



# Terraprobe

*Consulting Geotechnical & Environmental Engineering*

*Construction Materials Inspection & Testing*

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
BRAMPTON HYDRO ONE DUCT INSTALLATIONS  
HWY. 410/CLARK BOULEVARD INTERCHANGE  
THE CITY OF BRAMPTON, ONTARIO  
MINISTRY OF TRANSPORTATION  
CENTRAL REGION  
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## **PART A – FOUNDATION INVESTIGATION REPORT**

**HYDRO ONE BRAMPTON DUCT INSTALLATIONS  
HWY. 410/CLARK BOULEVARD INTERCHANGE  
THE CITY OF BRAMPTON, ONTARIO  
MINISTRY OF TRANSPORTATION  
CENTRAL REGION**



## 1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by Hydro One Brampton on behalf of NBM Engineering Inc. (NBM), to provide foundation engineering services in support of Hydro One duct installations below Highway 410 at the Clark Boulevard interchange in the City of Brampton, Ontario.

The scope of work for the foundation engineering services is outlined in Terraprobe's proposal titled *"Proposal for Geotechnical Engineering Services, Installation of Hydro One Duct below Hwy. 410 at Queen St. & Clarke Blvd., City of Brampton"* dated July 03, 2015. This report provides factual data on the subsurface conditions at the site.

## 2.0 SITE DESCRIPTION

The site is located at the Highway 410/Clark Boulevard interchange in the City of Brampton, Ontario. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location.

At this site Highway 410 is a divided freeway consisting of an unpaved median with three lanes in each direction, and fully paved inner and outer shoulders. The Clark Boulevard underpass is a two span bridge that links Clark Boulevard east and west of Highway 410. Heart Lake Road South is aligned parallel to and is located on the west side of Highway 410.

## 3.0 INVESTIGATION PROCEDURES

The fieldwork for this project was carried out on October 13 and 14, 2015. Four boreholes numbered Borehole C1 to C4 were drilled and sampled to depths ranging from 7.6 m to 8.1 m below ground surface at the approximate locations shown on Drawing 1. The boreholes were marked in the field by NBM who also provided Terraprobe with their coordinates and geodetic elevations. This data is summarized in the following table.

**Borehole Details**

Borehole No.	MTM NAD 83 Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH C1	4 840 104.2	286 038.7	215.5	7.6
BH C2	4 840 123.7	286 056.3	216.5	7.8
BH C3	4 840 144.9	286 092.2	215.8	8.1
BH C4	4 840 180.9	286 129.5	216.7	7.7

The boreholes were drilled with truck and track-mounted BOA-5M drill rigs supplied and operated by a specialist drilling contractor. Terraprobe's staff observed and recorded the drilling, sampling and in situ testing operations and logged the boreholes.

Samples of the overburden soils were generally obtained at intervals of 0.75 m depth using a 50 mm outer diameter (O.D.) split-spoon sampler in conjunction with the Standard Penetration Testing (SPT) procedures as specified in ASTM Method D 1586<sup>1</sup>.

1 ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

Ground water conditions in the open boreholes were observed during the drilling operations and standpipe piezometers were installed in Boreholes C1 and C4 to permit longer term ground water level monitoring. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The recovered soil samples were subjected to Visual Identification (VI). Select soil samples were also subjected to a laboratory testing programme consisting of natural moisture content, grain size distribution analyses and Atterberg limits determinations.

## **4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Regional Geology**

The site is located in the physiographic region of Southern Ontario referred to as the Peel Plain<sup>2</sup> and the youngest glacial deposit in this area is Halton Till. This area of low relief was affected by a succession of changing levels of glacial Lake Peel and within the Halton Till can be found intervening areas of glaciofluvial and glaciolacustrine deposits consisting of gravel, sand, silt and clay.

The overburden soils are underlain by bedrock of the Georgian Bay Formation. The Georgian Bay Formation is of Middle Ordovician Age and is predominantly shale with interbeds of calcareous sandstone, siltstone and grey argillaceous limestone.

### **4.2 Subsurface Conditions**

Reference is made to the Record of Borehole Sheets in Appendix A. Details of the encountered soil stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawing. An overall description of the stratigraphy is given in the following paragraphs.

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic section are inferred from non-continuous soil sampling and therefore represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In summary, a flexible pavement and fill soils consisting of compact to very dense gravelly sand to sand and gravel, and stiff to hard silty clay were encountered at the site. The native overburden deposits consist of compact to very dense sand and silt to silty sand till, and hard silty clay till.

#### **4.2.1 Topsoil**

Boreholes C3 and C4 encountered a layer of topsoil ranging from about 80 mm to 110 mm in thickness. Topsoil thickness may vary between and beyond the boreholes.

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2 Karrow, P.F. 2005, "Quaternary geology of the Brampton Area", Ontario Geological Survey, Report 257, 2005.

#### 4.2.2 Flexible Pavement

Boreholes C1 and C2 were drilled through the Heart Lake Road and Ramp Clark Blvd. E/W – 410S pavement. Both boreholes encountered a flexible pavement consisting of 175 mm and 100 mm thick asphalt concrete underlain by granular fill consisting of sand and gravel to gravelly sand. The locations, thicknesses and base elevations of the granular base fill are summarized in the following table.

**Borehole Data – Pavement Granular**

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH C1	485	214.8
BH C2	1600	214.8

Standard Penetration tests carried out in the sand and gravel and gravelly sand fill gave SPT N-values ranging from 24 to 58 blows for 0.3 m of penetration indicating a compact to very dense relative density. The natural water content of samples of the granular fill range from 3% to 5% by weight.

A sample of the gravelly sand fill was subjected to a grain size distribution test and the results are presented in Figure B1 in Appendix B. These results show a grain size distribution consisting of 29% gravel, 55% sand, 12% silt and 4% clay sized particles.

#### 4.2.3 Fill – Silty Clay

Silty clay fill was encountered in all of the boreholes and the locations, thicknesses, depths and base elevations of the silty clay fill are summarized in the following table.

**Borehole Data - Silty Clay Fill**

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH C1	0.7	1.4	214.1
BH C2	0.4	2.1	214.4
BH C3	2.0	2.1	213.7
BH C4	3.6	3.7	213.0

Standard Penetration tests in the silty clay fill gave SPT N-values ranging from 10 to 38 blows for 0.3 m of penetration indicating a stiff to hard consistency. The natural water content of samples of the silty clay fill range from 12% to 26% by weight.

The grain size distribution curve of a sample of the silty clay fill is depicted on Figure B2 in Appendix B. These results show a grain size distribution consisting of 3% gravel, 32% sand, 39% silt and 26% clay size particles.

The silty clay fill was also subjected to an Atterberg Limits test and the results are presented on Figure B3 in Appendix B. These results indicate that the fill is a low plasticity (CL) cohesive soil. The results from the Atterberg limits tests are summarized below:

Liquid Limit:	28%
Plastic Limit:	17%
Plasticity Index:	11%
Natural Moisture Content:	14%

#### 4.2.4 Sand and Silt to Silty Sand Till

Till units with a soil matrix composition of sand and silt to silty sand were encountered at this site and their locations, thicknesses, depths and base elevations are summarized in the following table.

**Borehole Data – Sand and Silt to Silty Sand Till**

Borehole No.	Sand and Silt to Silty Sand Till Thickness (m)	Sand and Silt to Silty Sand Till Depth (m)	Sand and Silt to Silty Sand Till Base Elevation (m)
BH C1	1.5	2.9	212.6
BH C2	1.3	3.4	213.1
BH C3	2.2	4.3	211.5
	2.1	7.8	208.0
BH C4	4.0	7.7*	209.0

\* Borehole termination depth.

Standard Penetration tests performed in this deposit measured SPT N-values ranging from 13 to more than 100 blows for 0.3 m of penetration indicating a compact to very dense relative density. The natural water content of samples of this deposit range from 6% to 14% by weight.

The grain size distribution plots of four samples of the sand and silt to silty sand till are depicted on Figure B4 in Appendix B. The results show a grain size distribution consisting of 6% to 12% gravel, 37% to 49% sand, 35% to 45% silt and, 7% to 10% clay sized particles. The field investigations also show resistance to augering in these till units which suggests that random cobble and boulder inclusions are present.

#### 4.2.5 Silty Clay Till

Layers of silty clay till were encountered at this site and the locations, thicknesses, depths and base elevations of the silty clay till are summarized in the following table.

**Borehole Data - Silty Clay Till**

Borehole No.	Silty Clay Till Thickness (m)	Silty Clay Till Depth (m)	Silty Clay Till Base Elevation (m)
BH C1	4.7	7.6*	207.9
BH C2	4.4	7.8*	208.7
BH C3	1.4	5.7	210.1
	0.3	8.1*	207.7

\* Borehole termination depth.

Standard Penetration tests performed in the silty clay till measured SPT N-values ranging from 71 to more than 100 blows for 0.3 m of penetration indicating a hard consistency. The natural water content of samples of the silty clay till range from 5% to 15% by weight.

The grain size distribution plots of five samples of the silty clay till are depicted on Figure B5 in Appendix B. The results show a grain size distribution consisting of 0% gravel, 9% to 34% sand, 40% to 69% silt and, 22% to 31% clay sized particles. The field investigations also show resistance to augering in these till units which suggests that random cobble and boulder inclusions are present.



Atterberg limits tests were carried out on five samples of the silty clay till and the results are presented in Figure B6 in Appendix B. These values indicate that the silty clay till is a low plasticity (CL) cohesive soil.

The results from the Atterberg limits tests are summarized below:

Liquid Limit:	25% to 31%
Plastic Limit:	15% to 20%
Plasticity Index:	9% to 12%
Natural Moisture Content:	5% to 10%

#### 4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were also installed in selected boreholes and the measured ground water levels in the piezometers are summarized in the following table:

**Ground Water Level Data**

Borehole No	Date	Water Levels	
		Depth (m)	Elevation (m)
BH C1	November 13, 2015	1.8	213.7
	November 27, 2015	1.8	213.7
BH C4	November 13, 2015	4.0	212.7
	December 1, 2015	4.0	212.7

Based on the recorded water levels in the standpipe piezometers, ground water observations during drilling and, the measured water contents of the soil samples, the estimated ground water table elevation is  $213.7 \pm$  m. The ground water level is expected to fluctuate seasonally and is expected to rise during wet periods of the year. Perched water can also be expected to occur where deposits of sands and silts and silty sand are underlain by relatively impermeable silty clay soils.



## 5.0 MISCELLANEOUS

The investigation was carried out using drilling equipment supplied and operated by Groundwork Drilling Inc. of Etobicoke, Ontario. The field operations were organized and monitored by Ms. Sepideh D-Monfared, MEng. The laboratory testing was carried out at Terraprobe's Brampton laboratory.

This report was prepared by Mr. Hussein Ahmed, P.Eng. and reviewed by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact conducted an independent quality control review.

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## **PART B – FOUNDATION DESIGN REPORT**

**HYDRO ONE BRAMPTON DUCT INSTALLATIONS  
HWY. 410/CLARK BOULEVARD INTERCHANGE  
THE CITY OF BRAMPTON, ONTARIO  
MINISTRY OF TRANSPORTATION  
CENTRAL REGION**



## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to carry out designs for Hydro One duct installations at the Highway 410/Clark Boulevard interchange. The discussion and recommendations presented in this report are based on our understanding of the project and our interpretation of the factual data obtained from the subsurface investigations.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Hydro One electrical feeder cables will be installed on the south side of Clark Boulevard below Highway 410, the Clark Boulevard interchange Ramp Clark Blvd. E/W – 410S and Heart Lake Road. Summarized below are the relevant design details:

- The cables will cross below Heart Lake Road and Ramp E/W-410S. Below Ramp E/W-410S, MTO requires a minimum vertical clearance of 3 m measured from top of pavement to top of pipe;
- The cables will cross below Highway 410 and, MTO requires a minimum vertical clearance of 5 m measured from top of pavement to top of pipe;
- The proposed installation is approximately 129± m long extending at offset distances of 73 m west and 56 m east of the Highway 410 median centre line. In this report the offset distances are reported as stations i.e. Station 0-73 (west limit) easterly to Station 0+0 (Hwy. 410 median centre line), and Station 0+56 (east limit); and
- Nine electrical cables will be installed in a bank of four and a bank of five, requiring two HDD drives. The HDD bore will not exceed 300 mm in diameter.

### **6.2 Installation Methods**

The diameter, length and anticipated subsurface conditions limits the range of installation techniques that are economically viable. Each method considered has advantages, disadvantages or limitations and these are discussed further below.

Ground behaviour will be, in part, dependent on the installation method adopted and this report provides guidance on the influence of ground behaviour on some possible installation methods. It should not be construed that the Contractor is restricted to the particular methods considered herein, and in the event of alternative methods, the Contractor must make his own interpretation of the anticipated ground behaviour, based on the factual information provided in this report under Part A, Foundation Investigation Report.

Trenchless installation methods such as microtunnelling were considered but this methodology is ruled out as being uneconomical and possibly impractical because two electrical ducts are being installed relatively close to each other. The construction methodologies evaluated and the recommendations for selecting the preferred method took into consideration the risks and consequences of each alternative, relative construction costs, as well as the need to minimize traffic disruptions and reduce user delay costs

during construction. A trenchless alternative is generally more costly compared to an open-cut excavation, and there is always a possibility that excavations may be required to retrieve tunnelling equipment or, equipment may have to be abandoned if adverse subsurface conditions are encountered. Practical alternatives are:

- Open Cut Excavation; and
- Horizontal Directional Drilling (HDD).

### **6.2.1 Open Cut Excavation**

An open cut excavation involves a trench excavation and excavation sidewall support; bedding placement, duct installation followed by cover material placement and trench backfilling. The open cut method reduces the potential for delays resulting from encountering obstructions and provides the least risk of unanticipated damage to the active roadways.

The major disadvantages with an open cut installation is the requirement for proper construction staging to minimize traffic disruption, the need for relatively large and deep excavations; and the potential for post construction settlement of the backfill materials. Open cut excavations are not recommended for installations below Highway 410 and the interchange ramp.

### **6.2.2 Horizontal Directional Drilling (HDD)**

Horizontal directional drilling involves drilling an initial pilot hole from an entry point to an exit point using drilling mud to support the drill hole sidewalls. Following completion of the pilot hole, the hole is reamed successively in increasing diameters until the drill hole is of sufficient size to permit installation of a product pipe. The product pipe is then typically installed by attaching it to the drill rods and pulling it back through the drill hole from the exit point to the entry point.

HDD can be carried out in both soil and rock, and there are no specific limitations below ground water. Some restrictions may apply in very loose coarse sand or gravel. These soils will have a tendency to collapse in the bore path causing either excessive spoil removal or in some cases stopping the installation. The accuracy of HDD is dependent on the accuracy in determining the drill head location and depth. Accuracy is typically 2% to 5% of the depth.

The HDD alignment is designed such that the radii of curvature of all sections of the alignment (including those which may involve complex curves), are sufficiently large such that the HDD drill rods can accommodate the proposed curvature and; the cables can be installed/pulled along the bore path without being overstressed. The minimum radius of curvature will be dependent on the contractor's drill rod size and length, and the flexibility of the cables.

Typically, the electrical cables are packaged and stored on reels. After the bore path is reamed to its desired size, the cable ends are tied together and pulled through the bore path while the reels are gradually unrolled. It is essential to minimize the cable installation time because the bore path has the potential to degrade with time.

### 6.3 Assessment of Tunnelling Alternatives

To reduce the risk of subsidence or heave, tunnelling installations require a minimum depth of overburden cover over the tunnel crown. As the depth of overburden cover decreases, the risk of concentrated subsidence or heave increases, as does the risk of extreme events such as sinkholes, or in the case of HDD, frac-outs forming at the ground surface. In Ontario, the general practice is to maintain a minimum depth of cover equivalent to 2 to 3 tunnel diameters.

There are inherent risks and consequences involved with trenchless installations that could include some or all the following:

- Obstructions within the tunnel reach that could increase the level of construction effort. Adequate equipment such as mandrels, pneumatic breakers or chisels, and augers are required to break and remove obstructions. If such efforts prove futile the tunnel will have to be abandoned or, an open cut excavation would be required to remove the obstruction.
- Inability to correct for line and grade within the design tolerances. If misalignment occurs, it may be necessary to abandon the bore and grout the open section. Alternatively, an open cut excavation may be the most efficient way of completing the installation.

Table 1 following the text of this report, provides a summary of the HDD Bore Path elevations, Bore Path dimension, the range of depths of overburden cover, the Depth of Cover to HDD Bore Path Diameter Ratio and; the subsurface conditions encountered in the boreholes from ground surface to the respective HDD bore path.

The SPT N-values and the coefficient of uniformity (which is an indication of how well the soil is graded, and is expressed as the ratio of the particle size at which 60% of the particles are finer than to the particle size at which 10% are finer than), assist in classifying the soil behaviour according to the Tunnelman's Ground Classification System (Terzaghi, 1950). This system is commonly used to describe the potential behaviour of an unsupported tunnel face during excavation and it uses qualitative "stand-up time" criteria to classify the ground at and above the tunnel face into the following principal categories: firm, slow ravelling, fast ravelling, squeezing, cohesive running, running, flowing and swelling. Efforts to predict soil behaviour must also be tempered by experience and engineering judgement.

The soil conditions within the HDD bore path are classified in Table 2 following the text of this report. The soil conditions generally range from "firm to slow ravelling" to "cohesive running". A comparison of the advantages, disadvantages, risks and consequences associated with an HDD installation and an open cut excavation is presented in Table 3 following the text of this report.

## 6.4 Preferred Alternative

The alternatives described in Section 6.2 are ranked on a numerical scale in order of preference with one (1) being assigned to the most preferred alternative and two (2) being assigned to the least preferred option. The preferred alternative was selected based on installation costs, the requirement to complete the installation in a single drive and the availability of construction equipment and local construction knowledge.

Alternative	Ranking of Alternatives
Horizontal Directional Drilling (HDD)	1
Open Cut Excavation	2

From a geotechnical perspective, we recommend installing the Hydro One ducts below Highway 410 and the interchange ramp by HDD techniques since it is the most feasible and practical alternative. Open cut excavations below the highway and ramp is not recommended. The advantages, disadvantages, risks and consequences of the installation methods are provided in Table 3. HDD shall be carried out in accordance with OPSS 450.

## 6.5 Settlement

The zone of influence of soils disturbed by the HDD operations will be about 2 tunnel diameters and construction of the HDD bore path will result in ground movements that will produce a settlement trough above and ahead of the bore path.

After a tunnel is constructed, the transverse settlement trough that develops can be described by a Gaussian distribution curve as:

$$S = S_{\max} \exp \left( \frac{-x^2}{2i^2} \right)$$

Where

- S = settlement observed at a distance x from the tunnel axis;
- $S_{\max}$  = maximum settlement above the tunnel axis;
- x = horizontal distance from the tunnel axis; and
- i = horizontal distance from the tunnel axis to the inflexion point on the settlement trough.

The settlement trough induced by tunnelling can be characterized by means of two parameters namely the volume of settlement per unit length of tunnel ( $V_s$ ) and the horizontal distance from the tunnel axis to the inflexion point (i).

The volume of the settlement trough ( $V_s$ ) is difficult to evaluate as this parameter is dependent on construction methods and workmanship. This parameter ( $V_s$ ), is usually compared to the volume of ground loss produced at the tunnel level and is expressed as a percentage of the theoretical volume of excavated soils ( $V_t$ ).



Correlations by Mair and Taylor (1997)<sup>3</sup> concluded that the parameter  $i$ , can be reasonably estimated using the following expression:

$$i = K Z$$

Where  $K$  = is the trough width parameter and its value is a function of ground type; and  
 $Z$  = depth to the tunnel centre line.

The equations outlined above were used to estimate settlement due to tunnelling below the highway and interchange ramp. A trough width parameter of 0.25 was selected for tunnels in non-cohesive soils. The estimated maximum range of settlement at the tunnel centreline for a 4% volume of ground loss is approximately 5 mm. This estimate is based on the assumption that the work will be carried out by experienced tunnellers with great care and good workmanship. However, more ground loss and settlement can occur if unanticipated conditions such as cobbles and boulders are encountered in the HDD bore path.

## 6.6 Instrumentation and Monitoring

Active roadway surfaces shall be monitored before, during and after construction. A precondition survey should also be carried out prior to construction, to document the existing conditions of the pavement and any nearby structures; for the purpose of determining any restoration that may be required due to construction impacts. An instrumentation and monitoring program has been developed for this project consistent with the *"Guidelines for Foundation Engineering Tunnelling Specialty for Corridor Encroachment Permit Application (MTO, 2008)"*, modified as appropriate and the inclusion of instrumentation monitoring arrays aligned perpendicular to the duct alignment. The instrumentation and monitoring program is required to:

- Document the effects of the installation on the overlying roadway;
- Obtain prior warning of ground movements that could occur due to construction methods and equipment or unforeseen ground condition;
- Verify the Contractor's compliance with the settlement limits imposed in the Contract; and,
- Allow adjustments to be made to the duct installation methodology such that the established settlement limits are not exceeded.

For a HDD installation, in-ground monitoring points shall be installed in the overburden soils and surface monitoring points shall be installed on roadways. These monitoring points are considered sufficient in providing an advance indicator of subsurface disturbance and the potential for settlement/heave at the ground surface due to the HDD operation. For open cut excavations temporary protection systems should be monitored for movement.

The Settlement Monitoring Plan presented in Appendix C illustrates the approximate locations of the monitoring instruments and provide typical instrument details. The monitoring point locations are approximate and must be confirmed by the Contractor in consultation with the Geotechnical Engineer prior to installation and construction, and may have to be adjusted in the field to suit local conditions/constraints.

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<sup>3</sup> Mair, R. J. and Taylor, R. N. (1997). Bored tunnelling in the urban environment, Theme Lecture, Plenary Session 4, 14<sup>th</sup> International Conference on Soil Mechanics and Foundation Engineering, Hamburg, 6-12 September.



Monitoring points should be installed under the supervision of a geotechnical engineer at least seven days prior to any excavation. All monitoring points should be surveyed for elevation at least three times on three separate days before the start of tunnelling to establish a pre-construction baseline. All points behind the face of the excavation and those within 10 m of the front of the face should be surveyed for elevation a minimum of three times per day over the duration of the tunnel drive. Monitoring is also required three times daily during off-shift and weekend periods.

A specialist surveying firm should be retained to confirm the set-up and to carry out the monitoring during construction. Their equipment and procedures must be capable of surveying the instruments laterally and vertically to within  $\pm 2$  mm. The survey data should be submitted by the surveyor to the Geotechnical Engineer, Contract Administrator, Owner and MTO on an ongoing basis, for prompt review.

For this project a Review Level of 10 mm and an Alert Level of 15 mm is considered appropriate for horizontal and vertical displacements. The following procedure should be followed if displacements reach the Review and Alert Levels.

- If the Review Level is reached the Contractor should be required to provide a formal plan that clearly states what measures will be taken to ensure that the Alert Level is not reached; and,
- If the Alert Level is reached, the Contractor shall stop all work and the Contract Administrator, the Owner and MTO shall have the authority to order the Contractor to alter the construction methodology to maintain integrity of existing conditions. The Contractor Administrator, the Owner and MTO shall also have the authority to order the Contractor to make the mined excavation stable and suspend all tunnelling until an approved mitigation solution is developed. The Contractor must have an emergency plan in place to ensure public safety.

## 6.7 Entry and Exit Point Excavations

Supported open excavations will be required at the entry and exit points of the HDD operation to contain the drilling fluid and to install utility manholes. Tabulated below are the approximate entry and exit point locations, a summary of the soil units at these locations and the anticipated soil conditions at the excavation base (outlined in bold letters). The estimated ground water level, the anticipated ground behaviour and suggested treatments that will be required to maintain base stability are also included in the table.

**Entry and Exit Point Excavations – Summarized Ground Conditions and Treatments**

Location	Approximate Station	Ground Water Level Relative to Excavation Base (m)	Remarks
East Limit	0+56	Below Base	<ul style="list-style-type: none"><li>■ <b>Stiff to hard silty clay fill.</b></li><li>■ Maintain dry excavation by pumping from filtered sumps.</li></ul>
West Limit	0-73	At Base	<ul style="list-style-type: none"><li>■ Very stiff silty clay fill, <b>dense sand and silt till.</b></li><li>■ Maintain dry excavation by pumping from filtered sumps.</li></ul>



## 6.8 Trenching, Backfilling and Compaction Requirements

The majority of the fill and native site soils are generally considered suitable for reuse as backfill in trenches and at entry and exit point excavations, provided they are free of topsoil, organic material or other deleterious material. Trench backfill materials should be placed in maximum 300 mm loose lifts and uniformly compacted to at least 95% of Standard Proctor Maximum Dry Density (SPMDD).

To achieve the specified compaction, soils must neither be too wet nor too dry of their optimum moisture content. Soils that are too wet cannot be used immediately because the material will have to be dried to a moisture content of  $\pm 2\%$  of optimum. If the construction operations are time sensitive, the use of imported granular material may be considered. Soils that are dry of optimum can be used immediately provided that the material is moisture conditioned (i.e. water added) to achieve a moisture content of  $\pm 2\%$  of optimum.

Normal post-construction settlement of the compacted backfill equivalent to about 1% of the backfill height should be anticipated. The majority of this settlement will take place within about six months following the completion of the backfilling operations. If this post-construction settlement cannot be tolerated, it is recommended that the trench be backfilled with Granular "B" Type I compacted to a minimum of 98% of the material's SPMDD at a moisture content within  $\pm 2\%$  of the optimum value.

The duct installation must conform to the requirements of OPSD 2100.010 (Cable Installation in Trenches). Additional bedding and backfill requirements that may be imposed by the pipe supplier and Hydro One must also be followed.

Prior to placing the sand bedding, any accumulation of water at the base of the excavation should be removed and any soft/loose soils should be subexcavated and replaced with compacted sand fill.

The bedding material should be placed in 150 mm thick loose lifts and uniformly compacted to at least 95% of the materials SPMDD using suitable vibratory compaction equipment. Trenching, backfilling and compacting should be carried out in accordance with OPSS 401.

## 6.9 Temporary Protection Systems

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary protection systems should be designed in accordance with OPSS.PROV 539 and the designs should be carried out by a licensed Professional Engineer experienced in shoring design. Support systems for shallower excavations should be installed in accordance with OPSS.404. All temporary protection systems installed within the MTO Right-Of-Way shall be removed after construction is complete.

The shape of the soil pressure distribution diagram behind a temporary protection system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The sequence of work will also alter the shape of the shoring pressure diagram during the various construction phases.

Earth pressure computations must also take into account the ground water level. Above the ground water level, earth pressure is computed using the bulk unit weight of the retained soil. Below the ground water level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained.

Flexible shoring should be designed on the basis of the active earth pressure coefficient ( $K_a$ ). In this case, the performance level should be Level 2 – Angular Distortion 1:200 but shall not be more than

25 mm. Where limited shoring movement (Performance Level 1A or 1B) is required the design should be based on the at rest earth pressure coefficient ( $K_o$ ). For “kick out” design the lateral resistance should be computed on the basis of the passive earth pressure coefficient ( $K_p$ ). It should be noted that the lateral earth pressure coefficients chosen for design require certain movements for the active and passive conditions to be mobilized.

The appropriate lateral earth pressure parameters for use in the design of temporary protection systems are provided in the following table. The lateral earth pressure coefficients are based on the assumption that the ground surface behind the temporary protection system is horizontal. Where the retained ground is sloping, the lateral earth pressure coefficients must be adjusted to account for the slope and, these earth pressure coefficients can be estimated from the equations provided on Figures C6.17 and C6.18 of the CHBDC 2006.

**Temporary Protection System Design Parameters**

Stratigraphic Unit	Friction Angle $\phi$ (degrees)	Unit Weight $\gamma$ (kN/m)	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			$K_a$	$K_o$	$K_p$
Fill - Silty Clay	28	19	0.36	0.53	2.77
Sand and Silt Till	33	21	0.29	0.46	3.39

## 6.10 Erosion Control

Proper erosion control measures should be implemented both during construction and permanently. Temporary erosion and sediment control must be provided in accordance with OPSS 805 and, excavated areas as well as areas disturbed by construction shall be reinstated with permanent erosion protection in accordance with OPSS 803 and OPSS.PROV 804.

## 6.11 Excavations

All excavations must be carried out in accordance with the guidelines outlined in the *Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects*. Where workers must enter excavations extending deeper than 1.2 m, the trench walls must be suitably sloped and/or braced in accordance with the OHSA. For the purposes of the OHSA the subsurface soils encountered at this site in the areas where open cut excavations are proposed are classified as follows:

- Stiff to hard silty clay fill – Type 3 soils; and
- Dense to very dense sand and silt till – Type 2 soil above the water table and Type 3 soils below the water table.

The side slopes of temporary excavations may be formed no steeper than 1H:1V for Type 2, and Type 3 soils. Excavations at steeper inclinations will require temporary support. Excavations should be undertaken in accordance with OPSS 401.



## **6.12 Ground Water Control**

Surface water and ground water control will be required to maintain sufficiently dry conditions during construction. Extensive dewatering techniques will not be required where open cut excavations are made through and into relatively impermeable silty clay soils or where perched water is encountered above the ground water table. For these conditions any surface water run-off into excavations as well as minor subsurface seepage from any wet seams within the overburden can be controlled by employing a system of gravity drainage and pumping from strategically placed filtered sumps.

The design, installation, operation and maintenance of the dewatering system is the Contractor's responsibility. A suitable dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

## **6.13 Design Frost Depth**

At this site a depth of 1.2 m of earth cover should be provided for protection from frost penetration (OPSD 3090.101 Foundation Frost Penetration Depths for Southern Ontario).

## **6.14 HDD Considerations**

It is anticipated that the HDD operation would be advanced through stiff to hard silty clay fill, dense to very dense silt and sand to sand and silt till and hard silty clay till.

There is a potential for deviation in the alignment when transitioning between soil deposits or if cobbles and boulders are encountered in the till soils. Therefore, the contractor should select appropriate equipment including drill bits and reamers in order to deal with these subsurface soil conditions.

High fluid pressures that may develop during drilling may also require the installation of pressure relief pits in order to mitigate "frac outs". Pressure relief pits will also minimize the potential for "hydrolock", which is a condition where circulation from the bore is lost due to cuttings inhibiting mud circulation which then causes pressure build-up ahead of the advancing pipe. The maximum drilling fluid pressures and static confining stresses should also be considered by the Contractor in the design of the HDD bore path.

Prior to construction the contractor should submit for review a comprehensive drilling plan that addresses all aspects of the HDD operation, such as equipment type, drilling fluids to be used, bore path design, pull back calculations and construction methodology.



## 7.0 CLOSURE

This report was prepared by Mr. Hussein Ahmed, P.Eng. and reviewed by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact conducted an independent quality control review.

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## Ontario Provincial Standard Specifications (OPSS)

OPSS 401	Construction Specification For Trenching, Backfilling and Compacting
OPSS 404	Construction Specification For Support Systems
OPSS 450	Construction Specification For Pipeline and Utility Installation In Soil By Horizontal Directional Drilling.
OPSS.PROV 539	Construction Specification For Temporary Protection Systems.
OPSS 803	Construction Specification For Sodding.
OPSS.PROV 804	Construction Specification For Seed and Cover.
OPSS 805	Construction Specification For Temporary Erosion And Sediment Control Measures.

## Ontario Provincial Standard Drawings (OPSD)

OPSD 2100.010	Cable Installation in Trenches.
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario.



TABLE 1  
CONSTRUCTIBILITY REVIEW OF HORIZONTAL DIRECTIONAL DRILLING

Tunnelling Method	Proposed Bore Path Diameter (mm)	Approximate Station and Bore Path Elevation (m)		Proposed HDD Length (m)	Depth of Overburden Cover (m)	Depth of Cover to HDD Bore Path Diameter Ratio	Anticipated Subsurface Conditions Within The Zone of Tunnelling	Estimated Groundwater Depth Relative to HDD Bore Path
		From	To					
Horizontal Directional Drilling	300	0+56 EL : 215.4±	0+30 EL : 211.5±	26 ±	1.3 to 5.2±	4.3 – 17.3	Stiff to hard silty clay fill; Dense to very dense silt and sand till.	1.7 m± below to 2.2 m± above
		0+30 EL : 211.5±	0-50 EL : 211.3±	80 ±	3.8 to 5.8±	12.7 – 19.3	Very dense silt and sand till; Hard silty clay till.	2.2 m± to 2.4 m± above
		0-50 EL : 211.3±	0-73 EL : 213.6±	23 ±	3.8 to 1.3±	12.7 – 4.3	Hard silty clay till; Dense to very dense sand and silt till; Very stiff silty clay fill.	2.4 m± above to 0.1 m± above



TABLE 2  
FEASIBILITY OF HORIZONTAL DIRECTIONAL DRILLING

Tunnelling Method	Proposed Bore Path Diameter (mm)	Approximate Station and Bore Path Elevation (m)		Depth of Overburden Cover (m)	Reference Borehole Number	Soil Conditions <sup>1</sup> (Ground surface to pipe invert)	Fines Content <sup>2</sup> (%)	SPT N-Values	Coefficient of Uniformity <sup>3</sup>	Soil Behaviour (Within the HDD Bore Path)
		From	To							
Horizontal Directional Drilling	300	0+56 EL : 215.4±	0+30 EL : 211.5±	1.3 to 5.2±	BH C4	Fill - Silty Clay	65	34, 16, 14, 10, 12	-. <sup>4</sup>	Firm to Slow Raveling
						Silt and Sand Till	52	37, 87, 100, >100/0.3m	22	Cohesive Running
		0+30 EL : 211.5±	0-50 EL : 211.3±	3.8 to 5.8±	BH C4	Fill - Silty Clay	65	34, 16, 14, 10, 12	-. <sup>4</sup>	
						Silt and Sand Till	52	37, 87, 100, >100/0.3m	22	Cohesive Running
					BH C3	Fill – Silty Clay	-. <sup>4</sup>	25, 38, 14	-. <sup>4</sup>	
						Sand and Silt Till	46	20, 82, >100/0.3m	> 50	Cohesive Running
						Silty Clay Till	85	90, >100/0.3m	-. <sup>4</sup>	Firm to Slow Raveling
						Silty Sand Till	44	>100/0.3m	> 50	
						Silty Clay Till	-. <sup>4</sup>	>100/0.3m	-. <sup>4</sup>	
					BH C2	Fill – Gravelly Sand	16	58, 24	> 50	
						Fill – Silty Clay	-. <sup>4</sup>	19	-. <sup>4</sup>	
						Sand and Silt Till	-. <sup>4</sup>	13, 31	-. <sup>4</sup>	
						Silty Clay Till	91, 66	75, >100/0.3m	-. <sup>4</sup>	Firm to Slow Raveling
		0-50 EL : 211.3±	0-73 EL : 213.6±	3.8 to 1.3±	BH C2	Fill – Gravelly Sand	16	58, 24	> 50	
						Fill – Silty Clay	-. <sup>4</sup>	19	-. <sup>4</sup>	
						Sand and Silt Till	-. <sup>4</sup>	13, 31	-. <sup>4</sup>	
						Silty Clay Till	91, 66	75, >100/0.3m	-. <sup>4</sup>	Firm to Slow Raveling
					BH C1	Fill – Sand and Gravel	-. <sup>4</sup>	41	-. <sup>4</sup>	
						Fill – Silty Clay	-. <sup>4</sup>	25	-. <sup>4</sup>	Firm to Slow Raveling
					BH C1	Sand and Silt Till	48	31, 61	43	Cohesive Running
						Silty Clay Till	87, 91	71, 78, >100/0.3m	-. <sup>4</sup>	Firm to Slow Raveling

1. Soil conditions from ground surface to pipe invert; bold soil conditions indicate soil conditions within tunnel horizon.  
2. Fines content is defined as the percentage by weight of soil particles passing the No. 200 Sieve and ♦ denotes fines content for respective soil type.  
3. Coefficient of uniformity of soil within and above the tunnel horizon reported for coarse grained soils only. Not applicable for fine grained soils.  
4. No tests performed within that area of interest or the Coefficient of Uniformity parameter is not applicable for that specific soil type.



TABLE 3  
EVALUATION OF INSTALLATION METHODS

Installation Method	Ranking	Advantages	Disadvantages	Risk/Consequences
Horizontal Directional Drilling	1	<p>Minimal traffic disruption.</p> <p>Precludes the requirement for large entry and exit pit footprint areas i.e. minimal impact on surrounding area.</p> <p>Long drilling distances as much as 1500 -1800 m achievable in a single bore.</p> <p>Does not require groundwater lowering since the drill path will be kept open with drill fluid.</p> <p>Can be used to install ducts at relative close spacing compared to other tunnelling methods.</p> <p>Less expensive than open cut excavation.</p>	<p>Requires a skilled construction workforce.</p> <p>Difficult to control vertical and horizontal alignment if drill bit deflections occur off cobbles and boulders.</p>	<p>Possible inadvertent returns (Frac-out) or surface heave due to poor subsurface soil conditions or high fluid pressure especially within 30m of the entry point, due to the reduced depth of the bore path below ground surface.</p>
Open Cut Excavation	2	<p>Best control of gradient and alignment.</p> <p>Reduced potential for delays due to obstructions.</p> <p>Least risk of unanticipated damage to active roadways.</p> <p>Equipment and skilled construction workforce readily available.</p>	<p>Requires construction staging in order to maintain traffic on roadways which could result in traffic delays.</p> <p>Relatively large and deep excavations required along some sections of the alignment.</p> <p>Dewatering required for installations at/below ground water level.</p>	<p>Increased traffic disruption.</p>

**DRAFT**  
**DRAWING**





**APPENDIX A**  
**Record of Borehole Sheets**



## EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg. FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS  $\bar{u}$ .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

**CONSISTENCY:** COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS:

$c_u$ (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

**DENSENESS:** COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

**RECOVERY:** SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

**MODIFIED RECOVERY:** SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_{\alpha}$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	- °	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	- °	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_r$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_S$	%	SHRINKAGE LIMIT	q	m <sup>2</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(w - w_p)/I_p$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>3</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

# RECORD OF BOREHOLE No C1

1 of 1

METRIC

G.W.P. \_\_\_\_\_ LOCATION \_\_\_\_\_ Coords: E:286038.7 N:4840104.2 ORIGINATED BY SD  
 DIST \_\_\_\_\_ HWY 410 BOREHOLE TYPE SOLID STEM AUGERS COMPILED BY SD  
 DATUM GEODETIC DATE 2015-10-13 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)								
								20    40    60    80    100								
							○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE					PLASTIC LIMIT    NATURAL MOISTURE CONTENT    LIQUID LIMIT w <sub>p</sub> w    w <sub>L</sub> WATER CONTENT (%)				
							20    40    60    80    100					10    20    30				
215.5	GROUND SURFACE															
215.3	175mm ASPHALTIC CONCRETE															
0.2	485mm FILL, sand and gravel, trace silt, dense, brown, dry		1	SS	41		215									
214.8	FILL, silty clay, trace sand, trace gravel, very stiff, grey, dry to moist		2	SS	25											
0.7																
214.1	SAND AND SILT, trace clay, trace gravel, dense to very dense, brown, moist to wet (GLACIAL TILL)		3	SS	31		214							6 46 39 9		
1.4			4	SS	61		213									
212.6	SILTY CLAY, trace to some sand, trace gravel, containing cobbles and boulders, hard, brown to 3.7m, grey below, dry (GLACIAL TILL)		5	SS	71		212							auger grinding		
2.9			6	SS	100 / 140mm									0 13 58 29		
			7	SS	100 / 100mm		211							auger grinding		
			8	SS	78		210							0 9 69 22		
			9	SS	130 / 250mm		209									
			10	SS	100 / 75mm		208									
207.9																

Piezometer installation consists of a 25mm diameter schedule 40PVC pipe with a 3.0m slotted screen.

Wet cave at 5.8 m below ground surface upon completion of drilling.

WATER LEVEL READINGS		
Date	Water Depth (m)	Elevation (m)
Nov 13, 2015	1.8	213.7
Nov 27, 2015	1.8	213.7



# RECORD OF BOREHOLE No C2

1 of 1

METRIC

G.W.P. \_\_\_\_\_ LOCATION \_\_\_\_\_ Coords: E:286056.3 N:4840123.7 ORIGINATED BY SD  
 DIST \_\_\_\_\_ HWY 410 BOREHOLE TYPE SOLID STEM AUGERS COMPILED BY SD  
 DATUM GEODETIC DATE 2015-10-13 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)							WATER CONTENT (%)		
								20 40 60 80 100							w <sub>p</sub> w w <sub>L</sub>		
								○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE							WATER CONTENT (%)		
216.5	GROUND SURFACE																
	100mm ASPHALTIC CONCRETE		1	SS	58		216							29 55 12 4			
	1600mm FILL, gravelly sand, some silt, trace clay, compact to very dense, brown, dry		2	SS	24												
214.8							215										
1.7	FILL, silty clay, trace to some sand, trace gravel, very stiff, grey, moist		3	SS	19												
214.4																	
2.1	SAND AND SILT, trace to some clay, trace gravel, compact to dense, brown, moist to wet (GLACIAL TILL)		4	SS	13		214										
213.1			5	SS	31		213										
3.4	SILTY CLAY, trace sand, trace gravel, containing cobbles and boulders, hard, grey, dry (GLACIAL TILL)		6	SS	126 / 250mm									auger grinding			
							212							0 9 64 27			
			7	SS	75												
							211							auger grinding			
			8	SS	100 / 50mm												
							210										
			9	SS	100 / 100mm												
	frequent sand seams		10	SS	167 / 225mm									0 34 40 26			
							209							auger grinding			
208.7			11	SS	100 / 25mm												
7.8																	

## END OF BOREHOLE

Borehole was dry and open upon completion of drilling.

# RECORD OF BOREHOLE No C3

1 of 1

METRIC

G.W.P. \_\_\_\_\_ LOCATION \_\_\_\_\_ Coords: E:286092.2 N:4840144.9 ORIGINATED BY SD  
 DIST \_\_\_\_\_ HWY 410 BOREHOLE TYPE SOLID STEM AUGERS COMPILED BY SD  
 DATUM GEODETIC DATE 2015-10-14 CHECKED BY RA

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)										WATER CONTENT (%)		
								20 40 60 80 100										10 20 30		
215.8	GROUND SURFACE																			
	80mm TOPSOIL		1	SS	25															
	FILL, silty clay, trace to some sand, trace gravel, very stiff to hard, brown, moist		2	SS	38															
	trace organics, stiff, grey		3	SS	14															
213.7			4	SS	20															
2.1	SAND AND SILT, trace clay, trace to some gravel, containing cobbles and boulders, compact to 2.9m, very dense below, brown, moist (GLACIAL TILL)		5	SS	82															
			6	SS	100 / 25mm															
211.5			7	SS	90															
4.3	SILTY CLAY, some sand, trace gravel, containing cobbles and boulders, hard, brown, dry (GLACIAL TILL)		8	SS	161 / 225mm															
210.1			9	SS	100 / 140mm															
5.7	SILTY SAND, trace clay, trace gravel, containing cobbles and boulders, very dense, grey, dry (GLACIAL TILL)		10	SS	100 / 125mm															
			11	SS	108															
208.0																				
7.8																				
207.7																				
8.1	SILTY CLAY, trace sand, trace gravel, hard, grey, dry (GLACIAL TILL)																			

## END OF BOREHOLE

Borehole was dry and open upon  
completion of drilling.



# RECORD OF BOREHOLE No C4

1 of 1

METRIC

G.W.P. \_\_\_\_\_ LOCATION \_\_\_\_\_ Coords: E:286129.5 N:4840180.9 ORIGINATED BY SD  
 DIST \_\_\_\_\_ HWY 410 BOREHOLE TYPE SOLID STEM AUGERS COMPILED BY SD  
 DATUM GEODETIC DATE 2015-10-14 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)							WATER CONTENT (%)		
								20 40 60 80 100							w <sub>p</sub> w w <sub>L</sub>		
							O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					WATER CONTENT (%)					
							20 40 60 80 100					10 20 30					
216.7	GROUND SURFACE																
	110mm TOPSOIL		1	SS	34												
	FILL, silty clay, trace sand to sandy, trace gravel, stiff to hard, brown to 2.9m, grey below, moist		2	SS	16												
			3	SS	14												
			4	SS	10												
			5	SS	12												
213.0	SILT AND SAND, trace clay, trace to some gravel, containing cobbles and boulders, dense to very dense, brown to 6.4m, grey below, moist (GLACIAL TILL)		6	SS	37												
3.7			7	SS	87												
			8	SS	100												
			9	SS	100 / 75mm												
			10	SS	100 / 75mm												
209.0																	

## END OF BOREHOLE

Piezometer installation consists of a  
25mm diameter schedule 40PVC pipe  
with a 3.0m slotted screen.

Wet cave at 4.7 m below ground  
surface upon completion of drilling.

## WATER LEVEL READINGS

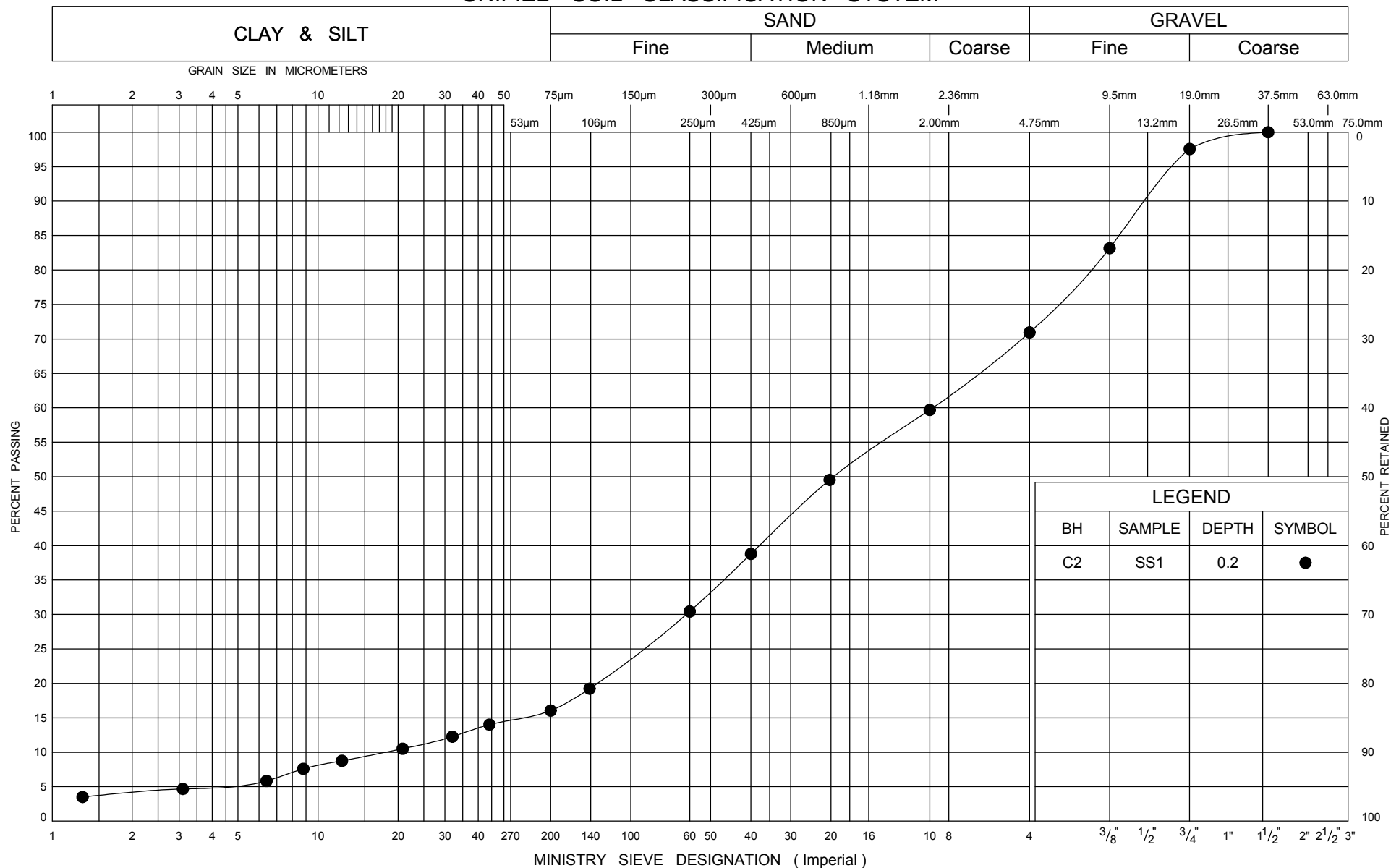
Date	Water Depth (m)	Elevation (m)
Nov 13, 2015	4.0	212.7
Dec 1, 2015	4.0	212.7

## **APPENDIX B**

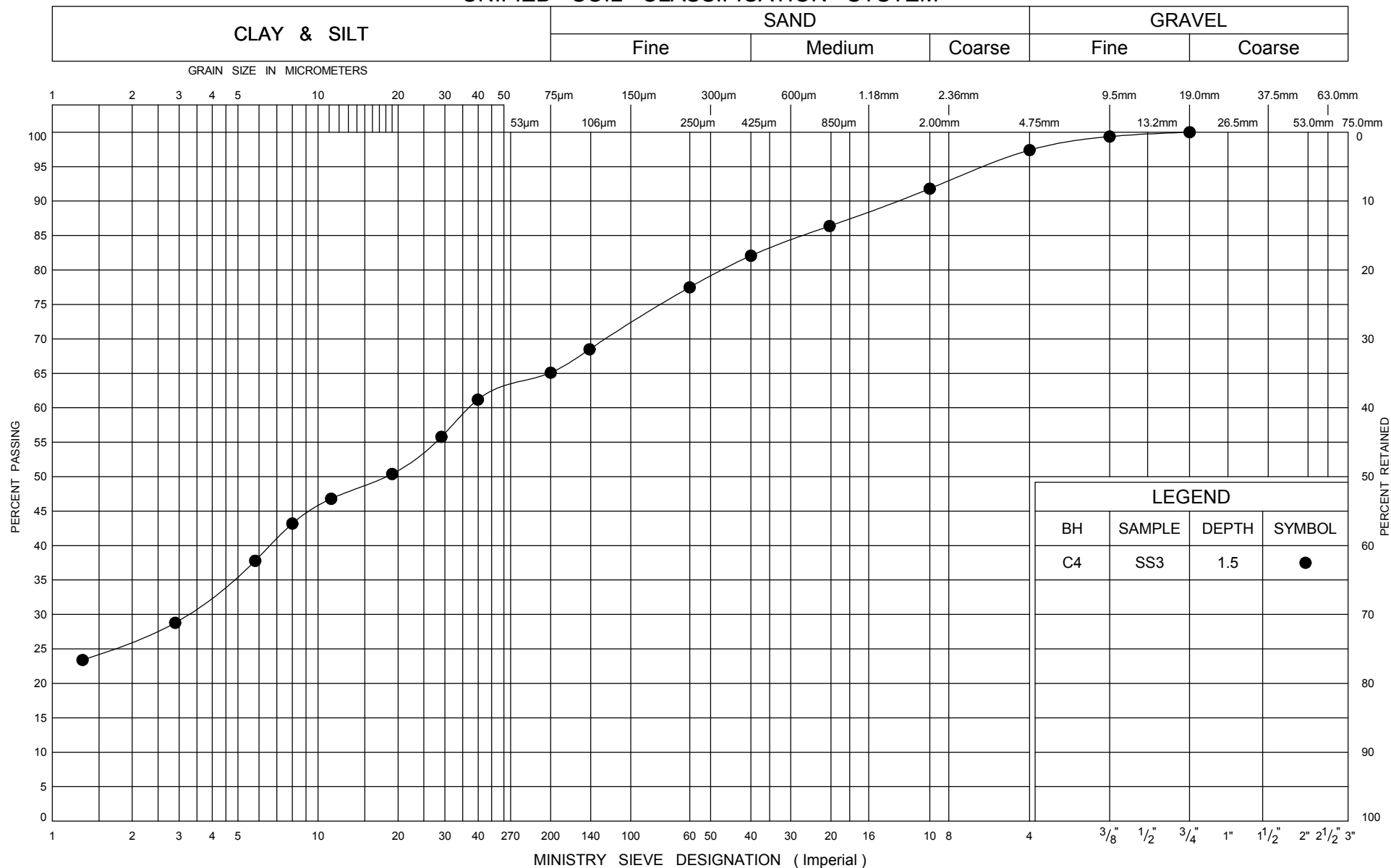
### **Laboratory Test Results**

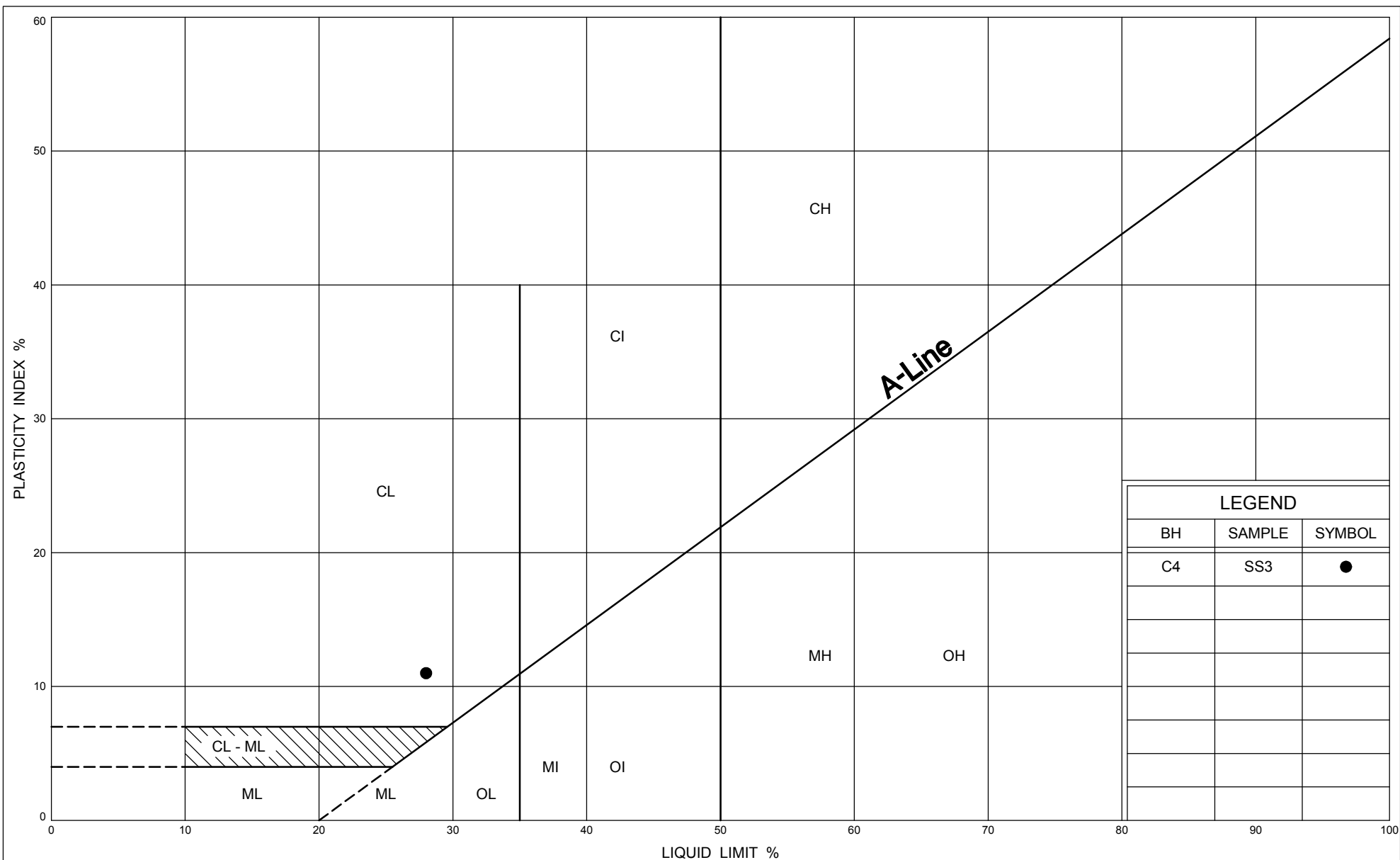


# UNIFIED SOIL CLASSIFICATION SYSTEM

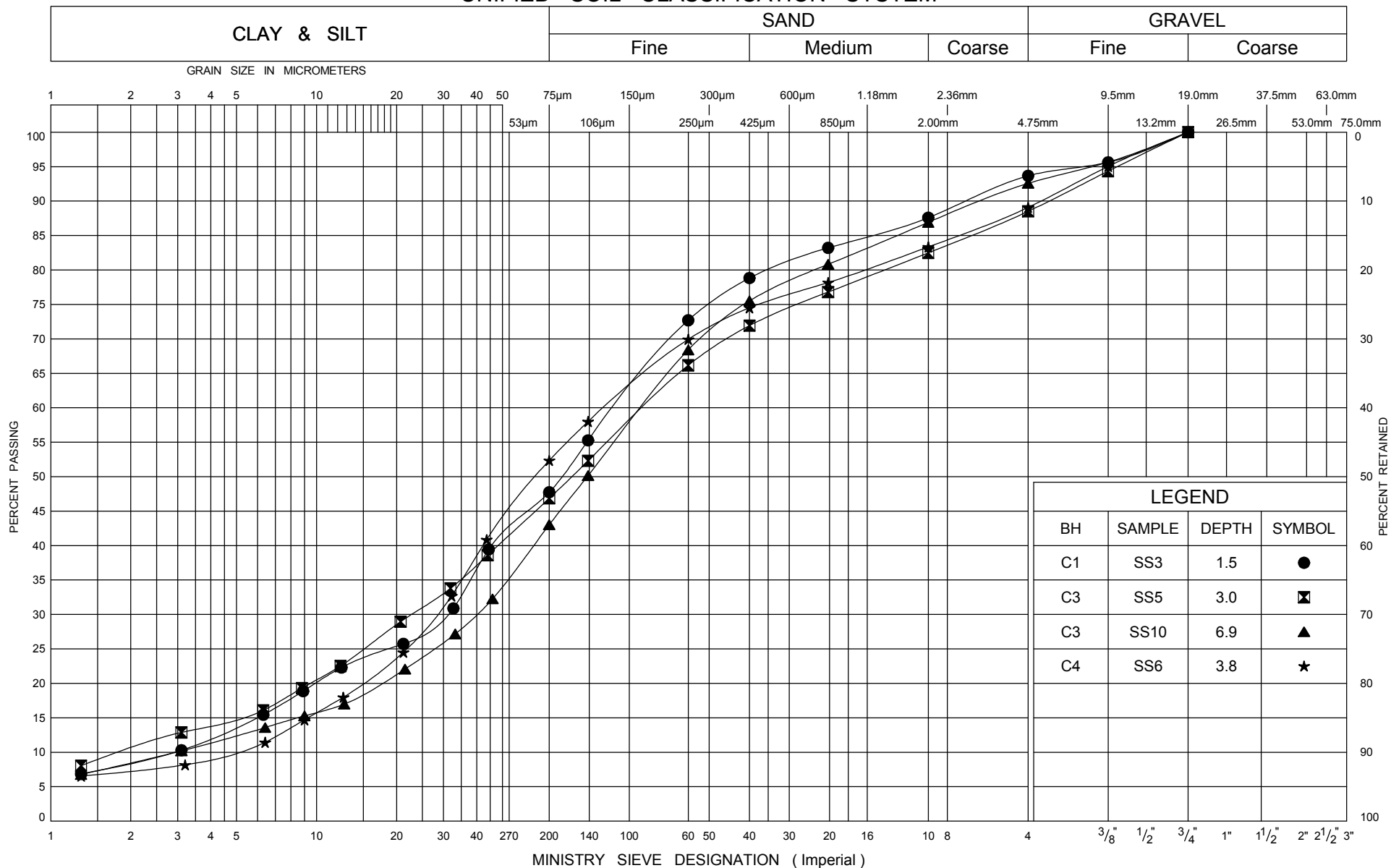


# UNIFIED SOIL CLASSIFICATION SYSTEM

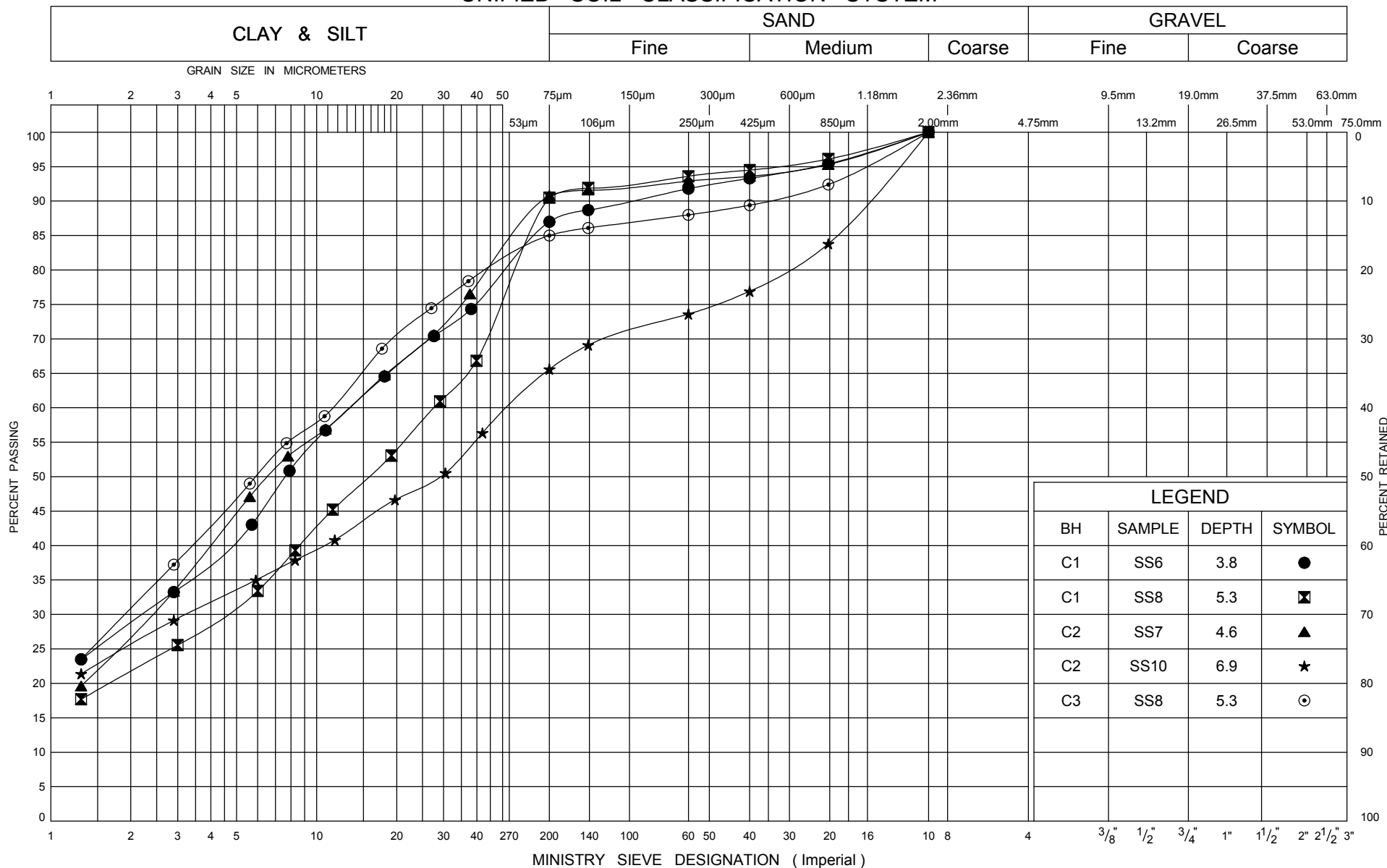




# UNIFIED SOIL CLASSIFICATION SYSTEM



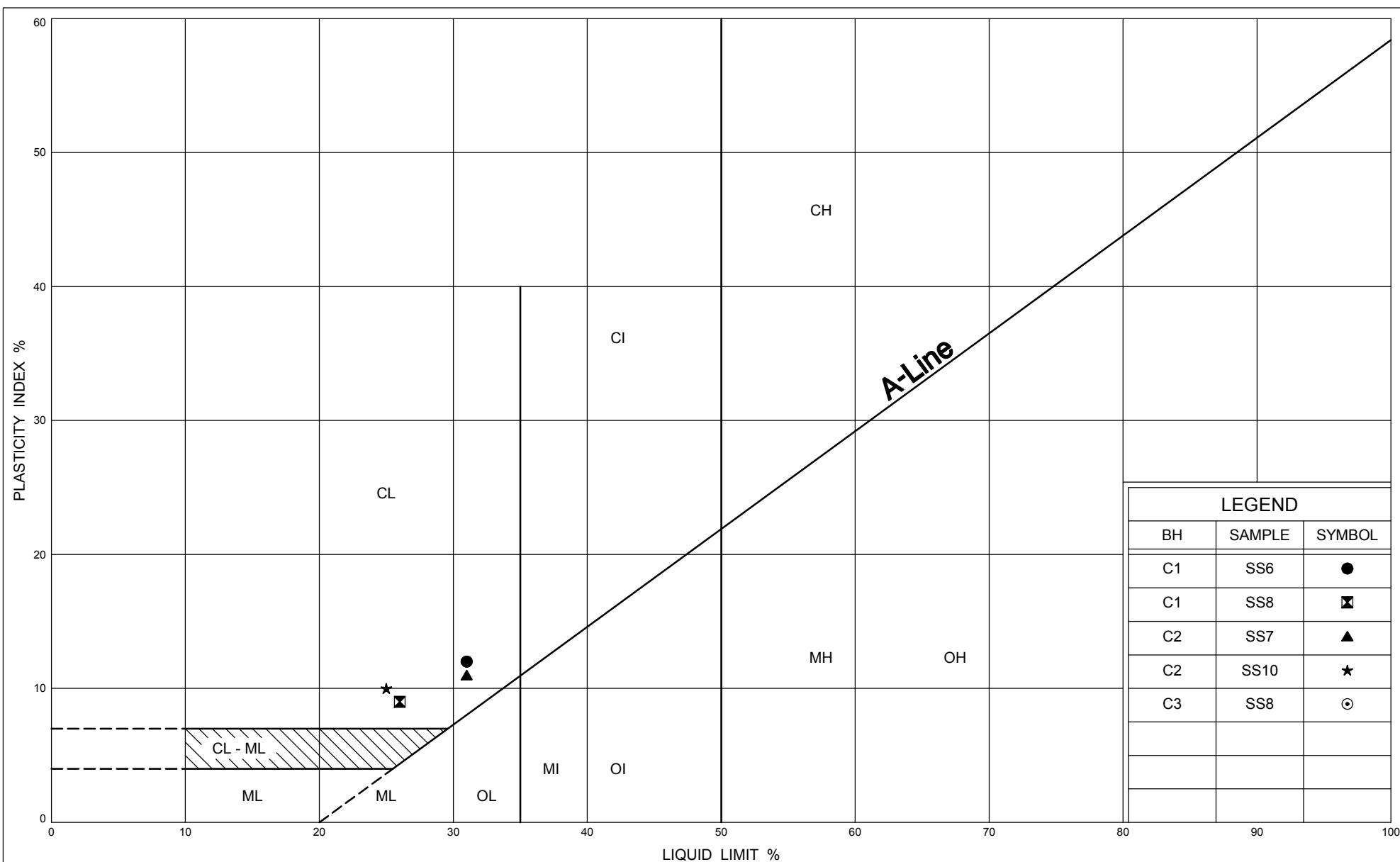
# UNIFIED SOIL CLASSIFICATION SYSTEM



## GRAIN SIZE DISTRIBUTION SILTY CLAY TILL

FIG No B5

G W P



Ministry of  
Transportation

# PLASTICITY CHART SILTY CLAY TILL

FIG No B6

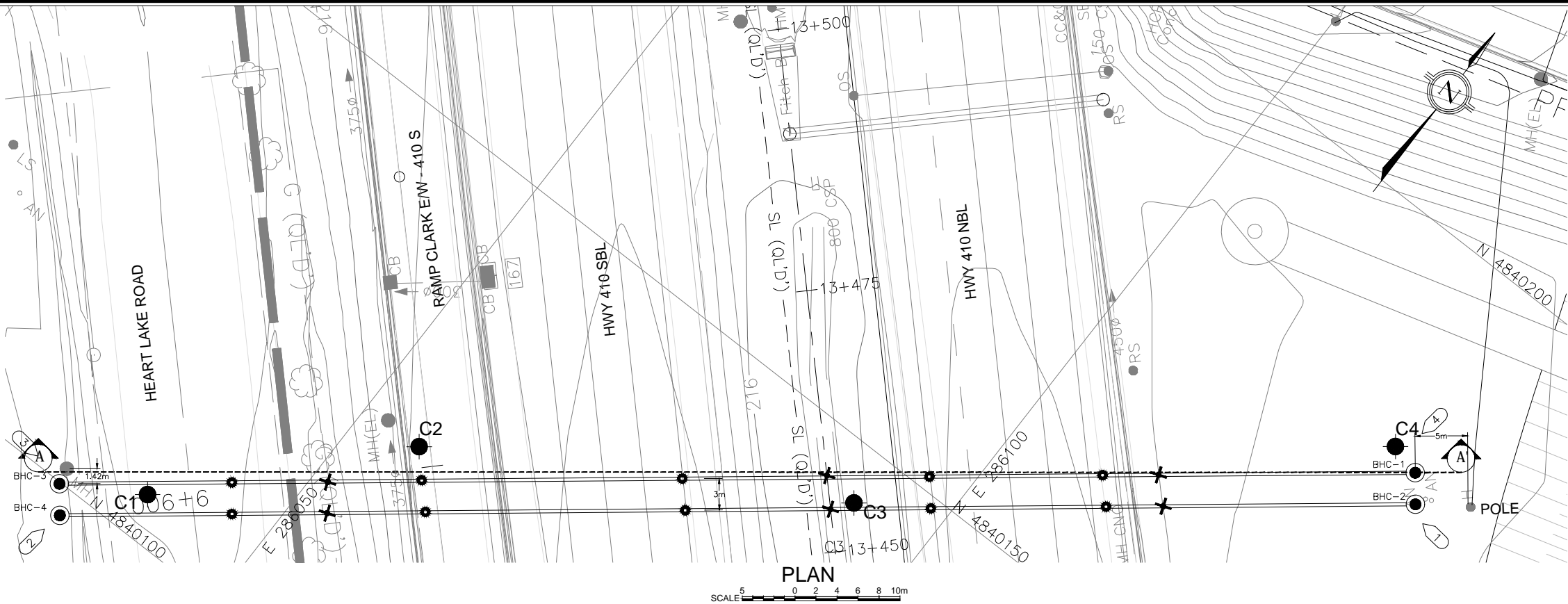
G W P



# **APPENDIX C**

## **Settlement Monitoring Programme**





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

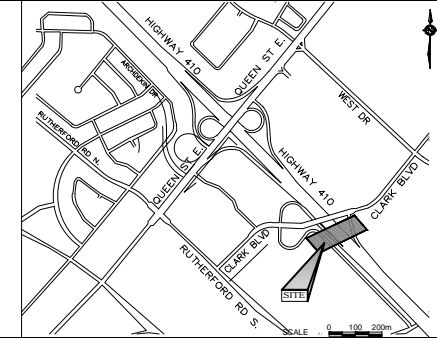
HYDRO ONE DUCT CROSSING,  
HWY 410 & CLARK BLVD BRAMPTON  
SETTLEMENT MONITORING PROGRAMME



SHEET  
---



**Terraprobe Inc.**  
Consulting Geotechnical & Environmental Engineering  
Construction Materials Engineering, Inspection & Testing  
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650



KEY PLAN

LEGEND

- BOREHOLE
- IN GROUND MONITORING POINT
- SURVEY MONITORING POINTS
- PAVED ROADWAYS

No	ELEV.	COORDINATES	
		NORTHING	EASTING
C1	215.5	4 840 104.2	286 038.7
C2	216.5	4 840 123.7	286 056.3
C3	215.8	4 840 144.9	286 092.2
C4	216.7	4 840 180.9	286 129.5

NOTE

THE PROPOSED DETAILS / WORKS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY AND MAY NOT BE CONSISTENT WITH FINAL DESIGN CONFIGURATION AS SHOWN ELSEWHERE IN THE CONTRACT DOCUMENTS.

REFERENCE

Drawings provided in digital format by NBM Engineering Inc. by email drawing files xNBH-H14-0073-HOB-Hwy 410 & Clarke Blvd -Bore Hole Locations ( For Tender) Rev0.0 received August 20, 2015 and Hwy410 (hwy401 - Queen St)\_Utilities@Queen+Clark\_topo received November 27, 2015

REVISIONS	DATE	BY	DESCRIPTION
HWY. 410	PROJECT No. 1-15-0536	DIST.	
SUBM'D.HA	CHKD. RA	DATE: JAN. 2016	SITE: --
DRAWN: KC	CHKD.	APPD: MT	DWG. 1

NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH TERRAPROBE INC. REPORT No. 1-15-0536, DATED JANUARY 2016.
- ALL MONITORING LOCATIONS SHOULD BE CONSIDERED APPROXIMATE AND MUST BE CONFIRMED BY THE CONTRACTOR IN CONSULTATION WITH THE CONTRACT ADMINSTRATOR, GEOTECHNICAL ENGINEER, PRIOR TO INSTALLATION/CONSTRUCTION AND MAY HAVE TO BE ADJUSTED IN THE FIELD TO SUIT LOCAL CONDITIONS/CONSTRAINTS.
- THE CONSULTANT SHALL RETAIN A SURVEYOR REGISTERED IN ONTARIO FOR ESTABLISHING AND SURVEYING THE MONITORING POINTS FOR THE DURATION OF CONSTRUCTION.
- ALL MONITORING INSTRUMENTS SHALL BE INSTALLED AT LEAST 7 DAYS PRIOR TO ANY EXCAVATION OR TUNNELLING TAKING PLACE.
- IN-GROUND MONITORING POINTS INSTALLED SHALL BE FOUNDED BELOW FROST PENETRATION DEPTH (1.2m), AT A DEPTH OF 1.5m BELOW EXISTING GRADE. SURFACE MONITORING POINTS ON PAVED ROADWAYS SHALL BE INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S INSTRUCTIONS.
- THE CONTRACTOR SHALL ESTABLISH TWO TEMPORARY BENCHMARKS OUTSIDE THE AREA OF CONSTRUCTION. THE CONTRACTOR SHALL SUBMIT THE PROPOSED BENCHMARK LOCATIONS TO THE CONTRACT ADMINISTRATOR FOR APPROVAL. PRIOR TO CONSTRUCTION ALL MONITORING POINTS SHALL BE SURVEYED FOR ELEVATION AND LOCATION TO A TOLERANCE OF NOT MORE THAN 2mm IN THE VERTICAL AND HORIZONTAL DIRECTION.
- DURING TUNNELING, ALL POINTS SHALL BE SURVEYED A MINIMUM OF 3 TIMES PER DAY.
- THE SPECIFIED SETTLEMENT REVIEW LEVEL IS 10mm AND THE SPECIFIED SETTLEMENT ALERT LEVEL IS 15mm RELATIVE TO THE BASELINE READING. IF SETTLEMENTS REACH THE REVIEW LEVEL THE CONTRACTOR SHALL PROVIDE A FORMAL PLAN TO ENSURE FURTHER SETTLEMENTS DO NOT OCCUR. IF SETTLEMENTS REACH THE ALERT LEVEL, THE CONTRACTOR SHALL SUSPEND TUNNELLING AND THE CONTRACTOR ADMINISTRATOR, OWNER AND MTO WILL HAVE THE AUTHORITY TO ORDER THE CONTRACTOR TO SUSPEND ALL TUNNELLING UNTIL AN APPROVED MITIGATION SOLUTION IS DEVELOPED.
- AFTER TUNNELLING HAS BEEN COMPLETED, THE CONTRACTOR SHALL SURVEY THE MONITORING POINTS ONCE PER DAY FOR 10 DAYS OR UNTIL DATA INDICATES THAT ALL MOVEMENTS HAVE ESSENTIALLY CEASED.
- WITHIN 24 HOURS OF COMPLETION OF ANY MEASUREMENT A COPY OF THE RESULTS SHALL BE MADE AVAILABLE TO THE CONTRACT ADMINISTRATOR, OWNER AND MTO.
- THE CONTRACTOR SHALL MAKE ALL ARRANGEMENTS FOR TRAFFIC CONTROL IN ACCORDANCE WITH ONTARIO TRAFFIC MANUAL BOOK 7.
- REMOVE ALL MONITORING POINTS ON COMPLETION OF SURVEY, SUBJECT TO APPROVAL FROM THE CONTRACT ADMINISTRATOR AND GEOTECHNICAL ENGINEER.

