Appendix M.8 – Foundations Report – Rouge River

Kennedy Road Environmental Assessment between Steeles Avenue and Major Mackenzie Drive

Preliminary Foundation Investigation and Design Report

Rouge River Bridge at Kennedy Road,
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
ROUGE RIVER BRIDGE AT KENNEDY ROAD
CLASS ENVIRONMENTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE, MARKHAM, ONTARIO

1.0 INTRODUCTION

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

The purpose of the investigation was to evaluate the subsurface soil and groundwater conditions at the Rouge River bridge by means of a limited number of boreholes and, based on our interpretation of the data, to provide preliminary foundations engineering design recommendations for the structures.

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

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

2.0 SITE DESCRIPTION

The existing Rouge River bridge at Kennedy Road consists of a single span structure carrying four lanes of vehicular traffic and sidewalks on both sides of Kennedy Road as shown on the Key Plan on Figure 1. The pavement surface along Kennedy Road at the Rouge River bridge varies from Elevation 173.0 m to 172.4 m. The drawings referenced below were provided to Golder by HDR on August 23, 2018:

- The Regional Municipality of York Engineering, drawing titled, "Regional Road No 3 Realignment Rouge River Bridge, Rouge River, Town of Markham, General Arrangement, Drawing No. S-208-7A, Cont. No. 81-15, Sheet Nos. 27", prepared by Totten Sims Hubicki Associates, dated March 1981.

The existing structure is approximately 30 m long and 19 m wide. Both abutments are supported by driven 324 mm diameter battered steel tube piles extending between 8.5 m and 9.1 m in length to reported Elevation 158.5 m. The undersides of the pile caps are reportedly at Elevation 167.6 m and 167.0 m at the north and south abutments, respectively. The above drawings indicate that the riverbed of Rouge River is at approximately Elevation 167 m and the river water level is at about Elevation 167.3 m, some 5 m below Kennedy Road.

The Denby Valley Park is located east and west of the Rouge River bridge and there are residential developments located south of the park. Based on observations of the embankment at the time of the subsurface investigation, the side slopes appear to be performing adequately with no visual evidence of surficial sloughing or slope instability.

3.0 INVESTIGATION PROCEDURES

The field work for the preliminary investigation was carried out between November 7 and November 19, 2018 during which time two boreholes (designated as Boreholes RR-1 and RR-2) were advanced near the existing bridge abutments to a depth of 15.7 m below ground surface. The locations of the boreholes are shown on the Borehole Location Plan, on Figure 2and the borehole records are provided in Appendix A.

The investigation was carried out using a truck-mounted drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced using hollow-stem auger and rotary tri-cone techniques by using 216 mm outside diameter (O.D) hollow-stem augers in the overburden soils and rotary tri-cone bits and casing from 6.1 m and 3.0 m below ground surface at Borehole RR-1 and RR-2, respectively. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm O.D split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)[1]. The split-spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions. The results of in situ field tests (i.e., SPT “N” values) as presented on the borehole records and in sub-sections of Section 4.2 are uncorrected.

Due to the anticipated artesian groundwater conditions (i.e., confined aquifer conditions), Borehole RR-1 was advanced from 6.1 m below ground surface using drilling mud and Borehole RR-2 was advanced from 3.0 m below ground surface using drilling mud. Groundwater conditions were noted during drilling and immediately following drilling operations. Groundwater observation wells were installed in both boreholes, in accordance with Ontario Regulation 903 (as amended), to permit observation of the groundwater level. The monitoring wells consist of a 50 mm diameter PVC pipe with a slotted screen sealed within the sandy gravel deposit in RR-1 and within the gravelly silty sand fill layer in RR-2. The wells were equipped with flush-mount casing. Details of the monitoring well installation and water level readings are presented on the borehole records in Appendix A.

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

The borehole locations and ground surface elevations were obtained using a mobile GPS unit (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and horizontal directions. The locations provided on the borehole records and shown on Figure 1relative 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.

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

Table 1: Borehole Coordinates, Ground Surface Elevation and Depth

Borehole No. and Location	Location (UTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
RR-1 (South Abutment)	4,858,302.08	636,266.16	172.0	15.7
RR-2 (North Abutment)	4,858,353.61	636,259.99	172.9	15.7

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

4.2 Subsurface Conditions

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

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

In general, the subsurface conditions generally consist of asphalt underlain by a thick layer of fill. The fill is underlain by sequential deposits of sandy silty clay and silty sand and gravel to sandy gravel. A sandy clayey silt glacial till deposit is present beneath the gravelly deposit in Borehole RR-1. Deposits of silty clay and silty sand are present beneath the gravelly deposit in Borehole RR-2. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

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

4.2.1 Pavement Structure

Approximately 125 mm thick asphalt was encountered at ground surface in both boreholes. A 1.2 m and 1.1 m thick layer of granular road base fill was encountered underlying the asphalt in Boreholes RR-1 1 and RR-2, respectively. The granular fill varies in composition from gravelly sand to sand and extended to depths of 1.4 m and 1.2 m below ground surface (Elevation 170.6 m and 171.7 m), respectively. The SPT "N" values measured within the granular fill were 16 blows and 46 blows per 0.3 m of penetration, indicating a compact to dense level of compaction. The natural water content measured on two samples of the granular fill are about 3 per cent.

4.2.2 Fill

A 1.5 m and 2.8 m thick layer of silty clay fill was encountered underlying the pavement structure in Boreholes RR-1 and RR-2 and extended to depths of 2.9 m and 4.0 m below ground surface (Elevations 169.1 m and 168.9 m), respectively. A 3.0 m thick layer of gravelly silty sand fill was encountered underlying the silty clay fill in Borehole RR-2 and extended to a depth of 7.1 m below ground surface (Elevation 165.8 m). The SPT "N" values measured within the silty clay fill range from 9 blows to 17 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. Two SPT "N" values measured within the gravelly silty sand fill are 4 blows and 14 blows per 0.3 m of penetration, indicating a very loose to compact level of compaction.

The natural water contents measured on two samples of the silty clay fill are about 12 per cent and 18 per cent. The natural water contents measured on the sample of gravelly silty sand fill is about 31 per cent. The results of a grain size distribution test carried out on one sample of gravelly silty sand fill is shown on Figure B-1 in Appendix B.

4.2.3 Sandy Silty Clay

A silty clay deposit was encountered below the fill at depths of 2.9 m and 7.1 m below ground surface and extended to depths of 7.1 m and 8.6 m below ground surface (Elevation 164.9 m and Elevation 164.3 m) in Boreholes RR-1 and RR-2, respectively. The SPT "N" values measured within the deposit range from 0 blows (i.e. weight of hammer) to 9 blows per 0.3 m of penetration, indicating a very soft to stiff consistency.

The results of grain size distribution testing carried out on a sample of the deposit is shown on Figure B-2 in Appendix B. Atterberg limits testing was completed on a sample of the deposit and the liquid limit is about 28 per cent, the plastic limit is about 14 per cent, and the corresponding plasticity index is about 14 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 of low plasticity. The natural water contents measured on samples of the sandy silty clay deposit range from about 18 per cent to 32 per cent.

4.2.4 Silty Sand and Gravel to Sandy Gravel

A granular deposit consisting of silty sand and gravel to sandy gravel was encountered below the silty clay at depths of 7.1 m and 8.5 m below ground surface and extended to a depth of 13.2 below ground surface (Elevation 158.8 m and Elevation 159.7 m) in Boreholes RR-1 and RR-2, respectively.

During drilling, the augers were grinding in Borehole RR-1 between 8.5 m and 8.8 m below ground surface (Elevation 163.5 m) and between 9.8 m and 10.1 m below ground surface (Elevation 162.2 m). It can be inferred that boulders and/or cobbles are present at the depths where the augers were grinding. Previous experience indicates that deposits that contain cobbles and boulders may not be identified by conventional drilling, sampling, and laboratory testing methods. The measured SPT "N" values range from 25 blows to 157 blows per 0.3 m of penetration, indicating a compact to very dense level of compaction.

The results of grain size distribution testing carried out on two samples of the deposit is shown on Figure B-4 in Appendix B. The water contents measured on samples of the silty sand and gravel to sandy gravel deposit range from about 8 per cent to about 19 per cent.

4.2.5 Silty Clay

A silty clay deposit was encountered below the silty sand and gravel deposit in Borehole RR-2. The deposit was encountered at 13.2 m below ground surface and extends to 14.7 m below ground surface (Elevation 158.2 m). One SPT "N" value measured within the deposit was 91 blows per 0.3 m of penetration, indicating a hard consistency.

The results of grain size distribution testing carried out on one sample from the deposit is shown on Figure B-5 in Appendix B. Atterberg limit testing was carried out on one sample of the deposit and the results indicate a liquid limit of about 27 per cent, a plastic limit of about 16 per cent, and a corresponding plasticity index of about 11 per cent. These test results, which are plotted on a plasticity chart on Figure B-6 in Appendix B, indicate the deposit is a silty clay of low plasticity. The natural water content measured on a sample of the silty clay is about 16 per cent.

4.2.6 Sandy Clayey Silt (Till)

A sandy clayey silt till deposit was encountered below the sandy gravel deposit in Borehole RR-1. The deposit was encountered at a depth of 13.2 m below ground surface and extends to the borehole termination depth of 15.7 m below ground surface (Elevation 156.3 m). Two SPT "N" values measured in the deposit are 56 blows and 76 blows per 0.3 m of penetration, indicating a hard consistency. Although not encountered when advancing the boreholes, cobbles and boulders are commonly encountered in glacially derived materials and should be expected within this deposit.

The results of grain size distribution testing carried out on a sample from the deposit is shown on Figure B-7 in Appendix B. Atterberg limit testing was carried out on one sample of the deposit and the results indicate a liquid limit of about 17 per cent, a plastic limit of about 12 per cent, and a corresponding plasticity index of about 5 per cent. These test results, which are plotted on a plasticity chart on Figure B-8 in Appendix B, indicate the clayey silt till has slight plasticity. The natural water contents measured on two samples of the sandy clayey silt till are about 9 per cent and 18 per cent.

4.2.7 Silty Sand

A silty sand deposit was encountered below the silty clay deposit in Borehole RR-2 at a depth of 14.7 m below ground surface and extends to the borehole termination depth of 15.7 m below ground surface (Elevation 157.2 m). One SPT "N" value measured within the deposit was 117 blows per 0.3 m of penetration, indicating a very dense level of compaction.

The results of grain size distribution testing carried out on a sample from the deposit is shown on Figure B-9 in Appendix B. The natural water content measured on a sample of the silty sand is about 19 per cent.

4.2.8 Groundwater Conditions

The overburden samples obtained from the boreholes were generally moist above 3 m below ground surface and wet below 3 m below ground surface. Details of the groundwater levels observed in the boreholes during and upon completion of drilling are summarized on the borehole records.

Upon completion of drilling the groundwater level in the observation well in Borehole RR-1 was 3.4 m below ground surface (Elevation 168.6 m) and in Borehole RR-2 the groundwater level was 4.4 m below ground surface (Elevation 168.5 m). The recorded groundwater level in the observation wells are summarized in Table 2.

Table 2: Depth and Elevation of Measured Groundwater Level

Borehole Number	Screened Stratigraphy	Ground Surface Elevation (m)	Groundwater Level Depth (m)	Groundwater Elevation (m)	Date of Observation
RR-1	Sandy Gravel	172.0	3.2	168.8	November 29, 2018
			3.4	168.6	January 11, 2019
RR-2	Gravelly Silty Sand Fill	172.9	4.0	168.9	November 29, 2018
			4.6	168.3	December 13, 2018

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

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder, and was reviewed Mr. Storer Boone, Ph.D., P.Eng., a senior geotechnical engineer and Principal of Golder.

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PART B

PRELIMNARY FOUNDATION DESIGN REPORT
ROUGE RIVER BRIDGE AT KENNEDY ROAD
CLASS ENVIRONEMNTAL ASSESSMENT STUDY FOR IMPROVEMENTS TO KENNEDY ROAD FROM STEELES AVENUE TO MAJOR MACKENZIE DRIVE
MARKHAM, ONTARIO

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

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

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

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

6.1 General

To accommodate the proposed widening of Kennedy Road, it is understood that there are currently two options being considered for the Rouge River bridge. The options are as follows:

- **Option 1:** Widening of the existing structure, including widening of the foundation elements, to accommodate six lanes of traffic and active transport paths; and,
- **Option 2**: Full replacement of the existing Rouge River bridge.

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 bridge 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, 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 6.7 below.

6.3 Foundation Options

Both shallow and deep foundations options have been considered for support of the abutments for the proposed Rouge River bridge (widening structure and full replacement) at Kennedy Road. Based on the existing structure geometry and the subsurface conditions encountered in the boreholes advanced at the site, the following foundations recommendations were considered for preliminary design:

Shallow Foundations

- **Strip/Spread Footings:** Considering the structural loads required to the support the proposed Rouge River bridge (widening structure and full replacement) structure, strip/spread footings are not considered suitable for support of the bridge foundations (abutments) at this site due to the presence of the relatively weak (soft to stiff) sandy silty clay deposit below the fill. In addition, the existing structure is founded on driven steel tube piles and, therefore, supporting the new structure on shallow foundations could result in unacceptable differential settlement between the existing and new structure elements. In addition, at both abutments the fill extends to about 5.0 to 6.0 m depth below ground surface and the footings would need to extend below the fill and soft sandy silty clay, result in excavations of about 7.0 to 8.6 m deep which is not practical for the bridge widening. For these reasons, supporting the bridge abutments on spread / strip footings is not recommended and is not discussed further.

Deep Foundations:

- **Steel H-piles or Pipe Piles:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments and would permit design of conventional abutments, semi-integral abutment (for tube piles) or integral abutments (for H-piles). For the full replacement option, the abutments may be constructed with a pile cap perched above the Rouge River grade; however, this will likely result in a longer span length. Staging options may permit open cut excavations for the pile cap construction, or temporary protection systems may be required, depending on the elevation of the underside of the pile cap. For a widened structure, it is recommended that the pile cap be founded at the same elevation as the existing pile cap; however, for this option cofferdams will be required to permit construction in reasonably dry conditions. Temporary protection systems would be required for the construction of the pile cap at about the same elevation as the existing pile cap. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense silty sand and gravel to hard sandy clayey silt deposits at the abutments.

- **Drilled shafts (caissons):** Drilled shafts are considered feasible for support of the abutments for the proposed new structure or widened structure; however, due to the presence of water-bearing granular soils (i.e., the silty sand and gravel and the interlayers or seems of gravel) care will be required during foundation drilling where it occurs adjacent to the existing structure to ensure that there is not any loss of ground adjacent to and/or below the existing steel pipe piles. Permanent liners filled with water or drilling fluids (i.e., slurry) at all times would be required during caisson installation to control the ground and groundwater within these water-bearing cohesionless zones, which would result in the caisson foundations being less cost-effective than the installation of driven steel H-piles. In this regard, if deep foundations are adopted, the use of driven piles would be preferred as compared to drilled shafts. Depending on the underside of the pile cap elevation at the abutments temporary protection systems may be required. This option would be somewhat more complicated to implement if the structure requires use of integral abutments.

Based on the above considerations, driven steel H-piles are preferred for the support of the new abutments for the widened structure / replacement structure of the Rouge River Bridge. Subsequent sections of this report provide preliminary recommendations for deep foundation options for widened structure / replacement structure of the Rouge River Bridge.

6.4 Deep Foundations

The following sections provide the recommended founding elevation and geotechnical resistances for driven steel H-piles, driven steel pipe piles and drilled shafts. Due to the presence of the firm to stiff cohesive fill material and the soft to stiff cohesive deposit at the abutments, settlement due to the additional loadings from the new approach embankments may induce downdrag on the existing steel tube piles and new deep foundations (this is further discussed in Section 6.4.3) depending on the relative timing of installation of the piles and construction of approach embankments. Recommendations for protection against frost penetration are provided in Section 6.4.4.

The recommendations provided in Sections 6.4.1 and 6.4.2, below, are provided for the widened structure / replacement structure; however Borehole RR-1 advanced near the south abutment did not penetrate into soil conditions that would provide full end-bearing resistance for driven piles and, therefore, it is during Detailed Design additional boreholes should be drilled, particularly at the south abutment, to confirm the elevation of conditions suitable for support of end-bearing piles and at least 3 m into such soils.

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

To facilitate widening of the existing bridge structure, the new structure elements may be supported on driven end bearing steel H-piles or driven steel pipe piles, founded at a minimum of 1 m within very dense soil (having SPT "N"-values of greater than 100 blows per 0.3 m of penetration). Based on the two boreholes advanced at the bridge site, the elevation of soils suitable to support end-bearing piles is variable. Borehole RR-1 did not encounter soils of sufficient thickness or density to confirm end-bearing pile support conditions. The elevation at which end bearing pile resistance may be achieved in the vicinity of RR-1 should be confirmed during Detailed Design with additional boreholes. For HP 310x110 piles driven to the recommended tip elevations and based on available subsurface information, the estimated factored ultimate axial geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) for design of the foundations is provided in Table 3. Geotechnical resistance values provided near borehole RR-1 are provided for the hard sandy clayey silt deposit, though higher resistance values at deeper tip elevations may be achieved, pending additional borehole exploration.

Table 3: Founding Elevation and Geotechnical Axial Resistance for Driven Steel H-Piles

Foundation Unit	Preliminary Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation	Factored Axial Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN) (for 25 mm of Settlement)
North Abutment	Very dense silty sand and gravel	160.0 (Borehole RR-2)	1,100	-[1]
South Abutment	Hard sandy clayey silt	157.0 (Borehole RR-1)	700	-[1]

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

The preliminary factored geotechnical resistances provided above will need to be re-evaluated and modified, as necessary, during Detailed Design in consideration of any additional subsurface investigation at the foundation elements.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a slightly higher risk of “hanging up” or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates for protection during driving in accordance with OPSS 903 (*Deep Foundations*) and OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*).

6.4.2 Drilled Shaft (Caisson) Foundations

As an alternative to driven piles, drilled shafts (caissons) founded within the very dense silty sand and gravel and hard sandy clayey silt deposit may be considered for support of the north and south abutments for a proposed full replacement or widened structure respectively.

Based on the available subsurface information, preliminary design of drilled shafts installed to the recommended tip elevations may be based on estimated factored ultimate axial geotechnical resistance and factored serviceability geotechnical resistance (for 25 mm of settlement) provided in the table below.

Table 4: Founding Elevation and Geotechnical Axial Resistance for Drilled Shafts

Foundation Unit	Preliminary Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation	Caisson Diameter	Factored Axial Ultimate Geotechnical Resistance (kN)	Factored Serviceability Geotechnical Resistance (kN) (for 25 mm of Settlement)
North Abutment	Very dense silty sand and gravel	160.0 (Borehole RR-2)	0.9 m	1,800	-[1]
			1.2 m	3,000	-[1]
			1.5 m	4,200	-[1]
South Abutment	Hard sandy clayey silt	157.0 (Borehole RR-1)	0.9 m	1,300	-[1]
			1.2 m	2,000	-[1]
			1.5 m	3,100	-[1]

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

The preliminary factored geotechnical resistances provided above will need to be re-evaluated and modified, as necessary, during Detailed Design in consideration of any additional subsurface investigation at the foundation elements.

For the installation of drilled shafts, consideration must be given to the potential presence of cobbles and boulders within the very dense silty sand and gravel and hard sandy clayey silt deposits. Appropriate construction equipment and techniques must be selected to penetrate the anticipated cobbles and boulders. Given the presence of saturated granular 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 silty sand and gravel materials and any interbedded granular layers (e.g., gravel layer identified in borehole RR-1) are anticipated to cause difficulties during caisson installation and a sufficient head of drilling slurry or water will likely 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.

6.4.3 Downdrag Loads

As a result of the loading from the new approach embankments and any proposed roadway grade increases, long-term consolidation settlement of the underlying cohesive fill material and the natural cohesive deposit will occur. 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 (new and existing) will result in the development of negative skin friction (“downdrag”) on the piles and drilled shafts (caissons).

If the piles/drilled shafts for the abutments are installed prior to the construction of the approach embankments, an assessment should be carried out in accordance with the requirements of the Canadian Foundation Engineering Manual (2006) 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 factored drag load. The magnitude of the drag loads should be assessed during Detailed Design.

For the option of constructing a widened structure either east and/or west of the existing structure the drag loads should be estimated at Detailed Design and the structural engineer must assess the structural capacity of the existing driven steel tube piles taking into account the factored dead load combined with the factored downdrag load induced by the adjacent new embankment loading. If the structural capacity of the existing piles is exceeded mitigation measures could include the use of light-weight fill such as Expanded Polystyrene (EPS) to construct the approach embankment and side slopes. If EPS is used for embankment construction, a concrete slab would be required above the EPS to minimize reflection cracking of the pavement arising from the joints in the EPS blocks. For the option of full replacement of the existing structure several other measures to mitigate against downdrag can be considered at Detailed Design such as preloading, preloading and surcharging, use of a pile section with a higher structural capacity, use of light-weight fill, or a combination of these.

6.4.4 Frost Protection

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).

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

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

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

- Free-draining granular fill meeting the specifications of OPSS.MUNI 1010 (*Aggregates*) Granular ‘A’ or Granular ‘B’ Type II should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill, as applicable. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.MUNI 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to subdrains and frost taper should be in accordance with OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*) for wingwall / end walls as applicable.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall, with limitations on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.MUNI 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.4 m behind the back of the wall in accordance with Figure C6.20(a) of the *Commentary to the CHBDC* (2014). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at flatter than 1 horizontal to 1 vertical (1H:1V) extending up and back from the rear face of the wall or footing, as applicable, in accordance with Figure C6.20(b) of the *Commentary to the CHBDC* (2014).

6.5.1 Static Lateral Earth Pressures for Design

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

- For a restrained wall, the pressures are based on the existing or proposed new embankment fill behind the granular backfill zone, and the following parameters (unfactored) may be used assuming the use of earth fill or OPSS.MUNI 1010 (*Aggregates*) Select Subgrade Material (SSM) for the general embankment fill:

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

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

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.

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

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

Fill for construction of the approach embankments should consist of Granular ‘A.’, Granular ‘B’ Type I or Type II meeting the specifications of OPSS. MUNI 1010 (Aggregates) or other pre-approved earth fill unless lightweight fill options are required to alleviate downdrag loads on piles. All earth fill embankments should be placed and compacted in accordance with OPSS.MUNI 501 (Compacting) and OPSS.PROV 206 (Grading). Embankment side slopes should be constructed no steeper than 2 Horizontal to 1 Vertical (2H:1V) in granular fill and properly benched and keyed into the existing embankment fill in accordance with OPSD 208.010.

To reduce surface water erosion on the granular embankment side slopes, topsoil and seeding as per OPSS 802 (Topsoil) and OPSS.MUNI 804 (Seed and Cover) should be carried out as soon as possible after construction of the embankments. If this slope protection is not in place before winter, then alternate protection measures, such as covering the slope with straw, or gravel sheeting as per OPSS.MUNI 511 (Rip Rap, Rock Protection and Granular Sheeting), and OPSS.MUNI 1004 (Aggregates) will be required to reduce the potential for erosion and to reduce the potential for the requirement of remedial works on the side slopes in the spring prior to topsoil dressing and seeding.

6.6.2 Global Stability

Based on observations at the time of the field work, we understand that the abutment slopes and approach embankment fill slopes at both end of the existing bridge are performing adequately with no visual evidence of surficial sloughing or slope instability.

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

6.6.3 Settlement

For new/widened approaches, fill will be required to be placed on and keyed into the existing slope. It is estimated that approximately up to 5 m to 6 m of fill may be required for the new approach embankments. The final width of the permanent approach embankment is not known at this time, but it is assumed that it will be in the order of 1.5 times the existing embankment width.

The settlement of the foundation soils under the additional fill is estimated to be in the range of 25 mm to 100 mm, with the greater values being associated with the thickest fill locations (highest and widest sections of new fill). For areas where predicted settlements are greater than 25 mm, settlement mitigation measures would be required. Mitigation measures may include preloading or use of lightweight fill. However, the estimated settlements should be reassessed during the Detailed Design stage, once the proposed structure and embankment geometry is available.

6.7 Construction Considerations

6.7.1 Removal of Existing Foundations

If drilled shafts (caissons) are adopted for a full bridge replacement, the existing piles should not be extracted from the ground to avoid disturbance of the extensive cohesive deposit encountered at the south approach. In addition, to avoid potential conflicts between the existing piles and the drilled shafts during construction, the new abutments should be offset from the existing abutment.

6.7.2 Open Cut Excavation

The construction of new pile caps will require excavations up to about 6.0 m below the existing Kennedy Road grade (Elevation 167.0 m) and will generally be made through the existing stiff to very stiff cohesive fill and gravelly silty sand fill at the north abutment and stiff cohesive fills to native firm to stiff sandy silty clay deposits at the south abutment. 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 would be classified as Type 3 soil and the sandy silty clay deposits would likely be categorized as 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 fill, would be categorized as Type 4 soils. 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. During wet periods of the year some local flattening of slopes may be required. Excavated material must be stockpiled at a distance away from the excavation equal to or greater than the height of the open cut excavation.

6.7.3 Temporary Protection Systems

At this preliminary stage, it is anticipated that temporary protection systems may be required along Kennedy Road, in order to facilitate the construction of the widened bridge structure and for the option of constructing a new structure, spanning the Rouge River. Cofferdams will be required in order to construct the pile caps.

Temporary excavation support systems should be designed and constructed in accordance with OPSS.MUNI 539 (*Temporary Protection Systems*). The lateral movement of the temporary protection systems should meet Performance Level 2 as specified in OPSS.MUNI 539, provided that the existing structures and any adjacent utilities can tolerate this magnitude of deformation. Although the selection and design of the protection systems will be the responsibility of the Contractor, for conceptual purposes, a driven, interlocking driven steel sheet pile system or soldier pile and timber lagging system should be suitable for the temporary excavation support at this structure site. The sheet pile wall would have to be socketed to sufficient depth to provide the necessary passive resistance for the retained soil height. Additional lateral support to the sheet pile wall or soldier pile wall, if required, could be provided in the form of rakers or temporary anchors. It is anticipated that cobbles and/or boulders may be encountered within the subsurface soils at the site, which may affect the installation of protection system elements.

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

6.7.4 Groundwater and Surface Water Control

The groundwater level measured in the monitoring well installed in Borehole RR-2 was screened in the gravelly silty sand fill deposit was measured at an elevation of about 168.9 m or about 4.0 m below ground surface.

Foundation excavations for the pile cap at the south and north abutment are anticipated to extend to at least Elevation 167.0 m and within the firm to stiff silty clay and gravelly silty sand fill deposit, respectively. Due to the low permeability of the silty clay material (south abutment) it is anticipated that water inflow from this material would be low; however, at the north abutment the surface of loose to compact gravelly silty sand fill was encountered at the elevation of the proposed pile cap and this fill is saturated as it is below the river level. The groundwater will be required to be lowered to 1 m below the base of the pile cap in order to construct the pile cap. As discussed in Section 6.7.3 cofferdams will be required adjacent to the river to permit construction of the pile caps.

Surface water seepage into the excavations should be expected and will be heavier during periods of sustained precipitation and all surface water should be directed away from the excavations.

6.7.5 Basal Stability

The surface silty sand deposit encountered at a depth of about 14.7 m below ground surface (Elevation 158.2 m) in Borehole RR-2 which is under some hydrostatic pressure. Although not encountered in Borehole RR-1, similar conditions from the silty sand and gravel deposit are anticipated. Excavations for the pile cap for deep foundations are anticipated to extend to about Elevation 167 m, approximately 8.8 m above the silty sand deposit. Based on this, the Factor of Safety against base instability should be greater than 1.2 and the silty sand interlayer underlying the silty clay deposit may not be required to be dewatered/depressurized prior to excavations for pile caps pending completion of additional explorations during Detailed Design.

6.7.6 Obstructions During Pile Driving

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

6.7.7 Vibration Monitoring During Temporary Protection System or Pile Installation

Structures near the site include the existing bridge (at the site) and residential homes (approximately 35 m from the site). A maximum partial peak velocity (PPV) of 100 mm/s is generally considered acceptable for bridge structures in good condition. Based on vibration monitoring experience, it is considered unlikely that vibrations induced by conventional construction activities (such as pile driving) will reach this threshold level, however, it is considered prudent that pre- and post-construction condition surveys and vibration monitoring at the nearby structures be considered to defend against potential damage claims associated with vibration-inducing activities at the site. A PPV threshold of 25 mm/s is generally considered applicable for residential buildings.

6.7.8 Ground and Groundwater Control for Drilled Shaft (Caisson) Construction

As discussed in Section 6.7.5, running or flowing of the water-bearing granular soil (interlayers within the till and possibly the granular till deposits) could occur during or after drilling of the caissons, and basal heave (inflow of soil under water pressures) could occur at the caisson base. If caisson foundations are adopted for support of any of the foundation elements, caisson liners would be required to support the soils during construction. In addition, in order to counter-balance the groundwater pressure, the liner must be advanced with water or controlled-density drilling fluids (e.g., slurry) inside the liner and the auger should be advanced beyond the tip of the liner at any time. The fluid level within the liner should be maintained at least 1 m above the groundwater levels and equipment withdrawal rates should be controlled to avoid developing suction forces at the bottom of the drilled holes. Further, placement of concrete by tremie methods and design of high-slump tremie concrete would be required to result in proper displacement of drilling fluids and any residual cuttings and that remain in the drilled hole.

7.0 RECOMMENDATIONS FOR FURTHER INVESTIGATION WORK DURING DETAILED DESIGN

Additional exploration and testing should be completed during the Detailed Design if the Rouge River Bridge structure is to be modified to accommodate the widening of Kennedy Road or if a new structure is to be constructed. Additional boreholes should be advanced within the footprint of the new / widened foundation elements to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report as follows:

- Assess the type and depth of fill present;
- Assess near surface soil deposits within the footprint of the proposed embankments for settlement analysis, where applicable;
- Advance the boreholes a minimum of 3 m into “100-blow material” (South Abutment) and confirm depths to materials suitable for end-bearing piles and drilled shafts and to confirm at the south abutment.
- Evaluate the lateral capacities of pile foundations;

- Evaluate the seismic Site Class and seismic hazard values;
- Test parameters used to assess the corrosive potential of the soil to concrete and buried steel;
- Confirm groundwater elevations in the till materials and the piezometric levels in any hydraulically significant deposits that may be encountered; 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 driven steel H-pile foundations.

8.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder, and was reviewed Mr. Storer Boone, Ph.D., 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.

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VN/SEMP/SJB/cr;mes

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

Canadian Standards Association 2014. Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S06-14 and Commentary.

Ontario Provincial Standard Drawings (OPSD)

OPSD 202.010	Slope Flattening
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3001.100	Steel Tube Pile Driving Shoe
OPSD 3090.101	Foundation, Frost Penetration Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Provincial Standard Specifications (OPSS)

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

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)

ASTM International:

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

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

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

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

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

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

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

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

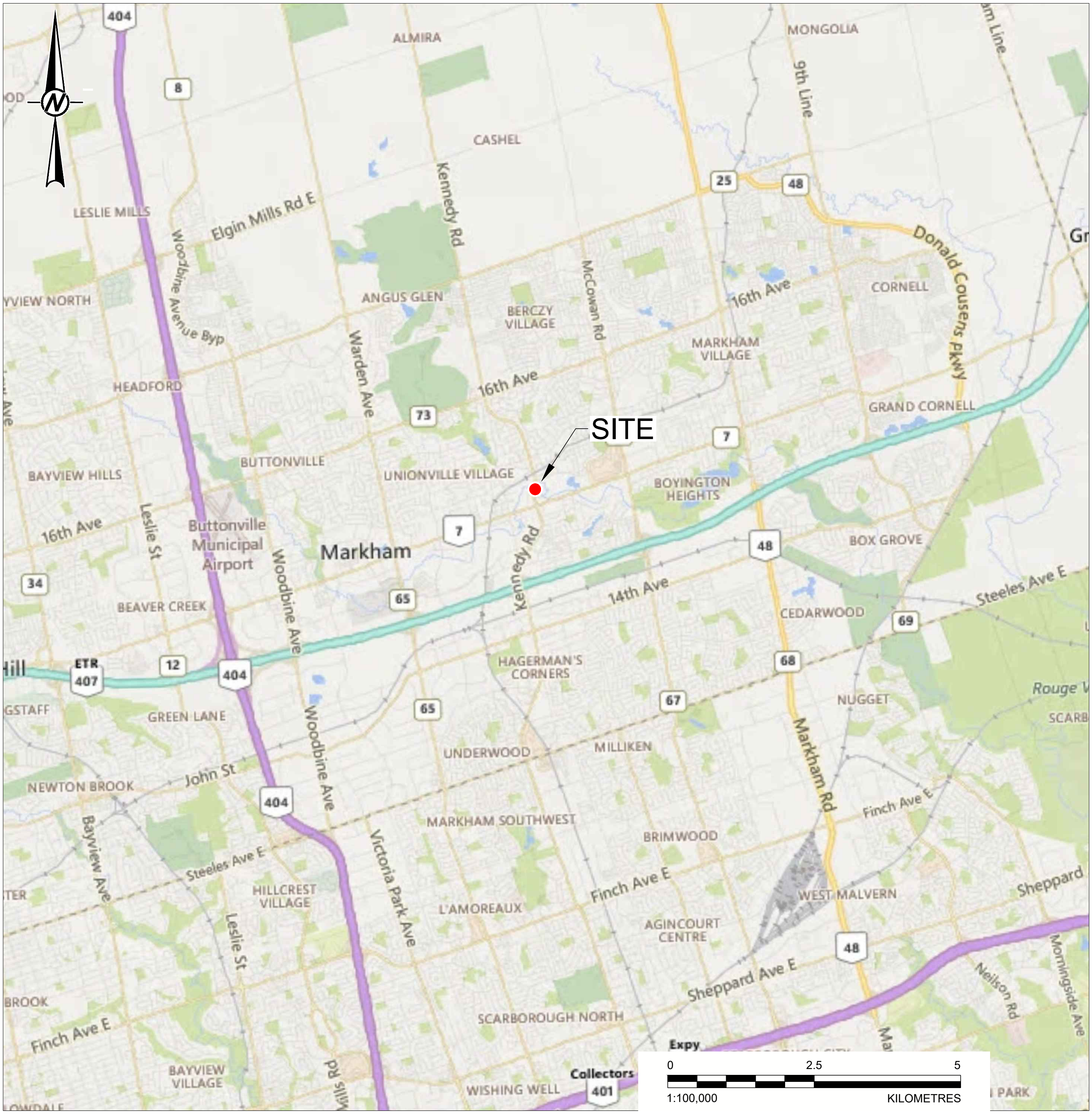
Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater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

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

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

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

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



CLIENT
HDR INC.

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

TITLE
KEY PLAN

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

CONSULTANT
GOLDER

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

PROJECT NO.	CONTROL	REV.	FIGURE
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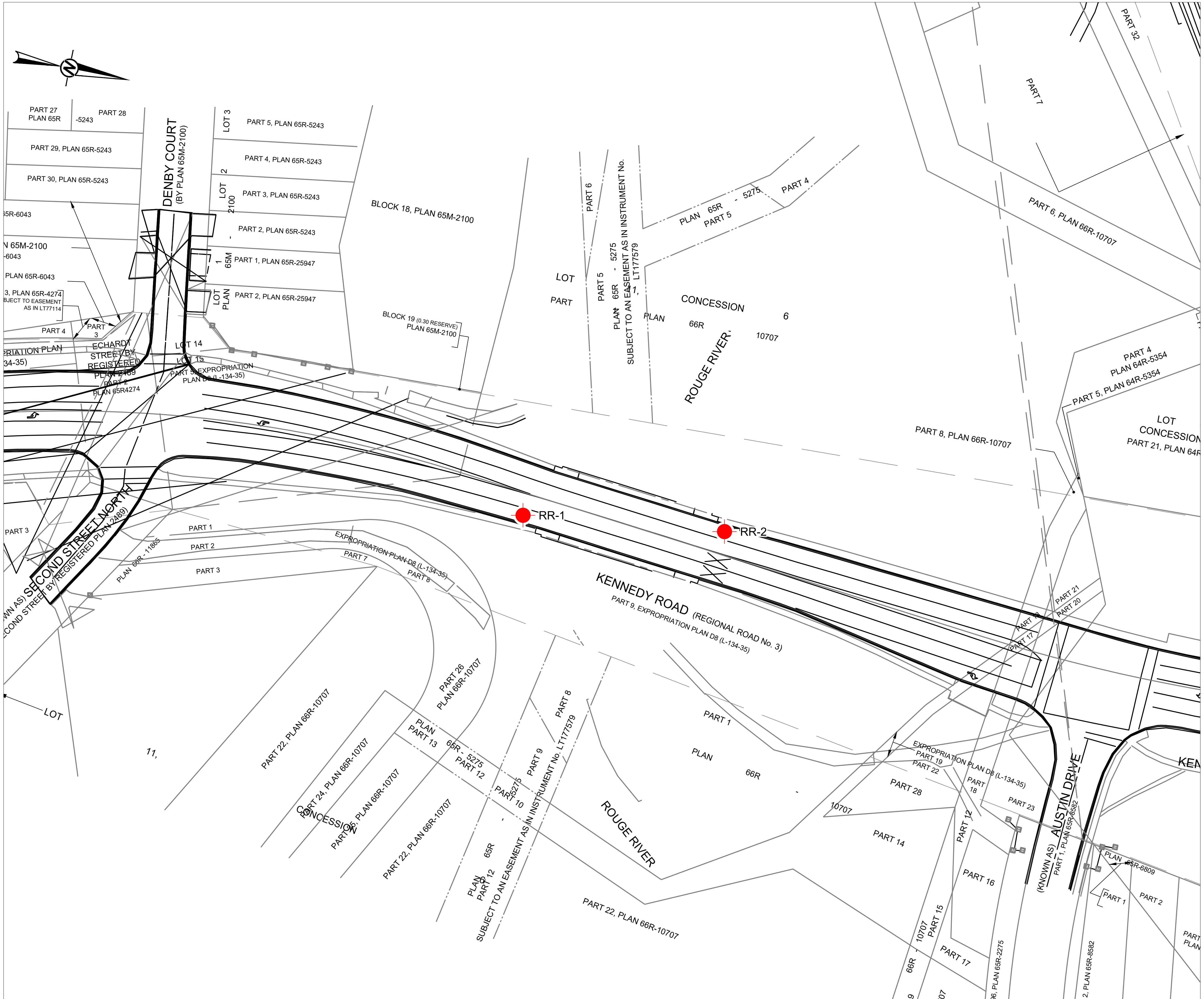

LEGEND

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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CLIENT

HDR INC.

PROJECT

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

TITLE

BOREHOLE LOCATION PLAN

CONSULTANT

GOLDER

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

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

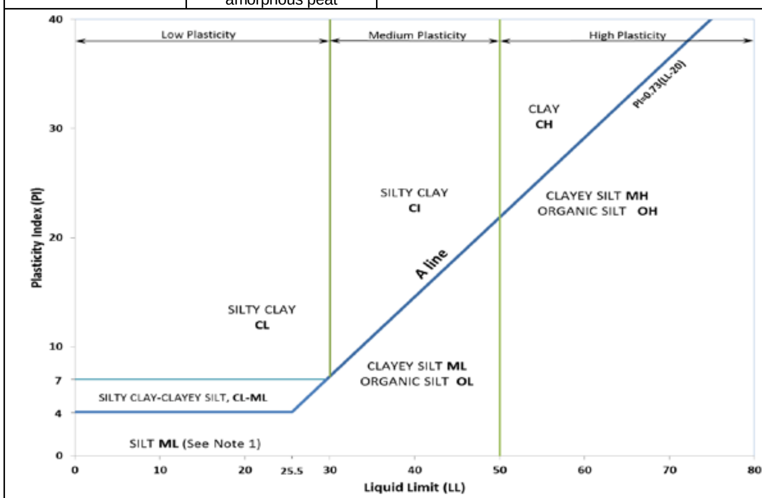
APPENDIX A

Lists of Symbols and Abbreviations
Record of Boreholes RR-1 and RR-2

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

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

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



Note 1 – Fine grained materials with PI and LL that plot in this area are named (ML) SILT with slight plasticity. Fine-grained materials which are non-plastic (i.e. a PL cannot be measured) are named SILT.
Note 2 – For soils with <5% organic content, include the descriptor "trace organics" for soils with between 5% and 30% organic content include the prefix "organic" before the Primary name.

Dual Symbol — A dual symbol is two symbols separated by a hyphen, for example, GP-GM, SW-SC and CL-ML.
For non-cohesive soils, the dual symbols must be used when the soil has between 5% and 12% fines (i.e. to identify transitional material between "clean" and "dirty" sand or gravel.
For cohesive soils, the dual symbol must be used when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart (see Plasticity Chart at left).

Borderline Symbol — A borderline symbol is two symbols separated by a slash, for example, CL/CI, GM/SM, CL/ML.
A borderline symbol should be used to indicate that the soil has been identified as having properties that are on the transition between similar materials. In addition, a borderline symbol may be used to indicate a range of similar soil types within a stratum.

ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES AND TEST PITS

PARTICLE SIZES OF CONSTITUENTS

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

MODIFIERS FOR SECONDARY AND MINOR CONSTITUENTS

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

PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) split-spoon sampler for a distance of 300 mm (12 in.). Values reported are as recorded in the field and are uncorrected.

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

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

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

SAMPLES

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

SOIL TESTS

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

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

NON-COHESIVE (COHESIONLESS) SOILS

Compactness[2]

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

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

Field Moisture Condition

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

COHESIVE SOILS

Consistency

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

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

Water Content

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

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan\delta$
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

PROJECT: 1664178 (2000)
LOCATION: N 4858302.08; E 636266.16

RECORD OF BOREHOLE: RR-1

BORING DATE: November 15 and 19, 2018

DATUM: Geodetic

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

HAMMER TYPE: AUTOMATIC

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE	172.00					
0	Power Auger, 216 mm O.D. Hollow Stem Augers	ASPHALT (125 mm)	0.00					50 mm Diameter PVC Monitoring Well (Flushmount)
		FILL - (SP) gravelly SAND, trace fines; brown; non-cohesive, moist, compact to dense	0.13	1	SS	46		
1				2	SS	16		
		FILL - (CL) SILTY CLAY, some sand, some gravel; brown to dark brown, oxidation staining; cohesive, w~PL, stiff	170.63 1.37	3	SS	13		Cuttings
2		- Oxidation staining to 2.9 m depth		4	SS	12		
3		(CL) sandy SILTY CLAY, trace gravel; grey; cohesive, w>PL, firm to stiff	169.10 2.90	5	SS	9		29-Nov-18
4–5				6	SS	6	MH	
6	Power Auger, Tri-cone / Mud-Rotary			7	SS	8		Bentonite
7		(SM) SILTY SAND and GRAVEL; grey; non-cohesive, moist, compact to very dense	164.91 7.09					
8		- Gravel seam from 7.9 m to 8.5 m depth		8A, 8B	SS	25		
		- Tri-cone grinding from 8.5 m to 8.8 m depth						
9				9	SS	61		
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: YS
CHECKED: AMP

RECORD OF BOREHOLE: RR-1

PROJECT: 1664178 (2000)
LOCATION: N 4858302.08; E 636266.16
BORING DATE: November 15 and 19, 2018
SHEET 2 OF 2
DATUM: Geodetic

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

DEPTH SCALE METRES	BORING METHOD	DESCRIPTION	ELEV. / DEPTH (m)	NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT PERCENT	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		--- CONTINUED FROM PREVIOUS PAGE ---							
10	Power Auger / Tri-cone / Mud-Rotary	(GP) sandy GRAVEL, trace fines; red, grey, and black; non-cohesive, wet, compact to dense	161.87 / 10.13						Bentonite
11		- Tri-cone grinding from 9.8 m to 10.1 m depth		10	SS	25			Silica Sand
12				11	SS	32		MH	Screen
13		(CL-ML) sandy CLAYEY SILT, trace gravel; grey, (TILL); cohesive, w<PL to w>PL, hard	158.82 / 13.18						Silica Sand
14				12	SS	76		MH	
15				13	SS	56			Bentonite
		END OF BOREHOLE	156.30 / 15.70						

NOTES:

1. Water level measured in monitoring well as follows:

Date	Depth (m)	Elev. (m)
19-Nov-18	3.4	168.6
29-Nov-18	3.2	168.8
11-Jan-19	3.4	168.6

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

DEPTH SCALE 1 : 50
GOLDER
LOGGED: YS
CHECKED: AMP

RECORD OF BOREHOLE: RR-2

PROJECT: 1664178 (2000)

LOCATION: N 4858353.61; E 636259.99

BORING DATE: November 7 and 12, 2018

DATUM: Geodetic

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

HAMMER TYPE: AUTOMATIC

BORING METHOD	SOIL PROFILE – DESCRIPTION	ELEV. / DEPTH (m)
	GROUND SURFACE	172.92
Power Auger – 216 mm O.D. Hollow Stem Augers	ASPHALT (125 mm)	0.00
	FILL - (SP) gravelly SAND; brown; non-cohesive, moist	0.13
	FILL - (SP) SAND, some gravel; brown; non-cohesive, moist	172.21 / 0.71
	FILL - (CL) SILTY CLAY, some sand, some gravel; brown to dark brown; cohesive, w<PL, stiff to very stiff - Trace organics from 2.1 m to 4.0 m depth	171.70 / 1.22
Power Auger – Tri-cone / Mud-Rotary	FILL - (SM) gravelly SILTY SAND; dark brown, trace rootlets; non-cohesive, moist to wet, very loose to compact	168.88 / 4.04
	(CL) sandy SILTY CLAY, trace gravel; grey; cohesive, w>PL, very soft	165.83 / 7.09
	(GP) sandy GRAVEL, trace fines; grey; non-cohesive, wet, very dense	164.31 / 8.61

SAMPLE NUMBER	TYPE	BLOWS/0.3m	WATER CONTENT (%)	ADDITIONAL LAB. TESTING
1	AS	-		
2	AS	-		
3	SS	9		
4	SS	13		
5	SS	17		
6	SS	4		
7	SS	14	~30	MH
8	SS	WH		
9	SS	53		

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

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

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

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

PIEZOMETER OR STANDPIPE INSTALLATION:
- 50 mm Diameter PVC Monitoring Well (Flushmount)
- Silica Sand
- Cuttings
- Bentonite
- Water level: 29-Nov-18
- Silica Sand
- Screen
- Silica Sand
- Bentonite

DEPTH SCALE METRES: 0–10

CONTINUED NEXT PAGE

DEPTH SCALE 1 : 50

GOLDER

LOGGED: YS

CHECKED: AMP

RECORD OF BOREHOLE: RR-2

PROJECT: 1664178 (2000)
LOCATION: N 4858353.61; E 636259.99
BORING DATE: November 7 and 12, 2018

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

--- CONTINUED FROM PREVIOUS PAGE ---

Boring method: Power Auger; Tri-cone / Mud-Rotary

Description	Elev. / Depth (m)	Sample No.	Type	Blows/0.3m	Additional Lab. Testing	Piezometer or Standpipe Installation
(SM) SILTY SAND and GRAVEL; grey; non-cohesive, wet, very dense	162.79 / 10.13	10	SS	157	MH	Bentonite
		11	SS	125		
(CL) SILTY CLAY, some sand, trace gravel; grey; cohesive, w~PL, hard	159.74 / 13.18	12	SS	91	MH	
(SM) SILTY SAND; grey; non-cohesive, wet, very dense	158.21 / 14.71	13	SS	117	MH	
END OF BOREHOLE	157.22 / 15.70					

NOTES:

1. Water level measured in monitoring well as follows:

Date	Depth (m)	Elev. (m)
12-Nov-18	4.4	168.5
29-Nov-18	4.0	168.9
13-Dec-18	4.6	168.3

DEPTH SCALE
1 : 50

GOLDER

LOGGED: YS
CHECKED: AMP

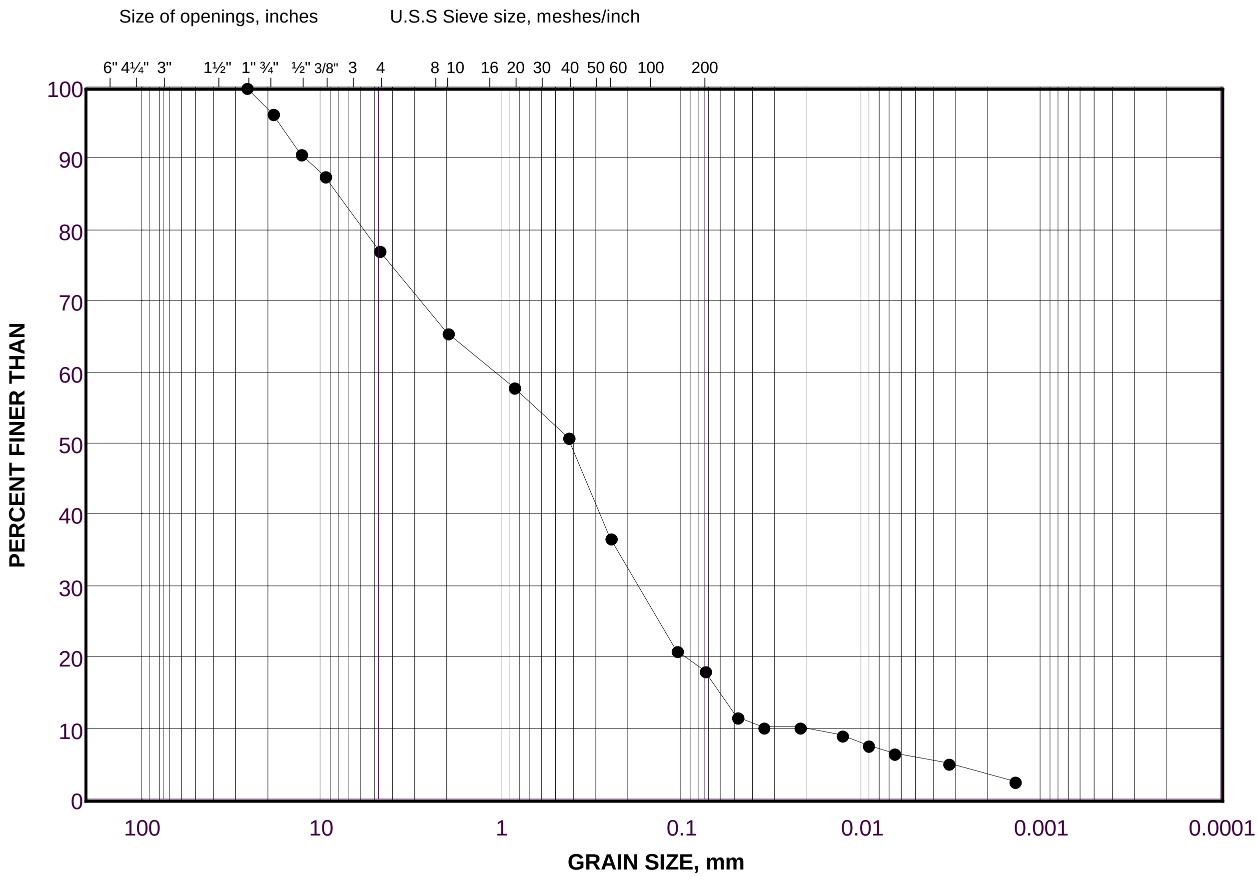
APPENDIX B

Geotechnical Laboratory Results

GRAIN SIZE DISTRIBUTION

FILL - (SM) gravelly SILTY 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)
●	RR-2	7	166.6

Project Number: 1664178 (2000)

Checked By: AMP

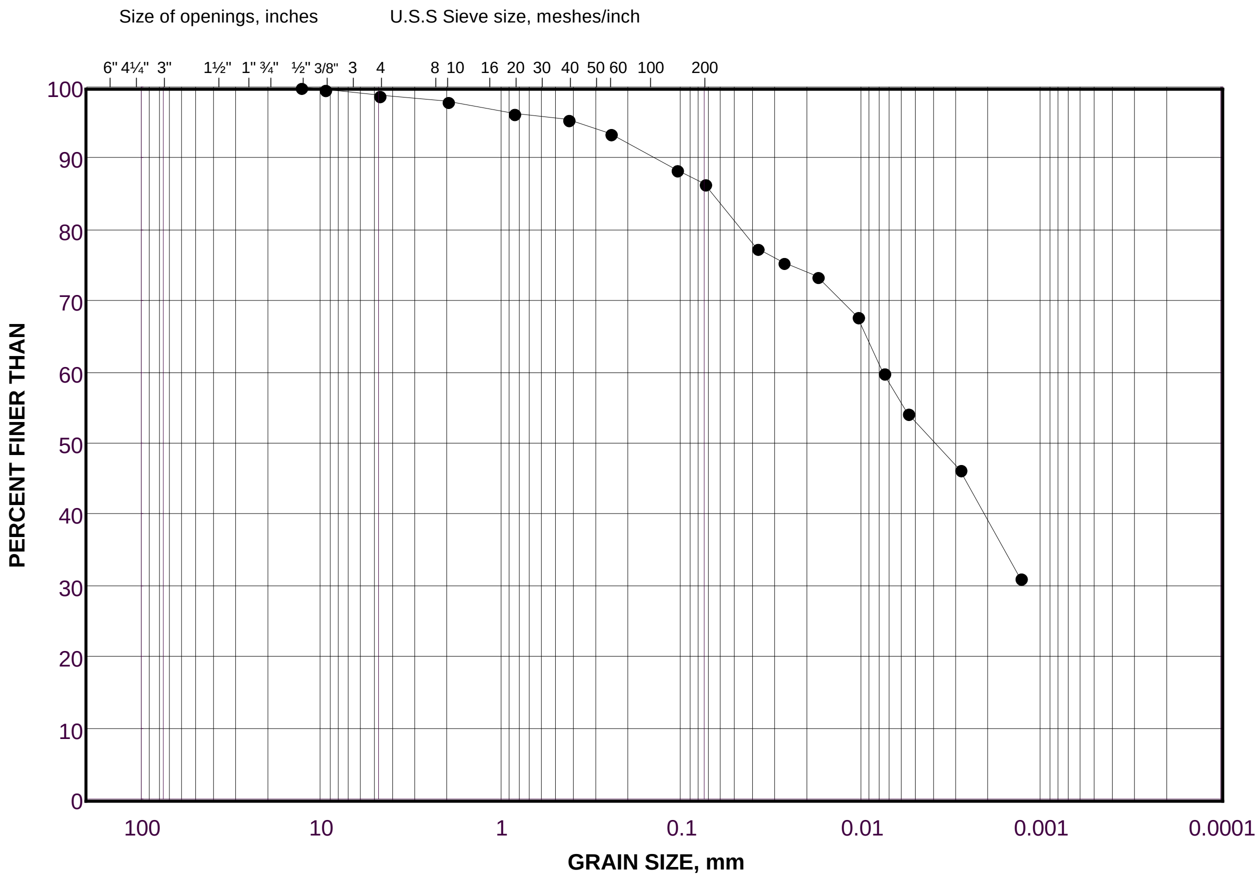
Golder Associates

Date: 03-Feb-19

GRAIN SIZE DISTRIBUTION

(CL) sandy 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)
●	RR-1	6	167.2

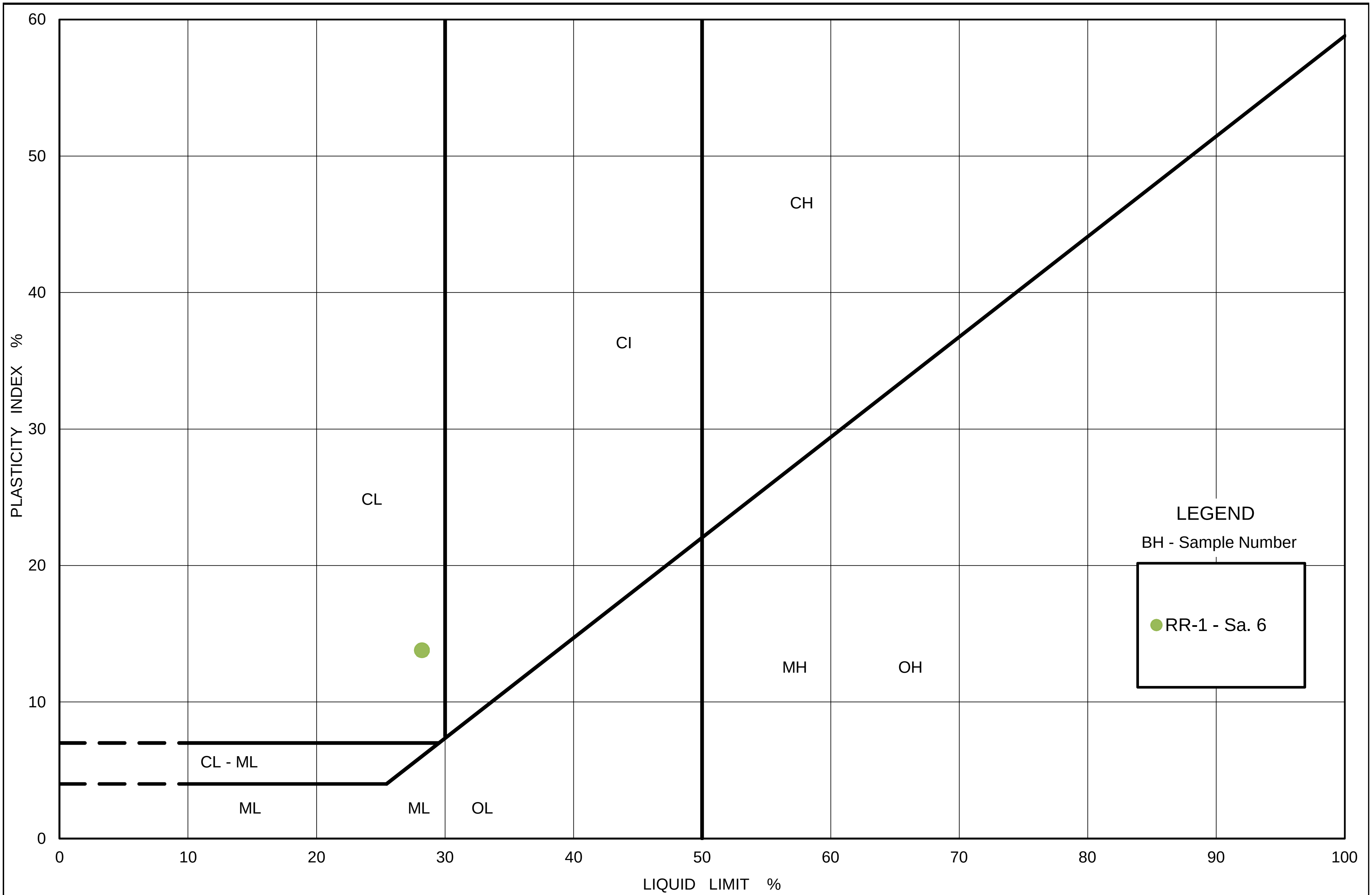
Project Number: 1664178 (2000)

Checked By: AMP

Golder Associates

Date: 03-Feb-19

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



GOLDER

PLASTICITY CHART

(CL) sandy SILTY CLAY

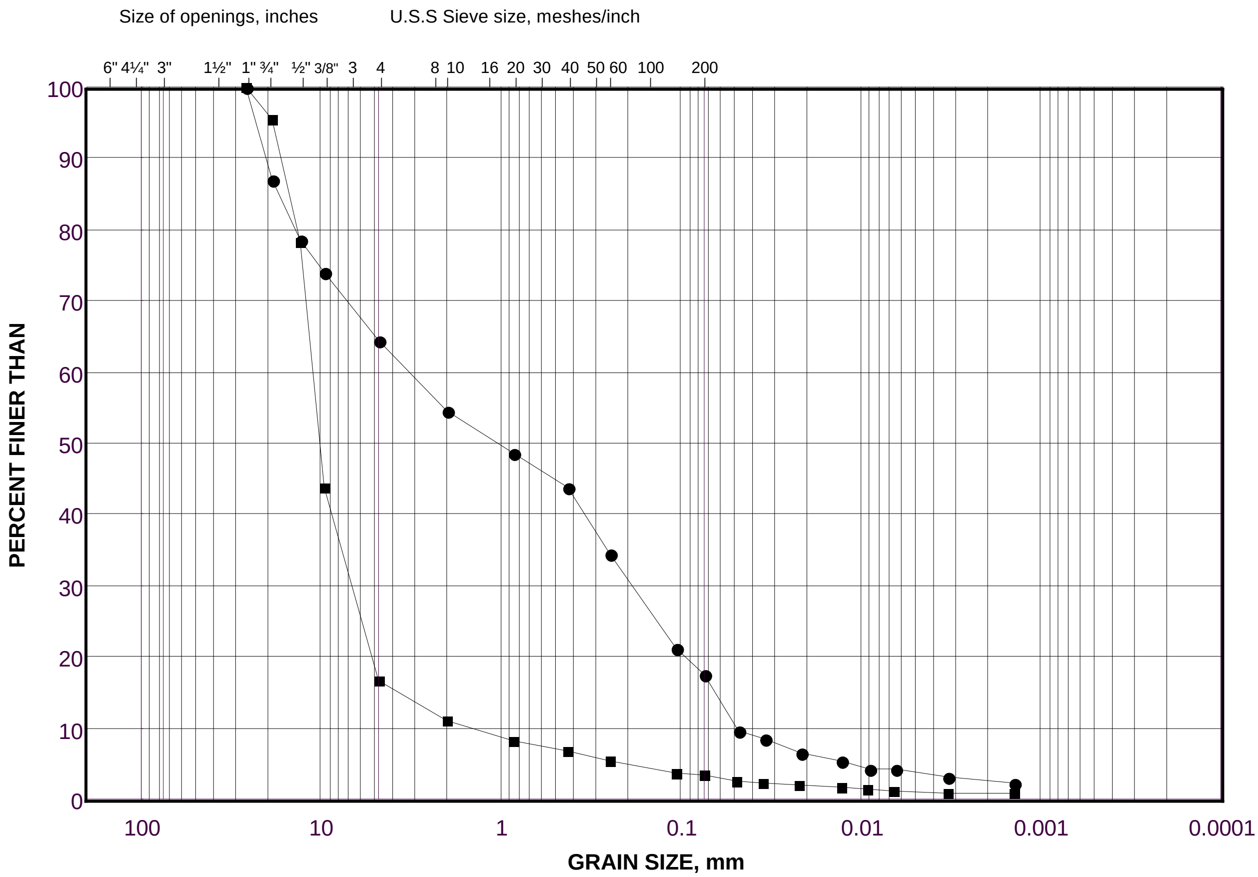
Figure No.: B-3

Project No.: 1664178 (2000)

Checked By: AMP

GRAIN SIZE DISTRIBUTION
(SM) SILTY SAND and GRAVEL to (GP) sandy GRAVEL

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)
●	RR-2	10	162.0
■	RR-1	11	159.6

Project Number: 1664178 (2000)

Checked By: AMP

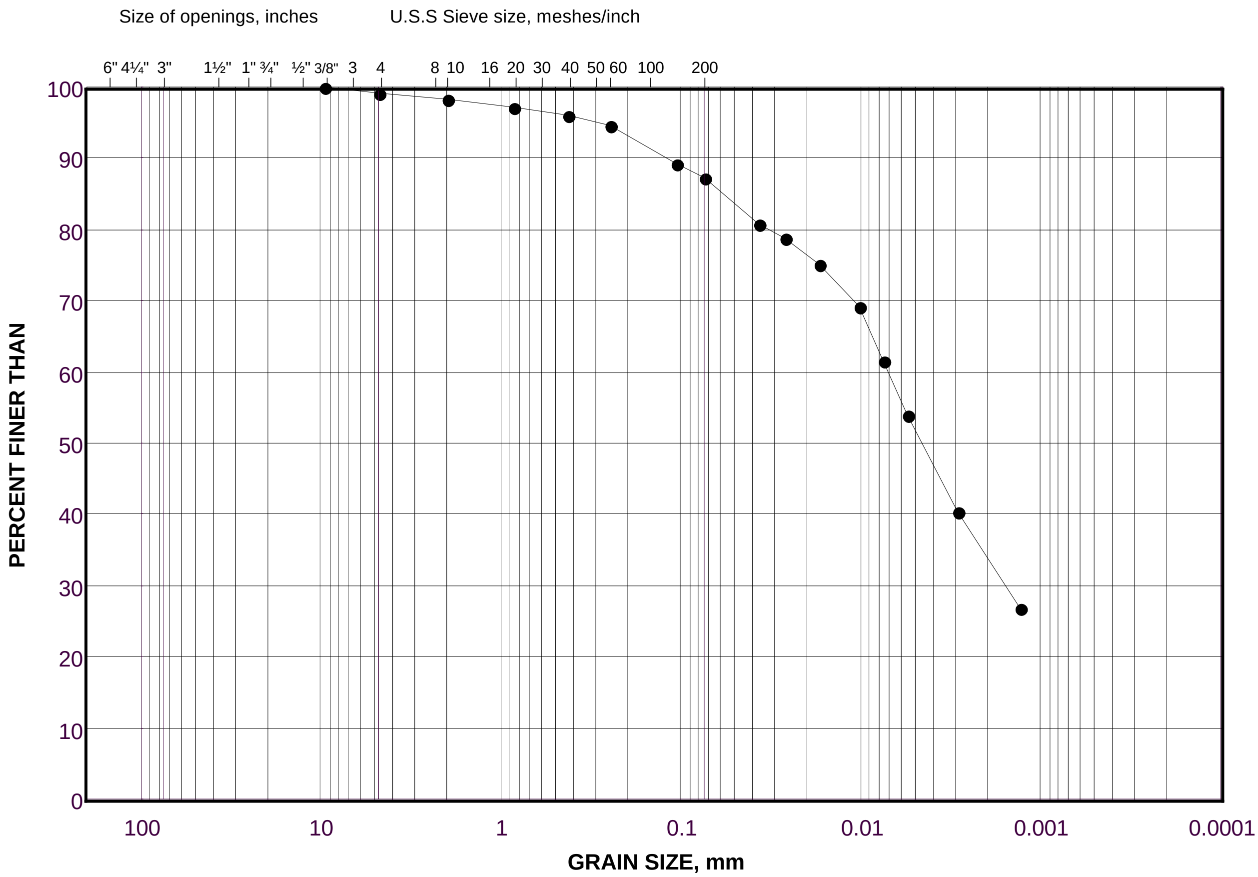
Golder Associates

Date: 03-Feb-19

GRAIN SIZE DISTRIBUTION

(CL) SILTY CLAY

FIGURE B-5

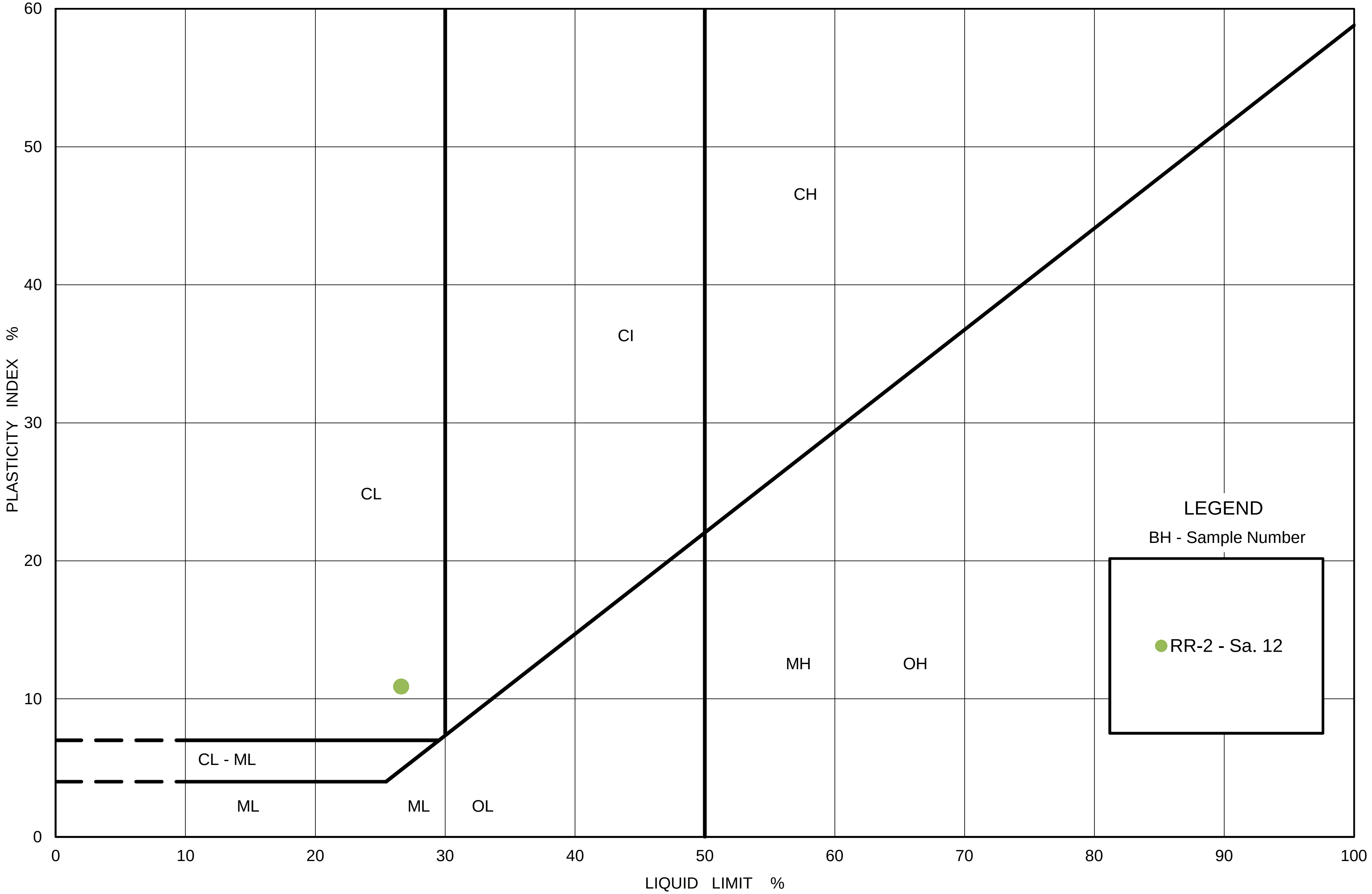


COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RR-2	12	159.0

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



GOLDER

PLASTICITY CHART

(CL) SILTY CLAY

Figure No.: B-6

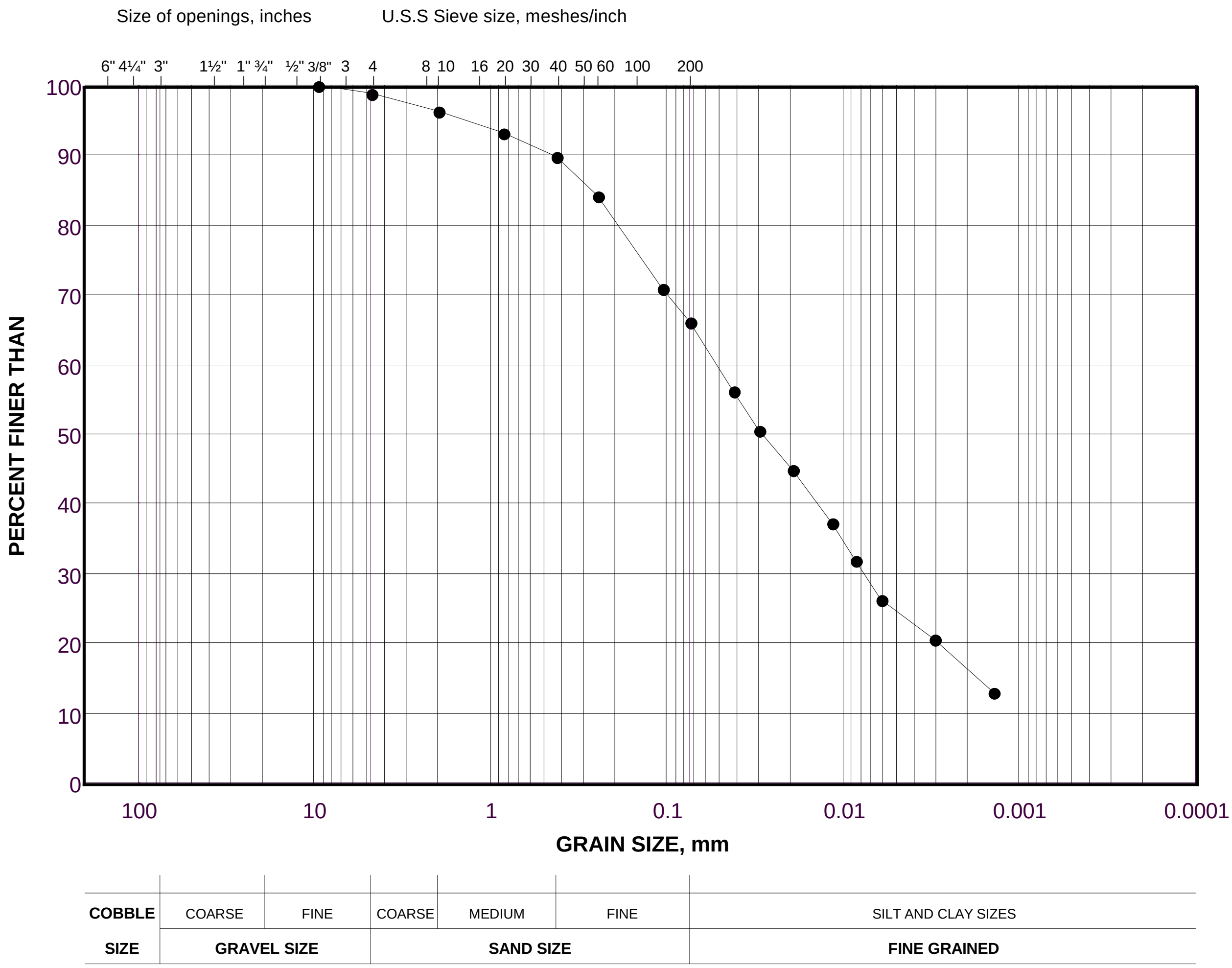
Project No.: 1664178 (2000)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

(CL-ML) sandy CLAYEY SILT (TILL)

FIGURE B-7



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RR-1	12	158.1

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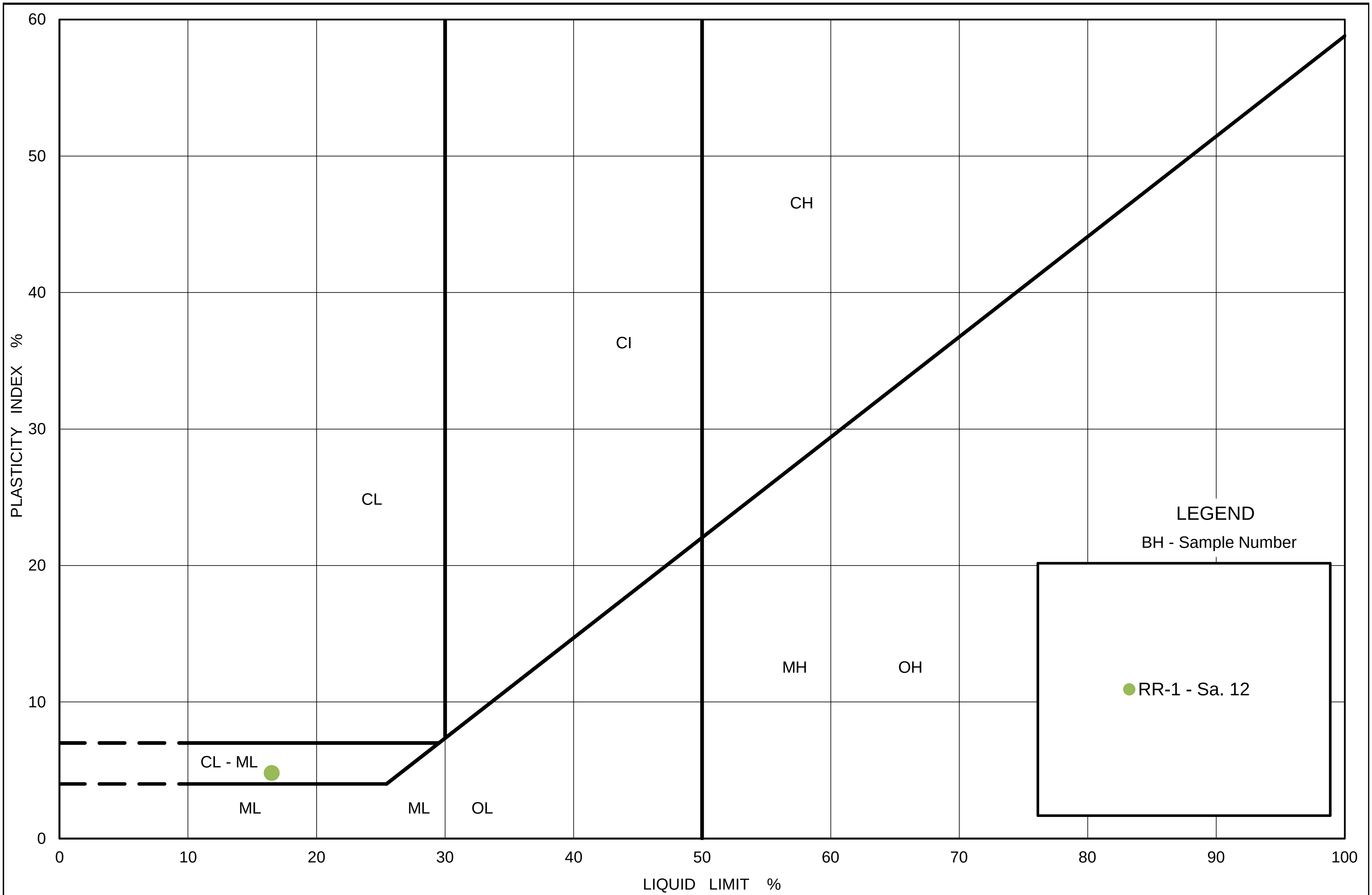
LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ASTM D4318)



GOLDER

PLASTICITY CHART

(CL-ML) sandy CLAYEY SILT (TILL)

Figure No.: B-8

Project No.: 1664178 (2000)

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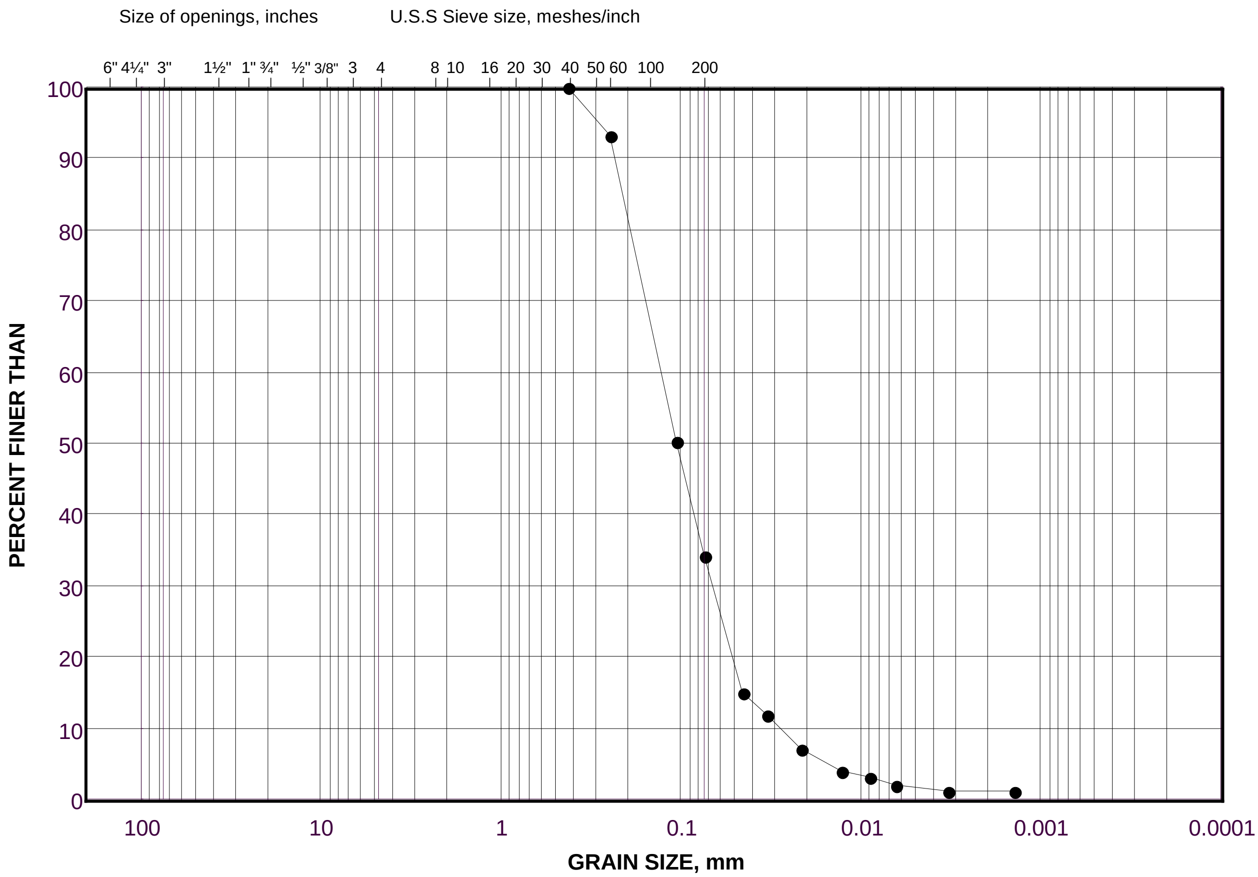
GRAIN SIZE DISTRIBUTION

(SM) SILTY SAND

FIGURE B-9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	RR-2	13	157.5

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