be selected to penetrate the anticipated cobbles and boulders. Given the presence of saturated cohesionless soil deposits, temporary steel liners will be required to stabilize the sides and base of the augered holes. The relatively high groundwater pressures in the non-cohesive materials may cause difficulties during caisson installation. A sufficient head of drilling slurry may need to be maintained within the caisson liner and concrete will need to be placed using tremie methods.

The performance of drilled shafts will depend upon the final cleaning and verification of the subgrade quality at the base of the drilled shaft. Each drilled shaft excavation should be carefully cleaned to remove all loosened debris to ensure that the concrete is in intimate contact with the competent bearing stratum. The inspection of the base of the drilled shafts can be accomplished by means of observing the base cleaning processes by qualified personnel, probing, using appropriate steel bar on a wireline and Shaft Inspection Devices (SID). Should the inspection indicate that loosened/unacceptable soil is present at the base of the drilled shaft, the base would need to be re-cleaned and re-inspected. A Foundation Engineer must confirm that the conditions encountered are consistent with the information obtained from the boreholes and that the drilled shaft holes and base have been properly prepared. Concrete must be placed using tremie methods immediately following cleaning and inspection of the base. Concrete placement should also be observed by qualified foundation engineering staff to observe mixed cuttings and concrete that rise to the surface as the high-slump concrete displaces these materials to the top of the column.

6.6.3 Downdrag

As a result of the loading from the new approach embankments and any proposed grade raise, consolidation settlement of the underlying cohesive soil deposits will occur over a time period of about 6 months following fill placement. The difference in the vertical movement between the thick overburden (i.e., from the consolidation settlement and creep of the cohesive deposits) and the piles may result in the development of negative skin friction and downdrag on the piles and drilled shafts (caissons), depending on construction staging.

If the piles/drilled shafts for the abutments are installed prior to construction of the approach embankments then in accordance with the requirements of the Canadian Foundation Engineering Manual (2006), an assessment is required to be carried out to estimate if the structural capacity of the steel H-pile would be exceeded when taking into account the factored dead load combined with the downdrag load. The magnitude of anticipated settlements within the likely construction staging schedule and the potential for development of downdrag loads should be assessed at detailed design.

If the structural capacity is exceeded mitigation measures can be considered at detailed design such as preloading, use of a pile section with a higher structural capacity, use of light-weight fill and high-grade steel piles.

6.7 Approach Embankments

For an overpass configuration, approach embankment pavement elevations may be as much as 10 m above the surrounding grades, and for a hybrid alternative, approach embankments are estimated to be about 4 m above the surrounding grade. Prior to construction of the new approach embankments any topsoil/organic soils and loosened/softened fill should be stripped from within the embankment footprint.

Fill for construction of the new embankments should consist of Granular ‘B’ Type I, Type II or Select Subgrade Material meeting the specifications of OPSS.MUNI 1010 (Aggregates). The embankment fill should be placed and compacted in accordance with OPSS.MUNI 501 (Compacting) and OPSS.MUNI 206 (Grading). Embankment side slopes should be constructed no steeper than 2 horizontal to 1 vertical (2H:1V) in granular or earth fill.

To control erosion of the side slopes, a minimum 2 m wide bench is recommended where embankment slopes are greater than 8 m in height, such that the uninterrupted slope height does not exceed 8 m, consistent with OPSD 202.010 (Slope Flattening). To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.MUNI 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw mats, 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.

Limit equilibrium slope stability analyses were performed on the approach embankment side slopes using the commercially available program “Slide V.2018” published by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum factored FoS of 1.5 is adopted for the design of embankment slopes under static conditions at the end of construction as per the CHBDC (2014). This FoS is considered adequate for the embankments at this site considering the design requirements provided that a suitable number of boreholes are completed during the Detailed Design stage to confirm the anticipated subsurface conditions as described herein. The stability analyses were performed to assess if the target minimum FoS was achieved for the design embankment height and geometries. For the new granular/earth fill and native soil deposits, effective stress parameters were employed in the analysis assuming drained conditions and the parameters were estimated from empirical correlations using the in situ SPT ‘N’-values. The correlations proposed by Terzaghi and Peck (1967) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils. The analysis indicates that the north and south approach embankments constructed of granular fill will have a factored FoS of 1.5 or greater against global instability.

Settlement of the founding soils under the north and south approach embankment areas can be expected as a result of the loading from the new fills on the existing fill and native soil deposits. Settlement of new granular fill that is properly placed and compacted for construction of the widened embankments would occur during construction.

To estimate the magnitude of the expected settlements of the subgrade material, analyses were carried out using hand and spreadsheet calculations. The immediate compression of the existing fill and native cohesionless deposits was modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). Based on the analysis carried out the settlement of the up to 10 m high approach/high fill embankment is expected to be about 150 mm, and settlement of the up to 4 m high approach embankment (for a hybrid alternative) is expected to be about 60 mm these settlements are expected to occur within about 6 months following construction.

6.8 Permanent Retaining Walls

Where the proposed grade separation profile is above the anticipated groundwater level, conventional open cut excavations with temporary side slopes excavated no steeper than an inclination of 1 horizontal to 1 vertical (1H:1V) are considered feasible. If permanent slopes are considered, they should be designed with inclinations no steeper than 3H:1V. Where the proposed profile extends below the groundwater table, excavation support (shoring) will likely be required to reduce the lateral extent of the excavations.

Retaining walls will be required to provide support of a depressed corridor to reduce the lateral extent of the excavations along the Kennedy Road and at the abutments for the undercrossing. Considering that competent soils are present at about 4 m to 5.5 m below existing ground surface along the length of the depressed corridor, the use of spread footings placed on the native gravelly silty sand deposit is considered to be the most practical option for foundation support for a majority of the retaining wall alignment. For shorter walls, with a lightly loaded foundation, strip footings founded within the firm to stiff silty clay deposit designed based on a factored ULS resistance of 100 kPa and a SLS resistance of 75 kPa could also be considered. As noted above a multi-level dewatering system for construction as well as either a permanent groundwater cut-off system (e.g., secant pile wall) with a supplementary internal drainage and pumping system, or a long-term groundwater drawdown pumping system will be required. Alternatively, consideration could be given to a tied back soldier pile and concrete panel wall for the retaining structure, if an easement for the soil anchors extending behind the walls can be obtained.

As previously noted, cobbles and boulders are also expected to be present throughout the native soil depsoits which may affect the installation of temporary or permanent retaining walls/excavation support systems.

6.8.1 Permanent Ground Anchors

Permanent anchors extending into the dense to very dense gravelly silty sand deposit or very stiff to hard silty clay and sandy silt may be used for support of the permanent retaining walls or as "tie-downs" for resistance to hydraulic uplift pressures on base slabs for the lowered roadway corridor. Soil anchors for retaining structures must be designed to accommodate the loads applied from lateral static and seismic earth pressures and surcharge pressures from area, line or point loads (such as train loading) and account for any sloping ground behind the retaining wall system. The retaining walls should be designed to provide adequate drainage behind the walls or hydrostatic pressures based on an assumed groundwater elevation of about 171 m should be considered. The ground anchors should be designed based on the Recommendations of Prestressed Rock and Soil Anchors by Post Tensioning Institute during the Detailed Design phase.

Anchors may be sized based on the following preliminary ultimate adhesion capacities acting between the grout and soil in Table 6 below.

Table 6: Preliminary Ultimate Adhesion Capacities Between Grout and Soil

Soil Type	Single-Stage Grouted Anchors	Secondary Grouted Anchors
Dense to very dense gravelly silty sand	125 kPa	300 kPa
Very stiff to hard silty clay and sandy silt	50 kPa	150 kPa

For design of the permanent anchors, a minimum Factor of Safety of 3.0 should be used. The sustained working load should not be greater than 60 percent of the ultimate tensile strength of the anchor tendons or bars. The fixed length (bond zone) of the anchors should be maintained behind a line drawn upward at 45 degrees from the base of the piles. The permanent soil/bedrock anchors should be provided with double or triple corrosion protection. In some cases, sacrificial steel thicknesses for casings and/or tension reinforcement can also be considered to reduce the risks associated with long-term corrosion potential. A fixed anchor length of at least 5 m but not greater than 10 m is recommended for soil anchors. Where multiple anchors are used, a minimum spacing between anchors in a line should be at least 4 anchor diameters to avoid group effects.

All the ground anchors shall be designed as per Post Tensioning Institute Manual. The soil anchor capacity should be confirmed by carrying out full-scale performance tests to 1.5 times the design load on a minimum of 10 percent of the total number of anchors. All anchors should be proof-loaded to 1.25 times the design load and locked off at 1.1 times the design load. Anchor testing should be supervised by a qualified geotechnical engineer.

The global stability of retaining structures will be dependent on the type of wall, its geometry and location relative to adjacent structures, and the engineering characteristics of the fill and native soils. Without details regarding planned cut depths and retaining wall geometry, it is not possible to appropriately assess specific global stability factors of safety for retaining structures at this site. However, given the native soil conditions, provided that groundwater is appropriately controlled for both temporary and permanent conditions, global stability factors of safety for retaining walls constructed at this site are expected to be satisfactory.

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

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

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

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

6.9.1 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 Select Subgrade Material for the general embankment fill:

Table 7: 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
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:

Table 8: 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. 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.

6.10 Construction Considerations

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

6.10.1 Temporary Excavation

Temporary excavations for the undercrossing and/or foundation construction will be made through the existing cohesive and non-cohesive fill and into the firm to very stiff silty clay, loose silty sand and dense to very dense gravelly silty sand deposits. 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. For preliminary planning purposes, the existing fill and loose or firm to stiff native soil deposits would be classified as Type 3 soil and the dense to very dense native deposits would generally be considered Type 2 soils, according to the OHSA, where these materials are above groundwater levels. Below groundwater levels, granular materials, such as the gravelly silty sand layer, would be categorized as Type 4 soils. The dense to very dense non-cohesive soils below the groundwater level may, on initial excavation behave similarly to soils categorized as Type 3 but will degrade toward a Type 4 depending on the time of exposure and conditions of any underlying soils (i.e., Type 4 soils will disturb and undermine overlying soils that would otherwise be more stable). Any categorization of materials made to address OHSA requirements must be reviewed and evaluated at the time the ground is exposed during construction since the construction operations (e.g., dewatering and drainage) and groundwater conditions at the time will influence actual soil behaviour. Temporary excavations above the water table or within effectively dewatered materials should be made with side slopes no steeper than 1H:1V extending upwards and outwards from the base of the excavation. Surface water should be directed away from all excavations.

If temporary excavation support is provided using trench liner boxes, it should be noted that the boxes are intended only for the protection of workers and do not prevent movements of the adjacent soil. Any voids between the outside of the liner box and the adjacent soil face should be filled immediately with free draining granular material. Support will be required for any existing infrastructure within the zone of influence of all excavations as defined by a line drawn upward and outward from the base of the excavation at an inclination of 1H:1V.

6.10.2 Groundwater and Surface Water Control

The groundwater level measured in the monitoring well installed in Borehole CNR-202B, which was screened in the gravelly silty and deposit, was measured at an elevation of about 170.6 m, or about 5.0 m below ground surface.

For deeper portions of the excavation (i.e., below elevations of about 170.5 m) proactive dewatering and depressurization of the non-cohesive water-bearing layers will be required, likely using vacuum well points, eductors, or pressure relief wells installed at properly spaced intervals. The well points and eductors or deep well systems will likely be required to control the groundwater flows from the granular soils into the open excavations.

Some relief of these dewatering requirements could be made should the project area below the anticipated water level be enclosed within a cut-off wall system that is socketted into the lower silty clay deposit found at elevations of about 167.8 m to 167.0 m. Provided the system is sufficiently “water tight”, a sump and pump operation may be sufficient to remove the water to complete construction in the dry; however, as water drains from permeable layers within the cut-off excavation, flowing ground conditions can develop, and construction planning would need to account for such conditions.

For an underpass configuration, permanent drainage of the roadway and pavements will be required to reduce the potential for hydraulic uplift. During subgrade preparation, a subdrain system could be installed beneath the pavement granular materials at an appropriate depth and spacing and of proper size to collect groundwater and direct it to a dedicated outlet. Depending on the retaining structure type selected to construct the underpass, effective drainage behind the walls may also be required. It is expected that either a deep gravity sewer or a pumping station will be required to manage groundwater flows. Further, it is highly recommended that a redundant means discharging the groundwater be incorporated into the design. Prior to final design, detailed hydrogeological explorations, testing and analyses will be needed to better define anticipated short-term and long-term flow rates.

If permanent dewatering/drainage is prohibited, the structure should be designed to resist uplift pressure based on a design water level at Elevation 170.6 m using either a thickened bottom slab or vertical anchors extending into the underlying very stiff to hard silty clay and sandy silt strata. Suitable anchor design capacities are noted above in Section 6.8.1.

Control of the surface water will be necessary to allow excavation and foundation construction to be carried out in dry conditions. Precipitation runoff in the construction area should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade or granular backfill / bedding material.

6.10.3 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 grade separation.

These temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary protection systems should meet Performance Level 2 as specified in OPSS 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 soldier pile and concrete panel system would be suitable for the temporary excavation support at this structure site, based on the anticipated subsurface soil and groundwater conditions, and provided that adequate groundwater control is in place.

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 phase.

6.10.4 Obstructions During Pile Driving or Drilling Operations

The glacially derived soils at the site should be expected to contain cobbles and boulders, which could affect the installation of deep foundations, soil anchors, dewatering systems and/or excavation protection/support systems. If driven pile foundations are adopted, it is recommended that driving shoes be used to facilitate pile driving to minimize damage to pile tips, as described above. If drilling methods are to be used for installation of foundations, soil anchors and dewatering systems, the equipment and methods should be selected to permit penetration of cobbles and boulders.

6.10.5 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 be placed on the subgrade of foundation excavations within four hours after preparation, inspection and approval of the subgrade.

6.10.6 Settlement and Vibration Monitoring During Construction

Settlement monitoring of existing, new or temporary railway tracks will be required during and following construction of the structure and/or approach embankments, as applicable. During the Detailed Design stage a settlement monitoring program should be designed in accordance with the latest version of the *American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering*.

There are existing residences and commercial complexes surrounding the site and while it is expected that vibration levels resulting from installation of piles or temporary protection systems will not reach these thresholds at this structure, it is considered prudent that pre- and post-construction condition surveys and vibration monitoring at or near the buildings should be considered to defend against potential damage claims associated with vibration-inducing activities at the site. A PPV threshold of 25 mm/s is generally considered applicable for residential buildings and 50 mm/s applicable for steel/concrete commercial buildings.

The owner of any utilities located within a 200 m radius of the site should be consulted to determine the sensitivity of the utilities to ground vibrations. Requirements for vibration monitoring and PPV thresholds should be developed in consultation with the utility owner, as required.

7.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION WORK DURING DETAILED DESIGN

Additional exploration and testing should be completed during the Detailed Design of the proposed grade separation. Additional boreholes should be advanced within the footprint of the proposed foundation elements and the approach embankments or depressed corridor 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 the organic content and environmental quality (for excess soil management/disposal) of the fill deposit;
- Confirm depths to materials suitable for end-bearing piles and drilled shafts;
- Evaluate the lateral capacities of pile foundations;
- Test parameters used to assess the corrosive potential of the soil to concrete and buried steel;
- Confirm the groundwater elevation in the non-cohesive deposits;
- Measure the in situ hydraulic conductivity and interconnection, if any, of interbedded granular layers; and
- 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 deep foundations.

8.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Matthew Kelly, P.Eng., and reviewed by Mr. Storer Boone, P.Eng. a senior geotechnical engineer and Principal 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.

Matthew Kelly, P.Eng.	Storer J. Boone, Ph.D., P.Eng.
Geotechnical Engineer	*MTO Tunneling Specialist, Principal*

MWK/SJB/cr/rb;mes

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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.

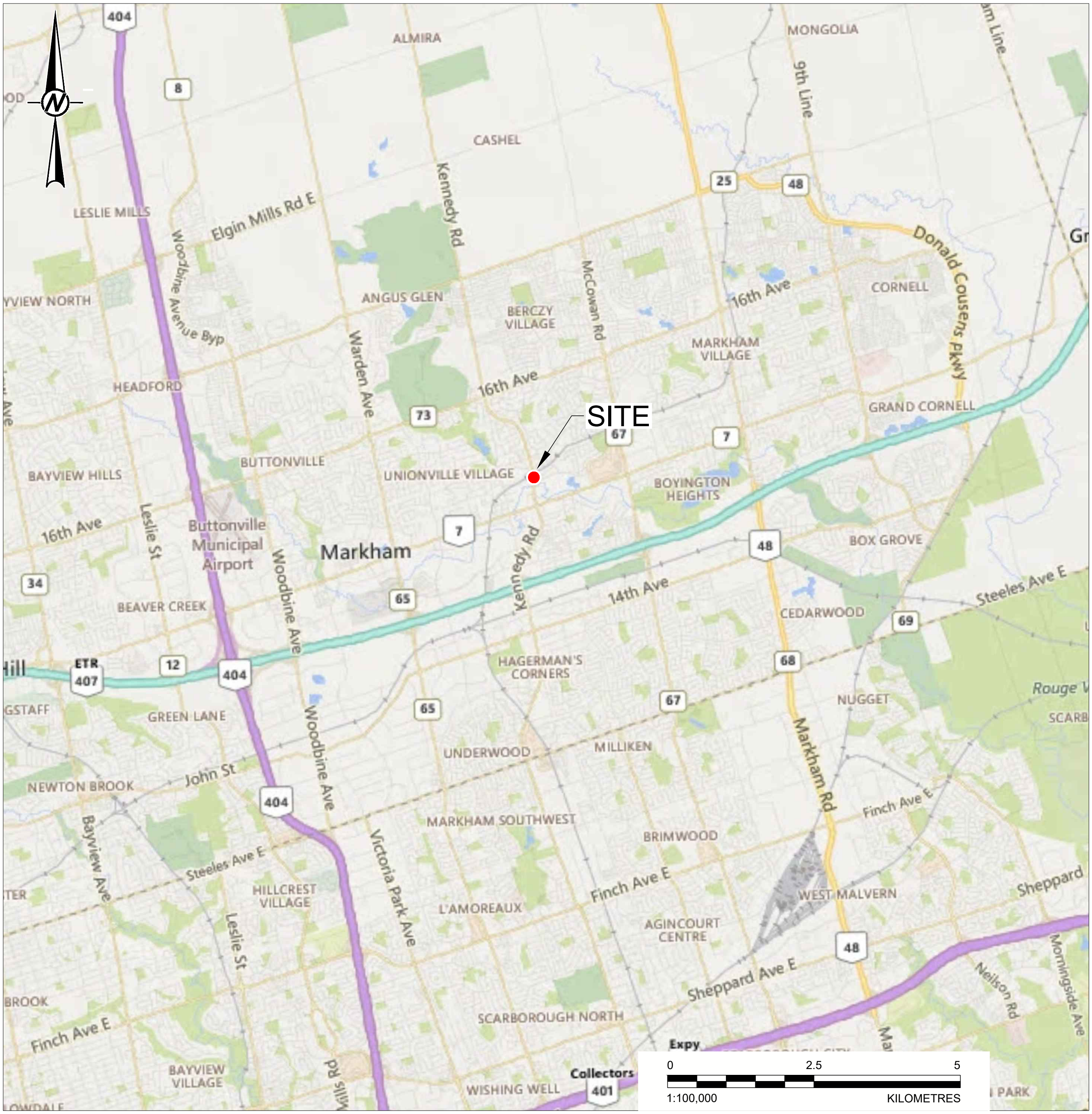
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater 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 AND GO RAIL CROSSING AT AUSTIN DRIVE
CLASS ENVIRONMENTAL ASSSESSMENT 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	2019-10-25
DESIGNED	
PREPARED	STB
REVIEWED	AMP
APPROVED	SEMP

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

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25 mm
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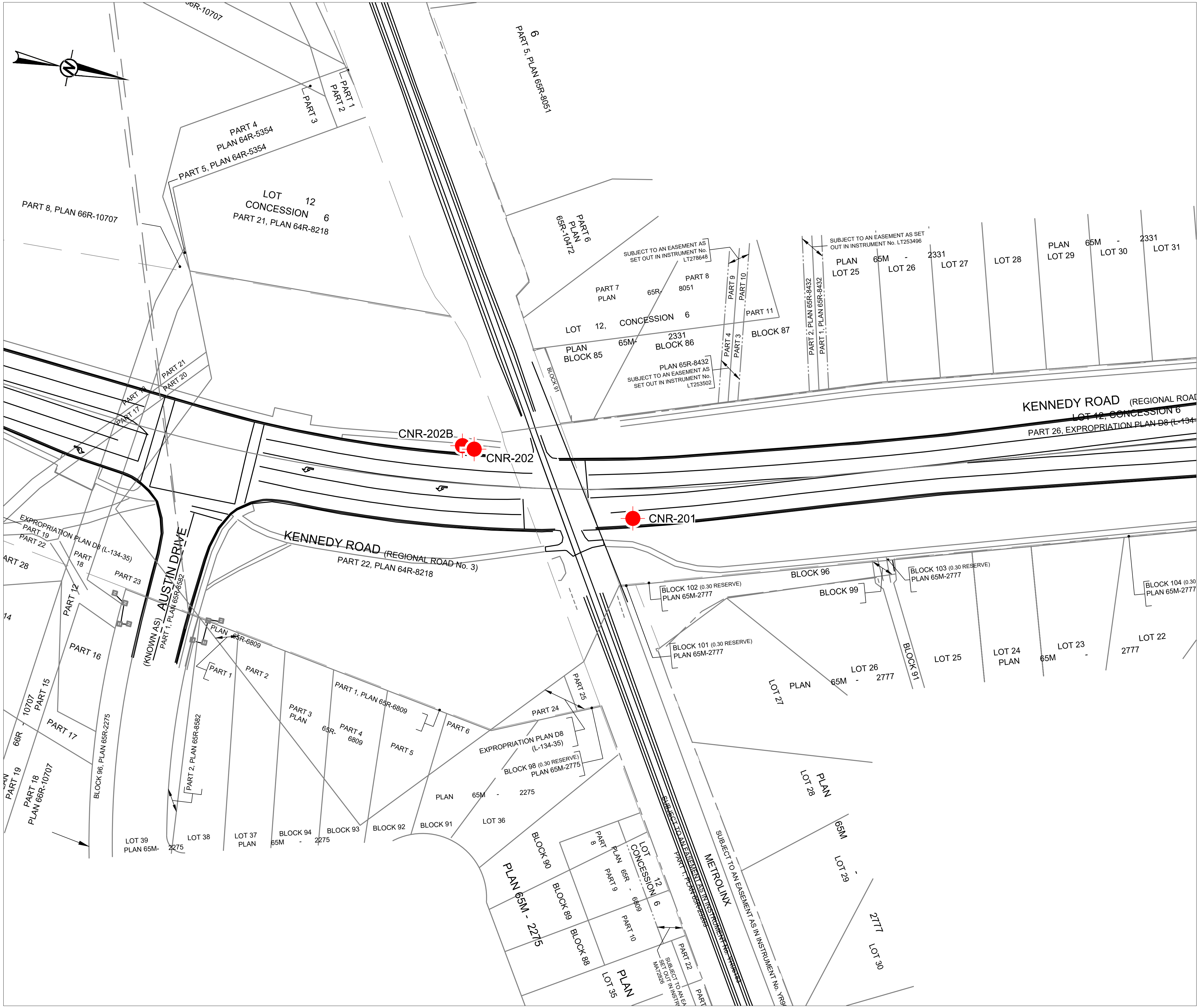

LEGEND

● 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 GO RAIL CROSSING AT AUSTIN DRIVE
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	2019-10-25
DESIGNED	
PREPARED	STB
REVIEWED	AMP
APPROVED	SEMP

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

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25 mm

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APPENDIX A

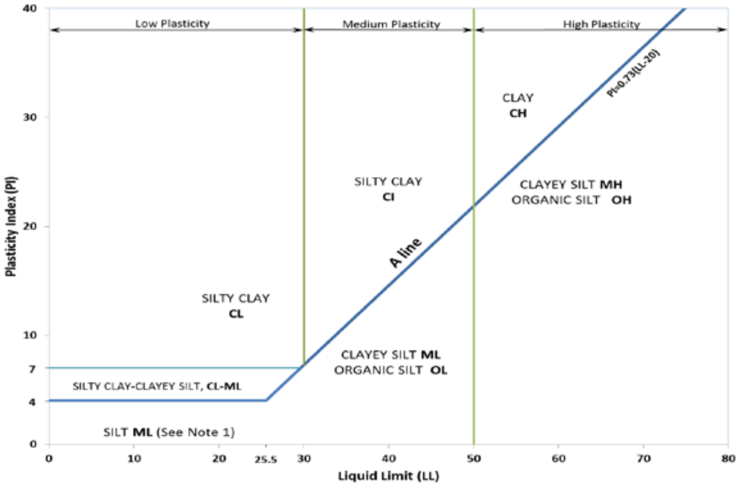
Borehole Records

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.

GOLDER

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

(d) Shear Strength

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

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)
LOCATION: N 4858566.68; E 636272.59

RECORD OF BOREHOLE: CNR-201

BORING DATE: November 20, 2018

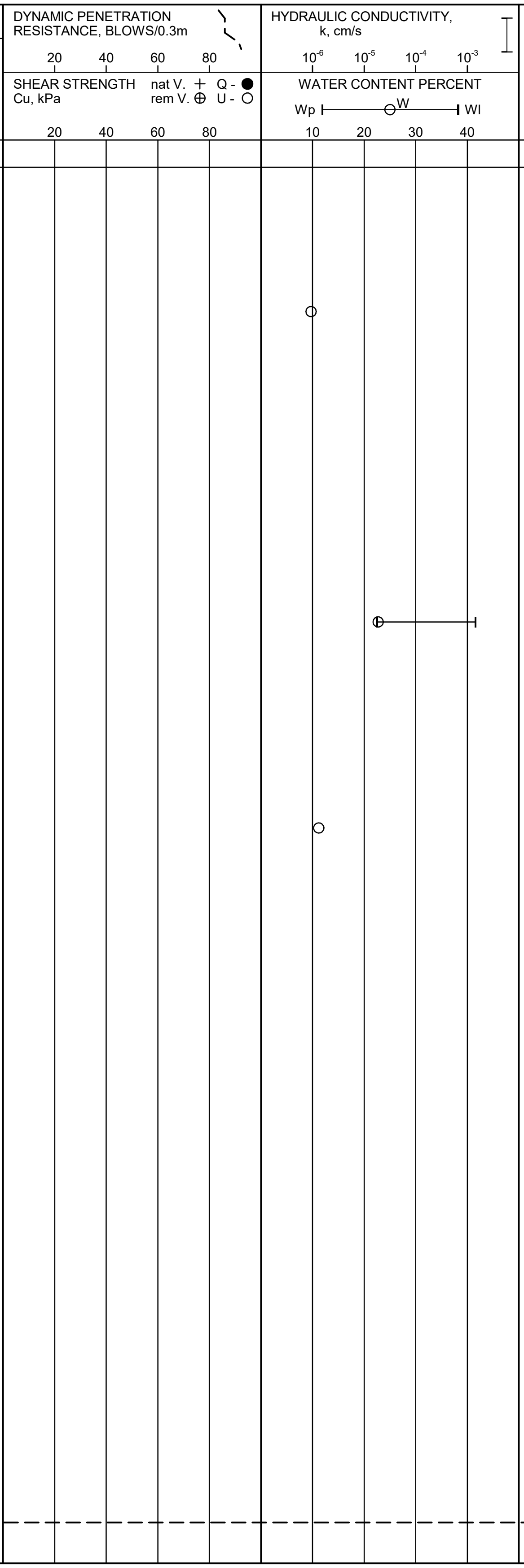
SHEET 1 OF 2
DATUM: Geodetic

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

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

DESCRIPTION	ELEV. / DEPTH (m)
GROUND SURFACE	175.74
ASPHALT (100 mm)	0.00
FILL - (SP) gravelly SAND, some fines; brown; non-cohesive, moist, loose to compact	0.10
(CI) SILTY CLAY, trace to some sand, trace to some gravel; brown to grey; cohesive, w~PL, firm to very stiff - Oxidation staining from 2.3 m to 2.9 m depth - Becoming grey at 3.1 m depth	174.29 1.45
(SM) SILTY fine SAND, some gravel; grey; non-cohesive, wet, loose	171.63 4.11
(SM) gravelly SILTY SAND; grey, trace cobble fragments; non-cohesive, moist, dense - Augers grinding at 7.0 m depth - No soil recovery from Sample 8	170.10 5.64
(CI and ML) SILTY CLAY and sandy SILT; grey, layered; cohesive, w~PL to w>PL, very stiff to hard	167.51 8.23

SAMPLES NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	SS	28	
2	SS	9	M
3	SS	8	
4	SS	10	
5	SS	19	MH
6	SS	8	MH
7	SS	30	
8	SS	33	
9	SS	32	



PIEZOMETER OR STANDPIPE INSTALLATION

DEPTH SCALE METRES: 0 1 2 3 4 5 6 7 8 9 10

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DEPTH SCALE
1 : 50

GOLDER

LOGGED: JS
CHECKED: AMP

PROJECT: 1664178 (2000)

LOCATION: N 4858566.68; E 636272.59

RECORD OF BOREHOLE: CNR-201

SHEET 2 OF 2

DATUM: Geodetic

BORING DATE: November 20, 2018

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

HAMMER TYPE: AUTOMATIC

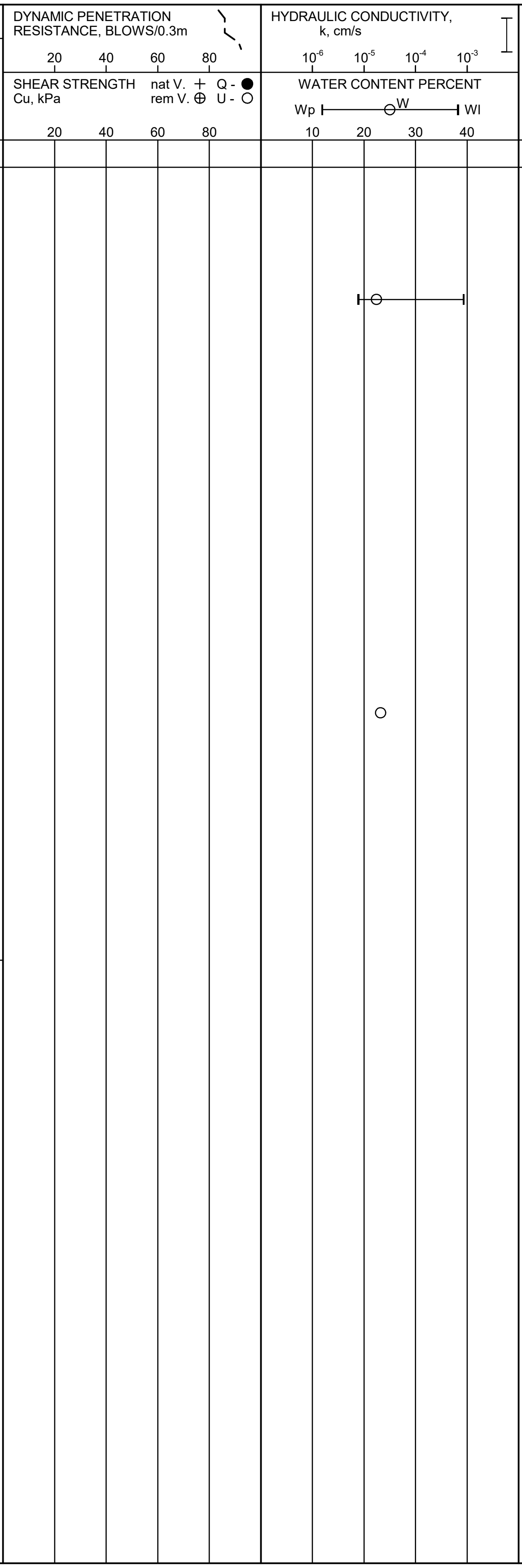
--- *CONTINUED FROM PREVIOUS PAGE* ---

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	PIEZOMETER OR STANDPIPE INSTALLATION
10	Power Auger 216 mm O.D. Hollow Stem Augers	(CI and ML) SILTY CLAY and sandy SILT; grey, layered; cohesive, w~PL to w>PL, very stiff to hard	▨					∇ 20-Nov-18
11					10	SS	23	
12					11	SS	28	
14					12	SS	35	
		(SW) SAND and SILT; grey; non-cohesive, wet, dense		160.96 14.78				
15					13	SS	37	
		END OF BOREHOLE		159.89 15.85				

NOTES:

1. Borehole open upon completion of drilling.

2. Groundwater measured in open borehole at a depth of 10.4 m below ground surface (Elev. 165.3 m) upon completion of drilling.



DEPTH SCALE

1 : 50

GOLDER

LOGGED: JS

CHECKED: AMP

PROJECT: 1664178 (2000)
LOCATION: N 4858523.09; E 636263.23

RECORD OF BOREHOLE: CNR-202

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	GROUND SURFACE	175.60					
0	TOPSOIL (50 mm)	0.00					
	FILL - (CL-ML) sandy CLAYEY SILT, some gravel; brown and grey, organic staining; cohesive, w<PL, stiff	0.05	1	SS	10		
1			2	SS	14		
	FILL - (SP) SAND, some gravel; grey; non-cohesive, wet, compact	174.15 1.45	3	SS	25		
2	(CI) SILTY CLAY, trace to some sand, trace to some gravel; brown to grey; cohesive, w~PL, firm to stiff	173.39 2.21	4	SS	9	MH	
3	- Becoming grey at 3.0 m depth		5	SS	9		
4							▽ 22-Nov-18
5			6	SS	5		
	(SM) gravelly SILTY SAND; grey; non-cohesive, wet, dense to very dense	169.96 5.64					
6	- Trace cobble fragments at 6.4 m depth		7	SS	41		
7	- Augers grinding at 7.3 m depth						
8			8	SS	181		
	- Augers grinding at 8.2 m depth						
	(CI and ML) SILTY CLAY and sandy SILT; grey and light brown, layered; cohesive, w>PL, very stiff to hard	166.99 8.61					
9			9	SS	47	MH	
10							

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

CONTINUED NEXT PAGE

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JS
CHECKED: AMP

PROJECT: 1664178 (2000)

LOCATION: N 4858523.09; E 636263.23

RECORD OF BOREHOLE: CNR-202

BORING DATE: November 22, 2018

SHEET 2 OF 2

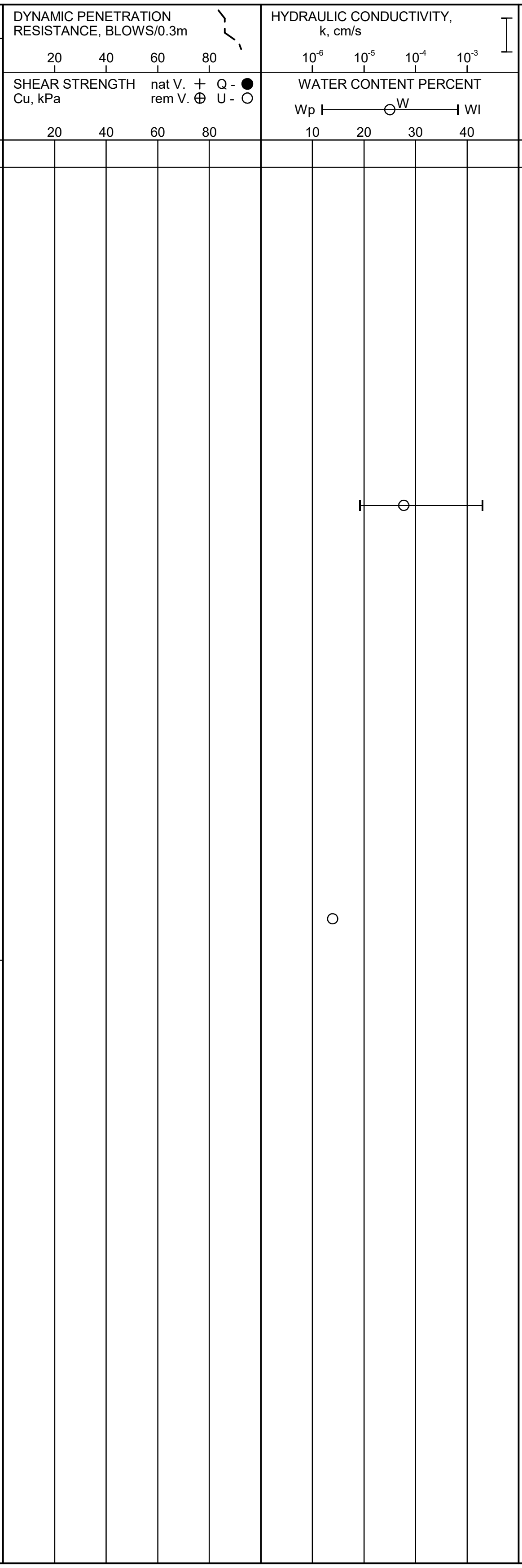
DATUM: Geodetic

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

HAMMER TYPE: AUTOMATIC

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

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	SAMPLES TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	--- CONTINUED FROM PREVIOUS PAGE ---							
10	(CI and ML) SILTY CLAY and sandy SILT; grey and light brown, layered; cohesive, w>PL, very stiff to hard	▨						
11				10	SS	27		
12				11	SS	20		
14			160.82 14.78	12	SS	43		
15	(SP) SAND, some fines; grey; non-cohesive, wet, compact - 1.8 m of heave inside augers; N value may not be representative		159.75 15.85	13	SS	12	MH NP	
16	END OF BOREHOLE							



NOTES:

1. Borehole caved to a depth of 14.4 m below ground surface upon completion of drilling.

2. Groundwater measured in open borehole at a depth of 4.3 m below ground surface (Elev. 171.3 m) upon completion of drilling.

3. NP = Non-plastic

DEPTH SCALE

1 : 50

GOLDER

LOGGED: JS

CHECKED: AMP

PROJECT: 1664178 (2000)
LOCATION: N 4858519.95; E 636262.96

RECORD OF BOREHOLE: CNR-202B

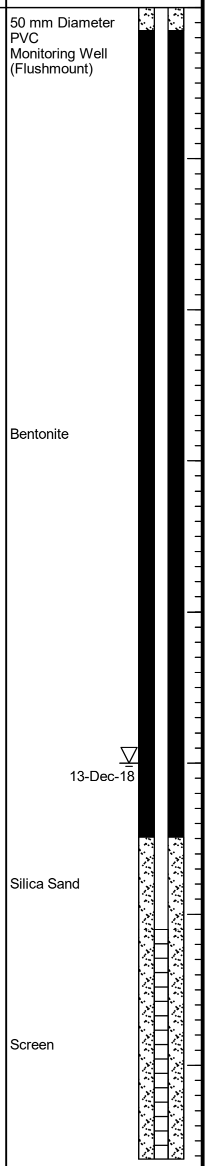
BORING DATE: November 23, 2018

SHEET 1 OF 1
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	SAMPLES: TYPE	SAMPLES: 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		175.57							
0	Power Auger 216 mm O.D. Hollow Stem Augers	Refer to Record of Borehole: CNR-202 for soil stratigraphy details from 0.0 m to 6.1 m depth - Augers grinding at 0.6 m		0.00							50 mm Diameter PVC Monitoring Well (Flushmount)
6		(SM) gravelly SILTY SAND; grey; non-cohesive, moist, compact to very dense		169.47 6.10	7B	SS	27		○	MH	
					8B	SS	183/ 0.28				
8		END OF BOREHOLE		167.52 8.05							



NOTES:

1. Water level measured in monitoring well as follows:

Date	Depth (m)	Elev. (m)
23-Nov-18	5.3	170.3
30-Nov-18	5.6	170.0
13-Dec-18	5.0	170.6

DEPTH SCALE
1 : 50

GOLDER

LOGGED: JS
CHECKED: AMP

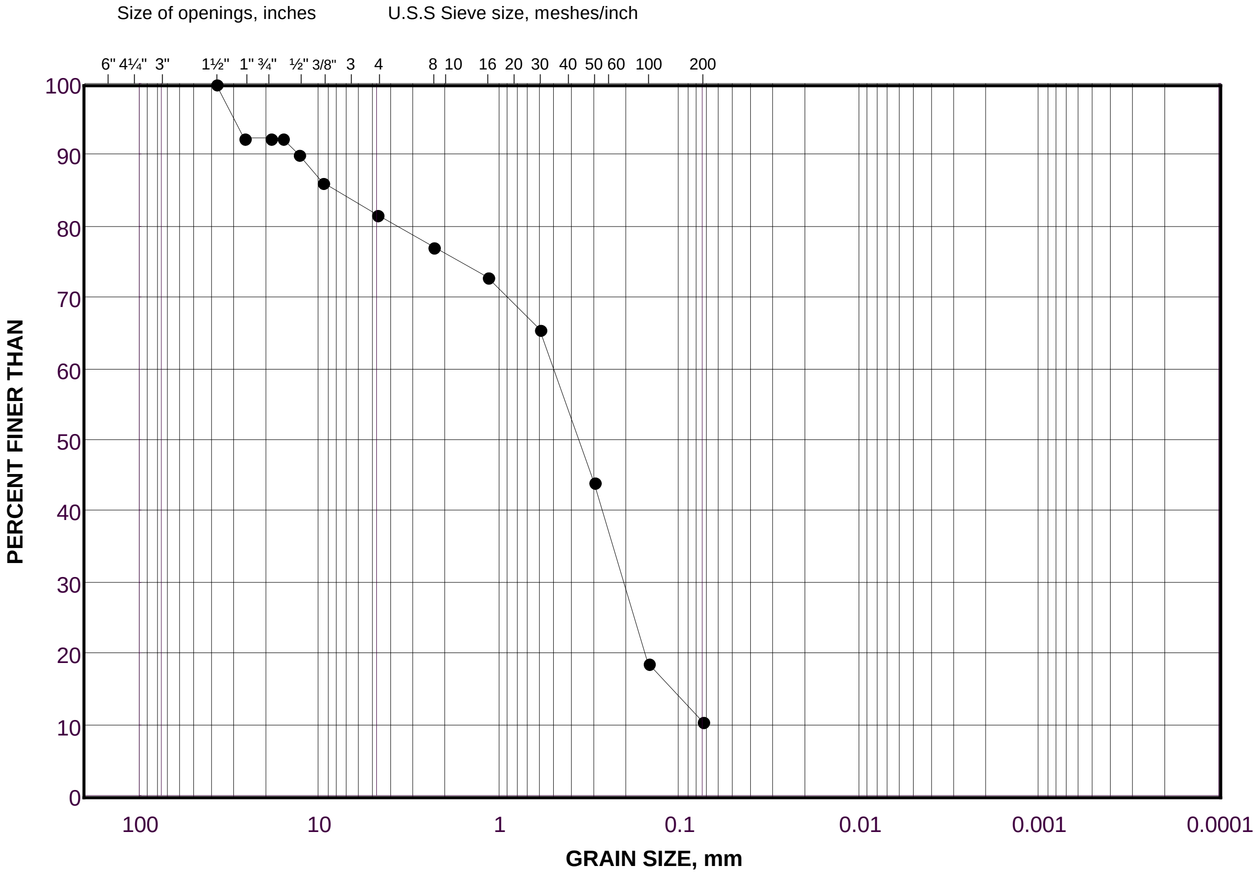
APPENDIX B

Geotechnical Laboratory Results

GRAIN SIZE DISTRIBUTION

FILL - (SP) Gravelly SAND

FIGURE B-1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	CNR-201	2	174.6

Project Number: 1664178 (2000)

Checked By: AMP

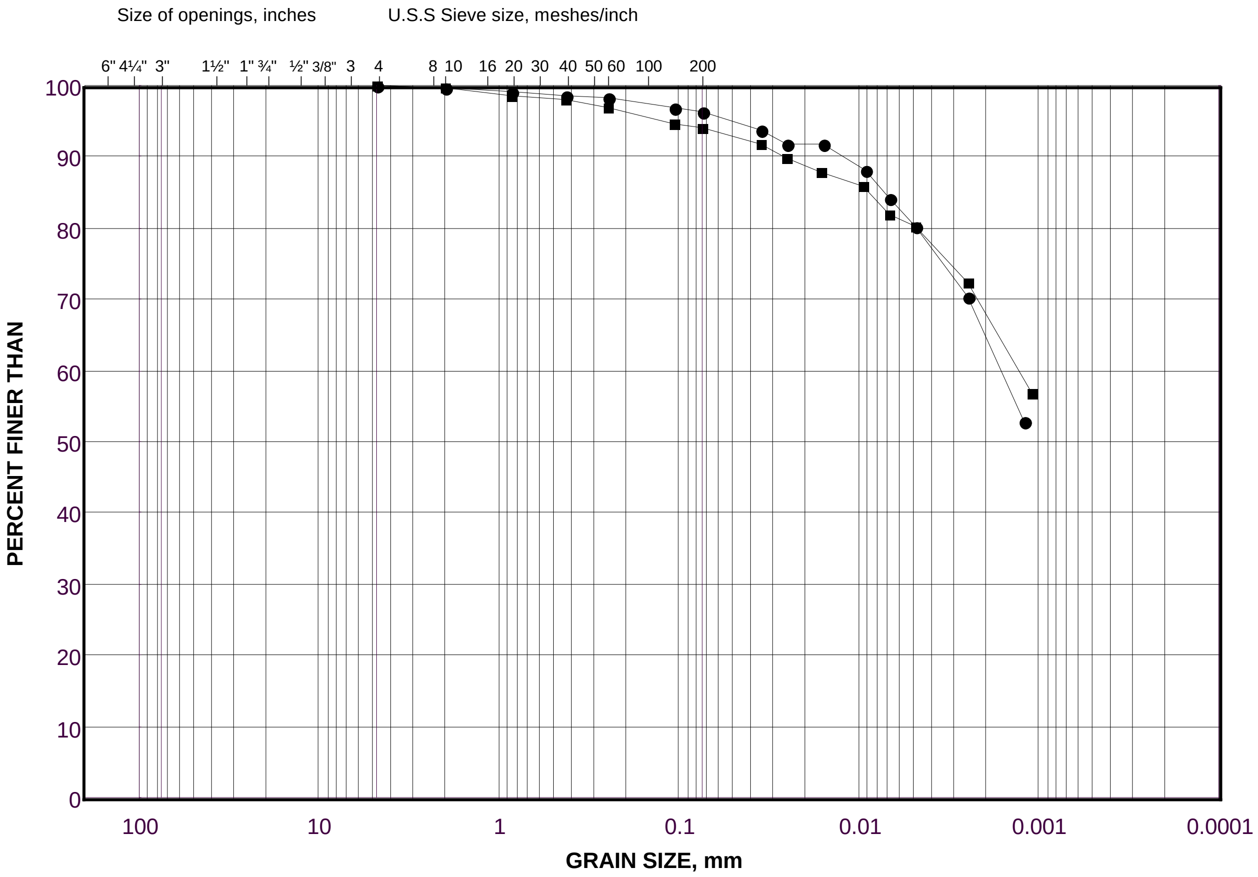
Golder Associates

Date: 05-Jan-19

GRAIN SIZE DISTRIBUTION

(CI) SILTY CLAY

FIGURE B-2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	CNR-202	4	173.0
■	CNR-201	5	172.4

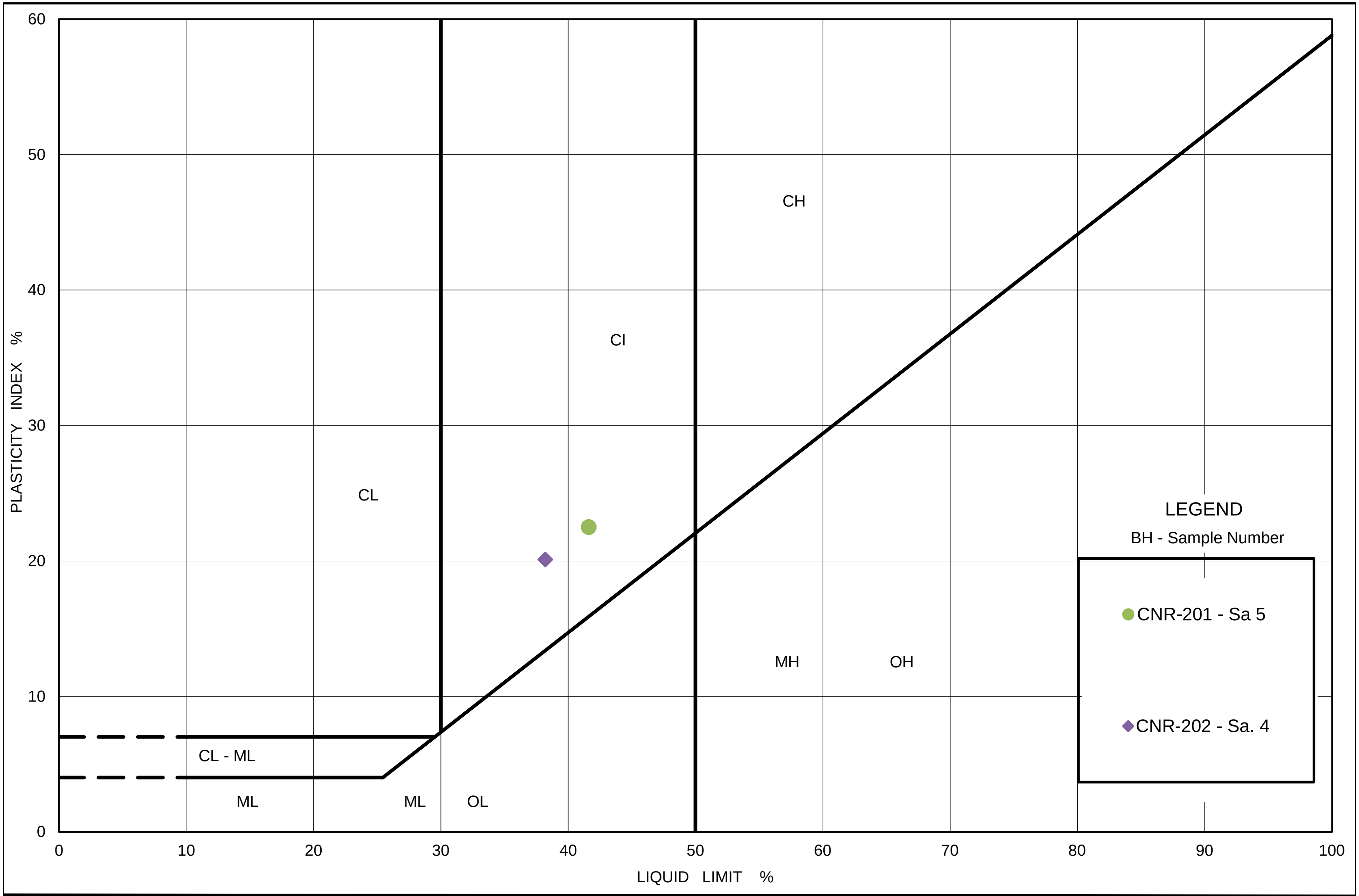
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 05-Jan-19

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

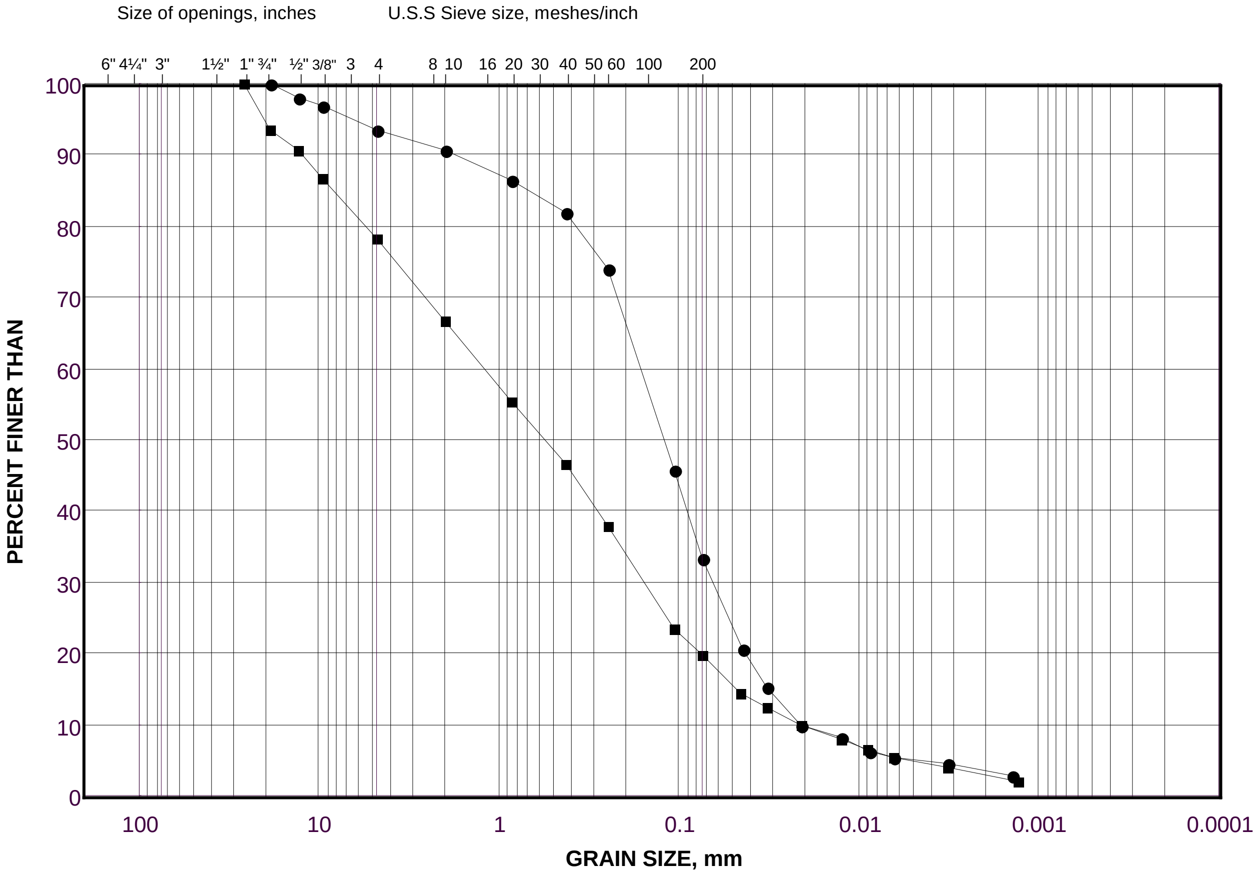


GOLDER	**PLASTICITY CHART**	Figure No.: B-3
	(CI) SILTY CLAY	Project No.: 1664178 (2000)
		Checked By: AMP

GRAIN SIZE DISTRIBUTION

(SM) SILTY SAND to (SM) gravelly SILTY SAND

FIGURE B-4



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	CNR-201	6	170.7
■	CNR-202B	7B	169.2

Project Number: 1664178 (2000)

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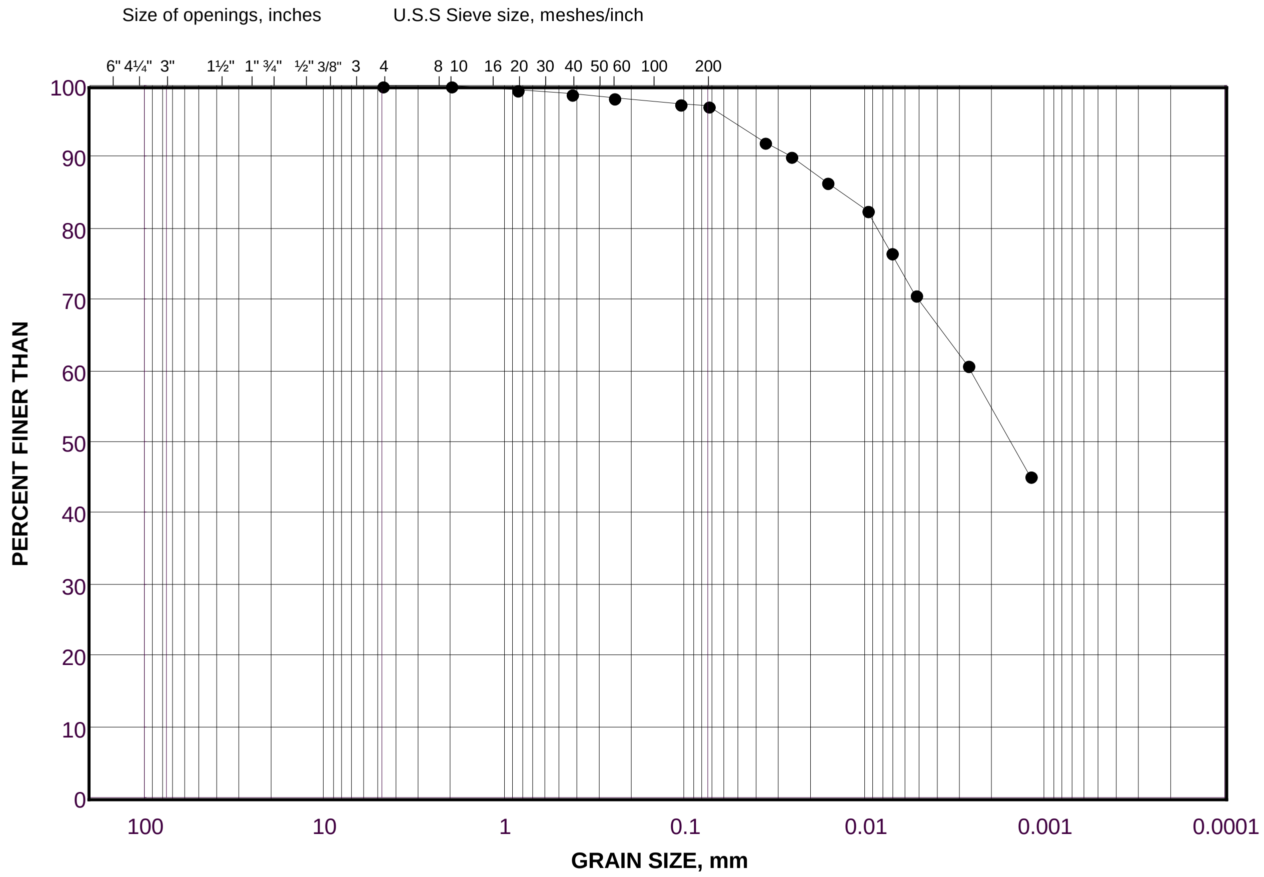
Date: 20-Feb-19

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-202	9	166.2

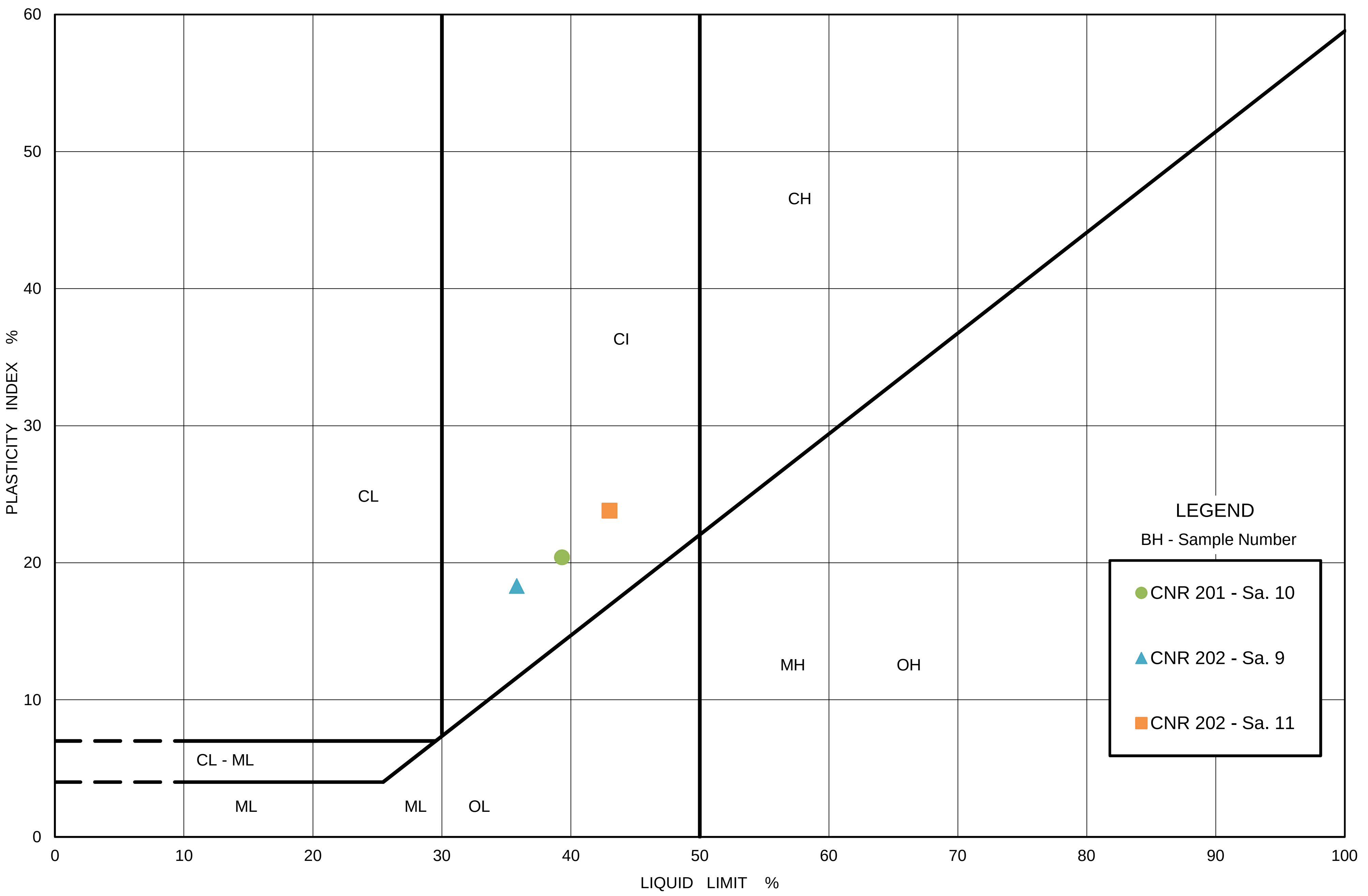
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 05-Jan-19

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



GOLDER

PLASTICITY CHART

(CI) SILTY CLAY

Figure No.: B-6

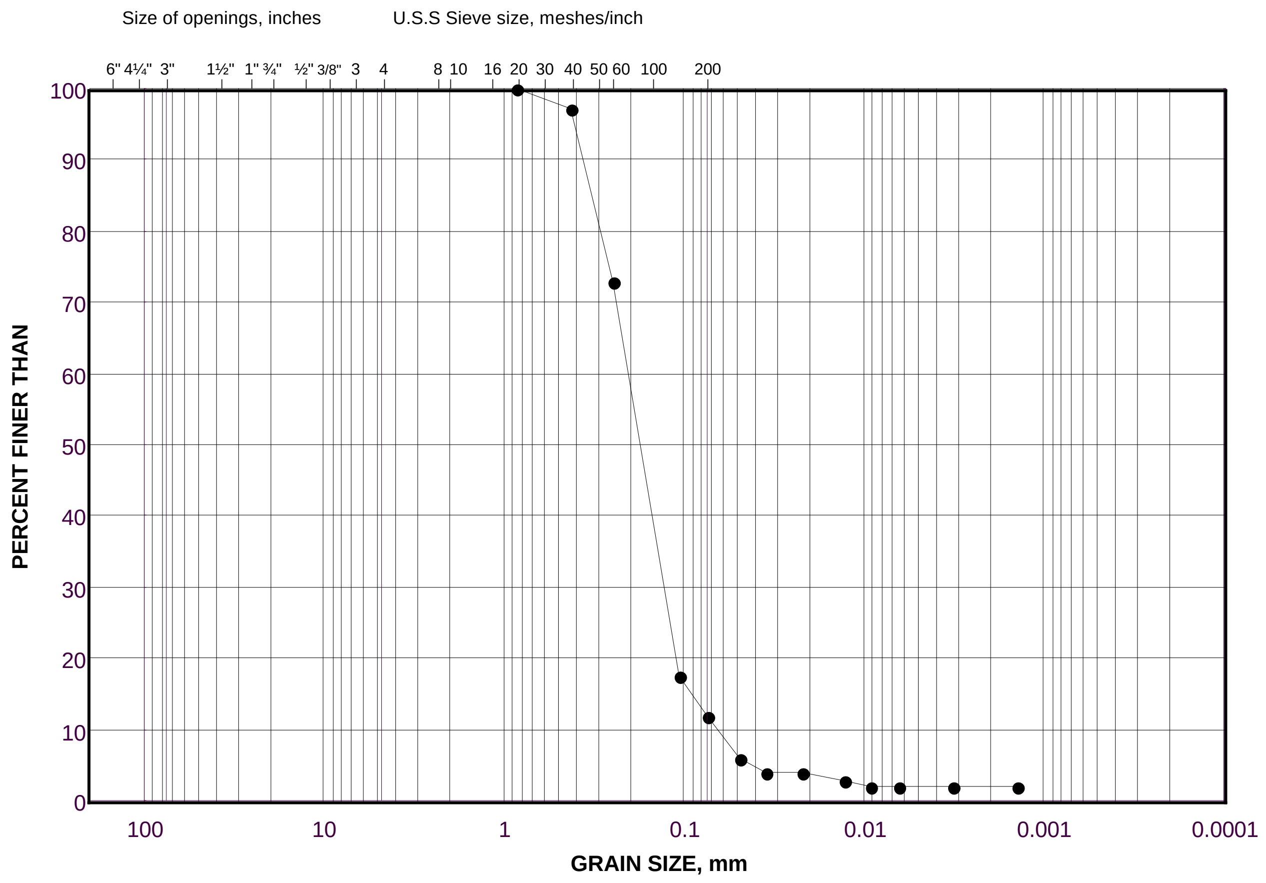
Project No.: 1664178 (2000)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

(SP) SAND

FIGURE B-7



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	CNR-202	13	160.1

Project Number: 1664178 (2000)

Checked By: AMP

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Date: 05-Jan-19

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