



**Preliminary Foundation  
Investigation and Design Report  
GWP 4059-17-00**

Nagle Road Interchange Study  
Town of Cobourg, Ontario

MTO Site No. 21X-0248/B0

Latitude 43.9935

Longitude -78.1445

G.W.P. 4059-17-00

Geocres No. 30M16-72

Prepared for:

Ministry of Transportation Ontario

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April 2021

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**PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT**

For  
G.W.P 4059-17-00

Nagle Road Interchange Study (Site No. 21X-0248/B0)

Northumberland County and Cobourg, Ontario

## **1.0 INTRODUCTION**

The Ministry of Transportation, Ontario (MTO) and the Town of Cobourg have retained Stantec Consulting Ltd. (Stantec) to undertake a Planning, Preliminary Design, and Class Environmental Assessment Study on Highway 401 for a new interchange near Nagle Road in the Town of Cobourg and the Township of Hamilton. This study is being completed concurrently with the Highway 401 Planning Study from Cobourg to Colborne (GWP 4060-11-00).

The project involves the replacement of the existing two-lane, three-span underpass on the same alignment as the existing bridge. The new underpass, which is being designed to accommodate the future widening of Highway 401 to an eight-lane configuration, is planned to consist of a two-span structure that will be longer and wider, to accommodate four lanes plus two turning lanes, than the existing bridge.

The purpose of the preliminary foundation investigation was to assess the subsurface conditions at the site of the bridge replacement by drilling 2 boreholes and carrying out in-situ testing to supplement existing borehole information and completing a laboratory testing program on selected soil samples obtained from the boreholes.

This Preliminary Foundation Investigation and Design Report (FIDR) has been prepared specifically and solely for the proposed bridge replacement project described above. This Preliminary Report is not to be used for the detail design of this project. A Detailed Foundation Investigation and Design Report will follow in the future after more site investigation is completed. The preliminary foundation recommendations presented in this preliminary report are subject to change, if necessary, based on the findings of the future site investigation.

## **2.0 SITE DESCRIPTION**

### **2.1 SITE LOCATION**

Nagle Road crosses over Highway 401 at approximately Station 13+429, approximately 4 km east of Cobourg, Ontario. The site location is shown on the Key Plan inset on the Borehole Locations and Soil Strata Plan, Drawing No. 1 in Appendix A.



## 2.2 SITE DESCRIPTION

Nagle Road is a two-lane undivided roadway that crosses over Highway 401 on a three-span underpass structure. At the bridge location, Highway 401 is a six-lane divided freeway with three lanes in each direction that is aligned in an approximate east-west orientation; as such, the structure will be referenced as being orientated south to north. The chainage on Highway 401 increases from west to east.

The elevations of the Nagle Road pavement surface at the underpass vary from approximately 146.7 m on the south side of the bridge to 150.1 m on the north side. At the bridge site, the asphalt surface on Highway 401 is at an elevation of just below 140.5 m. The original grade in the vicinity of the bridge was approximately 141 to 146 m indicating that the Highway 401 corridor was developed within a cut.

The existing bridge is a three-span structure constructed in the late 1950's (W.P. 88-57). The bridge has an overall length of approximately 78.8 m. The bridge north abutment and piers are supported on spread footings, while the south abutment is supported on piles. The structure underwent rehabilitation about 6 years ago. Additional details related to the existing bridge foundations are included in Section 8.2 of this report.

The ground surface at the Nagle Road underpass site is surrounded by relatively flat terrain within an overall slope towards the south. The lands surrounding the bridge site are undeveloped, typically contain grass and/or mature trees and are generally used for agricultural purposes.

Recent air-photos of the site show that ground modification work was carried out west/northwest of the existing underpass bridge, adjacent to the west bound lanes. The ground modification begins approximately 30 m west of the bridge and extends 200 m further to the west. This ground modification is associated with a cut slope (including construction access to the slope) that was constructed as part of the recent widening of the highway.

### 2.2.1 Site Reconnaissance

The following items were noted during a site visit on November 29<sup>th</sup>, 2019:

- No visible signs of settlement or deformation of the existing structure was noted. Areas of previous rehabilitation/patching were visible at numerous locations on the exposed concrete of the underpass structure.
- The asphalt on the bridge surface appeared to have been installed relatively recently and displayed minor cracking (longitudinal crack at centerline, transverse cracks near abutments). A photograph of the bridge looking north is provided below.





- No signs of embankment settlement or significant instability were observed. Some minor erosion of the embankment was noted in areas where surface water originating from the road appeared to have flowed over the slope face.
- The slope erosion protection beneath the abutments was in poor condition (e.g. overgrown with vegetation, displaced materials etc.).

### 2.2.2 Site Drainage

Regionally, surface water flow in the area of the site is typically from north to south towards Lake Ontario.

### 2.2.3 Geological Information

The site is located within the Iroquois Plain physiographic lowland region which borders the Lake Ontario and specifically within the subsection extending from Newcastle to Trenton. Chapman and Putnam (1984) states the following regarding this physiographic region: "From Cobourg to a point a few miles east of Colborne, the Iroquois Plain is about three and half miles wide and has a peculiar belted pattern. Through the center along Highway 2, numerous drumlins can be seen. They are large drumlins, some of them reaching heights of 150 feet or more, while some slopes have been over steepened by the waves of post-Iroquois waters. They have a southward alignment and the hollows between them are floored with silt. To the north along the route of Highway 401, the high shoreline of Lake Iroquois may be seen, the old waterplane rising steadily from 565 feet a.s.l. near Baltimore to 600 feet a.s.l. near Biddy Lake".

Existing geological mapping suggests the Nagle Road underpass structure is underlain by a sandy silt to silty sand-textured till. Coarse textured glaciolacustrine deposits consisting of sand and gravel with minor silt and clay are present to the north of the site.



Review of available water well records for wells located to the south of the bridge site indicates that bedrock was encountered at depths of approximately 25 m to 40 m below ground surface.

### 3.0 PREVIOUS INVESTIGATIONS / AVAILABLE INFORMATION

Prior to carrying out the subsurface investigation, Stantec reviewed subsurface information available within the MTO GEOCRE database. The following provides a summary of reports that provided information near to the proposed underpass replacement location.

- GEOCRE Report titled “Foundation Report on New Bridge at Highway No. 401, Line ‘E’ Underpassing Gravel Road between Lots 9 and 10, Township of Hamilton, Northeast of Cobourg” prepared by the Materials & Research Section – Downsview of the Department of Highways – Ontario in 1958 (GEOCRE Reference No. 30M16-019).

The investigation consisted of advancing two boreholes with adjacent dynamic cone penetration tests and an additional two dynamic cone penetration tests at the site between the dates of November 12<sup>th</sup> and 25<sup>th</sup>, 1957. Borings 1 and 3 were drilled by advancing BX casing using a skid-mounted core drill machine. A series of sleeve samples were collected at these locations. Borings 2 and 3 consisted solely of dynamic cone penetration tests. The borehole records indicate that a 250 lb (~113.6 kg) hammer was used with a drop height of 19.5 inches (~0.50 m) during sampling; it is noted that this is not the standard hammer size or drop height used for Standard Penetration Tests (SPTs) which uses a 140 lb (~63.5 kg) hammer with a 30 inch drop (~0.76 m). Therefore, the penetration resistances measured are considered to be similar to, but do not represent, SPT ‘N’ values. Penetration resistances of greater than 100 blows for 0.3 m of penetration of the sleeve sampler were measured at all except one location where a resistance of 44 blows per 0.3 m of penetration was recorded.

The subsurface stratigraphy encountered at the boreholes consisted of a surficial layer of topsoil underlain by an extensive deposit of glacial till. The till was encountered to the maximum depths investigated which extended to depths of 9.4 m below the original ground surface. The till deposit is described as a boulder “gravel - sandy loam till”. The deposit is reported to be made up of about 20% cohesive material, 45% fine aggregate, and 35% coarse aggregate with a moisture content of about 5.5%.

Dynamic cone penetration tests (DCPTs) were performed by driving a cone to refusal at each of the testing locations. As noted above, the borehole records indicate that a 250 lb (~113.6 kg) sampling hammer and a drop height of 19.5 inches (~0.50 m) was used during the testing. These tests were terminated upon encountering refusal at elevations ranging from approximately 137.7 m to 144.4 m. The DCPT records indicate that bouncing of the hammer was observed at the refusal depth at all locations; this may have been due to the presence of cobbles and/or boulders in the till soil at the site. Lower penetration resistances were recorded within the upper portions of the DCPTs, up to about 3 m from the original ground surface at some locations, however, these looser materials were removed as part of the original construction of the highway in order to attain the pavement subgrade level.

Piezometers/monitoring wells were not installed as part of this investigation.

The approximate locations of the boreholes and cone penetration tests are shown on the Borehole Location Plan (Drawing No. 1) provided in Appendix A based on the Drawing No. F-57-46A from the above-mentioned GEOCRE





report. The borehole records from the investigation and the original borehole location plan are included in Appendix B for reference.

## 4.0 SUBSURFACE CONDITIONS

### 4.1 FIELD INVESTIGATION PROCEDURES

The current subsurface investigation program consisted of advancing two boreholes, identified as BH20-1 and BH20-2, with one borehole advanced at each side of highway. Borehole BH20-1 was drilled to the east of the existing bridge in the south shoulder of the highway (i.e. immediately south of the east-bound lanes of Highway 401) while Borehole BH20-2 was drilled on east side of the Nagle Road at the north abutment of the proposed bridge. The locations of these boreholes are shown on the Borehole Locations and Soil Strata Plan, Drawing No. 1, in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of both private and public utilities.

The boreholes were advanced using truck or track-mounted drill rigs equipped for soil sampling between the dates of April 29<sup>th</sup> and May 5<sup>th</sup>, 2020. Borehole BH20-1 was carried out using continuous flight solid-stem augers while Borehole BH20-2 was advanced using continuous flight hollow-stem augers up to approximately 7.5 m depth and wash boring methods were used thereafter. Coring methods were used to advance this borehole from 8.5 m to 9.1 m below ground surface after casing refusal was encountered on an inferred boulder.

The subsurface stratigraphy encountered in each borehole was recorded in the field by a member of Stantec's geotechnical staff. Standard Penetration Tests (SPTs) were carried out in the boreholes and split spoon samples were collected at regular intervals. All recovered SPT samples were returned to our Ottawa laboratory for detailed classification and testing.

Groundwater conditions were observed during drilling and on completion of drilling of each borehole. After completion of drilling, the boreholes were sealed with bentonite. Borehole BH20-1 was provided with a surficial layer of cold patch asphalt.

### 4.2 LOCATION AND ELEVATION SURVEY

The boreholes were located in the field relative to the existing site features and the borehole locations and elevations were subsequently determined by Tulloch Geomatics, a licensed survey firm. The borehole coordinates and ground surface elevation information are considered accurate to 0.1 m. Table 4.1 below summarizes the borehole location information with the borehole ground surface elevations, depths and termination elevations.

**Table 4.1: Borehole Coordinate and Elevation Information**

Borehole	MTM Zone 11 Coordinates		Approximate Ground Surface Elevation (m)	Borehole Depth (m)	Borehole Termination Elevation (m)
	Northing	Easting			
BH20-1	4873498.6	413530.6	140.6	11.3	129.3
BH20-2	4873554.4	413513.4	149.0	12.8	136.2



## 4.3 LABORATORY TESTING

All samples were transported to Stantec's Ottawa laboratory where they were visually examined by a geotechnical engineer. The geotechnical laboratory testing program completed on the borehole samples is summarized in Table 4.2.

**Table 4.2: Geotechnical Laboratory Testing Program**

Test Description	Number of Tests
Moisture Content	25
Atterberg Limits	2
Grain Size Distribution (sieve & hydrometer)	8

Two soil samples, one from each borehole location, were also tested for pH, soluble sulphate content, chloride content, and resistivity by Paracel Laboratories Ltd. of Ottawa.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

## 4.4 OVERBURDEN STRATIGRAPHY

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are displayed on the Record of Borehole sheets contained in Appendix C. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix C. The results of geotechnical laboratory testing are presented in Appendix D. Drawing No. 1 in Appendix A illustrates the borehole locations and includes a stratigraphic section of the soils encountered in the boreholes.

The stratigraphic boundaries on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units.

In general, the subsurface stratigraphy encountered at the borehole locations consisted of a surficial layer of topsoil or asphalt underlain by a fill layer (Highway 401 pavement structure and approach embankments) underlain by a deposit of glacial till. The till is well-graded and predominantly generally granular in nature, ranging in composition from silty sand to silty sandy gravel, with the exception of the upper portion of the till at Borehole BH20-1 which is comprised of clayey silt with sand. Both boreholes were terminated within the till at depths of 11.3 m to 12.8 m below existing ground surface.

The following sections provide a summary of the subsurface conditions encountered during the investigation.

### 4.4.1 Surficial Materials

Boreholes BH20-1 was drilled through the existing asphalt in the south shoulder of the highway. The asphalt thickness at this borehole location was approximately 300 mm.

An approximately 75 mm thick, surficial deposit of topsoil was encountered at ground surface at the location of Borehole BH20-2.



#### 4.4.2 Fill

Cohesionless fill materials were encountered beneath the asphalt in Borehole BH20-1 and below the topsoil in Borehole BH20-2.

The fill material encountered in BH20-1 is associated with the Highway 401 pavement structure and consists of silty sandy gravel fill. Standard Penetration Test (SPT) N-values ranging from 25 to 49 blows per 0.3 m of penetration were measured within the pavement structure fill indicating it is in a compact to dense state. This pavement structure fill is approximately 1.0 m thick and extends to 1.3 m depth or an elevation of approximately 139.2 m.

The fill material encountered in BH20-2, which is inferred to be associated with the approach embankment to the Nagle Road underpass structure, varies in composition from silty sand near ground surface to gravelly sand near the base of the fill. The fill was noted to contain trace clay and rootlets and is inferred to contain cobbles and boulders based on grinding of augers noted at multiple locations within the fill. SPT 'N' values of between 5 and 9 blows per 0.3 m of penetration were measured within these fill materials indicating these materials are in a loose state. The fill at BH20-2 is approximately 3.0 m thick with the base of the fill at an elevation of approximately 146.0 m.

Laboratory testing of samples of the fill materials yielded moisture contents that ranged from approximately 4 to 10%, expressed as a percentage of the dry weight of the soil. Gradation analyses were carried out on two (2) representative samples of the fill materials. The results of the tests are illustrated on the gradation curves on Figure No. D1 in Appendix D. Based on the laboratory results, the USCS group symbol for this fill material will be silty sandy gravel (GM) or gravelly silty sand (SM).

#### 4.4.3 Clayey Silt with Sand (TILL)

A cohesive till layer comprised of clayey silt with sand containing cobbles and boulders was encountered below pavement structure fill layer in Borehole BH20-1. This cohesive till extended to approximately 1.3 m depth corresponding to a base elevation of about 138.4 m.

A SPT 'N' value of 19 blows per 0.3 m of penetration was measured within this deposit. Based on the SPT 'N' value and manual/tactile examination of a sample of this material, the clayey silt till is considered to have a very stiff consistency.

Laboratory testing of a sample of this deposit yielded a moisture content of 9%. The results of grain size distribution carried out on a sample of this material are shown on Figure No. D2 in Appendix C. The results of Atterberg Limit testing on the same sample, displayed on Figure D3, measured a Plastic Limit of 8 percent and Liquid Limit of 14 percent and corresponding Plasticity Index of 6. Based on the Atterberg Limit testing, the USCS group symbol for this deposit is CL-ML.

#### 4.4.4 Sandy Silt to Silty Sandy Gravel (TILL)

A predominantly granular glacial till deposit typically consisting of gravel, cobbles, and boulders in a matrix of silty sand to sandy silt was encountered beneath the fill materials or clayey silt till deposit at depths of approximately 2.2 m and 3.0 m below ground surface. The composition of this till varies from silty sand/sandy silt containing varying amounts of gravel to silty sandy gravel in Borehole BH20-1 and sandy silt/silty sand in Borehole BH20-2. The till also contains zones of sandy clayey silt.



Typically, cobbles and boulders are present throughout the till deposits of Southern Ontario. The presence of cobbles and/or boulders in the till deposit at this site is inferred based on frequent grinding of the augers/drill equipment throughout the deposits during drilling. In Borehole BH20-2, coring methods were used to advance the borehole from 8.5 m to 9.1 m below ground surface after encountering auger refusal was encountered. The coring confirmed the presence of cobbles and/or boulders within this deposit.

SPT 'N' values measured in this deposit ranged from 11 blows per 0.3 m of penetration to greater than 100 blows per 0.3 m of penetration. The lower SPT 'N' values were typically recorded within the upper portion of the deposit with 'N' values of 11 and 16 blows per 0.3 m of penetration recorded in the upper 1.5 m of the till in BH20-2 and a SPT 'N' value of 47 measured at a depth of about 2.5 m in Borehole BH20-1; these resistance values indicate the upper portion of the till is in a compact to dense state. The remainder of SPT 'N' values measured at greater depths were typically greater than 100 blows per less than 0.3 m of penetration indicating the till becomes very dense at depth.

Laboratory testing of samples of the till materials yielded moisture contents that ranged from approximately 3 to 14%. The results of grain size distribution testing carried out on five samples of this deposit are shown on Figure No. D4 in Appendix C.

The results of an Atterberg Limit test carried out on a finer-grained portion of the till from Borehole 20-2 is shown on Figure No. D5 in Appendix C. This test measured a Plastic Limit of 10 percent and Liquid Limit of 15 percent and corresponding Plasticity Index of 5.

The USCS group symbol for the till is considered to vary from silty sandy gravel (GM) to silty sand/sandy silt (SM to ML). The till also contains zones of sandy clayey silt (CL-ML).

The till extended to the termination depths of boreholes at 11.3 m and 12.8 m below ground surface (corresponding to elevations of 129.3 m and 136.2 m in Boreholes BH20-1 and BH20-2, respectively).

## **4.5 GROUNDWATER**

After completion of drilling of Borehole BH20-1, groundwater was observed at 2.4 m (~Elevation 138.2 m), while sloughing/cave-in was encountered at 6.1 m (~Elevation 134.4 m). No groundwater seepage was noted prior to initiation of coring/wash boring at depth of 7.8 m in Borehole BH20-2.

Groundwater levels at the site will be subject to fluctuations due to seasonal changes, snowmelt and precipitation events. The water levels should be expected to be higher during the spring season and during and following periods of heavy precipitation or snow melt. As Highway 401 was developed within a cut at the bridge site, groundwater levels are anticipated to be near ground surface and controlled by the water levels in the roadside ditches at the site.

## **5.0 CHEMICAL TESTING**

One representative sample of the subsurface soils was collected from each of the boreholes and was tested for pH, sulphate and chloride concentrations, and resistivity. The analysis results are provided in Table 6.1.



**Table 5.1: Results of Chemical Analysis**

Borehole No	Sample No.	Depth (m)	pH	Resistivity (Ohm-m)	Chloride (µg/g)	Sulphate (µg/g)
BH20-1	SS04	2.3-2.9	7.9	15.5	377	38
BH20-2	SS08	5.3-5.9	7.7	70.3	27	22

## 6.0 MISCELLANEOUS

The field work was carried out under the supervision of Kirby Lales under the direction of Kevin Nelson, P.Eng.

The utility locates for the boreholes were arranged by Stantec personnel.

The drilling equipment was supplied and operated by Downing Drilling Ltd. of Grenville-sur-la-Rouge, Quebec.

The location and elevation survey of the boreholes was completed by Tulloch Geomatics Inc.

Traffic control service was provided by On Track Safety Ltd. of Thornhill, Ontario.

Geotechnical laboratory testing was carried out at Stantec's Ottawa laboratory. The chemical testing for pH, soluble sulphate and chloride contents, and soil resistivity was carried out by Paracel Laboratories Ltd. of Ottawa.

This report was prepared by Ramin Ghassemi, P.Eng., Ph.D., and reviewed by Kevin Nelson, P.Eng., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.



## 7.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted;

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PART B - PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

For  
G.W.P 4059-17-00

Nagle Road Interchange Study (Site No. 21X-0248/B0)

Northumberland County and Cobourg, Ontario

## 8.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

### 8.1 OVERVIEW

This section of the report provides preliminary foundation design input related to the proposed replacement of the underpass structure located at the crossing of Nagle Road over Highway 401 (Site No. 21X-0248/B0). The new underpass is being designed to facilitate the construction of a new interchange and to accommodate the ultimate 8-lane configuration.

The recommendations provided herein are based on interpretation of the factual data obtained from Stantec geotechnical investigation carried out on April 29, 2020, and May 5, 2020.

The interpretation and recommendations provided in this report are intended solely to provide the designers with information to assess the feasible foundation alternatives for the proposed underpass replacement. As such, where comments are made on construction aspects of the project, they are provided only to highlight those aspects which could affect the preliminary design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

Additional subsurface investigation will be required to meet minimum MTO foundation investigation requirements for the detail design of the replacement underpass structure. This Preliminary Report is not to be used for the detail design of this project. A Detailed Foundation Investigation and Design Report will follow in the future after more site investigation is completed. The preliminary foundation recommendations presented in this preliminary report are subject to change, if necessary, based on the findings of the future site investigation.

### 8.2 PROJECT DESCRIPTION AND BACKGROUND

#### 8.2.1 Project Description

The project involves the Planning, Preliminary Design, and Class Environmental Assessment Study for a new interchange at Nagle Road and Highway 401 in the Town of Cobourg and the Township of Hamilton. This study is being completed concurrently with the Highway 401 Planning Study from Cobourg to Colborne (GWP 4060-11-00).

Based on preliminary design information, the new interchange at Highway 401 and Nagle Road will require a new underpass with spans that will accommodate an 8-lane highway configuration (4 lanes in each direction) plus ramp lanes as well as four-lanes of traffic on Nagle Road. This will require a new underpass structure that is wider and



longer than the existing bridge. Nagle Road is planned to be closed during construction of the new underpass structure.

### 8.2.2 Existing Underpass Structure

The existing underpass is an approximately 66.9 m long, three-span structure constructed in the late 1950's (W.P. 88-57). The pavement surface elevations on the Nagle Road underpass vary from approximately 146.7 m (south side) to 150.1 m (north side) while the asphalt surface on Highway 401 is at an elevation of just below 141 m.

The following details on the existing bridge foundations are provided based on the information shown on the available structural design drawings for the existing bridge structure and GEOCREC documentation related to the site:

- The foundation investigation report for this site, contained in the MTO GEOCREC database, indicates that the bridge can be supported on 'spread footing type foundations' and that a bearing value of 3 tons per square foot (~287 kPa) could be used for footings founded at an elevation in the vicinity of 453 feet (138.1 m). This bearing value is inferred to correspond to an allowable bearing pressure using a working stress design philosophy.
- The bridge north abutment and piers are supported on shallow foundations (i.e. large strip and spread/pad footings), while the south abutment is supported on a series of piles.
- The north abutment is founded on a U-shaped foundation that also supports the associated abutment wingwalls. The foundation is approximately 10.6 m long, 3.4 m wide and founded at an elevation of approximately 144.8 m with approximately 2.3 m long by 0.9 m wide extensions beneath each wingwall.
- The south abutment pile cap is approximately 10.6 m long, 4.0 m wide and founded at an elevation of approximately 142.2 m. This foundation is supported on three rows of 12BP53 (310x79) H-piles. The two rows of piles closest to the highway (6 piles in first row, 5 piles in second/middle row) are inclined at 1H:3V towards the highway and the remaining row of 4 piles are vertical. The bridge design drawings indicated piles were to be driven to a capacity of 45 tons (~400 kN); no information on the pile lengths is provided.
- Each bridge pier is supported on two, approximately 3.1 m by 3.1 m square footings founded at an elevation of approximately 137.4 m (north pier) and 138.5 m (south pier). The design drawings indicate that the footings were designed using an allowable bearing pressure of 6000 psf (~287 kPa).
- Approximately 5.2 m and 6.7 m long approach slabs are present adjacent to south and north abutments, respectively.

### 8.2.3 Proposed Structure Modifications and Replacement

Based on preliminary investigations, we understand that the Nagle Road structure is planned to be replaced to accommodate the future 8-laning of Highway 401 and the development of an interchange with Highway 401. The new underpass would consist of a new 2-span structure on the same alignment as the existing structure.

The new bridge is planned to be wider than the existing bridge in order to accommodate four lanes of through traffic and two turning lanes. The length of the new bridge will be increased to approximately 78 m. The existing approach embankments will be raised slightly (by less than 1 m on the north, and by about 2 m on the south, sides of the bridge) and will be widened to accommodate the wider bridge. The approach embankments are planned to be constructed with 2H:1V sideslopes.

The new bridge will incorporate either integral abutments, if the ground conditions are conducive, or semi-integral abutments.





## 8.3 DEGREE OF SITE AND PREDICTION MODEL UNDERSTANDING

The Canadian Highway Bridge Design Code (CHBDC) [December, 2014] requires an assessment of the “degree of site and prediction model understanding” as a component of the geotechnical engineering investigation and/or services. The site and prediction model understanding includes the geotechnical properties on the site and the accuracy and degree of confidence regarding the numerical performance prediction models to be used to estimate the geotechnical serviceability limit states reactions and ultimate limit states resistances.

Based on the scope of subsurface investigations completed and available subsurface information related to this site, a “Typical Understanding” has been adopted for foundation design assessment purposes. The consequence classification has been selected as “Typical Consequence” as per Section 6.5 of the Commentary on CSA S6-14, Canadian Highway Bridge Design Code (CHBDC), (S6.1-14).

## 8.4 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered consisted of a surficial layer of topsoil or asphalt underlain by a fill layer (Highway 401 pavement structure and/or approach embankment fill) underlain by a deposit of glacial till. The till is broadly-graded typically consisting of sandy silt/silty sand to silty sandy gravel and contains cobbles and boulders. The groundwater level was recorded at approximately 2.4 m depth (~Elevation 138.2 m) after completion of drilling of Borehole BH20-1.

The soil profiles identified in Tables 8.1 to 8.3 below and Drawing Nos. E1 to E3 in Appendix E can be used for the preliminary design of the bridge replacement. The geotechnical parameters identified in the soil profile were developed based on a synthesis of the borehole data, the measured penetration resistance values and laboratory index test results (including moisture contents) of soil samples obtained in the investigation.

**Table 8.1: Representative Soil Profile North Abutment - Nagle Rd. Underpass**

Elevation (m)		Soil Type	Design Parameters		
From	To		Total Unit Weight $\gamma$ , (kN/m <sup>3</sup> )	Friction Angle <sup>2</sup> $\phi'$ , (°)	Soil Modulus E, (MPa)
149.0	146.0	FILL: Silty SAND to GRAVELLY SAND (SM/SP), loose	21.0	30	10
146.0	144.5	Compact SANDY SILT (ML) to Silty SAND (ML/SM) (TILL). Contains frequent cobbles and boulders.	22.5	32	40
144.6	136.2	Very dense SANDY SILT (ML) to Silty SAND (ML/SM) (TILL). Contains frequent cobbles and boulders.	24.0	35	150

Note:

- (1) Groundwater is assumed to be at an Elevation of 139 m for preliminary design purposes. Submerged unit weights ( $\gamma'$ ) should be used below the groundwater level.
- (2) The friction angles are applicable to drained conditions only.



**Table 8.2: Representative Soil Profile – Central Pier - Nagle Rd. Underpass**

Elevation (m)		Soil Type	Design Parameters			
From	To		Total Unit Weight $\gamma$ , (kN/m <sup>3</sup> )	Friction Angle <sup>2</sup> $\phi'$ , (°)	Undrained Shear Strength <sup>2</sup> $S_u$ , (kPa)	Soil Modulus $E$ , (MPa)
Ground surface	139.2	FILL: Silty SANDY GRAVEL (GM), compact to dense	21.5	32	-	25
139.2	138.4	Very Stiff CLAYEY SILT with sand (CL-ML) (TILL). Contains frequent cobbles and boulders.	21.5	32	150	50
138.4	129.3	Dense to very dense, SANDY SILT to silty sandy GRAVEL (ML/SM/GM) (TILL). Contains frequent cobbles and boulders.	24.0	36	-	150

Note:

- (1) Groundwater is assumed to be at an Elevation of 139 m for preliminary design purposes. Submerged unit weights ( $\gamma$ ) should be used below the groundwater level.
- (2) The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only.

**Table 8.3: Representative Soil Profile – South Abutment - Nagle Rd. Underpass**

Elevation <sup>1</sup> (m)		Soil Type <sup>1</sup>	Design Parameters			
From	To		Total Unit Weight $\gamma$ , (kN/m <sup>3</sup> )	Friction Angle <sup>3</sup> $\phi'$ , (°)	Undrained Shear Strength <sup>3</sup> $S_u$ , (kPa)	Soil Modulus $E$ , (MPa)
146.2	141	FILL: Composition to be determined during detailed design	TBD	TBD	TBD	TBD
141	138.4	Very Stiff CLAYEY SILT with sand (CL-ML) (TILL). Contains frequent cobbles and boulders.	21.5	32	150	50
138.4	129.3	Dense to very dense, SANDY SILT to silty sandy GRAVEL (ML/SM/GM) (TILL). Contains frequent cobbles and boulders.	24.0	36	-	150

Note:

- (1) The future detail design investigation will need to verify and validate or adjust the above soil types, including the type/depth of fill and the thickness of the clayey silt till deposit which may have influenced the use of pile foundations to support the existing south abutment.
- (2) Groundwater is assumed to be at an Elevation of 139 m for preliminary design purposes. Submerged unit weights ( $\gamma$ ) should be used below the groundwater level.
- (3) The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only.

Soft soils were not encountered within the boreholes drilled for this preliminary investigation and are not considered in the soil profile tables. Should soft soils be encountered during the detail design investigation, the geotechnical models presented above will need to be adjusted accordingly.



## 8.5 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations,  $f$ , at the site is 1.4 m. Therefore, all footings and pile caps should be provided with a minimum of 1.4 m of soil cover or equivalent insulation for protection against frost heaving.

This depth of frost penetration should also be considered in the design of frost tapers adjacent to the bridge abutment and retaining wall backfill zones.

## 8.6 SEISMIC CONDITIONS

### 8.6.1 Site Class

The available subsurface information indicates the site is underlain at shallow depth by very dense till deposits. Water well records from nearby sites suggest these types of materials are underlain by bedrock at depths of approximately 25 m to 40 m below ground surface. Based on these conditions, it is recommended that Site Class C as defined in Section 4.4.3 of the CHBDC (2014) be used for preliminary design purposes.

### 8.6.2 Peak Ground Acceleration (PGA)

Seismic hazard values for this site were obtained from Natural Resources Canada (2015 National Building Code). The 2015 NBC Seismic Hazard calculation sheet for this site is provided in Appendix F. Table 8.4 summarizes the parameters based on a 2475-year return period to be used in forced based design.

**Table 8.4: Peak Ground Acceleration Data**

<i>PGA</i>	<i>S<sub>a</sub>(0.2)</i>	<i>PGA<sub>ref</sub></i>	<b>Site Class</b>	Site Adjusted <i>PGA</i>
0.11g	0.175g	0.088g	C	0.11g

### 8.6.3 Liquefaction Potential

Liquefaction of the glacial till deposit that underlies the site is not considered to be a concern due to the age of the deposit, the typically very dense nature of the till and the relatively low peak ground acceleration that applies for the site location.

## 8.7 PRELIMINARY FOUNDATION ENGINEERING DESIGN INPUT

The following sections provide preliminary foundation engineering input related to the foundations for a replacement underpass structure at this site. The input provided in this section of the report is preliminary in nature and should be reviewed, and modified as necessary during detail design, once further subsurface investigation is completed and the loading conditions for the new foundations are determined.

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2014).



## 8.7.1 Foundation Options

Both shallow and deep foundation options were evaluated for the proposed replacement bridge structure. Table 8.5 presents the advantages, disadvantages, relative costs and risks/consequences for various foundation options for the Nagle Road replacement bridge.

**Table 8.5: Comparison of Foundation Options for Nagle Road Underpass Structure**

Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
<b>H-Piles</b> Driven to Practical Refusal in Very Dense Till (Integral Abutment)	<ul style="list-style-type: none"> <li>Allows for use of Integral Abutments</li> <li>Reduced settlement</li> <li>Reduces depth of excavations and requirements for temporary support systems.</li> <li>More suitable than pipe piles for difficult driving conditions</li> </ul>	<ul style="list-style-type: none"> <li>Pre-drilling in till will be required prior to pile installation in order to provide sufficient pile lengths for integral abutments</li> <li>Pile capacity may not be fully utilized due to difficult driving conditions leading to limited pile lengths/shaft friction.</li> </ul>	Medium	<ul style="list-style-type: none"> <li>Pile damage during installation</li> <li>Potential for shallow refusal of piles on cobbles and boulders requires pre-drilling</li> </ul>
<b>Shallow Foundations</b> Founded on Very Dense Till (Semi-Integral Abutment and/or Central Pier)	<ul style="list-style-type: none"> <li>Lower foundation costs than deep foundations</li> <li>Existing structure supported on shallow foundations with suitable foundation performance</li> <li>Drilling through difficult deposits avoided</li> </ul>	<ul style="list-style-type: none"> <li>Not suitable for integral abutments (Semi-integral abutments possible)</li> <li>Larger foundation areas required compared to integral abutments or drilled piers</li> <li>Deeper excavations needed at abutments to found below existing fill materials.</li> <li>Groundwater control anticipated to be required during construction of pier</li> </ul>	Low to medium	<ul style="list-style-type: none"> <li>Potential for increased differential settlement</li> </ul>
<b>Drilled Piers/ Caissons</b>	<ul style="list-style-type: none"> <li>Can transmit very large axial and lateral loads</li> <li>Shorter construction time than shallow foundations</li> </ul>	<ul style="list-style-type: none"> <li>Difficult to drill and advance liners in granular till deposits containing boulders and cobbles</li> <li>Requires use of drilling mud to balance water pressures; cannot be visually inspected</li> <li>Not suitable for integral bridge abutment</li> </ul>	High	<ul style="list-style-type: none"> <li>Liners and/or drilling mud required to mitigate groundwater issues.</li> <li>Installation of liners to maintain sidewall stability may not be practical without specialized equipment.</li> </ul>

Due to the presence of very dense till at shallow depth beneath the existing highway grade, support of the bridge pier on shallow foundation systems is considered to be more economical and practical than the use of deep foundation systems. Shallow foundations may also be used for the abutments. Shallow foundations, if used, should be founded on/within the dense to very dense portion of the till deposits. This would require removal of all approach embankment fill materials at the future abutment locations and founding the pier footing(s) below the very stiff clayey silt till.

For an integral abutment design, steel H-pile foundations would be a suitable foundation option. The piles would be driven to practical refusal in the very dense till deposits and would develop most of their load carrying capacity from tip resistance/end-bearing. Where integral abutments are adopted, the upper portion of the piles are installed within sand-filled, corrugated steel pipe (CSP) liners to provide suitable flexibility of the steel H-piles. Driven piles may “hang up”/encounter refusal within the very dense till particularly where cobbles and/or boulders are encountered. In this regard, pre-drilling would be required both to permit installation of the corrugated steel pipe (CSP) liners and to obtain sufficient pile embedment/fixity below the CSP liners.



Drilled piers/caisson foundations could be considered for support of the central pier but would require either temporary liners or drilling mud to mitigate the potential risks of ground loss or collapse within the water-bearing, predominantly granular till soils during construction. Liner installation would be hindered by the presence of cobbles and boulders. The use of churn drills and possibly rock coring techniques may be required to penetrate these obstructions within the glacial till. Furthermore, the use of “wet” installation methods would preclude the ability to review/confirm the materials present at the base of the caissons and assess the potential for reduced capacity (due to changing subsurface conditions) increasing the risk of unsuitable foundation performance.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the central pier of the proposed bridge on shallow foundations and to support the bridge abutments on either shallow foundations or driven steel H-piles if an integral abutment is preferred.

### **8.7.2 Shallow Foundations**

The use of shallow strip/pad footings is the preferred foundation alternative to support the pier of the replacement underpass and is a suitable option for the abutment foundations.

#### **8.7.2.1 Foundation Subgrade Preparation**

As per Ontario Provincial Standard Drawing (OPSD) 3090.101 (Foundation Frost Depths for Southern Ontario), the frost penetration depth in the area is 1.4 m. Therefore, the footings should be provided with a minimum of 1.4 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration.

Strip/pad footings that support the pier and/or abutments of the replacement underpass should be founded on the undisturbed, very dense portion of the till deposits. The central pier of the new structure would be located in the area of the existing highway median. In preparation for construction of the new underpass foundations, all existing fill materials, existing infrastructure and any loose, wet, and/or otherwise disturbed native material should be removed from within the footprint of the proposed footing and any other settlement sensitive areas.

Following completion of the preparation of the founding surface, a milestone inspection should be conducted by foundation/geotechnical personnel arranged for by the Contract Administrator in accordance with SP109S12.

The subgrade soils are susceptible to disturbance and degradation on exposure to water and construction traffic. It is recommended that a 100 mm thick concrete working slab be placed immediately following inspection and approval of the subgrade, to protect the subgrade from softening.

#### **8.7.2.2 Geotechnical Resistances and Reactions**

For preliminary design purposes, the factored geotechnical resistances at ULS<sub>r</sub> and geotechnical reactions at SLS provided in Tables 8.6 to 8.8 below may be used in the design of strip/pad footings for the central pier and north abutment foundations for the replacement underpass structure that are founded on the undisturbed glacial till deposits. The design values provided for the central pier could also be used for preliminary assessment of a shallow foundation system at the south abutment; however, additional investigation would be required to confirm the depth to the till bearing stratum and the associated design parameters prior to the detailed design stage. The foundations should be constructed on properly prepared subgrade soils at the founding elevations indicated in the tables.



**Table 8.6: Geotechnical Resistance & Reaction - Shallow Foundations - North Abutment**

Founding Element	Founding Elevation / Subgrade Material Type	Footing Width (m)	Factored Geotechnical Resistance at ULS <sub>f</sub> (kPa) $\phi_{gu} = 0.5$	Factored Geotechnical Reaction at SLS <sub>f</sub> (kPa) $\phi_{gs} = 0.8$
Strip/Pad Footings	144.5 m or below / Very Dense TILL	3.5 to 4	400	450

**Table 8.7: Geotechnical Resistance & Reaction - Shallow Foundations – Central Pier**

Founding Element	Founding Elevation / Subgrade Material Type	Footing Width (m)	Factored Geotechnical Resistance at ULS <sub>f</sub> (kPa) $\phi_{gu} = 0.5$	Factored Geotechnical Reaction at SLS <sub>f</sub> (kPa) $\phi_{gs} = 0.8$
	Above 138.4 m / Clayey Silt TILL (CL-ML)		Not suitable as bearing stratum	
Strip/Pad Footings	138.4 m or below / Very Dense TILL	2.5 to 4	800	450

**Table 8.8: Geotechnical Resistance & Reaction - Shallow Foundations - South Abutment**

Founding Element	Founding Elevation / Subgrade Material Type	Footing Width (m)	Factored Geotechnical Resistance at ULS <sub>f</sub> (kPa) $\phi_{gu} = 0.5$	Factored Geotechnical Reaction at SLS <sub>f</sub> (kPa) $\phi_{gs} = 0.8$
	Above 138 m / Clayey Silt TILL (CL-ML)		Not suitable as bearing stratum	
Strip/Pad Footings	138 m or below / Very Dense TILL	2.5 to 4	800	450

Notes:

1. The factored geotechnical resistance at ULS for the north abutment has been calculated assuming a 2H:1V slope adjacent to/in front of the footing.
2. ULS resistances and foundation settlement are dependent on the footing size, configuration and applied loads. The geotechnical resistances and reactions are applicable to the assumed founding elevations and range of footing widths identified in the above table and should be re-evaluated if founding elevations or sizes are different than assumed.
3. The clayey silt till (CL-ML) material encountered above the very dense silty sand/sandy gravel till in BH20-1 is not a suitable bearing stratum. The proposed founding elevation for the south abutment is approximately 8 m below the Nagle Road grade at that location. The continuity/lateral extension of the CL-ML material and the associated founding elevation for shallow footings at the south abutment will need to be evaluated after further foundation investigation during the detailed design stage.

In accordance with Table 6.1 in the CHBDC, the ULS<sub>f</sub> and SLS<sub>f</sub> Geotechnical Resistances were determined based on a consequence level of “Typical” with a consequence factor equal to 1.

In accordance with Table 6.2 of Section 6.9.1 in the CHBDC and the site and prediction model understanding classification of “Typical”, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS<sub>f</sub>).

In accordance with Table 6.2 of Section 6.9.1 in the CHBDC and the consequence and site understanding classification of “Typical”, a resistance factor of 0.8 has been applied in calculating the geotechnical reaction at Serviceability Limit State (SLS<sub>f</sub>) corresponding to a maximum settlement of 25 mm.



The geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable inclination of the load should be considered in accordance with Section 6.10.2 of the *Canadian Highway Bridge Design Code (CHBDC 2014) and its Commentary*. The structural engineer must verify that the selected footing widths are sufficient to resist overturning.

The preliminary geotechnical resistance and reaction values provided above must be re-evaluated and modified as necessary during detailed design based on the actual footing sizes.

### 8.7.3 Driven Pile Foundations – Bridge Abutments

#### 8.7.3.1 Design Considerations

Pile foundations consisting of steel H-piles that are driven to effective refusal in the very dense till deposits, and that derive the majority of their capacity from end-bearing, can be used to support the integral abutments of the proposed replacement bridge. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their design orientation due to the presence of cobbles and/or boulders within till deposits. Therefore, H-piles are recommended for use at this site.

Given the variability in the composition of the till deposits, the depth of pile penetration into the till is expected to vary; Effective refusal could be encountered at shallower depth within the very dense portions of the till deposits particularly if cobbles and/or boulders are encountered. The use of integral abutments requires relatively flexible piles with little horizontal constraint for a depth of 3 m below the pile cap level; the piles are installed within a sand-filled CSP liner in this zone. The piles would need to be installed a further approximately 5 m, beyond which lateral load transfers would no longer be anticipated. Therefore, for preliminary design purposes, predrilling is recommended to be carried out down to a level approximately 8 m below the pile caps at all abutment pile locations in order to obtain sufficient pile embedment/satisfy the minimum pile length requirements to obtain the condition of pile fixity. Piles are anticipated to need to be driven several meters into the very dense till below the base of the pre-drilled zone in order to be driven to effective refusal. If the predrilling method results in removal of soil in the predrilled zone (rather than the existing soils being left in place), the section of the pre-drilled holes below the bottom of the CSP liners should be filled with sand prior to pile installation.

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS.PROV 903 – Construction Specification for Deep Foundations.

#### 8.7.3.2 Geotechnical Axial Resistance

A factored geotechnical resistance at Ultimate Limit States ( $ULS_f$ ) of 1,400 kN may be used in the design of HP 310x110 piles driven to practical refusal within the very dense glacial till.

The estimated geotechnical reaction at  $SLS_f$  (factored) for 25 mm of vertical settlement for a HP 310x110 pile driven to effective refusal in the till exceeds the geotechnical reaction at  $ULS_f$  (factored). Therefore, the  $ULS_f$  (factored) resistance will govern.

#### 8.7.3.3 Downdrag and Relaxation of Piles

The proposed bridge replacement structure will be constructed in the same location as the existing bridge. The existing highway embankments are anticipated to be widened and raised approximately 1 m (north side) to 2 m



(south side) above existing site grades. However, as the native site soils underlying the abutment locations consist predominantly of very dense till deposits, the proposed grade raises are not anticipated to result in substantial settlement of the site soils. Based on these conditions, the piles are not anticipated to be subjected to significant downdrag loads.

Relaxation and reduction of pile capacity will not be of concern for H-piles that are driven to the refusal within the very dense till deposits.

#### **8.7.3.4 Pile Driving and Capacity Testing Considerations**

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS.PROV 903 – Construction Specification for Deep Foundations.

The site soils generally consist of embankment fill over dense to very dense till deposits containing cobbles and boulders. Pre-drilling should be carried out to the depths described previously and the piles should be provided with driving shoes such as Titus “H” Bearing Pile Point (Standard Model) or equivalent to facilitate penetration into/through the dense overburden and reduce the potential for damage to the piles during driving.

The capacity of each pile should be verified in the field by the use of either the Hiley Formula (MTO Standard Structural Drawing SS-103-11) or high-strain dynamic testing (i.e. Pile Driving Analyzer (PDA) testing) to confirm that the specified ultimate capacity is achieved.

#### **8.7.4 Geotechnical Horizontal Resistance (Sliding)**

The unfactored horizontal resistance to sliding of the strip footings for the piers and/or abutments may be calculated using the following unfactored coefficient of friction:

- 0.55      between OPSS Granular A and cast-in-place concrete
- 0.45      between silty sand/sandy silt/silty sandy gravel till and cast-in-place concrete

In accordance with Table 6.2 of the CHBDC and the consequence and site understanding classification of “Typical”, a resistance factor against sliding of 0.8 (frictional) should be applied to obtain the resistance at ULS<sub>r</sub>.

### **8.8 LATERAL EARTH PRESSURES**

#### **8.8.1 Abutment Backfill**

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge/underpass abutments. The materials used as backfill behind the abutments of the replacement underpass structure should consist of free-draining granular fill placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this report, it is assumed that backfill materials meeting the requirements of OPSS Granular B (Type I or Type II) or Granular A materials will be used.

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures. Backfill materials should be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.





## 8.8.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and retained soil systems (if any). Computation of earth pressures should be in accordance with Section 6.17.3 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

The total at rest ( $P_O$ ), active ( $P_A$ ), and passive ( $P_P$ ) thrusts can be calculated using the following equations:

$$P_O = \frac{1}{2} K_o \gamma H^2$$

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

where  $H$  is the height of the wall and  $\gamma$  is the unit weight of the backfill soil. Values for  $K_a$ ,  $K_p$ ,  $K_o$  and  $\gamma$  are provided in Tables 8.7 and 8.8 for horizontal and sloping (2H:1V) backfill conditions, respectively. For the purposes of preliminary design, a friction angle of 30 degrees has been assumed for the existing embankment fill materials at the site; this value will need to be confirmed and/or reassessed once further subsurface investigation is completed prior to detailed design. The thrust acts at a point one third up the height of the wall.

**Table 8.9: Recommended Static Earth Pressure Parameters (Horizontal Backfill)**

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21	22	21
Effective Friction Angle	32°	35°	30°
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.47	0.43	0.50
Coefficient of Active Earth Pressure ( $K_a$ )	0.31	0.27	0.33
Coefficient of Passive Earth Pressure ( $K_p$ )	3.25	3.69	3.00

**Table 8.10: Recommended Static Earth Pressure Parameters (2H:1V Backfill)**

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21	22	21
Effective Friction Angle	32°	35°	30°
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.68	0.62	0.72
Coefficient of Active Earth Pressure ( $K_a$ )	0.47	0.39	0.54



### 8.8.3 Seismic Lateral Earth Pressures

The following design parameters are provided for use in assessing the earth pressures induced on the bridge abutment and wingwalls under seismic loading conditions.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where:

- $K_{AE}$  = active earth pressure coefficient (combined static and seismic)
- $K_{PE}$  = passive earth pressure coefficient (combined static and seismic)
- $H$  = height of wall
- $k_h$  = horizontal acceleration coefficient
- $k_v$  = vertical acceleration coefficient
- $\gamma$  = total unit weight

The seismic earth pressures for structures with horizontal backfill behind the walls may be calculated using the parameters provided in Table 8.9. Table 8.10 provides seismic earth pressures for yielding walls with 2H:1V backfill slopes behind the walls.

For this site, the following design parameters were used to develop the recommended  $K_{AE}$  and  $K_{PE}$  values as per CHBDC 2014.

**Table 8.11: Seismic Design Parameters to Estimate Lateral Earth Pressures**

Site Adjusted <i>PGA</i>	Horizontal Acceleration Coefficient, $k_{ho}$	Horizontal Acceleration Coefficient, $k_h$
	Non-Yielding	Yielding ( <i>wall movements of 25 mm to 50 mm</i> )
0.11g	0.11	0.055
Note: $k_{ho}$ is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the site-adjusted <i>PGA</i> estimated at ground surface. The vertical acceleration coefficient ( $k_v$ ) should be ignored in the calculations as per CHBDC 2014, section C4.6.5.		

As noted above, a friction angle of 30 degrees has been assumed for the existing embankment fill materials at the site for the purposes of preliminary design; this value will need to be confirmed and/or reassessed once further subsurface investigation is completed prior to detailed design.

The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.



**Table 8.12: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)**

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21	22	21
Effective Friction Angle	32°	35°	30°
Passive Earth Pressure, ( $K_{PE}$ )	3.1	3.6	2.9
Height of Application of $P_{PE}$ from base as a ratio of wall height, (H)	0.325	0.325	0.325
<b>Yielding Wall</b>			
Active Earth Pressure ( $K_{AE}$ ) for Yielding Wall	0.34	0.30	0.37
Height of Application of $P_{AE}$ from base as a ratio of wall height, (H) for Yielding Wall	0.36	0.36	0.36
<b>Non-Yielding Wall</b>			
Active Earth Pressure ( $K_{AE}$ ) for Non-Yielding Wall	0.37	0.33	0.40
Height of Application of $P_{AE}$ from base as a ratio of wall height, (H) for Non-Yielding Wall	0.38	0.38	0.38

**Table 8.13: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)**

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, $\gamma$ (kN/m <sup>3</sup> )	21	22	21
Effective Friction Angle	32°	35°	30°
<b>Yielding Wall</b>			
Active Earth Pressure ( $K_{AE}$ ) for Yielding Wall	0.58	0.47	0.72
Height of Application of $P_{AE}$ from base as a ratio of wall height, (H) for Yielding Wall	0.38	0.38	0.40

## 8.9 EMBANKMENT STABILITY AND SETTLEMENT

### 8.9.1 Stability of Approach Embankments

The existing Nagle Road Underpass north and south approach embankments are approximately 3 m and 6 m high (above original grades), respectively, have crest-to-crest widths of approximately 12 m and have sideslopes of approximately 2H:1V.

The approach embankments are proposed to be raised slightly (i.e. by approximately 1 m at the north abutment and about 2 m at the south abutment). The embankments are planned to be widened by approximately 5 m to 6 m on both sides of the existing embankment to accommodate the wider bridge structure.

Based on the existing boreholes advanced at the site, the native subgrade materials present beneath the approach embankments are expected to consist of predominantly of very dense (compact to dense near ground surface) glacial till deposits that would provide adequate subgrade support for the proposed widened/raised embankments.



Furthermore, the existing embankments do not exhibit signs of instability or other distress. Based on these conditions, no issues with embankment instability are expected to occur for the slightly higher approach embankments proposed for the new bridge structure provided that the alterations to the existing embankments are constructed with conventional 2H:1V sideslopes using engineered fill materials that are placed and compacted in accordance with applicable Ontario Provincial Standard Specification requirements.

Any embankment widening associated with the grade raise should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

### **8.9.2 Embankment Settlement**

The existing approach embankments do not exhibit signs of significant settlement and the native subgrade materials present beneath the approach embankments consist of compact to very dense glacial till deposits that are not highly compressible.

Based on the above conditions, no issues with embankment settlement are expected to occur as a result of the minor raising and widening of the approach embankments required for the underpass reconstruction provided that all surficial topsoil and/or organic materials are removed from beneath areas of new fill placement and that the existing embankments are constructed using engineered fill materials that are placed and compacted in accordance with applicable Ontario Provincial Standard Specification requirements.

## **8.10 CEMENT TYPE AND CORROSION POTENTIAL**

One soil sample from each borehole location were submitted to Paracel Laboratories for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results are summarized in Table 5.1.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations the samples tested were 38 and 22 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH of the samples tested were 7.9 and 7.7 which is within the normal pH range for soil (5.5 to 9.0). However, reported resistivity values of 15.5 and 70.3 (ohm-m) suggest a high and moderate degree of corrosiveness for steel.

The test results provided in Table 6.1 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.



## 9.0 CONSTRUCTION CONSIDERATIONS

### 9.1 CONSTRUCTION STAGING AND DETOUR

A local detour is anticipated to be required for the construction of the new underpass structure as Nagle Road is planned to be closed to traffic during the construction of the new bridge.

The construction of the foundations for the new central pier of the bridge is anticipated to involve staging and/or shifting of traffic lanes away from the Highway 401 median using appropriate traffic control. The use of a temporary roadway protection system may be required near the centerline of existing Highway 401 to permit the foundation construction.

### 9.2 TEMPORARY ROADWAY PROTECTION

Temporary roadway protection may be required to protect traffic on Highway 401 or maintain traffic on Nagle Road during excavations for the foundations of the replacement underpass structure.

The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters.

The following table compares the available roadway protection options considered for the proposed rehabilitation:

**Table 9.1: Comparison of Roadway Protection Systems**

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Soldier piles with timber lagging; (struts/rakers as required)	<ul style="list-style-type: none"> <li>Simple installation process</li> </ul>	<ul style="list-style-type: none"> <li>Additional labour required</li> <li>Groundwater seepage into the excavation can occur without groundwater control</li> <li>Removal of soldier piles can be difficult</li> </ul>	Low	<ul style="list-style-type: none"> <li>Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented</li> <li>Potential for minor loss of ground at rear of lagging</li> </ul>
Steel sheet piles (SSP)	<ul style="list-style-type: none"> <li>Simple installation process</li> <li>Provides cut-off to groundwater seepage from sides of excavation</li> <li>Can be incorporated with groundwater cut-off system for excavation for removal of weak soils at west abutment of TMB</li> </ul>	<ul style="list-style-type: none"> <li>Difficult to drive/install in hard or very dense till particularly where cobbles/boulders are present</li> <li>May require large sections where cantilever design is adopted</li> </ul>	Medium	<ul style="list-style-type: none"> <li>Potential for sheet piles to either be damaged, deflected or meet refusal due to obstructions (e.g. boulders within the till) during driving</li> </ul>

Due to the potential difficulties installing sheet piles into the native ground conditions (i.e. very dense glacial till containing cobbles and boulders) at the site, the use of a soldier pile and lagging protection system is considered to be the more viable option for this site. The soldier pile and lagging walls should be supported with struts or rakers from the construction side or tie-backs/ground anchors.



Roadway protection design should meet the requirements of Performance Level 2 in accordance with OPSS.PROV 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Strut, raker, or tie-back design, if and as required, must be designed not to exceed these limits. Horizontal movement of the temporary roadway protection system should be monitored throughout the bridge replacement process as described in OPSS.PROV 539.

## 9.3 EXCAVATION AND BACKFILLING

Excavation and backfilling for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures.

Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath proposed the bridge foundations and any associated retaining/wing walls. Where deleterious materials are encountered at foundation subgrade level, the materials should be excavated, removed and replaced with compacted granular fill materials. The lateral extent of the zone of subexcavation (and replacement) should include all deleterious material within the influence zone of the above foundation elements.

Grading work should be carried out in accordance with OPSS.PROV 206 Construction Specification for Grading and SP 206S03. Where existing embankments are to be widened, the new fill materials should be benched into the existing embankments in accordance with OPSD 208.010.

All side slopes for open cut excavations should conform to the Occupational Health and Safety Act regulations for Construction Projects (OHSA). The excavations required for construction of the new pier and abutment foundations would extend to several meters depth and be developed through the existing highway and Nagle Road approach embankment fill. These excavations are expected to encounter fill materials and the native, the compact to very dense till deposits. Where space permits, these excavations may be developed using open-cut methods. The dense to very dense till would be classified as Type 2 soil while the compact till and the fill materials above the water table would be classified as Type 3 soils.

OHSA indicates that temporary excavations made within Type 3 soils that are above the water table and/or dewatered prior to excavation should be developed with side slopes no steeper than 1H:1V. Granular soils (fill materials and/or native overburden) below the water table, if encountered, would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements.

## 9.4 TEMPORARY GROUNDWATER CONTROL

The groundwater level was observed at approximately elevation 138.2 m at the time of the investigation. As the highway was developed within a cut, it is anticipated that groundwater levels may rise to near ground surface within the highway corridor and temporary unwatering may be needed for the excavation for the central pier.

The Highway 401 grade in the area of the underpass is at an elevation of approximately 140 m and the available General Arrangement drawing for the bridge identifies a founding elevation of approximately 138 m for the central pier of the new underpass.

For excavations that are developed within glacial till soils and extend slightly (i.e. less than 1 m to 2m) below the water table, temporary unwatering using conventional sump and pump techniques is generally considered appropriate.



The requirements for unwatering/dewatering should be further reassessed during the detailed design stage once additional information on the site soils at the central pier location is available.

All groundwater control systems required for the construction of the replacement bridge should be designed and implemented in accordance with NSSP FOUN0003.

## 9.5 EXISTING FILL MATERIALS

The results of the investigation have identified the presence of loose sandy fill materials with a thickness of about 3 m in the area of the north abutment. Additionally, fill materials of unknown composition are present within the approach embankment for the south abutment which has a height of about 6 m above original site grades.

There are no construction records identifying the type, source and/or level of compaction of the existing fill. Due to this lack of information, as well as historic highway construction practices involving the use of differing materials in the core and sideslopes of embankments, existing fill materials could pose design and/or construction challenges (e.g. difficult excavation conditions, need for removal of fill containing organic or debris materials etc.). Sufficient investigation of the existing fills should be carried out during the detailed design stage to identify the type of fills and assess these issues.

## 9.6 EXISTING SOUTH ABUTMENT PILES

The south abutment of the existing bridge is supported on steel H-piles including battered piles inclined towards the existing highway. As the south abutment of the new bridge will be located to the south of the existing abutment, these piles are not expected to interfere with the construction of the south abutment foundation. However, the battered piles are expected to be encountered within the excavation zone (i.e. within the cut slope) for the widened highway corridor and may extend near to the asphalt-surfaced shoulder of the ultimate widened highway configuration. Piles extending into this area should either be extracted or cut-off a minimum of 1.0 m below the pavement subgrade level.

## 9.7 OBSTRUCTIONS

Cobbles and/or boulders are present in the till deposits at this site. These materials could obstruct excavations and the installation of pile foundations and temporary roadway protections systems. In addition, the existing pile foundations supporting the south abutment of the existing bridge will also obstruct the excavation for the widening of the highway. A Non-Standard Special Provision (NSSP) should be developed during the detailed design stage for inclusion in the contract to address this issue.



## 10.0 FURTHER WORK FOR DETAILED DESIGN

Based on the subsurface conditions encountered in the current investigation, driven pile foundations at the abutments and shallow foundations at the central pier are the preferred foundation types to be used in the preliminary design of the underpass replacement at this site.

The following foundation engineering related items should be completed prior to, or as part of, the detailed design to confirm and/or further assess the preliminary recommendations provided in this report:

- Additional subsurface investigation, and associated laboratory testing, should be completed for the bridge structure. The standard minimum MTO foundation investigation for a bridge structure (i.e. two boreholes at each foundation unit advanced to 3 m below refusal, defined as material for which SPT 'N' values are greater than 100 blows per 0.3 m of penetration) is considered appropriate provided that shallow foundations are to be used for the new structures. Based on the available information, the above noted MTO refusal criteria is anticipated to be encountered at relatively shallow depth at the Nagle Road site. Therefore, if deep foundations are to be considered, the boreholes would need to be advanced beyond the typical borehole refusal depth in order to provide information over the length and influence zone of the deep foundation units.
- One borehole within 20 m of the new bridge abutments in the area of each approach embankment.
- 'Soft' soils were not encountered in the borehole advanced near the south abutment. However, as the south abutment of the existing bridge is supported on piles, two boreholes should also be advanced near the toe of the higher/wider approach embankments to determine if softer/compressible soils are present in those areas.
- Boreholes should be advanced through the existing approach embankments to determine the type, thickness and consistency/density of the existing fill materials and their potential impact on the design of the new bridge.
- Additional boreholes should be also advanced as per MTO Standards for any retaining walls or temporary roadway protection systems required for construction staging purposes.
- Piezometers/monitoring wells should be installed in critical areas (e.g. boreholes located within the excavation areas for planned foundation units which will be shifted laterally away from existing foundations due to proposed widening of the highway, on the north side of the highway where the highway corridor was developed in a cut etc.) to permit determination of the stabilized water level. Additionally, consideration should be given to installing dataloggers within the piezometers/wells to permit collection of seasonal groundwater monitoring data.
- Additional gradation analyses of the site soils and/or in situ permeability testing in wells installed in excavation areas should be completed to permit evaluation of the unwatering/dewatering requirements.
- For a new interchange, ramps located on the north side of the highway would require deep excavations that would pass through the newly cut slope or disturbed area located west of the existing structure, adjacent to the westbound lanes. The future foundation investigation for the interchange should include sufficient boreholes along ramp alignments to meet minimum MTO Foundation investigation requirements for 'deep cuts'.
- Following completion of the additional investigation and laboratory testing, the soil design parameters outlined in this report should be re-evaluated and a detailed assessment of the potential for differential settlement be undertaken if differing foundation types (i.e. shallow and deep foundations) are planned to be used.
- A Final Foundation Investigation and Design Report meeting MTO's standard requirements for foundation engineering assignments should be prepared based on the final structure configuration.





## 11.0 SPECIFICATIONS

The following specifications are referenced in this report:

**Table 11.1: Specifications Referenced in Report**

Document	Title
NSSP FOUN0003	Dewatering Structure Excavations
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls, abutment, backfill – Minimum Granular Requirements
OPSS 206	Earth Excavation, Grading
OPSS 501	OPSS 501 Construction Specification for Compaction
OPSS 902	Construction Specification for Excavation and Backfilling – Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 501	Construction Specification for Compacting
OPSS. PROV 539	Construction Specification for Temporary Protection System
OPSS.PROV 1010	Material Specification for Aggregates
SP517F01	Amendment to OPSS 517, July 2017
SP105S10	Construction Specification for Compaction
SP109S12	Amendment to OPSS 902, November 2010
SP 206S03	Earth Excavation, Grading



## 12.0 CLOSURE

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered by others at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

This report was prepared by Ramin Ghassemi, Ph. D, P.Eng. and reviewed by Kevin Nelson, P.Eng., and Raymond Haché, M.Sc., P.Eng., Designated Principal MTO Foundation Contact.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

**STANTEC CONSULTING LTD.**



Ramin Ghassemi, P.Eng., Ph.D.  
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Principal, Senior Geotechnical Engineer



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Designated Principal MTO Foundations Contact



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## 13.0 REFERENCES

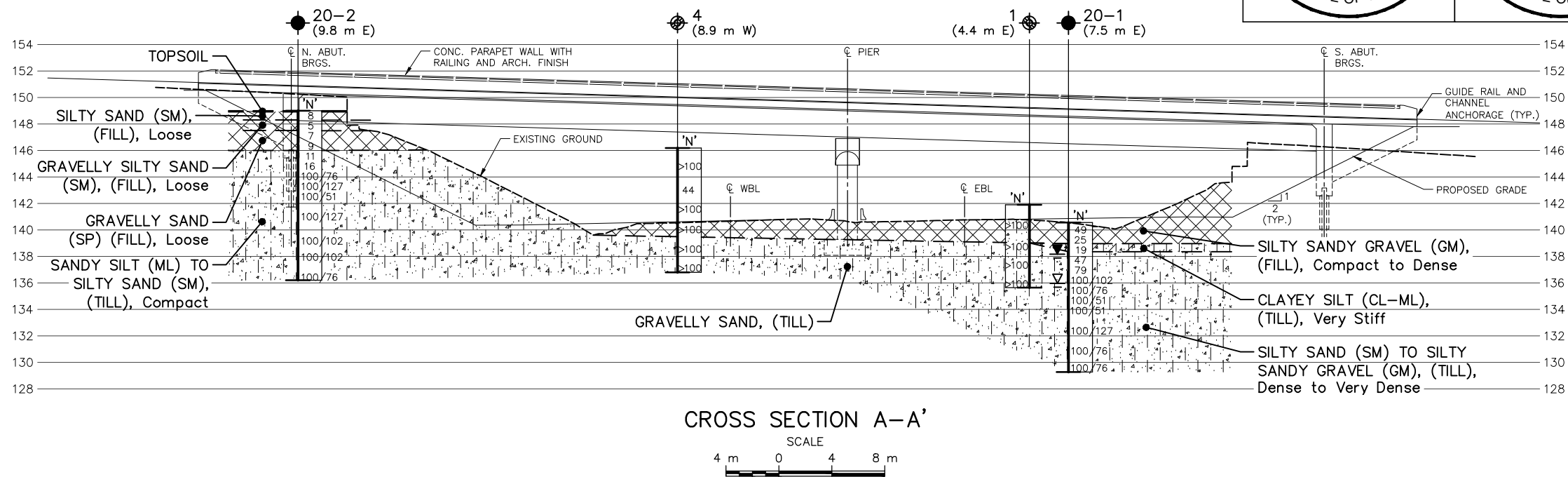
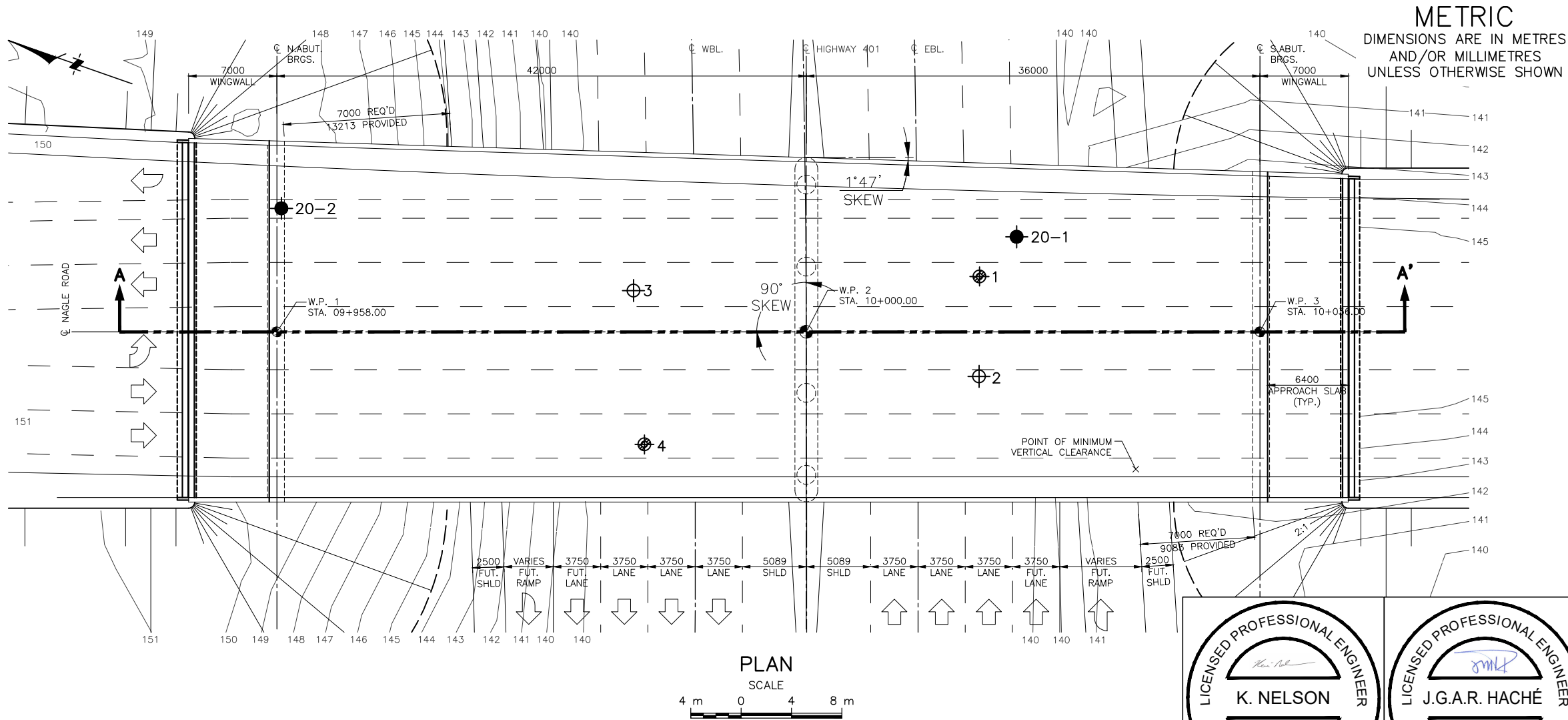
- A.M. Lount and Associates. 1958. Bridge Design Drawing. Hamilton Twp. Bridge #13 Highway No. 401 - Drawing No. D-4128-2 – Layout of Footings and Bore Holes
- Chapman, L.J. and D.F. Putnam. 1984. The Physiography of Southern Ontario, Ontario Geologic Survey
- CHBDC. 2014. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.
- NBC. 2015. National Building Code of Canada Vol.1. National Research Council of Canada, Ottawa, Ontario.
- OHSA. 2015. Occupational Health and Safety Act Regulations for Construction Projects. Carswell, Toronto Ontario
- Ontario – Department of Highways. 1958. Foundation Report. New Bridge at Highway 401. Crossing road between Lots 28 and 29 (Con. I), about two miles North West of Grafton, Haldimand Township, W.P. 90-57, W.J. F58-36. (GEOCRETS Reference No. 30M16-022)
- Ontario Geological Survey. 2010. Surficial Geology of Southern Ontario GIS data set.



## Appendix A

### **A.1 DRAWING NO. 1 – BOREHOLE LOCATION PLAN AND SOIL STRATA PLOT**





METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

PLATE No  
**CONT**  
**GWP 4059-17-00**

HIGHWAY 401  
NAGLE ROAD  
BOREHOLE LOCATIONS & SOIL STRATA

**Stantec**

**KEY PLAN**  
1 km 0 1 2 km

LEGEND				
	Borehole (Stantec, 2020)			
	Borehole (MTO, 1958)			
	Dynamic Cone Penetration Test (MTO, 1958)			
(x.x m)	Offset from Cross Section Line in meters			
N	Blows/0.3m (Std Pen Test, 475 J/blow)			
	WL at time of investigation May 2020			
	WL Measured on May 2020			
No	ELEVATION	MTM ZONE 10 NORTH	COORDINATES EAST	
20-1	140.6	4 873 498.6	413 530.6	
20-2	149.0	4 873 554.4	413 513.4	
1	145.3	4 873 500.4	413 526.7	
2	145.4	4 873 497.8	413 519.2	
3	145.6	4 873 525.9	413 516.5	
4	145.7	4 873 521.0	413 505.3	

**NOTES**

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

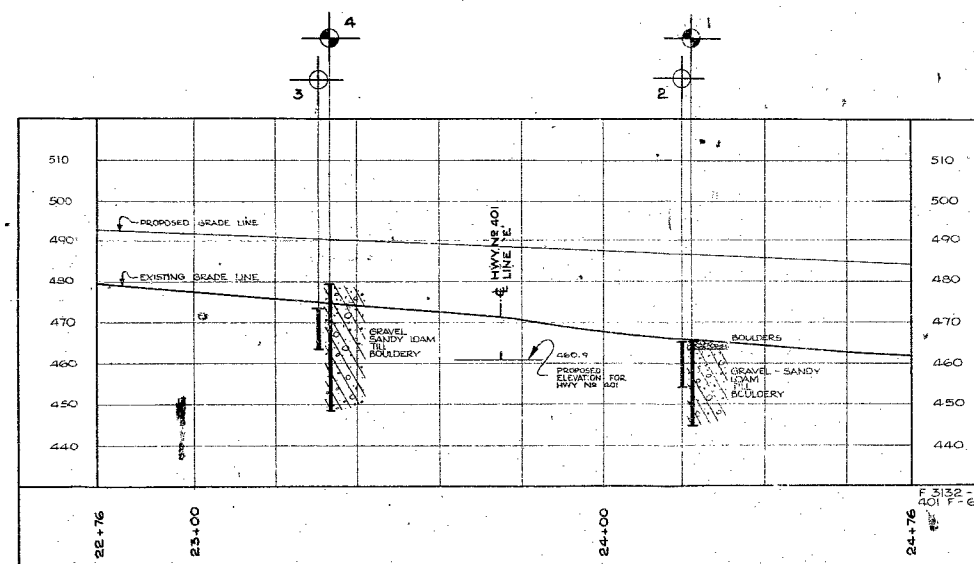
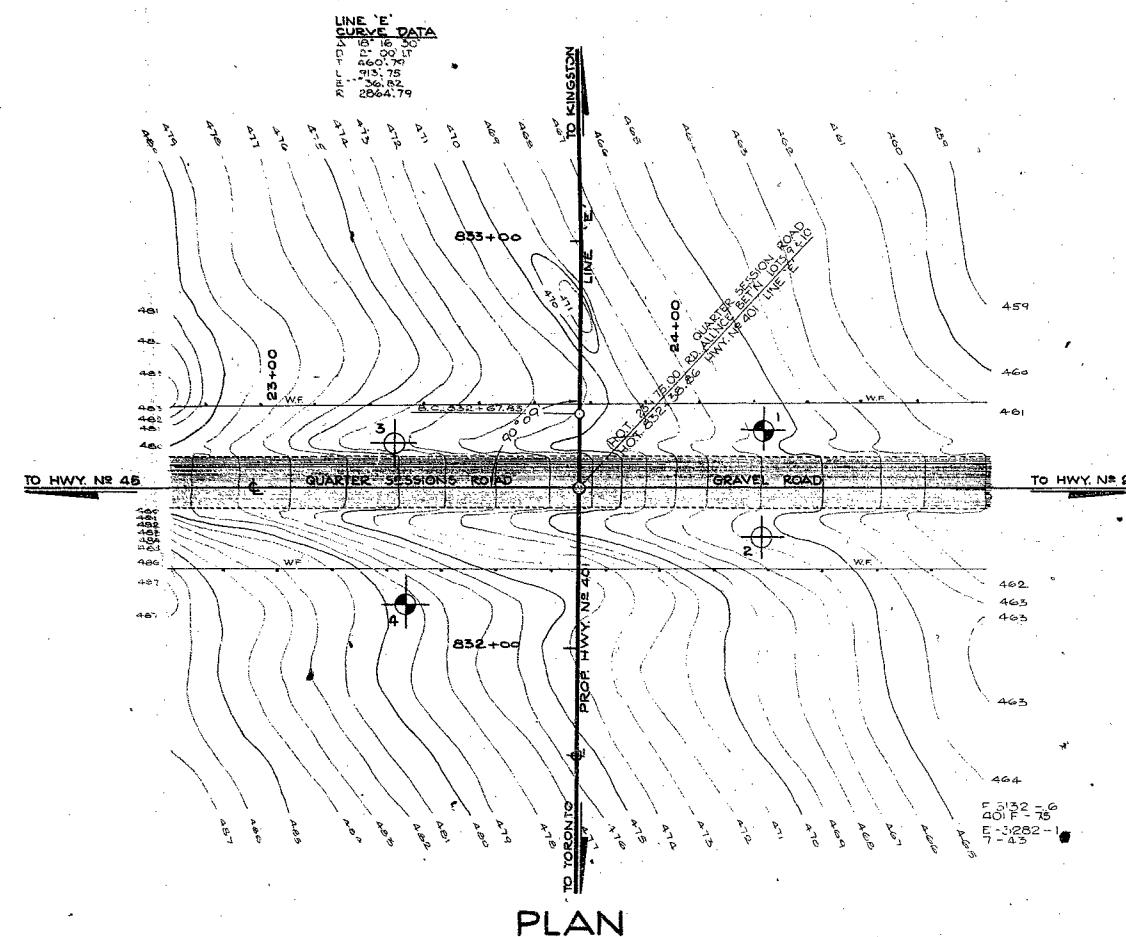
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REVISIONS			DATE		BY	DESCRIPTION
GEOCRETS No			30M16-72			
HWY No 401			DIST			
SUBM'D KN			CHECKED	DATE 2021-04-16	SITE 21X-0248/B0	DWG 1
DRAWN GGB			CHECKED	APPROVED		

## Appendix B

### **B.1 AVAILABLE GEOCRETS INFORMATION INCLUDING SOIL STRATA PLOT AND BOREHOLE RECORDS**





LEGEND			
BORE HOLE			
PENETRATION HOLE			
BORE & PENETRATION HOLE			
HOLE NO.	ELEVATION	STATION	DISTANCE FROM Q.
1	465.5'	832+54'	45' RT.
2	465.5'	832+28'	45' RT.
3	473.5'	832+50'	45' LT.
4	479.5'	832+10'	42' LT.

NOTE  
 THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS - ONTARIO		
MATERIALS & RESEARCH SECTION - DOWNSVIEW		
<b>GRAVEL ROAD          PROPOSED CROSSING          2 MILES N.E. OF COBourg</b> SHOWING POSITION & ELEVATION OF HOLES		
HWY. NR. 401	W.P. 88-57	DIV. NR. 7
CO. NORTHUMBERLAND	LOT 10	CON. 1
TWP. HAMILTON		
SCALE 1 IN. = 20 FT.	SUBMITTED BY	DATE 3 MARCH 58
DRAWN BY R.E.F.	APPROVED BY	DRAWING NO. F-57-46A

DEPARTMENT OF HIGHWAYS - ONTARIO  
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION BORE & PENET'N JOB T-57-46 W.P. 88-57 BORING L STA. 832+54 (45' RT)  
 CASING 3X (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC 1957  
 SAMPLER HAMMER WT. 250 LBS. DROP 19 1/2 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 13 NOV 1957

## ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION  
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING  
 QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL  $\gamma$  - UNIT WEIGHT

## SAMPLE TYPES

CS - CHUNK S.S. - SLEEVE SAMPLE  
 DO - DRIVE OPEN PS - PISTON SAMPLE  
 DF - DRIVE FOOT VALVE WS - WASHED SAMPLE  
 TO - THIN WALLED OPEN RC - ROCK CORE

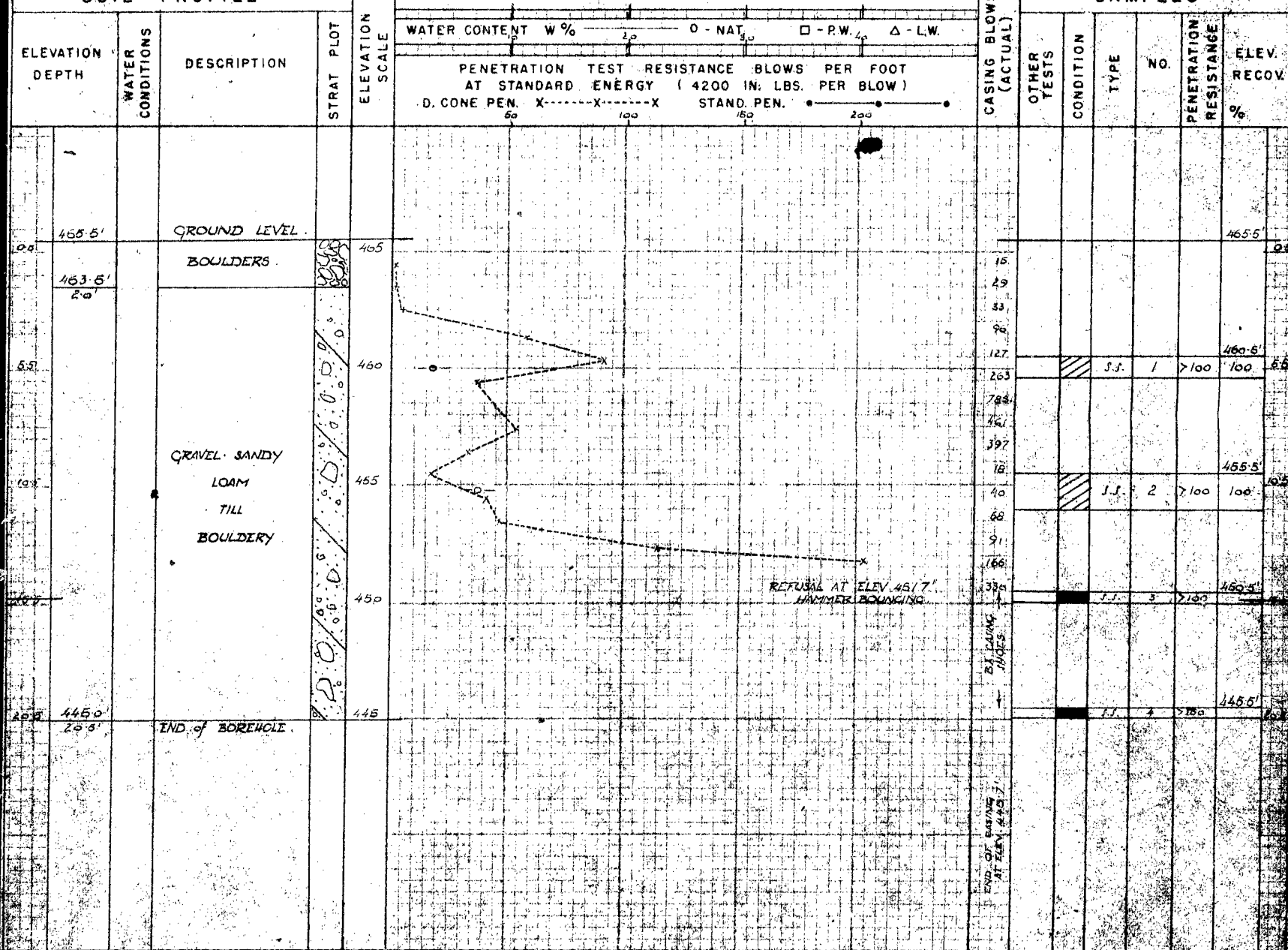
## SAMPLE CONDITION



- DISTURBED  
 - FAIR  
 - GOOD  
 - LOST

## SOIL PROFILE

## SAMPLES





DEPARTMENT OF HIGHWAYS - ONTARIO  
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION PENETRATION JOB F-57-46 W.P. 88-37 BORING 2 STA. 832+28 (45' RT)  
CASING BK (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957  
SAMPLER HAMMER WT. 250 LBS. DROP 19 1/2 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 19. NOV. 1957

**ABBREVIATIONS**

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION  
U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING  
QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL  $\gamma$  - UNIT WEIGHT

**SAMPLE TYPES**

C.S. - CHUNK S.S. - SLEEVE SAMPLE  
D.O. - DRIVE OPEN P.S. - PISTON SAMPLE  
D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE  
T.O. - THIN WALLED OPEN R.C. - ROCK CORE

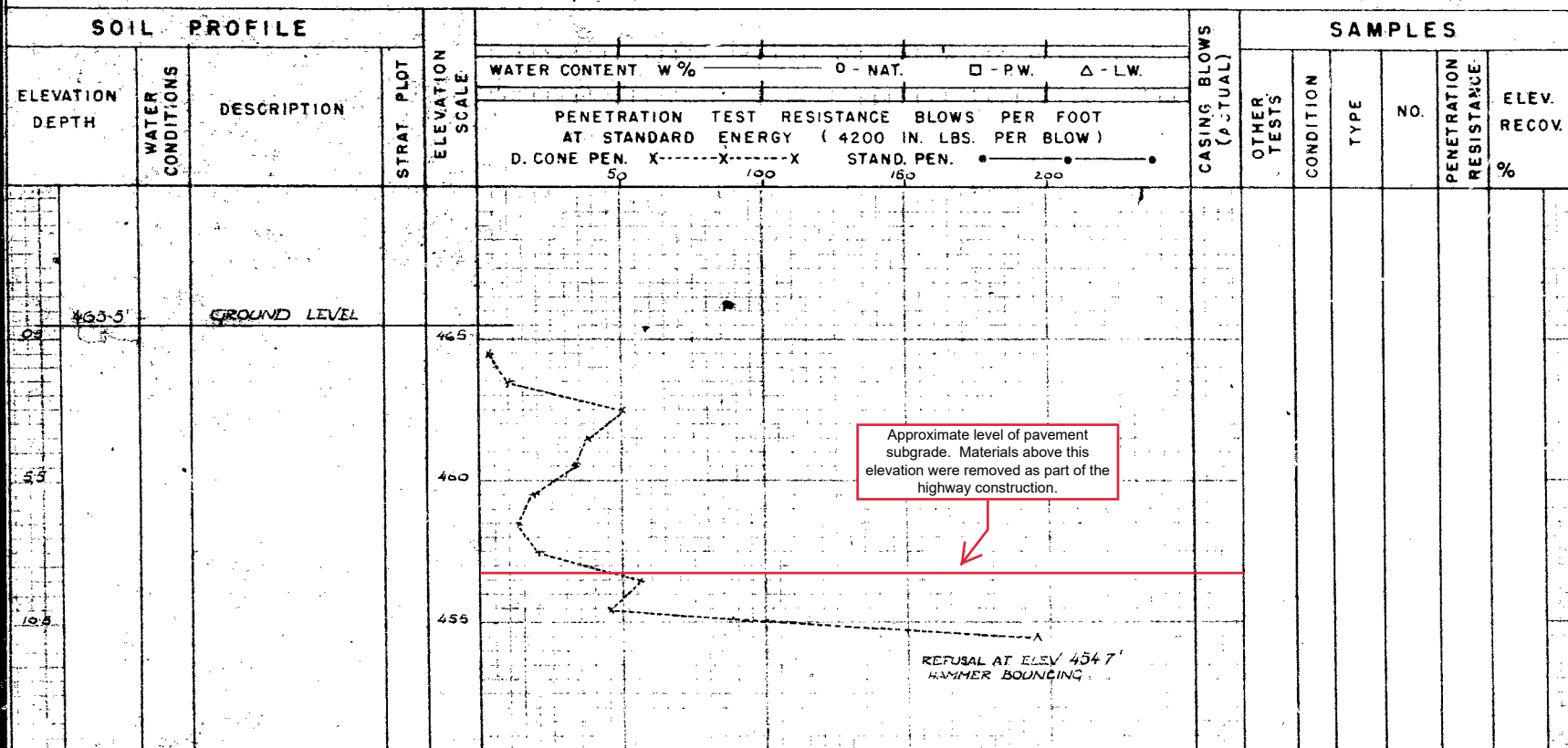
**SAMPLE CONDITION**



- DISTURBED  
- FAIR  
- GOOD  
- LOST

**SOIL PROFILE**

**SAMPLES**



DEPARTMENT OF HIGHWAYS - ONTARIO  
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION PENETRATION JOB F-57-46 W.P. 88-57 BORING 3 STA. 832.50 (45' LT)  
 CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957  
 SAMPLER HAMMER WT. 250 LBS. DROP 19 1/2 INCHES COMPILED BY HJ CHECKED BY AL DATE BORING NOV. 19 1957

## ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION  
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING  
 QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL  $\gamma$  - UNIT WEIGHT

## SAMPLE TYPES

C.S. - CHUNK S.S. - SLEEVE SAMPLE  
 D.O. - DRIVE OPEN P.S. - PISTON SAMPLE  
 D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE  
 T.O. - THIN WALLED OPEN R.C. - ROCK CORE

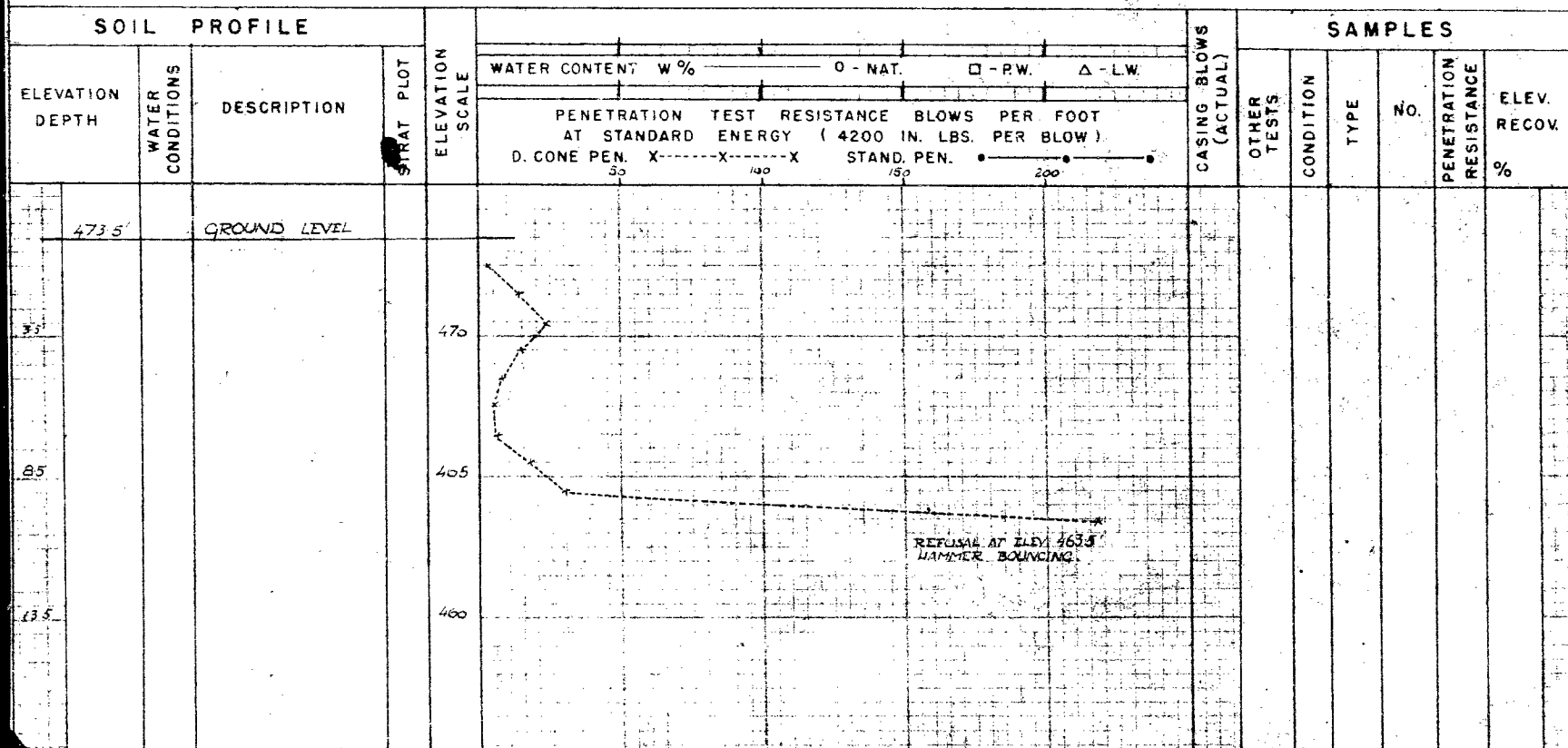
## SAMPLE CONDITION



- DISTURBED  
 - FAIR  
 - GOOD  
 - LOST

## SOIL PROFILE

## SAMPLES



DEPARTMENT OF HIGHWAYS - ONTARIO  
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW  
**OFFICE REPORT ON SOIL EXPLORATION**

DRILL RIG 54-1 OPERATION BORE & PENET'N JOB T-57-46 WP 88-57 BORING 4 STA. 832+10 (42' LT.)  
 CASING BX (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT DEC. 1957  
 SAMPLER HAMMER WT. 250 LBS. DROP 19 1/4 INCHES COMPILED BY H.S. CHECKED BY A.L. DATE BORING 21 NOV. 1957

## ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY  
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 Q<sub>c</sub> - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL  $\gamma$  - UNIT WEIGHT

## SAMPLE TYPES

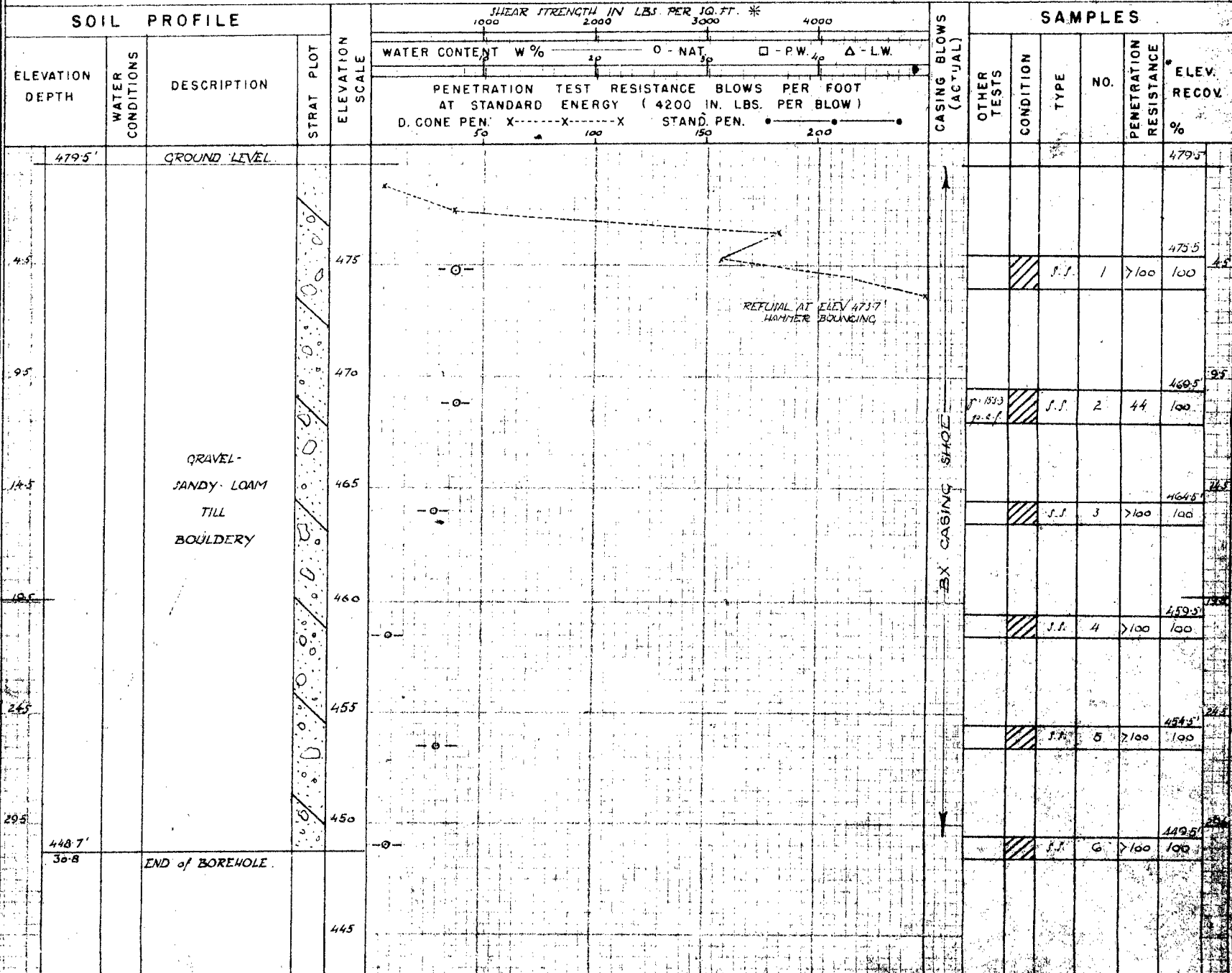
C.S. - CHUNK S.S. - SLEEVE SAMPLE  
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 T.O. - THIN WALLED OPEN R.C. - ROCK CORE

## SAMPLE CONDITION



- DISTURBED  
 - FAIR  
 - GOOD  
 - LOST

## SOIL PROFILE



## Appendix C

### **C.1 SYMBOLS AND TERMS USED ON BOREHOLE RECORDS**

### **C.2 BOREHOLE RECORDS (CURRENT INVESTIGATION)**



## SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

### SOIL DESCRIPTION

#### Terminology describing common soil genesis:

<i>Rootmat</i>	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

#### Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

#### Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4<sup>th</sup> Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

#### Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

#### Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

#### Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
<i>Very Soft</i>	<0.25	<12.5	<2
<i>Soft</i>	0.25 - 0.5	12.5 - 25	2-4
<i>Firm</i>	0.5 - 1.0	25 - 50	4-8
<i>Stiff</i>	1.0 - 2.0	50 - 100	8-15
<i>Very Stiff</i>	2.0 - 4.0	100 - 200	15-30
<i>Hard</i>	>4.0	>200	>30

## ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

### Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	Very Poor Quality
25-50	Poor Quality
50-75	Fair Quality
75-90	Good Quality
90-100	Excellent Quality

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	Crushed
Severely Fractured	Shattered or Very Blocky
Fractured	Blocky
Moderately Jointed	Sound
Intact	Very Sound

**RQD (Rock Quality Designation)** denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

**SCR (Solid Core Recovery)** denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

**Fracture Index (FI)** is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

### Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	Extremely Wide	-
2000-6000	Very Wide	Very Thick
600-2000	Wide	Thick
200-600	Moderate	Medium
60-200	Close	Thin
20-60	Very Close	Very Thin
<20	Extremely Close	Laminated
<6	-	Thinly Laminated

### Terminology describing rock strength:

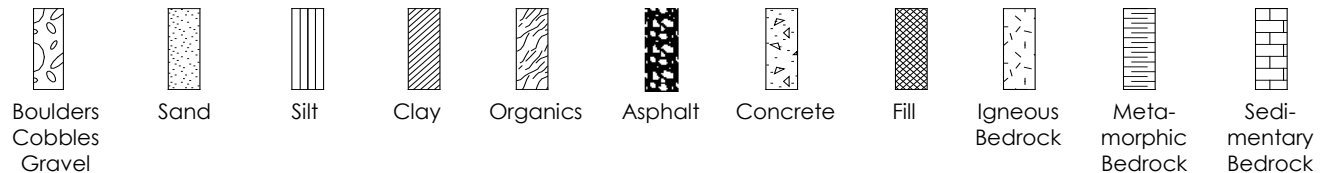
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

### Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

## STRATA PLOT

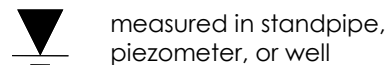
Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



## SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

## WATER LEVEL MEASUREMENT



## RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

## N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

## DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

## OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
$\gamma$	Unit weight
$G_s$	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
$Q_u$	Unconfined compression
$I_p$	Point Load Index ( $I_p$ on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

# RECORD OF BOREHOLE No BH20-1

1 OF 1

METRIC

W.P. 4059-17-00 LOCATION Highway 401 - Nagle Road Underpass ORIGINATED BY KL  
 DIST East HWY 401 BOREHOLE TYPE 150 mm SOLID STEM AUGERS COMPILED BY RG  
 DATUM Geodetic DATE 2020.04.29 LATITUDE 43.99343043 LONGITUDE -78.14437991 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE					
140.6	ASPHALTIC CONCRETE						20	40	60	80	100	20	40	60		GR SA SI CL
0.0	300 mm ASPHALTIC CONCRETE															
140.3																
0.3	Silty SANDY GRAVEL (GM) (FILL) Compact to dense Brown Moist		1	SS	49											36 33 30 1
			2	SS	25											
139.2																
1.3	CLAYEY SILT with sand (CL-ML) (TILL) Contains frequent cobbles and boulders Very stiff Brown		3	SS	19											0 45 38 17
138.4	Dry to moist															
2.2	Silty SAND (SM), trace to some clay, trace gravel to silty sandy GRAVEL (GM), trace clay (TILL) Contains frequent cobbles and boulders Dense to very dense Brown Moist Auger grinding at 3.5 m depth		4	SS	47											
			5	SS	79											8 42 33 17
			6	SS	100/ 102 mm											
			7	SS	100/ 76 mm											
	Grey below 4.6 m depth															
			8	SS	100/ 61 mm											
			9	SS	100/ 61 mm											
	Auger grinding at 6.9 m depth															
			10	SS	100/ 127 mm											51 25 17 7
	Auger grinding at 7.9 m depth															
	Auger grinding at 8.5 m depth															
			11	SS	100/ 76 mm											3 64 26 7
	SS11 contains 0.25 m thick pocket of SAND with silt, trace gravel															
	Auger grinding at 10.4 m depth															
			12	SS	100/ 76 mm											
129.3																
11.3	End of Borehole															
	After completion of drilling groundwater was observed at 2.4 m (~Elev. 138.2 m) and sloughing of borehole sidewalls occurred at 6.1 m (~Elev. 134.4 m).															

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONTARIO MTO 165001106 HWY\_401\_NAGLE\_RD.GPJ ONTARIO MTO.GDT 7/10/20



# RECORD OF BOREHOLE No BH20-2

1 OF 1

METRIC

W.P. 4059-17-00 LOCATION Highway 401 - Nagle Road Underpass ORIGINATED BY KL  
 DIST East HWY 401 BOREHOLE TYPE 200 mm HOLLOW STEM AUGERS; Wash Boring below 7.8 m. COMPILED BY RG  
 DATUM Geodetic DATE 2019.05.05 LATITUDE 43.99393486 LONGITUDE -78.14458308 CHECKED BY KN

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE							
								20 40 60 80 100							
						20 40 60 80 100			20 40 60			WATER CONTENT (%)			
149.0	TOPSOIL & GRASS														
0.1	75 mm TOPSOIL														
148.3	SILTY SAND (SM), trace clay, gravel, and rootlets (FILL)		1	SS	8										
0.7	Loose Brown Moist														
147.5	Gravelly SILTY SAND (SM), trace clay and rootlets (FILL)		2	SS	5		148								22 48 23 7
1.5	Loose Brown Moist														
	Auger grinding at 0.8 m depth		3	SS	7		147								
	GRAVELLY SAND (SP), trace silt (FILL)														
	Loose Brown Moist														
146.0	Auger grinding at 2.7 m depth		4	SS	9										
3.0	SANDY SILT (ML) to Silty SAND (SM), trace to some clay and gravel (TILL)						146								
	Contains frequent cobbles and boulders		5	SS	11										18 45 26 11
	Compact Brown Moist														
	Auger grinding at 3.7 m depth		6	SS	16		145								
	Very dense below 4.4 m depth Auger grinding from 4.7 m to 5.2 m depth		7	SS	100/ 76 mm		144								
	Auger grinding at 5.6 m depth		8	SS	100/ 127 mm										
	Auger grinding from 6.1 m to 7.6 m depth		9	SS	100/ 61 mm		143								7 42 36 15
	Auger grinding at 7.6 m depth Become grey below 7.6 m depth		10	SS	100/ 127 mm		142								
	Borehole advanced from 8.5 m to 9.1 m by coring after encountering casing refusal		11	SS	100/ 102 mm		141								
							140								
			12	SS	100/ 102 mm		139								
			13	SS	100/ 76 mm		138								
136.2	End of Borehole						137								
12.8	No groundwater seepage noted prior to initiation of wash boring.														

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

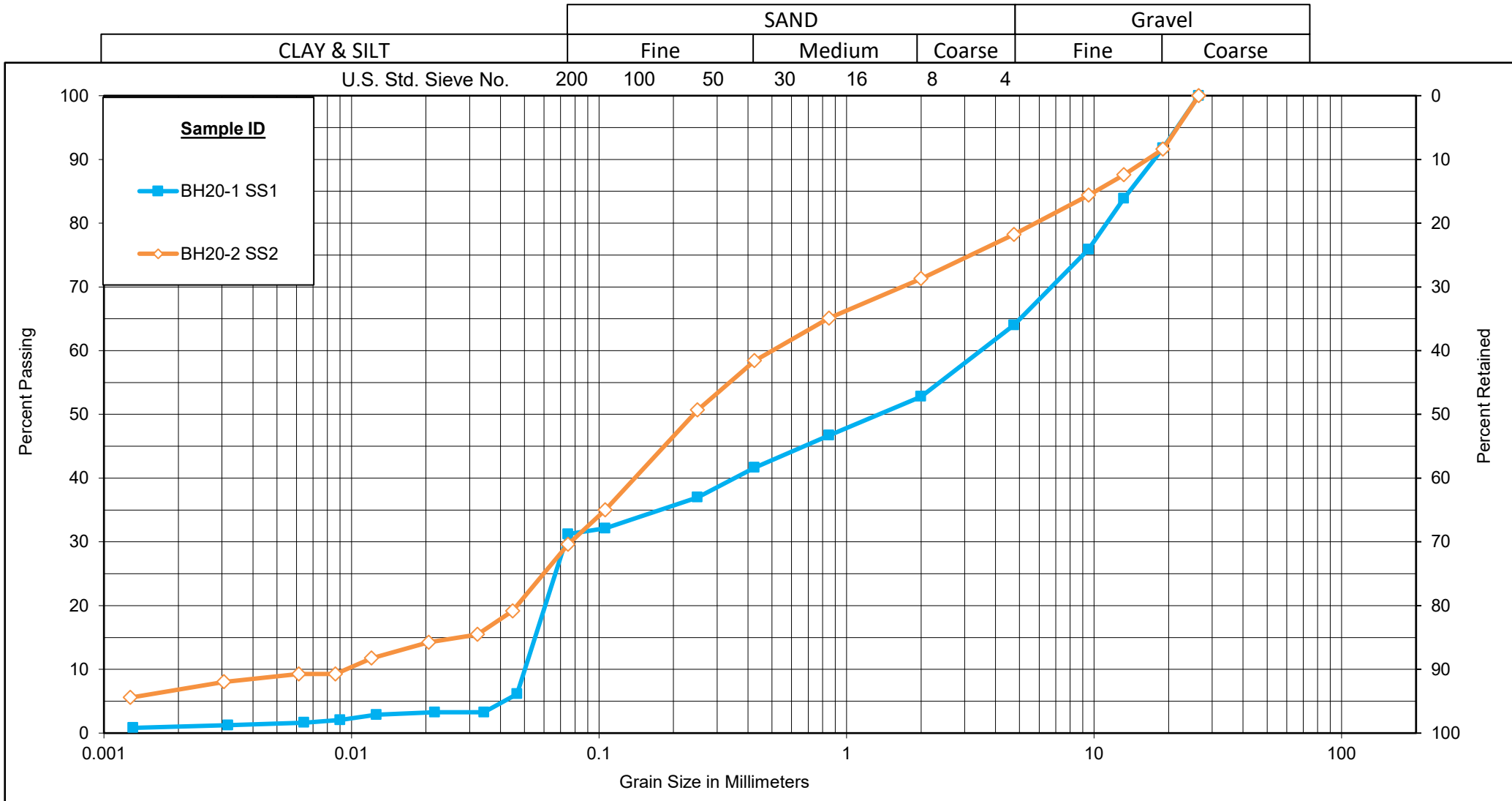
ONTARIO MTO 165001106 HWY 401\_NAGLE\_RD.GPJ ONTARIO MTO.GDT 7/10/20

## Appendix D

### D.1 LABORATORY TEST RESULTS



# Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH20-1 SS1	0'-2'	36.0	32.8	30.2	1.0
BH20-2 SS2	5'-7'	22.0	48.4	22.6	7.0



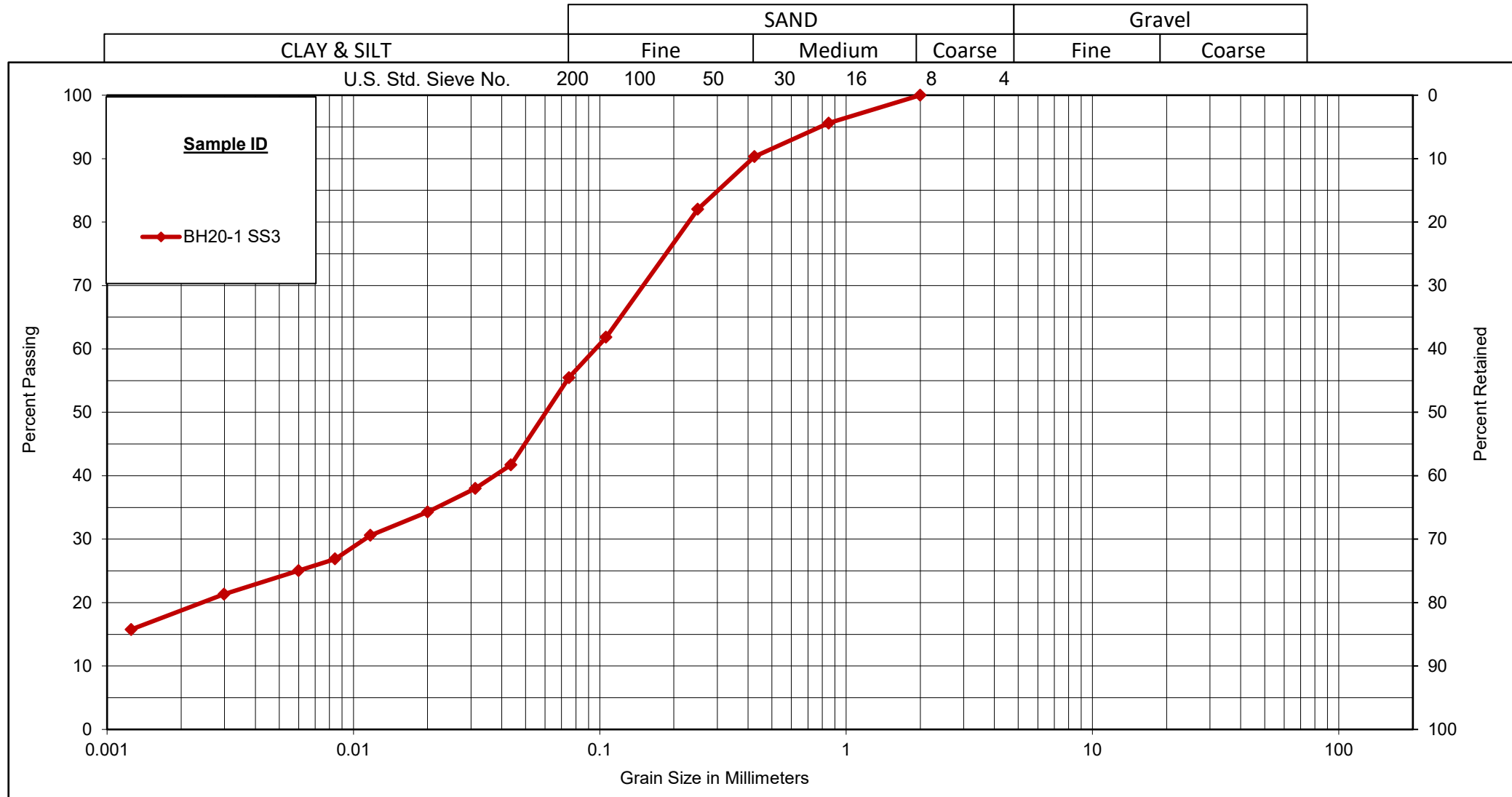
## GRAIN SIZE DISTRIBUTION

FILL: Silty Sandy GRAVEL (GM) to Gravelly Silty SAND (SM)  
Highway 401/Nagle Road Underpass

Figure No. D1

Project No. 165001106

# Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH20-1 SS3	5'-7'	0.0	44.5	37.5	18.0

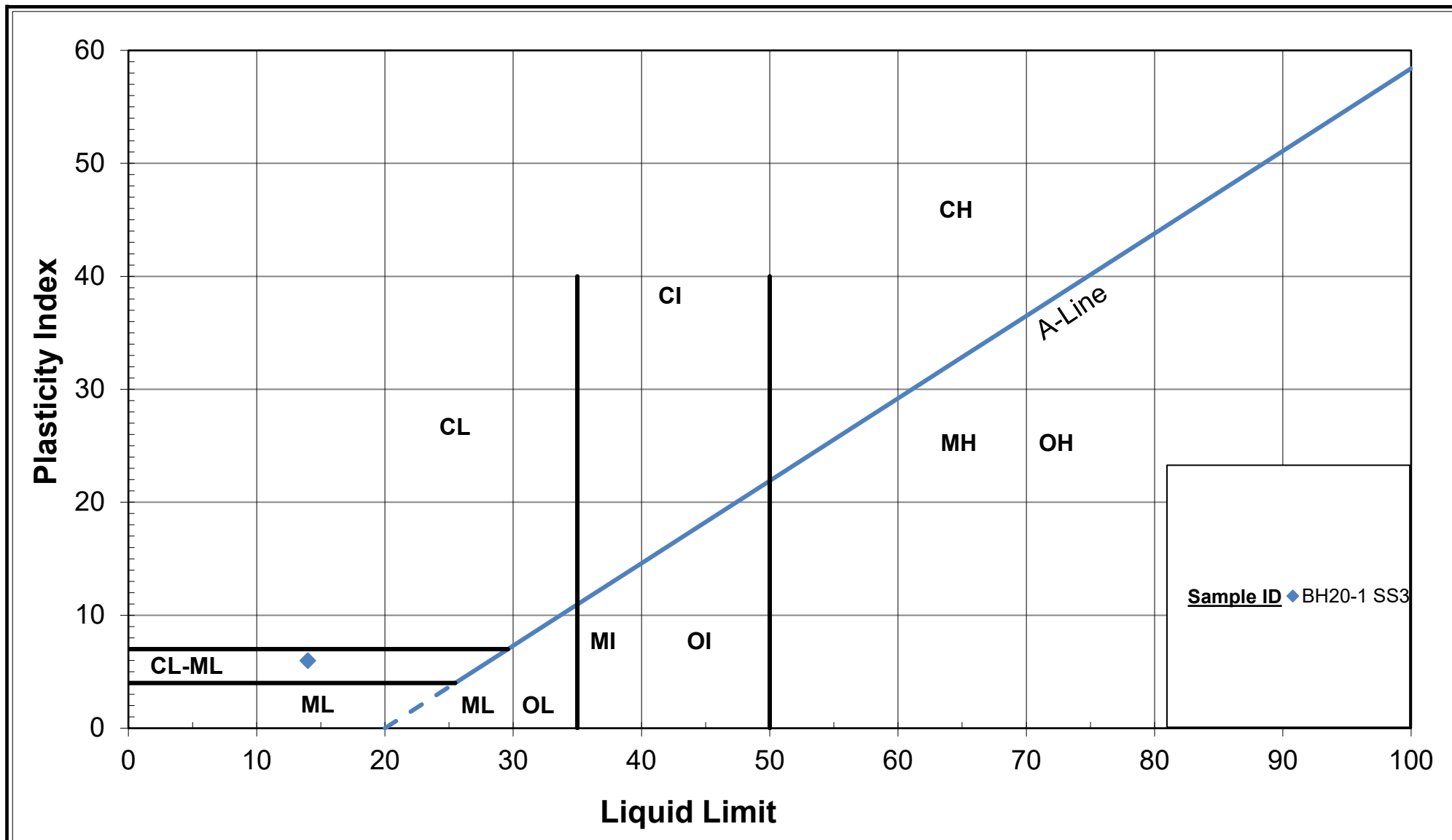


## GRAIN SIZE DISTRIBUTION

CLAYEY SILT with Sand (TILL) - (CL-ML)  
Highway 401/Nagle Road Underpass

Figure No. D2

Project No. 165001106



CLAYEY SILT with Sand (TILL) - (CL-ML)

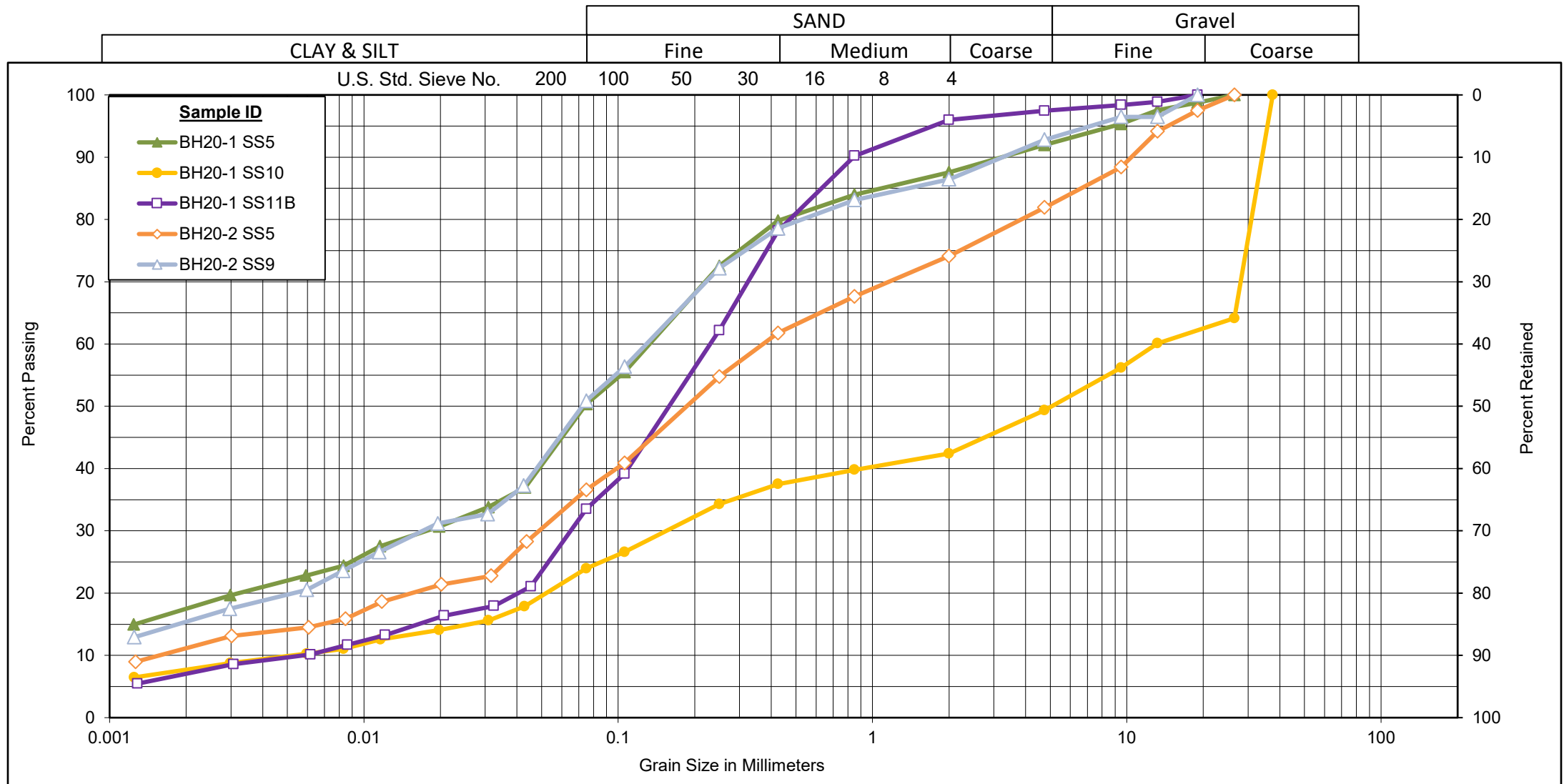
Highway 401 / Nagle Road Underpass

# PLASTICITY CHART

Figure No. D3

Project No. 165001106

# Unified Soil Classification System



Sample ID	Depth	% Gravel	% Sand	% Silt	% Clay
BH20-1 SS5	10'-12'	8.0	41.6	33.4	17.0
BH20-1 SS10	22'6"-24'6"	51.0	25.0	17.0	7.0
BH20-1 SS11B	27'6"-29'6"	3.0	63.5	26.5	7.0
BH20-2 SS5	10'-12'	18.0	45.4	25.6	11.0
BH20-2 SS9	20'-22'	7.0	42.1	35.9	15.0

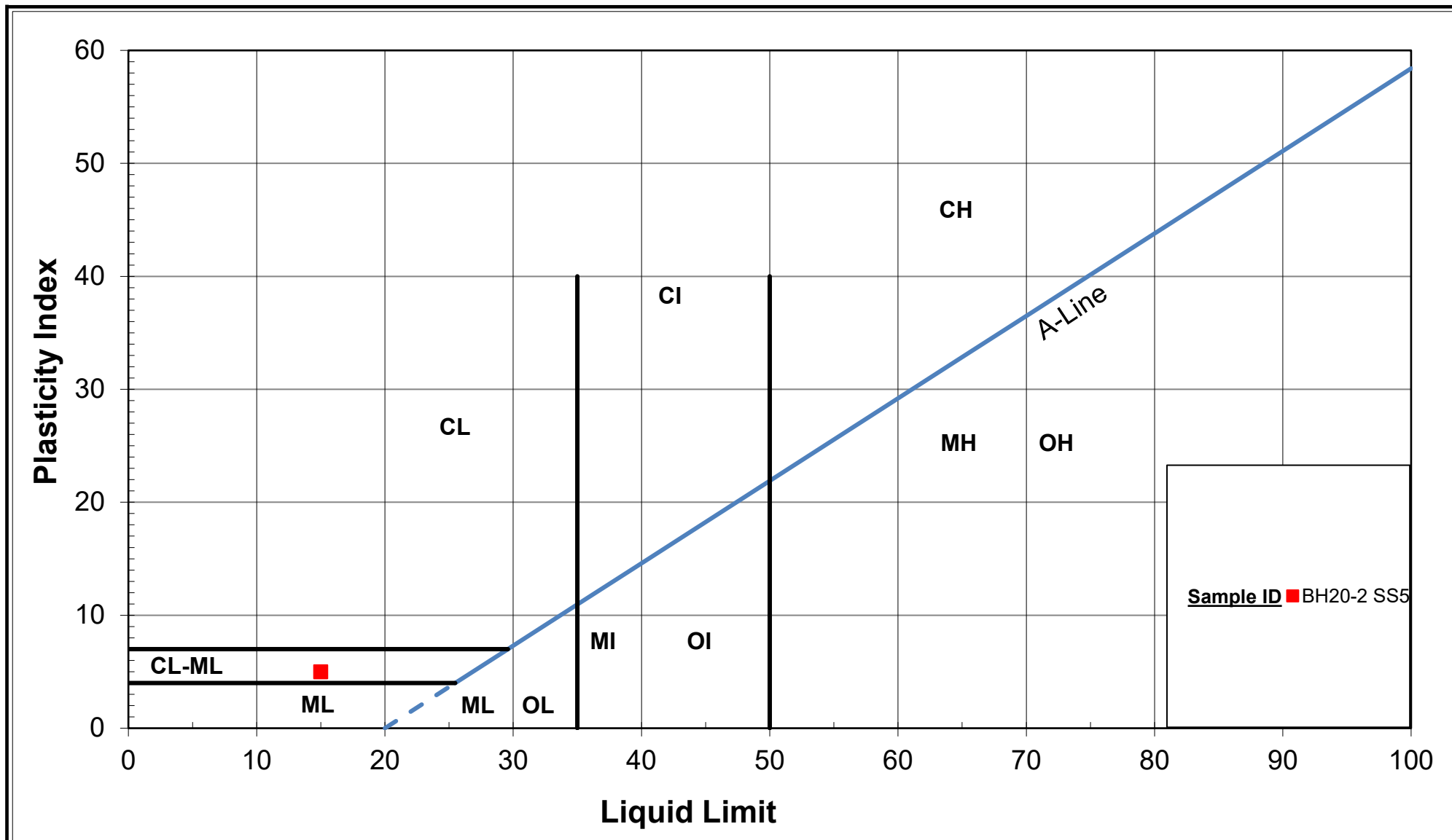


## GRAIN SIZE DISTRIBUTION

TILL: Silty Sandy Gravel (GM) to Silty Sand/Sandy Silt (SM to ML)  
Highway 401/Nagle Road Underpass

Figure No. D4

Project No. 165001106



Sandy CLAYEY SILT (TILL) - (CL-ML)

Highway 401 / Nagle Road Underpass

**PLASTICITY CHART**

Figure No. D5

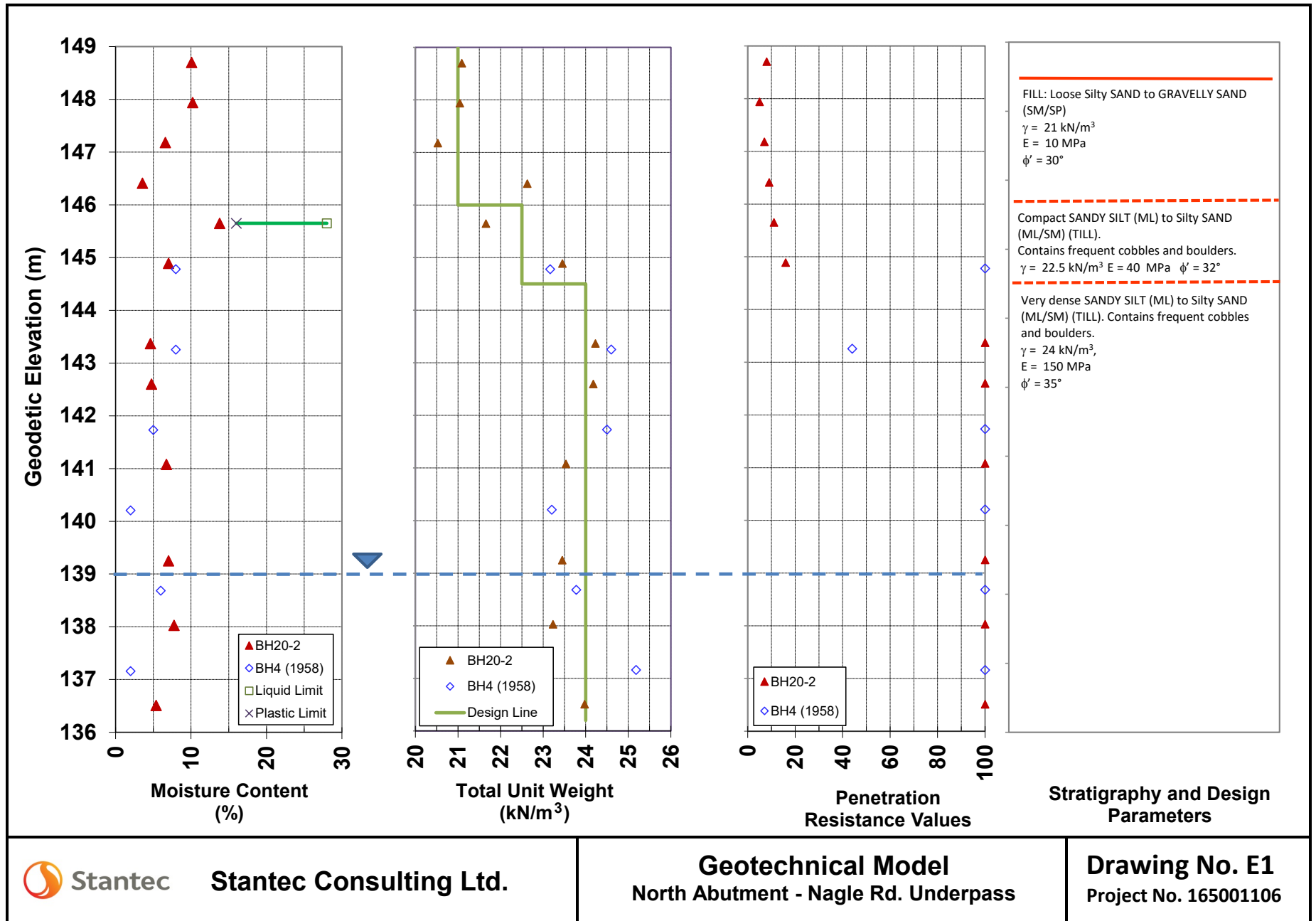
Project No. 165001106

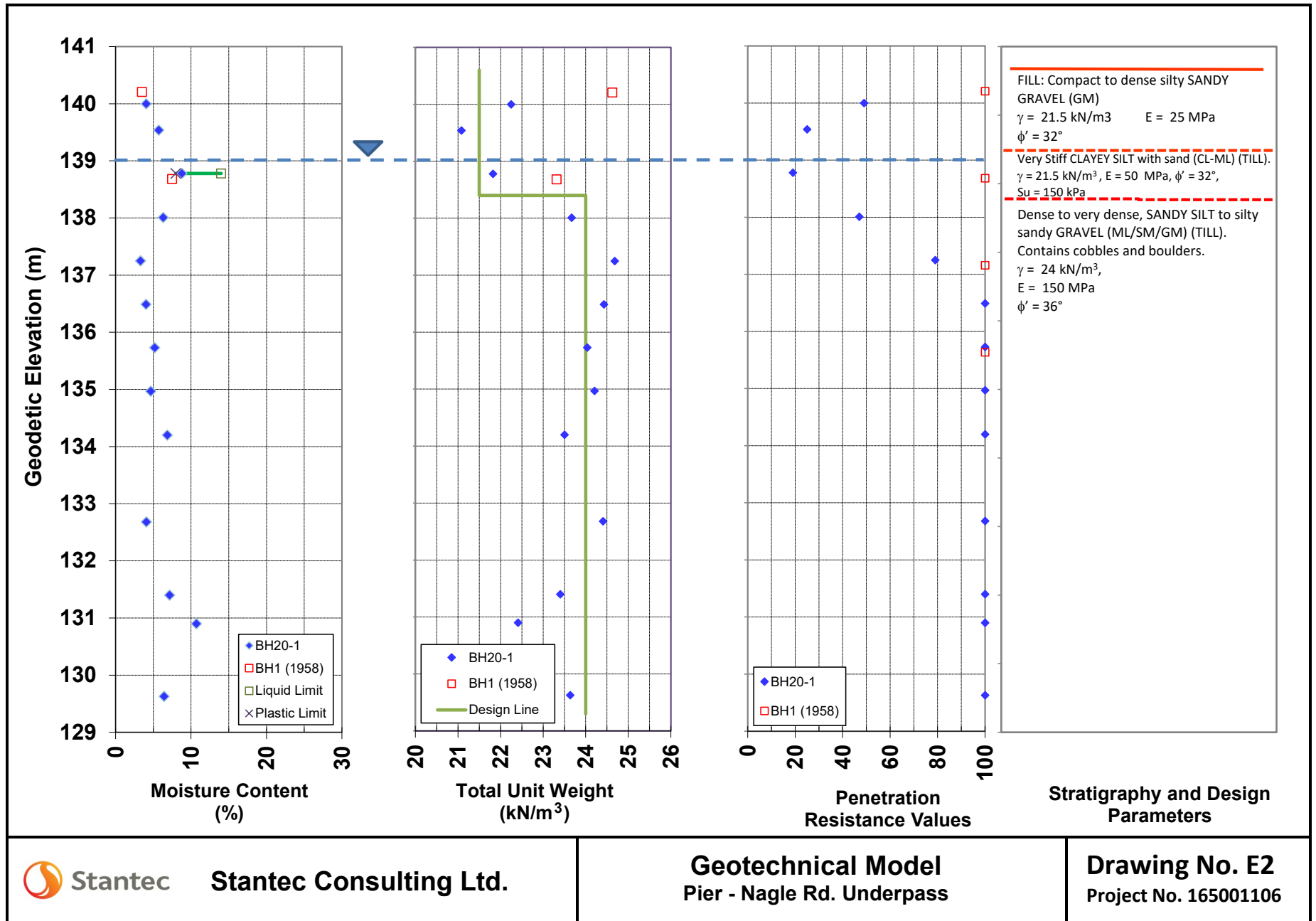
## Appendix E

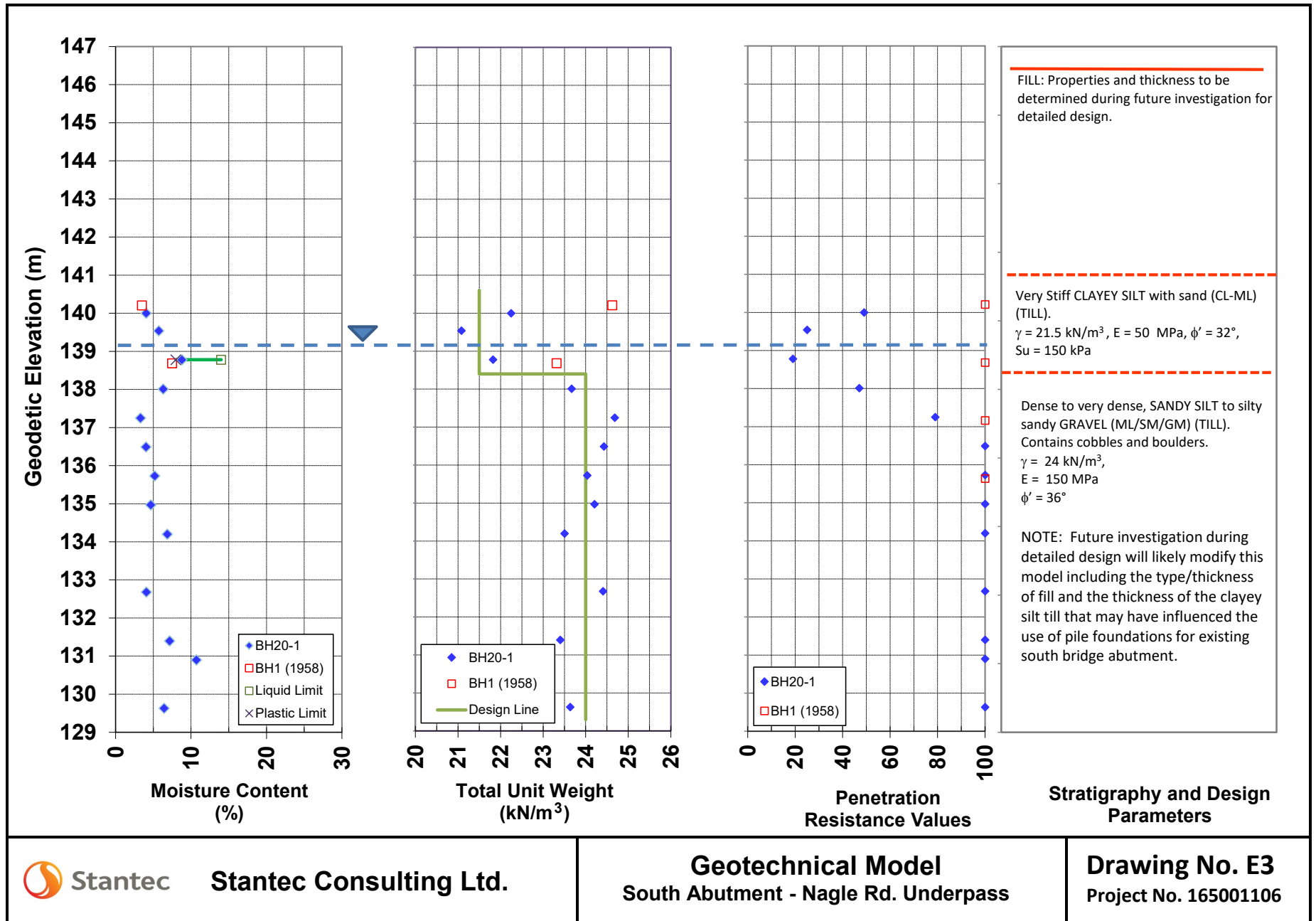
### E.1 DRAWINGS E1 TO E3 - GEOTECHNICAL SOIL MODELS











## Appendix F

### F.1 2015 NATIONAL BUILDING CODE SEISMIC HAZARD CALCULATIONS



# 2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836  
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Site: 43.994N 78.145W

User File Reference: Nagle Road Underpass

2020-06-10 19:22 UT

Probability of exceedance per annum	0.000404	0.001	0.0021	0.01
Probability of exceedance in 50 years	2 %	5 %	10 %	40 %
Sa (0.05)	0.160	0.084	0.049	0.014
Sa (0.1)	0.201	0.112	0.068	0.022
Sa (0.2)	0.175	0.103	0.066	0.023
Sa (0.3)	0.139	0.085	0.056	0.021
Sa (0.5)	0.105	0.067	0.045	0.016
Sa (1.0)	0.059	0.038	0.025	0.008
Sa (2.0)	0.030	0.019	0.012	0.003
Sa (5.0)	0.007	0.004	0.003	0.001
Sa (10.0)	0.003	0.002	0.001	0.001
PGA (g)	0.110	0.062	0.038	0.012
PGV (m/s)	0.087	0.052	0.033	0.010

**Notes:** Spectral ( $S_a(T)$ , where  $T$  is the period in seconds) and peak ground acceleration (PGA) values are given in units of  $g$  ( $9.81 \text{ m/s}^2$ ). Peak ground velocity is given in  $\text{m/s}$ . Values are for "firm ground" (NBCC2015 Site Class C, average shear wave velocity  $450 \text{ m/s}$ ). NBCC2015 and CSAS6-14 values are highlighted in yellow. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. **These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.**

## References

**National Building Code of Canada 2015 NRCC no. 56190;** Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

**Structural Commentaries (User's Guide - NBC 2015: Part 4 of Division B)**  
**Commentary J:** Design for Seismic Effects

**Geological Survey of Canada Open File 7893** Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites [www.EarthquakesCanada.ca](http://www.EarthquakesCanada.ca) and [www.nationalcodes.ca](http://www.nationalcodes.ca) for more information



Natural Resources  
Canada

Ressources naturelles  
Canada

Canada