100% Foundation Investigation and Design Report

Highway 401 Rail Tunnel
Kitchener Rail Corridor
Between Islington and Kipling Avenue
Toronto, ON

Project # TPB175141
Metrolinx Project No. 16-498
Geocres Number: 30M11-295

Prepared for:

Toronto Tunnel Partners
1004 Middlegate Road, Suite 1000, Mississauga, Ontario L4Y 1M4

24-Oct-19

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24-Oct-19

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Part A:

100% FOUNDATION INVESTIGATION REPORT
HIGHWAY 401 RAIL TUNNEL
KITCHENER CORRIDOR BETWEEN ISLINGTON AND KIPLING AVE.
CITY OF TORONTO

1.0 Introduction

Wood Environment & Infrastructure Solutions, a Division of Wood Canada Limited ("Wood") was retained by Toronto Tunnel Partners ("TTP" or the "Client") to conduct a supplemental geotechnical investigation for the proposed rail tunnel beneath Highway 401 between Kipling Avenue and Islington Avenue along the Kitchener Rail Corridor in the City of Toronto, Ontario.

The purpose of this investigation was to provide additional subsurface soil information within the proposed rail tunnel and along the proposed alignment of the retaining wall and based on this supplementary information together with previous investigations, to provide geotechnical recommendations pertaining to the design and construction of the proposed rail tunnel, site services and the retaining walls at west and east sides of the tunnel.

For the purpose of the supplemental geotechnical investigation planning, the proposed east and west retaining wall layout was provided by WSP Canada. This 90% geotechnical investigation and design report should be read in conjunction with supplementary geotechnical investigation work plan memo issued 26th January 2018.

As part of geotechnical investigation of the proposed Highway 401 tunnel in Kitchener rail corridor, subsurface investigations were carried out by or on behalf of the Ministry of Transportation Ontario (MTO) and Metrolinx. The subsurface information within the study area is listed below:

MTO GEOCRES, No. 30M11-065: Report titled "C.N.R. Overhead between Kipling Ave. and Islington Ave., Hwy. #401, District #6. W.J. 64-F-21, W.P. 239-60," by Department of Highways – Ontario, Materials and Research Division, Foundation Section, dated April 1964. Borehole 1 and 2 were drilled at the grade of Highway 401, through the backfill of the south side of the existing structure. Copies of these borehole reports are provided in Appendix E.

MTO GEOCRES, No. 30M11-066: Report titled "Foundation Investigation Report for Proposed Overhead Structures at the Intersection of C.N.R. and Hwy. 401, Twp. of Etobicoke, County of York, District No. 6, Toronto. W.J. 64-F-31, W.P. 239-60," by Department of Highways – Ontario, Materials and Research Division, Foundation Section, dated June 22, 1964. Boreholes 1 to 4 were drilled at the toe of the highway embankment on the east and west side of the location of the tunnel in 1964 and these borehole reports are provided in Appendix F.

Peto MacCallum LTD. (PML), Report No. 09TF014: Report titled "Preliminary Geotechnical Investigation, Islington Avenue to Kipling Avenue (M10.3 to 11.3), GO Transit Weston Rail Corridor" under GO Transit Tender No. ITC-2009-GT-008, dated March 31, 2010. In total twelve (12) boreholes (Borehole IK-1 to IK-12 and IK-14) were completed from west of Kipling Avenue to east of Islington Avenue. Copies of these borehole reports are provided in Appendix I.

Thurber Engineering Ltd. (TEL), Two Reports, both No. 19-1605-138: Reports titled "DRAFT Hydrogeology Assessment, Highway 401 & 409 Sewer Crossing, Georgetown South Project, Toronto, Ontario," dated August 2, 2013 and "Foundation Investigation and Design Report, Rail Grade Lowering at Existing Tunnel Crossing Highways 401 and 409 Between Kipling Avenue and

Islington Avenue, Toronto, Ontario," dated February 11, 2013. In total four (4) boreholes (designated Boreholes 12-01, 12-02, 12-03, and 12-04) were drilled from track grade, within the existing tunnel, approximately 50 meters apart. Copies of these borehole reports are provided in Appendix J.

Golder Associates Ltd., Report No. 13-1111-0035-1: Report titled "Geotechnical Data Report Rail Tunnel Beneath Highway 401 between Islington Avenue and Kipling Avenue, Kitchener Rail Corridor", dated June 17, 2016. In total, 16 boreholes (designated BH-2013-1 to 4 and BH2013-3A drilled on Highway 401 near the existing tunnel structure, adjacent to the proposed rail tunnel alignment and BH2014-R1 to R9, R12, and BH2015-R1 drilled along the existing railway from Kipling Ave. to west side of Hwy 401 lanes and from east side of 401 to Islington Ave.) were completed, to assess the extent and nature of underlain soil layers. A copy of these borehole reports is provided in Appendix K.

This 90% foundation investigation and design report is prepared based on the results of the previous investigations and the supplementary geotechnical investigation as outlined below. The scope of the fieldwork for this supplemental geotechnical investigation included eight (8) boreholes to depths ranging from 15.8 m to 23.9 m below existing grade in the footprint of the proposed retaining wall. Four additional Cone Penetration Tests (CPT's) have been completed as part of the supplemental geotechnical investigation program. Additional sampling and classification of the fill and native soils will be carried out during the installation of the vertical inclinometers and vibrating wire piezometers, as part of the Ground Monitoring Instrument Installation program. The information and recommendations provided in this report are for the purpose of ongoing design of the proposed structures.

This report contains the findings of the previous factual information and Wood's supplemental geotechnical investigation, together with recommendations and comments. The recommendations and comments are based on factual information at the test locations and intended primarily for use by design engineers. The number of boreholes may not be sufficient to determine all of the factors that may affect construction methods and costs. Subsurface and groundwater conditions between and beyond the boreholes may differ from those encountered at the borehole locations, and conditions may become apparent during construction that could not be detected or anticipated at the time of the site investigation.

The anticipated construction conditions are also discussed, but only to the extent that they may influence the design decisions. The feasible construction methods, however, express our opinion and are not intended to direct contractors on how they carry out construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all factors that may have effect upon construction.

This report has been prepared with the assumption that the design will be in accordance with good engineering practices, applicable regulations of jurisdictional authorities, and applicable standards and regulations. Further, the recommendations and opinions in this report are applicable only to the proposed project. Hydrogeological or environmental considerations will be provided under separate cover.

The scope of this report is strictly limited to the geotechnical and foundation aspects of the proposed works. As part of the Design-Build project, there will be ongoing liaison with other members of the Design-Build team during the construction phase of this project to confirm that the recommendations in this report have been interpreted and implemented as intended.

2.0 Investigation Procedure

2.1 Supplementary Foundation Investigation and Monitoring Well Installation

A total of eight (8) boreholes and four (4) Cone Penetration Testing (CPT) were advanced during the supplementary investigation program. The drilling investigation was performed by Geo-Environmental Drilling Inc. utilizing Hollow Stem Augers under the direct technical supervision of Wood personnel. The CPT investigation was performed by ConeTec Investigation Ltd. under direct supervision of Wood personnel.

Prior to drilling, utility locates were carried out to obtain clearances for existing underground utilities. Wood coordinated the utility locates with TTP and several public and private utilities locate companies prior to advancing the boreholes. The field work for the investigation was carried out between March 12 and July 24, 2018. Boreholes were advanced at 8 locations (BH2017-01 to BH2017-08) to depths ranging from 15.8 m to 23.9 m below ground surface (bgs). Three (3) boreholes were instrumented with monitoring wells.

The CPT soundings were advanced using a 30-ton thrust capacity truck rig and were advanced until the maximum safe capacity of the equipment was reached (i.e. refusal). The CPT's were advanced to depths ranging between 4.5 m to 14.1 m bgs. Early refusal was typically confirmed by advancing a secondary CPT sounding at a closely offset location. Prior to advancing the CPT soundings, pavement surface was cored using a nominal 100 mm diameter diamond core drill.

The general site location, and the borehole, test pit and CPT locations are shown on Drawing 1.

All of the boreholes were located and marked by our geotechnical engineering staff in consultation with TTP. Encroachment permits for boreholes located on MTO's right of way was provided by TTP. The elevations and MTM NAD 83 coordinates at each borehole location were recorded by TTP using a Global Positioning System (GPS) device, Leica Viva GS16 capable of decimetre accuracy. The project area was surveyed by TTP and the base plan was provided to Wood. The ground surface elevations, MTM NAD83 coordinates, and detailed subsurface conditions encountered in the boreholes are provided on the borehole logs attached in Appendix A.

The boreholes were advanced utilizing a track-mounted CME-75 drill rig equipped with 150 mm outside diameter hollow stem augers and conventional soil sampling tools. The boreholes were not advanced using mud rotary drilling, as Wood's supplemental investigation also included obtaining environmental samples for the chemical analyses of a limited number of soil samples and the use of mud in the drilling fluid would contaminate the samples. Soil samples were obtained at 0.75, 1.5 and 3.0 m intervals via the Standard Penetration Test method, ASTM D1586.

The results of the penetration tests are reported as 'N' values on each borehole log at the corresponding depths and have been used to infer the conditions of the subsurface soils. The soil stratigraphy within each borehole was visually examined and classified at the time of drilling in accordance with the M.T.C Soil Classification Manual (1980). Soil samples were collected from each borehole at selected depths and retained in sealed plastic bags for further detailed examination and laboratory testing.

Soil samples were transported to Wood's Soil Laboratory in Cambridge for further review and laboratory testing. A geotechnical laboratory testing program was carried out on selected soil and from the boreholes. The purpose of the geotechnical laboratory tests was to determine engineering properties of subsoils for use in design and analysis. Laboratory testing consisted of:

- 170 moisture content measurements (ASTM D2216);
- 37 sieve/hydrometer analysis (ASTM D421 and ASTM D422);
- 6 Atterberg Limits (ASTM D4318-10 and ASTM D2487-11);

Geotechnical laboratory test results are included in Appendix B and are summarized on the corresponding Record of Borehole sheets, attached.

Cone Penetration Tests were performed with a portable integrated electronic piezocone manufactured by ConeTec. The piezocone used was a compression model cone with a cross sectional area of 15 cm^2 and a 225 cm^2 friction sleeve. The piezocone dimensions, sensor characteristics and operating procedure were in accordance with ASTM D5778. ConeTec's portable CPT system takes data readings of tip resistance (q_t), sleeve friction (f_s) and dynamic pore pressure (u_t) as the cone penetrates the sediments. The combination of q_t and friction ratio ($R_f = f_S/q_t$) is used to differentiate between Soil Behaviour Types (SBT), as proposed by Robertson (1990).

Pore pressure dissipation tests were also carried in conjunction with the CPT soundings. The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the cone penetration was stopped to allow the dynamic pore pressures to stabilize. ConeTec provided Wood with raw data in excel format for further data interpretation. For additional information regarding CPT data interpretation, refer to ConeTec's CPT Investigation report and Interpreted Soil Parameters provided in Appendix D.

2.2 Monitoring Well Construction

As part of the supplementary geotechnical investigation, monitoring wells were constructed in three boreholes with a 51-mm diameter (2-inch) PVC riser pipe and 10-slot screen 1.5 and 3.0 m in length, respectively. The well screens were set below the groundwater table in a representative stratigraphic unit based on Wood's observations and interpretation of the stratigraphy during borehole drilling. The monitoring wells were completed with lockable J-plug caps and stick-up protective casings embedded in concrete at ground surface. All monitoring wells were tagged and labelled in accordance with Ontario Regulation 903 (Wells) by the well drillers, Water well installation record was submitted to the Ontario Ministry of Environment and Climate Change

(MOECC) by the drilling subcontractors. Well construction details are presented on the corresponding borehole logs attached and summarized in Table 1 below. Monitoring well locations are shown in Drawing 1.

Table 1: Summary of Monitoring Well Construction Details

BH/MW#	Diameter (mm)	Screen Interval (mbgs)	Screened Strata
2017-03	50	4.0 – 5.5	Fill/Sandy Silt Till
2017-05	50	8.2 - 11.2	Sand & Silt Till
2017-08	50	9.75 - 11.2	Sand & Silt

2.3 Groundwater Level Monitoring and Single Well Response Tests (SWRT's)

During the field investigation, groundwater levels were measured upon completion of borehole drilling/installation of monitoring wells. Two additional suites of groundwater monitoring were conducted as part of the groundwater monitoring program. The first round of groundwater level monitoring was conducted on March 26, 2018 prior to hydrogeological testing. The second round of groundwater level monitoring was conducted on April 19, 2018 prior to groundwater sampling. The results of the groundwater level readings are shown on the corresponding borehole logs attached and summarized in table 2 below.

Table 2: Summary of Groundwater Level Observations in Monitoring Wells

BH/MW#	Ground Surface Elevation (m)	Groundwater Level on March 26, 2018		Groundwater Level on April 19, 2018	
		Measured depth to Water (m)	Groundwater Elevation (m)	Measured depth to Water (m)	Groundwater Elevation (m)
2017-03	158.3	Dry	-	3.40	154.9
2017-05	158.6	7.81	150.79	7.53	151.1
2017-08	154.5	4.19	150.31	3.83	150.7

Fluctuations in groundwater table elevations should be expected in response to seasonal conditions and extreme weather events.

The response data from the single well response tests (SWRT's) was analyzed using Aqtesolv software, an advanced aquifer test analysis software, to estimate the hydraulic conductivity of the aquifer(s). The hydrogeological testing results and implications will be presented by Wood under a separate cover.

2.4 Analytical Soil Sampling and Testing

All laboratory chemical analyses were conducted by ALS Laboratory, accredited by the Canadian Association for Laboratory Accreditation Inc. (CALA) in accordance with ISO/IEC 17025:1999 – General Requirements for the Competence of Testing and Calibration Laboratories for the tested parameters set out in the Soil, Ground Water and Sediment Standards document (MOE, 15 April 2011).

The soil analytical sampling and testing programs are provided in Table 3, below. The laboratory certificates of analysis are presented in Appendix C.

Twenty soil samples from the proposed area of construction were submitted for analysis of petroleum hydrocarbon compounds fractions 1 to 4 (PHC F1 – F4), volatile organic compounds (VOCs), ad metals and inorganics including Sodium Adsorption Ratio (SAR) and Electrical Conductivity (EC). Ten of twenty samples were also analyzed for polycyclic aromatic hydrocarbons (PAHs) and/or polychlorinated biphenyl (PCBs). The soil samples were delivered to ALS under chain of custody documentation.

The shallow soil samples selected for analysis were collected from within the fill (sand and silt) or sand and silt material at a depth of approximately 0.8 to 6.2 m bgs. The deep soil samples selected for analysis were collected from within the fill (sand and silt) or native material at a depth of approximately 4.0 to 8.5 m bgs. The samples selected for analysis are presented in Table 3 below.

Table 3: Summary of Soil Samples Submitted for Chemical Analysis

Borehole No.	Sample ID	Sample Description
2017-01	SS2	Fill – sandy silt
2017-01	SS7	Sandy Silt Till
2017-02	SS1	Fill – sand & gravel
2017-02	SS4/SS5	Fill – silt
2017-02	SS7	Sandy Silt Till
2017-03 & DUP-1	SS1/SS2	Fill – sandy & silt
2017-03	SS7	Sandy Silt Till
2017-04	SS2	Fill – sandy silt to silty sand
2017-04	SS7	Sandy Silt Till
2017-05	SS2	Fill – sandy silt to silty sand
2017-05 & DUP-2	SS6/SS7	Fill – sandy silt to silty sand
2017-06	SS1/SS2	Fill – sandy silt
2017-06	SS9	Clayey Sandy Silt Till
2017-07	SS8/SS9	Fill – sandy silt to silty sand
2017-07	SS12	Sandy Silt to Silty Sand Till
2017-08	SS1/SS2	Fill (upper) – sandy silt
2017-08	SS4/SS5	Fill (lower) – sandy silt
2017-08	SS7/SS8	Sandy Silt Till

2.5 Groundwater Sampling

Prior to sampling, monitoring wells BH/MW 2017-03, BH/MW 2017-05 and BH/MW 2017-08 were micro purged using a peristaltic pump prior to collecting the groundwater samples. Prior to collecting groundwater samples, three well volumes were purged from the well to remove any stagnant water. The groundwater sample was collected using low-flow protocol on April 19, 2018 and submitted to ALS laboratory for analysis of PHC F1-F4, VOCs, sVOCs, metals and inorganics, general chemistry (total sulfides, magnesium, ammonium, nitrates, calcium, sodium, potassium, sulfate and chloride) and the City of Toronto Sanitary and Storm Sewer By-Law.

A summary of the groundwater samples submitted for chemical analysis is provided in Table 4 below.

Table 4: Summary of Groundwater Samples Submitted for Chemical Analysis

Groundwater Sample	Analyzed Parameters	Groundwater Sample Submission Rationale
BH/MW 2017-03	PHC F1-F4, VOCs, sVOCs and metals / inorganics, general chemistry, City of Toronto Sanitary/Storm Sewer By-Law	General overview of groundwater quality
BH/MW 2017-05 and DUP-1	PHC F1-F4, VOCs, sVOCs and metals / inorganics, general chemistry, City of Toronto Sanitary/Storm Sewer By-Law	General overview of groundwater quality
BH/MW 2017-08	PHC F1-F4, VOCs, sVOCs and metals / inorganics, general chemistry, City of Toronto Sanitary/Storm Sewer By-Law	General overview of groundwater quality

2.6 Corrosivity Analysis

Sixteen (16) selected soil samples from borehole BH2017-01, BH2017-04, BH2017-07 and BH2017-08 were submitted for corrosivity analysis (pH, moisture, redox potential, resistivity, chloride, sulphide and sulphate). The samples were selected in direct consultation with WSP and TTP. A summary of the results is included in Section 4.1 below.

2.7 Soil and Groundwater Waste

The soil cuttings from the boreholes were collected and placed in several metal drums. A composite sample of the soils was collected from the drums for a TCLP analysis and was determined to be non-hazardous. The soil and ground water wastes were removed by Ground Force Environmental Inc on a daily basis. The laboratory certificates of analysis are presented in Appendix C.

TORONTO TUNNEL PARTNERS TTP

3.0 Subsurface Conditions

3.1 Regional Geology

This project area is located in the Peel Plain physiographic region, as delineated in The Physiography of Southern Ontario. The surface of the Peel Plain is characterized by level to gently rolling topography, with a consistent, gradual slope toward Lake Ontario. The Plain is made up of deep deposits of dense, limestone and shale imbued Young Till derived from the Late Wisconsin glaciation in North America, surrounded with Modern River deposits including sand, silt, minor gravel and organic materials. Generally, the Young Till consists of clayey silt till and sandy silt till, all covered by a shallow layer of clay sediment as noted in the geological literature of the area (Czurda and Quigley 1973).

3.2 Subsurface Conditions Adjacent to Railway Corridor

Based on the soil conditions encountered in the railway corridor boreholes (BH2017-01 to BH2017-08), the soil profile generally consists of loose/firm to very dense/hard variable fill underlain by compact to very dense glacial till consisting of clay, sand and silt matrix. Cobbles encountered during the drilling within the glacial till required extended drilling effort to penetrate due to split spoon refusal. The glacial till, in turn, overlie granular soils comprised of sand, gravel and silt. The granular soils beneath the till cap are groundwater bearing as confirmed by select borehole logs from the field investigation.

Underlying the overburden soils is the interbedded shale and limestone of the Dundas formation. Shale bedrock was encountered at depths ranging from 20.7 m to 23.5 m bgs.

The stratigraphic units and groundwater conditions are discussed in the following sections and are presented on the corresponding borehole logs attached in Appendix A. Soil stratigraphy profiles along the rail corridor are presented on Drawings 2A and 2B.

The following summary is intended to assist the designers of the project with an understanding of the anticipated soil conditions within the proposed project area. However, it should be highlighted that the soil and groundwater conditions may vary between and beyond the borehole locations.

3.2.1 Fill

Fill materials consisting of clay, sand, silt and minor gravels were encountered at the surface of all the boreholes. The fill extended to depths ranging from surface to 7.8 m below existing ground surface. For cohesive fill materials, SPT ‘N’ values ranging from 8 to 49 blows per 300 mm of penetration was recorded indicating firm to hard consistency. For cohesionless fill materials, SPT ‘N’ values ranging from 7 to 40 blows per 300 mm of penetration was recorded indicating loose to dense relative density. The moisture content of samples of fill ranged between 9% and 21%.

CPT sounding at CPT-18-02 is interpreted to comprise soft clays and/or organic soils at a depth between 5.0 and 6.0 m bgs. Based on CPT data, the material behavior was mainly classified as clays and organic soils. This soft/loose deposit is inferred to be encountered at the interface of embankment fill – foundation soils.

Eight (8) grain size analysis were carried out on selected samples from all the boreholes. The laboratory testing results show a grain size distribution consisting of 0% to 33% gravel, 22% to 50% sand, 19% to 52% silt and 10% to 25% clay. These results demonstrate the variability of this layer.

One (1) Atterberg Limits test was also conducted on selected sample from BH2017-02. The laboratory testing result indicated Liquid Limit of 25, Plastic Limit of 16 and Plasticity Index of 9. The results of the Atterberg Limits tests indicate that the deposit can be described as a clay of low plasticity.

A summary of the fill thickness, average SPT "N" values and average moisture contents is included in Table 5 below:

Table 5: Summary of Fill Thickness, Average SPT N Values and Average Moisture Content

BH #	Approx. Elevation (m)	Fill Material	Thickness (m)	Approx. Bottom Elevation (m)	Average SPT N Values*	Average Moisture Content (%)	Underlying Material
2017-01	158.7	Clayey Silt with Sand	4.1	154.6	17	13.0	Clayey Silt with Sand Till
2017-02	158.4	Clayey Sand with gravel/silt / Clayey Silt with Sand	4.3	154.1	18	13.0	Clayey Silt with Sand Till
2017-03	158.4	Sand & Gravel / Clayey Silt with Sand	4.1	154.3	26	13.0	Clayey Silt with Sand Till
2017-04	158.3	Silty Sandy Gravel	3.7	154.6	13	14.0	Clayey Silt with Sand Till
2017-05	158.6	Gravelly Sand mixed with Clayey Silt	7.8	150.8	13	15.0	Sand & Silt Till
2017-06	156.2	Clayey Silt with Sand	5.3	151.0	17	13.0	Sandy Silt with Clay Till
2017-07	159.8	Clayey Silt with Sand	8.7	151.2	16	13.0	Sand and Silt Till
2017-08	154.5	Clayey Silt with Sand	3.7	150.8	13	14.0	Clayey Silt with Sand Till
	Min		**3.7**	**150.8**	**13**	**13.0**	
	Max		**8.7**	**154.6**	**26**	**15.0**	

3.2.2 Glacial Till

Glacial till deposits comprising sand, silt and clay mixtures were encountered in all the boreholes below the highway embankment fills. All the boreholes were terminated in the till deposit with the exception of BH2017-05, BH2017-07 and 2017-08. The till deposit was encountered at depths between 3.7 m and 8.7 m below ground surface (between Elev. 150.8 m and 154.6 m). The till was determined to be compact to very dense as indicated by the SPT 'N' values that ranged from 11 blows to over 50 blows per 300 mm of penetration. Split spoon refusal and hard/very dense, slow drilling conditions were noted in several boreholes, which was inferred to be due to cobbles within the till. Cobbles and boulders should be expected to be present within the till deposit. The moisture content of the samples of till ranged between 6% and 17%.

Twenty-two (22) grain size analyses were conducted on selected samples from this zone and the results are summarized in Table 6 below:

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Table 6: Glacial Till Soil Gradations Summary

Borehole # / Sample, Depth (m)	Percentage by Weight			
	Gravel	Sand	Silt	Clay
BH2017-01/SS9,6.1	8	30	47	15
BH2017-01/SS14&15,9.9	7	35	43	15
BH2017-02/SS7,4.6	6	33	46	15
BH2017-02/SS12,8.4	7	35	51	7
BH2017-03/SS11,7.6	6	30	45	19
BH2017-03/SS12,8.4	3	30	50	17
BH2017-03/SS15,10.7	1	18	59	22
BH2017-04/SS6&7,3.8	3	24	56	17
BH2017-04/SS10,6.8	3	22	43	32
BH2017-04/SS14&15	14	34	42	10
BH2017-05/SS11,7.6	7	41	40	12
BH2017-05/SS12,8.4	5	33	54	8
BH2017-05/SS14&15,9.9	15	36	42	7
BH2017-05/SS22&23,19.8	2	19	66	13
BH2017-06/SS8,5.3	4	27	50	19
BH2017-06/SS10&11,6.8	9	36	48	7
BH2017-06/SS17,15.2	5	69	24	2
BH2017-07/SS14&15,9.9	6	31	52	11
BH2017-07/SS20,18.2	11	26	58	5
BH2017-08/SS6&7,3.8	6	26	50	18
BH2017-08/SS16&17,11.4	10	23	62	5
BH2017-08/SS21&22,18.2	8	19	68	5

Five (5) Atterberg limits tests were also conducted on selected samples from this zone and are summarized in Table 7 below:

Table 7: Glacial Till Atterberg Limits Summary

Borehole # / Sample, Depth (m)	Atterberg Limits Tests			
	Liquid Limit	Plastic Limit	Plasticity Index	Soil Class
BH2017-06/SS8,5.3	21	13	8	CL
BH2017-01 /SS9, 6.1	20	13	7	ML
BH2017-03/SS12,8.4	18	12	6	ML
BH2017-04/SS10,6.8	27	15	12	CL
BH2017-02/SS9,6.1	19	13	6	ML

Within the till unit, granular soils comprised of silty fine to fine and medium sand and gravel were encountered at BH2017-02, BH2017-05, BH2017-06, BH2017-07 and BH2018-08. These granular soils may be water bearing as observed during the drilling program. Frequent heaving conditions were encountered during drilling due to confined hydrostatic pressure from groundwater attributed to granular soils beneath the till unit.

This layer was encountered at depths between 9.6 m and 19.4 m below ground surface (between Elev. 139.0 m and 144.9 m) with thickness varying from 1.1 to 4.1 m. The moisture content of the samples from this stratum ranged between 8% and 19%.

Four (4) grain size analyses were conducted on selected samples from this zone and the results are summarized in Table 8 below.

Table 8: Granular Soil Gradations Summary

Borehole # / Sample, Depth (m)	Soil Percentage by Weight			
	Gravel	Sand	Silt	Clay
BH2017-02/SS21,19.8	13	54	30	3
BH2017-06/SS17,15.2	5	69	24	2
BH2017-07/SS18,15.2	38	52	9	1
BH2017-08/SS14,9.9	9	49	40	2

3.2.3 Shale Bedrock

Shale Bedrock from the Georgian Bay formation was encountered below the glacial till. The shale bedrock was encountered at depths between 20.7 and 23.5 m below ground surface (between Elev. 133.8 m and 138.8 m). Boreholes BH2017-05, BH2017-07 and BH2017-08 were terminated in the bedrock unit. Split spoon refusal and very dense, slow drilling conditions were noted in the boreholes within the shale unit. Although, the SPT ‘N’ values indicate a very stiff unit, the shale is considered a weak and friable bedrock.

3.3 Subsurface Conditions from Previous Investigation – 401 Highway Corridor

Golder Associates Ltd. conducted a geotechnical investigation and laboratory testing program from 2013 to 2016, the results of which are presented in the Geotechnical Data Report, Rail Tunnel Beneath Highway 401 Between Islington Avenue and Kipling Avenue, Kitchener Rail Corridor, Toronto, Ontario, Report No. 13-1111-0035-1, dated June 17, 2016. Subsequently, Golder Associates Ltd. completed cone penetrometer testing (CPT) at selected locations along the proposed tunnel alignment. The recommendations provided in this foundation investigation and design report are based on the Data Report and subsequent CPT results reported by Golder. Wood has completed five (5) Cone Penetration Testing along the proposed tunnel alignment. Interpretation and discussion on these test results will be provided in the 90% submission.

In general, the subsurface conditions in the area of the proposed rail tunnel consist of asphalt or concrete pavement underlain by non-cohesive granular fills and cohesive fills associated with the Highway 401 and Highway 409 embankments. Clayey silt was encountered below the fill at one borehole location. The fills and clayey silt are underlain by glacial till comprised of clayey silt till, which grades to a sand and silt till. Cobbles and boulders are expected to be present within the glacial tills. Table 9 shows the soil strata in the new tunnel area, according to the previous borehole logs.

The approximate borehole locations, ground surface elevations and interpreted stratigraphic conditions for the new tunnel sections, based on the previous investigations at the site, are presented on Drawing 3.

3.3.1 Asphalt

Asphalt or Portland cement concrete pavement was present at the surface of the boreholes drilled within the travel areas of the Highway 401 and Highway 409 Right of Ways. The thickness of the asphalt pavement varied from 120 mm to 400 mm; the Portland cement concrete pavement was 430 mm, as measured at one borehole location. Topsoil was present at the test pit locations advanced along the landscaped embankment slopes. The thickness of the topsoil was 100 mm. All boreholes drilled through the travel lanes of the highways encountered pavement granular fill materials below the asphalt or Portland cement concrete pavements.

3.3.2 Fill

Embankment fill was encountered below the pavement granular fills at all the borehole locations. The embankment fills generally consisted of non-cohesive granular fills comprised of sand and gravel overlying cohesive clayey silt. The surface of the non-cohesive granular embankment fill was encountered at depths between 0.8 m to 1.2 m, corresponding to Elev. 162.6 m to 160.8 m. The thickness of the granular embankment fill varied from 2.4 m to 5.5 m. SPT "N" values recorded within the granular embankment fill during the borehole investigation varied from 6 blows to 40 blows per 0.3 m of penetration, indicating a loose to dense state of compactness. Measured moisture contents ranged from 8% to 12%.

The granular embankment fill was fully penetrated at depths between 3.5 m and 6.7 m below ground surface (between Elev. 159.6 m and 156.4 m) and was underlain by cohesive fill generally comprised of clayey silt with sand, and trace to some gravel. The Atterberg limit test results indicate the cohesive fill is low plasticity. SPT N values recorded in the cohesive embankment fill ranged from 2 blows to 18 blows per 0.3 m of penetration. In situ vane tests carried out in the cohesive fill measured undrained shear strengths between 45 kPa and 55 kPa. The SPT and vane test results indicate the consistency of the cohesive fill ranged from firm to very stiff. Measured moisture contents ranged from 7% to 27%. The base of the cohesive embankment fill extended to depths between 6.9 m and 9.0 m below ground surface (between Elev. 155.5 m and 154.1 m). The thickness of the cohesive embankment fill typically ranged from about 2.3 m to 4.8 m at the borehole locations.

3.3.3 Glacial Till

Glacial till deposits were encountered below the embankment fills in all the boreholes. The glacial tills generally consisted of cohesive clayey silt till underlain by non-cohesive silt to sand and silt till. In the boreholes advanced along the highway grade, the clayey silt till was encountered at depths between 6.9 m and 9.0 m below the highway surface (between Elev. 154.1 m and 155.0 m). The thickness of the clayey silt till was about 3.6 m to 5.3 m at the borehole locations. SPT "N" values recorded within the clayey silt till ranged from 13 blows to greater than 80 blows per 0.3 m of penetration, indicating the consistency of the clayey silt till was stiff to hard. Atterberg limit test results indicated that the cohesive till was of low plasticity. Moisture contents ranged from 17% to 23% for the tested samples of clayey silt till.

The cohesive clayey silt till was underlain by non-cohesive silt/sand and silt till. The surface of the non-cohesive till was encountered at depths between 12.2 m and 13.2 m below the highway grade (between Elev. 150.5 m and 149.2 m). All the boreholes were terminated in the non-cohesive till at depths between 20.4 m and 21.5 m (between Elev. 141.9 m and 140.9 m). Auger grinding was observed in one borehole, which was inferred to indicate that cobbles and/or boulders were present within the non-cohesive till. SPT "N" values measured within the non-cohesive till varied from 50 blows to greater than 154 blows per 0.3 m of penetration, indicating a very dense state of compactness. Moisture contents ranged from 6% to 13% for the tested samples of non-cohesive till.

Localized deposits of clayey silt were present at some of the borehole and test pit locations. SPT N values recorded in the clayey silt ranged from 21 blows to 44 blows per 0.3 m of penetration, indicating a very stiff to hard consistency. Atterberg limit test results indicated the clayey silt is of low plasticity.

Also, a sand deposit layer is encountered in one of the Peto MacCallum boreholes (IK-3), located east of the proposed rail tunnel that had been drilled in 2009. In PML Borehole IK-3, located east of the proposed rail tunnel, a layer of sand is noted on the Record of Borehole sheet within the sandy silt till at a depth of about 7.0 m below ground surface (148.2 m). The layer of sand extended to a depth of about 11.0 m below ground surface (Elevation 142.2 m). The layer of sand is described as wet and contains trace gravel below a depth of about 9 m below ground surface

(Elevation 144.2 m). The natural water content is reported to be between about 7 percent and 10 percent and no other geotechnical classification testing was reported as being carried out on any of the samples obtained from the sand layer. The SPT "N" values measured within the very dense sand layer ranged are reported as 21 and 128 per 0.3 m of penetration and 85 per 0.25 m of penetration, indicating a compact to very dense state.

Table 9 provides a summary of the subsurface stratigraphy identified in the available geotechnical reports.

Table 9: Subsurface Conditions at the Highway 401 Tunnel

<table>
<tr><th>Soil Layer</th><th>Sub-Layer</th><th>From Depth (m)</th><th>Thickness</th><th>SPT "N" value (blows/0.3m)</th><th>Water Content (%)</th><th>In-situ Tests</th></tr>
<tr><td rowspan="3">Top Layer</td><td>Asphalt</td><td rowspan="3">0</td><td>120-400 mm</td><td>-</td><td>-</td><td></td></tr>
<tr><td>Portland Cement Concrete</td><td>430 mm</td><td>-</td><td>-</td><td></td></tr>
<tr><td>Top Soil</td><td>100 mm</td><td>-</td><td>-</td><td></td></tr>
<tr><td rowspan="2">Fill</td><td>Granular Fill (Sand and Gravel)</td><td>0.8 - 1.2</td><td>2.4-5.5 m</td><td>6 to 40</td><td>8% - 12%</td><td></td></tr>
<tr><td>Cohesive Fill (Clayey Silt with Sand and trace Gravel)</td><td>3.5 - 6.7</td><td>2.3-4.8 m</td><td>2 to 8</td><td>7% - 27%</td><td>S_u = 45 to 55 kPa (From Vane Shear Test)</td></tr>
<tr><td rowspan="2">Glacial Till</td><td>Cohesive Clayey Till</td><td>6.9 - 9.0</td><td>3.6-5.3 m</td><td>13 to 80</td><td>17% - 23%</td><td></td></tr>
<tr><td>Non-Cohesive Silt to Sand and Silt Till</td><td>12.2 - 13.2</td><td>-</td><td>50 to 154</td><td>6% - 13%</td><td></td></tr>
<tr><td colspan="2">End of Boreholes</td><td>20.4 - 21.5</td><td colspan="4"></td></tr>
</table>

4.0 Laboratory Results

4.1 Summary of Corrosivity Results

To assess the corrosion potential of the site soils, sixteen (16) selected soil sample was submitted for analytical testing to ALS Laboratories in Waterloo, Ontario. The analytical laboratory report is presented in Appendix C. Table 10 below summarizes the reported results.

Table 10: Summary of Soil Corrosivity Laboratory Results

Borehole / Split Spoon Sample	Soil Type	Moisture (%)	pH	Redox Potential (mV)	Pore Water Resistivity (ohms/cm)	Chlorides (µg/g)	Sulphide (mg/kg)	Sulphates (µg/g)
BH2017-01 SS3	Fill – sandy silt	10.3	7.81	258	3270	106	0.85	41
BH2017-01 SS6	Fill – sandy silt	10.9	7.81	280	2790	128	0.27	31
BH2017-01 SS11	Clayey Sandy Silt Till	9.12	7.91	194	3470	<5.0	0.21	105
BH2017-01 SS16	Sandy Silt Till	7.98	7.98	190	3460	<5.0	0.34	182
BH2017-04 SS3	Fill – sandy silt to silty sand	15.4	7.59	266	5020	6.9	1.91	47
BH2017-04 SS6	Sandy Silt Till	9.93	7.84	281	4930	12.9	<0.20	36
BH2017-04 SS11	Clayey Sandy Silt Till	9.19	7.93	186	3160	<5.0	0.24	126
BH2017-04 SS16	Silty Sand to Sandy Silt Till	7.93	8.04	224	3550	<5.0	0.47	224
BH2017-07 SS3	Fill – sandy silt to silty sand	11.3	7.68	245	1200	296	0.47	289

Borehole / Split Spoon Sample	Soil Type	Moisture (%)	pH	Redox Potential (mV)	Pore Water Resistivity (ohms/cm)	Chlorides (µg/g)	Sulphide (mg/kg)	Sulphates (µg/g)
BH2017-07 SS6	Fill – sandy silt to silty sand	11.9	7.72	249	1360	243	<0.20	375
BH2017-07 SS13	Sandy Silt to Silty Sand Till	8.07	7.92	202	4220	40.8	<0.20	32
BH2017-07 SS19	Sand and Gravel	6.74	8.12	216	4830	15.9	<0.20	90
BH2017-08 SS3	Fill – sandy silt	12.7	7.69	289	6160	<5.0	<0.20	37
BH2017-08 SS6/SS7	Sandy Silt Till	0.26	7.47	232	4290	<5.0	<0.20	110
BH2017-08 SS10	Sandy Silt Till	7.47	7.99	186	5330	5.8	<0.20	32
BH2017-08 SS14	Sand	0.12	7.95	217	8120	<5.0	0.30	36

It is understood that WSP will be carrying out a detailed corrosivity analysis for the project area.

4.2 Soil and Groundwater Framework

Wood has prepared an Excavated Material Management Plan and is presented under separate cover, as Document Number: S121-ENV-3383-008 dated June 06, 2018.

The results of the soil and groundwater analyses carried out as part of the supplementary investigation are provided in Appendix C.

4.3 Hydrogeological Investigation Preliminary Recommendations

A Dewatering Management Plan has been prepared by Wood and is presented under separate cover, as Document Number: S121-ENV-3383-011 dated July 26, 2018. Groundwater levels were measured in boreholes and monitoring wells during each geotechnical investigation between 2015 and 2018. These groundwater levels have ranged between 3.4 mbgs and 7.8 mbgs (elevations of 154.9 m and 150.3 m).

Seasonal fluctuation of groundwater levels should be expected and some of the measured groundwater levels were collected in March and April, which may represent seasonal high groundwater conditions. Seasonal low groundwater conditions would be expected during the summer months (July and August) and to a lesser degree in the winter months (January and February). The highest recorded groundwater levels were used in estimating dewatering rates for the site.

4.4 CPT Test Results

Cone Penetrometer Tests (CPTs) were completed by ConeTec on behalf of Golder Associates as part of the preliminary investigation for the data report. A supplementary CPT program was completed by ConeTec, on behalf of TTP between July 23 and July 25, 2018. The supplementary CPT program included completing CPTs at four locations and conducting pore pressure dissipation tests at select depths to supplement previous CPT investigation results.

The CPT results from the Golder Associates and TTP investigations indicate that soil friction angle for the fill between Elev. 155 m and Elev. 165 m is approximately 33° and for the till between Elev. 150 m and Elev. 155 m is approximately 36°. Additionally, the CPT results indicate values of about 20 MPa and 75 MPa for the small strain shear modulus (G_{max}) of the fill and till layers respectively. Figures 1 and 2 below show undrained shear strength (S_u), soil friction angle, relative density, shear wave velocity, and G_{max} based on CPT tests.

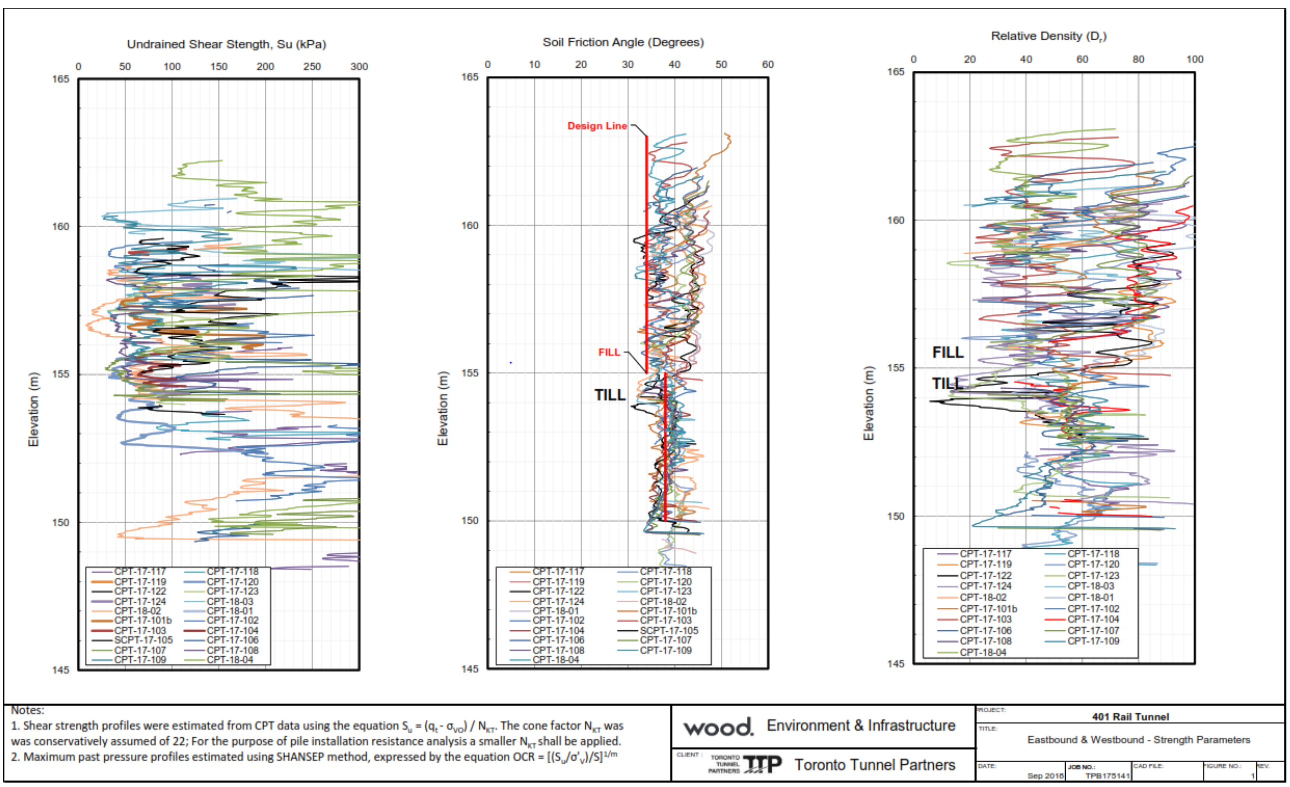

Figure 1 – CPT Strength Parameters

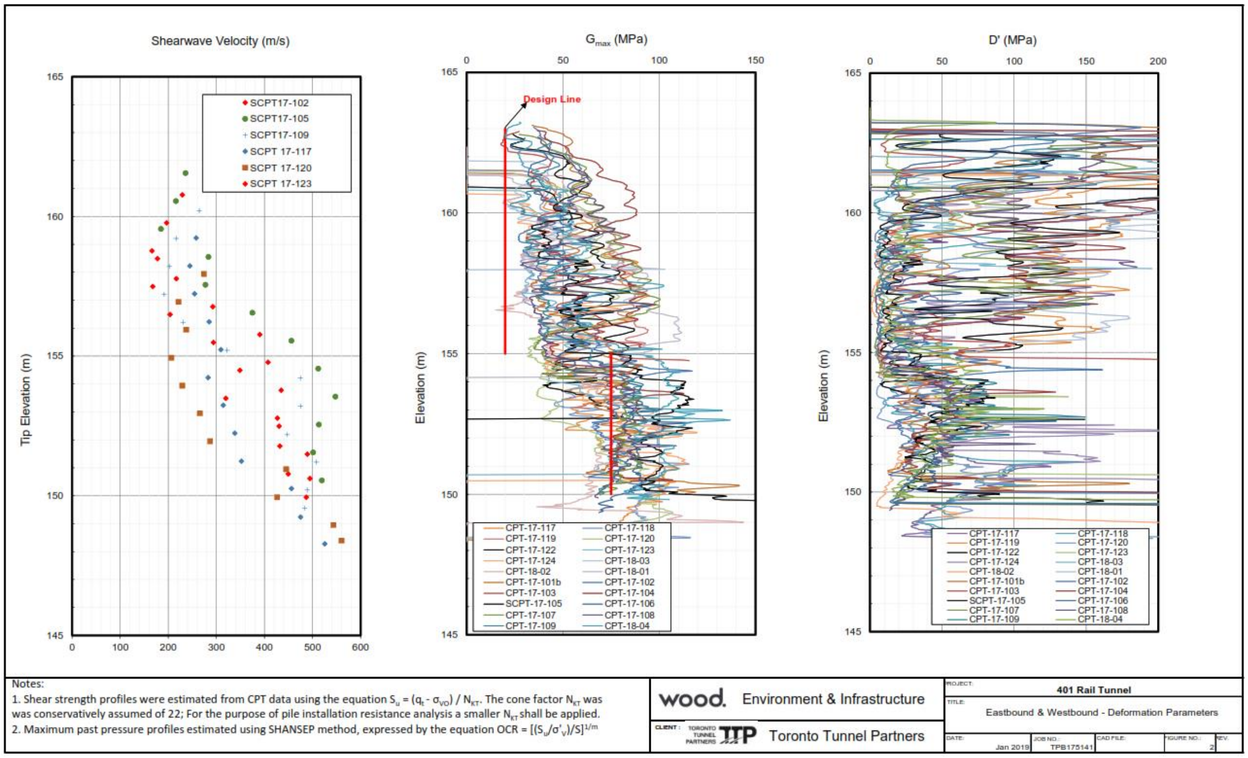

Figure 2 – CPT Deformation Parameters

Figures 1 and 2 are also provided in Appendix D.

5.0 Closure

This 100% Foundation Investigation Report was prepared by Vishu Vasisht, B.Sc., Geotechnical Project Coordinator and Mathi Shan, M.Sc., P.Eng., Senior Geotechnical Engineer. Mr. Ty Garde, M.Eng., P.Eng., Principal Geotechnical Engineer and a Designated MTO Foundations Contact for Wood, conducted an independent review of this report.

Sincerely,

Wood Environment & Infrastructure Solutions,
a Division of Wood Canada Limited

Prepared By:

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Part B:

100% FOUNDATION DESIGN REPORT
HIGHWAY 401 RAIL TUNNEL
KITCHENER CORRIDOR BETWEEN ISLINGTON AND KIPLING AVE.
CITY OF TORONTO

6.0 Design Recommendations for Tunneling

This section of the report provides foundation design recommendations for the proposed twin rail tunnels below Highway 401 and 409. The recommendations are based on interpretation of the factual data obtained from the previous and supplementary investigations on site.

The existing tunnel consists of a 12.5 m wide by 170 m long underpass below Highway 401 and 409, with a clearance of 7 meters from the top of the rails to the roof of the tunnel. Based on the planning study completed to date, the future tunnel will be located north of the existing tunnel.

Geotechnical recommendations for the tunnel design are discussed below. Several options and their advantages and disadvantages, for the design and construction of the tunnel, have been considered and are discussed in the following sections.

6.1 Options For Constructing The New Rail Tunnel

Several issues must be considered in the selection of the preferred conceptual tunnel design. The temporary support frame and final tunnel configuration must meet the railway geometric design and other design requirements allowing for all equipment and clearances, while fitting within the limited available space. This limits the minimum overall "size" of the tunnel and generally defines its position relative to existing features and constraints. Furthermore, interlocking support may be required to control deformations and ground loss during construction of any closed box system. Another consideration is that the method for installing a consistent waterproofing system needs to be compatible with the tunneling method. The tunnel alignment must accommodate a curve, which is difficult to construct in a straight box alignment. Temporary shafts would be needed in the highway median to permit construction of tunnels. Four tunneling options have been considered to address these issues. Table 11 provides a summary of the advantages and disadvantages of the four primary options, as well as potential variations to the primary options.

6.1.1 Microtunnelling

Microtunnelling would rely on temporary roof support provided by DN500/800 microtunnels that would be filled with concrete, and a shotcrete layer. For one option, a center beam would be constructed on a foundation to evenly distribute the loads and to control settlements. A temporary base slab could be placed shortly after excavation. A second microtunneling option would connect the microtunnel to the existing tunnel. A third option would be to construct an arched roof by installing tie-rods with a membrane waterproofing system between the initial and final lining.

6.1.2 Box Jacking

Box jacking involves the advancement of a prefabricated tunnel box structure from one end of the future tunnel to the other, by progressively excavating the soil in advance of the tunnel and jacking the box structure forward along the new tunnel alignment. A shaft may be required in the highway medians to reduce the driving lengths of the tunnel sections.

6.1.3 Sequential Excavation Method (SEM)- Single Tunnel

Single SEM tunnel construction would require temporary support provided by a grouted pipe canopy (or equivalent), shotcrete and lattice girders to control settlements. Various drift options are available to control settlements. The final lining would be constructed using fiber reinforced concrete.

6.1.4 Sequential Excavation Method (SEM)- Twin Tunnel

The design and construction of side by side tunnels with a defined distance between the new tunnels and between the new tunnels and the existing rail tunnel is achievable. For this option, temporary support is provided by a grouted pipe canopy (or equivalent), shotcrete and lattice girders to control settlements. Three drift options are available to control settlements. The final lining is constructed using fiber reinforced concrete.

6.1.5 Advantages and Disadvantages of the Considered Tunnelling Methods

Advantages and disadvantages of each considered tunneling method has been summarized in Table 11, below.

Table 11: Advantages and Disadvantages of the Considered Tunnel Methods

Tunneling Method	Advantages	Disadvantages
Micro-tunneling with Centre Support	▪ Center support will allow for a smaller tunnel in comparison to support frame ▪ Faster advance rate due to less steel erection ▪ Maximizes cover to Highway 401/409 surface	▪ Additional work to cut center support beam ▪ Tight space to install support beam right at face ▪ Ground inconsistency could cause problems with installation of support, hence jeopardize the structural model ▪ Long drives are needed for the microtunnels (accuracy) ▪ Median shaft must be constructed to allow microtunnel installation
Micro-tunneling with Box Connection	▪ Proximity to the existing rail alignment will help monitor movements ▪ Closer alignment to the tunnel will bring advantage in the overall project alignment ▪ Possibility to stabilize existing structure from new Highway 401 tunnel	▪ Underpinning of existing rail tunnel foundation will be required ▪ Possible conflict with existing foundation ▪ Ground inconsistency could cause problems with installation of support, hence jeopardize the structural model ▪ Long drives for the microtunnels needed (accuracy) ▪ Median shaft must be constructed to allow microtunnel installation

Micro-tunneling with Arch Roof	▪ Arch shape roof will eliminate the need for a steel support frame ▪ Possibility to utilize structural capacity of arch roof to minimize final lining thickness ▪	▪ Long drives are needed for the microtunnels (accuracy) ▪ Median shafts must be constructed to allow microtunnel installation
Box jacking with anti-drag system:	▪ One way pass will allow for quick construction ▪ Final box structure is built ahead of time and at ideal conditions (when space is available)	▪ Median shafts would be needed to allow installation ▪ It is unlikely to achieve limits of movement for settlement, heaving and movements restrictions on Highway 401/409 and train tunnel ▪ Possible additional loads on existing rail tunnel due to horizontal displacement of soil ▪ Continuous Jacking operation cannot be guaranteed (friction, obstacles, stop and go) ▪ High vertical amplitude tolerances
Box jacking in combination with micro-tunneling roof support	▪ One way pass will allow for quick construction ▪ Final box structure is built ahead of time and at ideal conditions (when space is available)	▪ Long drives for the microtunnels needed thus more difficult to maintain alignment vs other methods ▪ Median shafts would be needed to allow microtunnel installation ▪ Jacking box has larger clearance to allow for jacking tolerance, thus larger overall disturbance of ground and settlement is expected ▪ Installation of temporary steel support system is challenging
SEM single tunnel	▪ A very lean construction type due to the optimal structural shape which in turn allows the alignment to follow the rail alignment. ▪ All works could be done from within the tunnel, hence no temporary construction shafts needed in highway median ▪ Various pre-support measures available to stabilize ground prior to tunneling ▪ Tool Box measures available to mitigate unforeseen ground	▪ A sophisticated design and construction method which will requires experienced site personnel

	conditions and obstructions of any kind ▪ Adjustment of shape to actual requirements (ventilation etc.) ▪ Optimized structural thickness due to a round shape ▪ Very limited exposure during excavation (due to drift excavation)	
SEM twin tunnel with micro tunnelling	▪ A very lean construction type due to the optimal structural shape which in turn allows the alignment to follow the rail alignment. ▪ All works could be done from within the tunnel, hence no temporary construction shafts are needed in highway median ▪ Various pre-support measures available to stabilize ground prior to tunneling ▪ Tool Box measures available to mitigate unforeseen ground conditions and obstructions of any kind ▪ Allows for even better shape than a single tunnel resulting greater depth of cover ▪ Optimized structural thickness due to a round shape ▪ Very limited exposure during excavation (due to drift excavation) ▪ Grouted umbrella and benched excavation process provides highest control and lowest occurring settlements of all options considered ▪ Provides the most reliable approach to deal with challenging or changing ground conditions	▪ Adjustment of horizontal rail alignment required ▪ Sophisticated method of design and construction will require experienced site personnel. TTP meets this requirement

6.1.6 Selected Tunnelling Method

Figure 3 – Twin Tunnels

Based on TTP's assessment of the advantages and disadvantages of the tunnel methods described above, the twin tunnel SEM method is the preferred alternative for the design and construction of the Highway 401 Rail Tunnel. An ovoid tunnel shape will be used to optimize the structural design and the lining thickness, while various ground support, face support, pre-support and ground improvement measures will be installed as required to ensure the stability of the open-face SEM excavation, to improve the ground strength and stiffness of the soil surrounding the new rail tunnels and to keep movement of adjacent structures and utilities within maximum settlement and distortion limits. This excavation method can control settlements by minimizing the dimensions of the exposed faces and leads to maximum control of ground and reduced settlements. This method provides for a response to unexpected settlement magnitude and allows for adjustment of the advance length (prior to shotcreting), as needed to control displacements. It also can help in reducing face size by pocket excavation and increase of ground stiffness by permeation grouting.

According to the subsurface conditions and the proposed depth for the new tunnel, the heading of the twin tunnel will be drilled mostly in fill materials and the benching will be in Till.

6.2 Pre-Construction Considerations

6.2.1 Ground Improvement

The construction sequencing for tunneling advancement involves a two-stage operation:

- Pre-Support,
- Excavation

Pre-support methods are required to improve soil cohesion, reduce soil permeability and provide stand-up time during excavation to control deformations of the existing and new infrastructure. Existing infrastructure with limited allowable deformation include the existing highway pavement structure, the existing Rail Tunnel, and the existing retaining walls.

6.2.2 Methods, Materials, Equipment and Sequences Used to Pre-Support Ground Beneath the Highway Pavements Prior to Excavation

In the low overburden areas at the portal sections of the tunneling works, the tunnel roof pre-support will include the installation of 813 mm diameter steel pipes (Type 1 roof support). In the central part of the tunneling works, pre-support will be provided by a grouted pipe roof umbrella utilizing 139 mm diameter grouted pipes (Type 2 roof support), which will be installed before any excavation takes place. The pre-support will increase the stiffness of the tunnel overburden, to control ground displacements and settlement above the tunnel. The pipe roof umbrella will increase the stability in the working area by transferring loads in the longitudinal direction and decrease excavation induced deformations.

As significant jacking pressures (approximately 300 metric tons are currently estimated) are required for the Type 1 pipe installation, a rigid work platform and jacking block will be constructed at the pipe roof elevation ahead of the tunnel portals.

The pre-support will be installed in the crown of the tunnel using specialized equipment. The excavation operation of each tunnel will commence after the pre-support has been established, with the top heading excavation and the initial lining support of the exposed ground constructed first, followed by the bench-invert drift.

A drill boomer will be used to install the Type 2 pipe umbrellas for the SEM excavation. After a number of pipes are set in place, the grouting plant will be mobilized inside the tunnel to carry out the grouting effort. Grout will be injected via the grouting plant, through the grout tube where it will travel along the pipe sleeve and into the soil. The packer will be set to isolate one or more pipes at a time. Grout will be injected slowly into the soil and also under low pressures (0.0 to 0.3 Bar) to avoid grout traveling too far from the injected area. The spread of the grouting material is governed by the, setting time, grout type, grouting pressure, in situ density of the grouted soil and other soil parameters. Therefore, the grouting plant will be equipped with an array of piston pumps with high sensitive pressure gauges that will be used to allow for a smooth process and recording of the grouting process. Additionally, the grouting plant will be furnished with a mixer and agitator unit that will keep the grout mix from setting until it is injected through the Tube-a-Manchette pipes into the soil.

Based on the available geotechnical information, permeation grouting is currently envisaged as required, by installing tube-a-manchette pipes ("TAMs") in a pre-determined pattern and injecting silica base grout mix to permeate the embankment fill layer and improve soil strength and stiffness. Permeation grouting can be performed from within the tunnel with conventional drilling and grouting equipment, whilst pressure, flow and grout volumes are recorded to evaluate and determine the direction of the ground improvement program. Grouting pressures must be controlled to prevent encroachment of the permeation grouting into the restricted zone 1.5 m below the highway pavement.

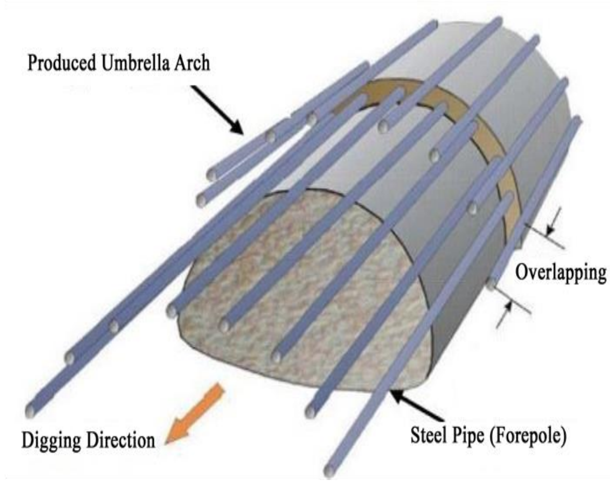

Figure 4 – Grouting Pipe Roof Umbrella

6.3 Construction Considerations

6.3.1 Alignment Control

The SEM allows for construction adjustment due to its "design confirmation as you dig" approach. Alignment control is derived from continuous automatic total station surveys which provide real time information related to the horizontal and vertical tunnel alignment as part of the in-tunnel instrumentation and monitoring program. The SEM allows for continuous alignment corrections as the excavation proceeds in small increments (approximately 1 m length), in contrast to other methods such a box jacking that have little segmental alignment control.

6.3.2 Length and Diameter Constraints

Length and diameter constraints are not significant factors with the SEM. Unlimited tunnel lengths can be achieved because the tunnel liner is built as the tunnel progresses. The SEM cross section can be designed to expand to very large dimensions of 25 m or greater. Transit station caverns such as for the Eglinton Crosstown LRT project, are routinely constructed using SEM.

6.3.3 Controlling Soil Excavation Faces at Atmospheric Pressure

The tunnel staging plan for advancing the tunnels to control the soil excavation faces is based on the currently available information. This staging will be confirmed based on interpreted geotechnical characterization along the tunnel alignment and the resultant improved 3D FE analysis during the detail design. If necessary, the contingency measures will be available and in place during excavation to control ground loss at the excavation face.

These contingency measures would include:

Face Support Wedge: This is a simple and effective face support measure, achieved by leaving an earth wedge in place during the excavation in order to support the face against raveling and moving inward;

Shotcrete Face Support: A 50–75 mm layer of fiber reinforced shotcrete is sprayed against the exposed excavation face in order to seal the ground and prevent raveling, running or flowing of the ground. Shotcrete face support is often used in combination with an earth wedge or pocket excavation;

Sub-Division of Excavation Faces: For larger cross sections, the most effective face stabilization technique is to reduce the cross-sectional area of the excavation by subdividing the face into multiple drifts;

Pocket Excavation: Pocket excavation can be used to handle even the most challenging ground conditions. It is achieved by excavating a series of small pockets (one to a few excavator buckets in size) with immediate shotcrete support as each pocket is exposed. This procedure is continued until the full round is completed;

Face Bolting: Horizontal bolts are drilled and grouted in place in order to prevent block failure at the tunnel face. The most practical type and installation method is using self-drilling, hollow fiberglass bolts with a length of up to 12 m, and an overlap of 3-6 m. The fiberglass bolts in the tunnel face can be easily removed using standard tunneling equipment;

Permeation Grouting: Cementitious or chemical grout is injected into the soil using perforated pipes or tube-a-manchette pipes in order to increase the cohesion of the soil and to stabilize loose or granular soils; and

Dewatering: This involves drilling perforated or slotted dewatering pipes into the ground ahead of the excavation face to limit groundwater seepage, disturbance to the exposed soils and reduce hydrostatic pressures at the face.

Further details of the contingency/mitigation measures, specific to the subsurface conditions anticipated to be encountered are presented in the Tunnel Design Report.

6.4 Settlement and Heave Estimation

Ground deformations must be controlled to meet the Limits of Movement listed in Schedule 15 Part 3-7 of the Project Agreement (PA). Analysis of settlements have been completed by Dr. Sauer & Partners Ltd. using 3D finite element analysis. This modelling assessed the overall impact on surface, subsurface(utilities) and existing tunnel infrastructure. The surface and subsurface conditions were also evaluated at completion of the first tunnel prior to start of excavation work in the second tunnel. A refined 3D analysis has been completed and the results of this modelling are presented in the submittal 90% Tunnel Structural Design Report (S104A-TUN3383-001 dated March 5, 2019). The following factors have been considered in the settlement analysis:

- Existing tunnel is part of the model
- Minimum overburden (1.5 m)

- Subsurface soil and groundwater conditions as noted in part 3.2 and 3.3
- Excavation and relaxation ahead of the advancing tunnel face is modelled by reduction of stiffness of material inside the tunnel cross section
- Results are used to obtain a preliminary assessment of lining thickness and surface deformation and to assess the impacts of the sequencing of the tunnel construction on the existing structure and the settlements

Ground deformations will be managed by proper installation of the pre-support, excavation sequencing, face support and the use of contingency measures as discussed above.

The output of the preliminary 3D modelling indicates surface settlements for the highways are estimated at or near the review level Limit of Movement Based on the analysis, the highway surface settlement after the completion of both tunnel construction is estimated to be less than 10 mm. The vertical deformation of existing rail tunnel after completion of both tunnels is estimated to be less than 5 mm.

6.4.1 Geotechnical Deformation Parameters for 3D FEM Modelling

The 3D FEM model has been completed by the tunnel designer, Dr. Sauer & Partners, and is presented in the submittal titled “Tunnel Structural Design Report”. The following geotechnical deformation parameters have been provided for use in developing the 3D FEM model.

The shear modulus (G) at different shear stress ratios of failure can be determined using the following relationship:

$$G = G_{max}[1 - \left(\frac{\tau}{\tau_f}\right)^{0.3}]$$

Where τ is the predicted shear stress and τf is the failure shear stress and Young’s modulus can be calculated using the following relationship:

$$E = 2G(1 + \nu)$$

Where ν is the Poisson’s ratio. Hence, considering Poisson’s ratio of 0.3, estimated stiffness at mobilized shear stress ratios of 25% and 50% of failure i.e. τ/τ_f =0.25 and 0.5 are presented in Table 12 below.

Table 12 - Stiffness Properties of Non-Cohesive Materials

Material	Design G_{max}	E_0	G_{25}	E_{25}	G_{50}	E_{50}	D’
	(MPa)						
Fill	20	50	7	18	4	10	10
Till	75	195	25	65	15	35	35

Where:

G_{max}: Maximum shear moduli
E_0: Young’s moduli

G_{25} and E_{25}: Shear and Young's moduli at mobilized shear stress ratio and deviator stress ratio of 25% of failure, respectively
G_{50} and E_{50}: Shear and Young's moduli at mobilized shear stress ratio and deviator stress ratio of 50% of failure, respectively
D': Constraint Modulus

To provide typical values for elastic moduli, soil elastic parameters were deduced from:

i. Triaxial tests (complete history of the degradation of soil stiffness with increase in strain level); and

ii. Cone Penetration Tests.

Elastic Moduli parameters were deduced from Consolidated Undrained (CU) triaxial tests conducted on the samples of silty clay till during the previous geotechnical investigations (Thurber, 2009). The results of the testing are provided in geotechnical reports in Appendix G and Appendix H.

Considering Poisson's ratio during constant volume change (v_u) of 0.5, undrained Young's moduli for cohesive materials are presented in Table 13.

Table 13 - Stiffness Properties of Cohesive Materials

Material	Design G_{max}	E_{u0}	G_{25}	E_{u25}	G_{50}	E_{u50}	E_{ur}	D
	(MPa)							
Fill	20	60	7	21	4	12	36	8
Till	75	225	25	75	15	40	160	40

The void ratio of the fill layer is estimated to be between 0.26 and 0.5.

Additional soil parameters are provided in Table 14, below.

6.5 Tunneling Construction Considerations

The twin tunnel SEM does not require the design or construction of exit or intermediate shafts to advance the tunnel. Therefore, temporary shoring and construction dewatering is not expected to be required during tunnel construction. Groundwater seepage into the tunnel during construction should be handled by properly filtered sumps and ditches.

During SEM excavation, direct measurement of ground loss, for instance by weighing spoils from tunnel excavation is not feasible, as the exact shape of the tunnel excavation, as well as materials brought into the tunnel, such as backfill material as a temporary roadbed or temporary invert shotcrete cannot be accurately measured. In order to control face loss, or more precisely deformations in the soil above and next to the newly built tunnel, a surface and subsurface monitoring system is utilized. Deformation monitoring gives a clear picture of the behaviour of the ground-structure interaction system of tunnel and surrounding soil. Monitoring frequency will be achieved at the highest possible frequency obtainable based on the instrument limitations.

Refer to TTP's document S-102A, Geotechnical Instrumentation and Monitoring Plan (GIMP) for the detailed description on instrumentation, monitoring frequency, and response action plan.

The surface and sub-surface monitoring system will also detect deformation, or ground loss, during any ground improvement measures, as the monitoring system can be installed prior to start of any construction activity. Mitigation measures for excessive ground loss during tunneling or ground improvement installation are reduction of face size, reduction of round length, change of pre-support system or increasing the stiffness of the ground ahead of the advancing tunnel face.

It is understood that the auger boring method is the preferred method for installing the roof support in advance of the SEM tunnel construction. A common problem associated with this method is cutting a hole slightly larger than the steel casing (overcutting), which will require grouting to fill any voids that may be formed to prevent excessive settlement. During auger boring operations, the bore pipe should be advanced as far ahead of the augers as possible such that the auger is maintained behind the leading edge of the bore pipe creating a plug of soil material at the face. It is recommended that the auger head maintains a sufficient distance behind the leading edge of the bore pipe, as determined by the tunnel designer. It is recommended that the pipe and auger should be advanced in sequence, rather than simultaneously, to minimize the potential for excessive ground losses associated with running granular soils.

To assist in the development of the auger boring work plan, the following geotechnical parameters are recommended for the non-cohesive and cohesive fill materials understood to be present between the Highway 401 pavement and the top of the new rail tunnels, at approximately 3 m depth below the pavement.

Table 14: Parameters for Auger Boring Plan

Soil	Angle of Internal Friction	Friction Angle between Soil and Steel Pipe	Angle of Repose
Non-Cohesive Fill	35° - 50°	27° - 38°	30° - 40°
Cohesive Fill	28° - 32° (1)	18° - 21° (2)	25° - 30°

(1) Undrained shear strength of cohesive fill = 130 kPa to 300 kPa
(2) Adhesion of cohesive fill = 55 kPa to 105 kPa

An analysis of the maximum grouting pressure to avoid hyrdofracturing (frac-out) of the grout, for depths of cover of 1.5 m and 3.0 m and for two fill compositions (upper layer non-cohesive/lower layer cohesive fill, and all non-cohesive fill) has been completed. The results of this assessment are summarized in the following table. No factor of safety has been applied to the results. An appropriate factor of safety should be selected by the auger boring and grouting designer considering potential for frac-out conditions and risk to existing structures and utilities.

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Table 15: Maximum Grout Pressures for Auger Boring Plan

Thickness and Type of Soil Cover	Maximum Grouting Pressure to Avoid Frac-out (kPa) (FS =1)
1.5 m of Non-cohesive fill	50
1.5 m of Non-cohesive fill and 1.5 m of Cohesive Fill (3.0 m total cover)	200
3.0 m of Non-cohesive Fill (3.0 m total cover)	130

Best practices will be followed by the auger boring contractor. It is anticipated that properly installed pipe supports using the intended auger boring method will not result in ground loss. TTP will be conducting test borings in simulated ground conditions to confirm the selected method will provide the required control of ground movements.

6.6 Existing Structures Along Tunnel Alignment (Retaining Wall #8)

It is understood that the new tunnels will intersect with the foundation of Highway 409 ramp retaining wall which is supported on driven steel H-piles. A combination of vertical piles and battered piles (with a batter inclination of 3V:1H) support the retaining wall #8. While design basis and construction records for the pile foundation are not available, it is understood the battered piles might have been provided to enhance the structural rigidity of the wall foundation. Some of the battered and vertical piles supporting the retaining wall will be partially removed and permanently supported on the tunnel lining during and after the top heading construction of the tunnels. This section provides information on total ultimate soil resistance (unfactored) for the existing HP310 x 79 piles. In terms of driven pile founding soil and elevation and associated soil resistance capacities there may be two scenarios as noted below:

a) Scenario 1: Based on the available subsurface information from the investigations, and assuming a pile tip elevation of 151.1 m, the pile tips are anticipated to be founded in hard sandy clayey silt till ("N" value: 33-38). It is assumed that the subsurface conditions at the pile locations are consistent with those encountered during the geotechnical investigations. Considering this scenario, it is envisaged that the piles were designed considering both toe resistance and shaft resistance. For piles founded in the hard sandy clayey silt till at elevation 151.1 m, an unfactored total ultimate soil resistance of 1100 kN in vertical compression may be used in the structural analysis.
   In this scenario, it should be noted that the tip elevation of the existing piles would be above the proposed tunnels bottom elevation. Therefore, to maintain the available soil capacity for the piles, care must be taken during excavation to ensure soils surrounding the piles and below the piles are not disturbed.
b) Scenario 2: Based on our understanding of typical MTO practice, it is possible that the piles were driven into underlying hard/very dense till founded at elevation 149.5 m to obtain a

higher pile resistance ("N" value: >70). In this scenario, a total ultimate soil resistance (unfactored) of 1650 kN in vertical compression can be adopted for structural analysis.
Additional information such as pile driving records for verification of founding soil type and pile tip elevation must be reviewed by Wood to confirm the actual soil capacities. Lacking this information, the more conservative soil capacity estimate associated with Scenario 1 should be utilized.

As noted above, some of the existing piles supporting Retaining Wall #8 will be exposed by the tunneling operation. As indicated in the analysis performed by WSP Canada Group Inc ("WSP"), the exposed piles will have essentially zero capacity to carry foundation loads and the loads will be transferred to the adjacent undisturbed existing piles. Load re-distribution analysis performed by WSP indicates a maximum 640 kN vertical load will be experienced by the vertical undisturbed piles, entirely supported in the glacial till, during the tunnel construction.

Based on the pile tip elevation discussed in Scenario 1, the total unfactored ultimate bearing capacity for an existing vertical HP310x79 steel pile is estimated to be 950 kN in compression. This bearing capacity assumes the soils surrounding the pile remain undisturbed between the pile tip elevation of 151.1 m and elevation 154.1 m (approximately 3 m above the pile tip), and also below the pile tip elevation. The unfactored ultimate bearing capacity is comprised of an unfactored ultimate tip resistance of 650 kN and a skin friction component of 300 kN. Although the skin friction component is not necessary to support the design vertical load of 640 kN (if no factor of safety is applied to the ultimate value), the soil between 154.1 m and 151.m must remain undisturbed to preserve the pile tip resistance.

If it is possible to expose one or more existing piles during tunnel construction to determine if Scenario 2 represents the as-built pile termination, the elevation range of the undisturbed soil zone surrounding and below the pile can be revised to reflect the confirmed as-built pile tip elevation. The retaining wall will be monitored during and following construction as part of the GIMP.

7.0 Shaft construction considerations for auger boring

An access shaft will be required in the median of Highway 401/409 to provide access for the auger boring machine to allow jacking the tunnel steel pipe roof support. The excavation configuration will accommodate the auger boring machine, length of steel pipe to be jacked into place, in addition to space for workers and appurtenant facilities.

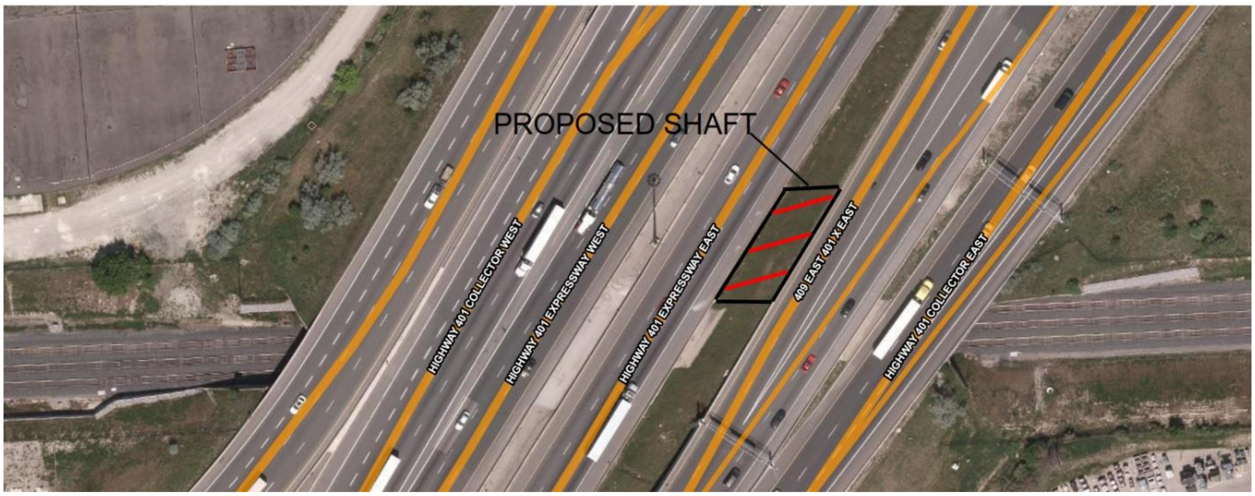

Figure 5 – Proposed Access Shaft Location

7.1 Open Cut Excavations

The excavations should be carried out with side slopes in accordance with the latest version of O. Reg. 213/91 as amended i.e. the Ontario Occupational Health and Safety Act and Regulations for Construction Projects. The soils found at the site are classified as follows:

Table 16: Soil Classification for Open Cut Excavations

Soil Type	Soil Classification
Granular Fill	Type 3
Fill, silty, sandy, clayey	Type 3
Loose to Compact, SAND & GRAVEL, SAND above the groundwater table	Type 3
Loose to Compact, SAND & GRAVEL, SAND below the groundwater table	Type 4

It should be noted that excavations within multiple soil types should be advanced following the requirements of the least optimum soil type.

When workmen must enter excavations advanced deeper that 1.2 m, the excavation walls must be suitably sloped and/or braced in accordance with O.Reg. 213/91 of the Occupational Health and Safety Act. The regulations stipulate maximum slopes of excavation by soil types as follows:

Table 17: Maximum Slope Inclination Based on Soil Types

Soil Type	Base of Slope	Maximum Slope Inclination
3	From bottom of excavation	1H:1V
4	From bottom of excavation	3H:1V

Minimum support system requirements for steeper excavations are stipulated in the Occupational Health and Safety Act, including provision for timbering, shoring, and moveable trench boxes

7.2 Lateral Earth Pressures for Shaft

It is understood that driven sheet piles will be used for temporary support of the shaft excavation. It is also understood that the shaft will be in use for approximately 6 months. The temporary shoring system must be designed to resist the lateral earth surcharge and hydrostatic pressures which could occur during construction.

It is understood internal bracing will be used instead of anchors due to the proximity to active highway corridor. The temporary shoring system, including internal bracing support should be designed and constructed in accordance with applicable standards such as, but not limited to OPSS 539 and current Ontario Health and Safety Regulations.

The shoring should be designed to account for lateral earth pressures resulting from the weight of the retained soil and other dead and live surcharge loads. The earth pressure distribution used for shoring design is depended on the specific wall design and on the nature of the lateral support provided.

The buoyant unit weight and associated hydrostatic pressure should be used below the groundwater table.

Surcharges at the ground surface should be added in accordance with applicable soil mechanics methods such as described in the Canadian Foundation Engineering Manual.

The design earth pressures in compacted backfill should be augmented with the dynamic effects of the compaction efforts, which typically are taken as a uniform 12 kilopascal (kPa) pressure over the entire depth below grade where the calculated earth pressure based on the above earth pressure factors is less than 12 kPa.

For shoring wall on stiff to hard cohesive soil, consisting of interlocking steel sheet piling supported by internal bracing, the system should be designed to resist a ‘trapezoidal’ earth pressure distribution. The following equation may be used (CFEM, 2006):

$$\sigma_h = 0.4\ (\gamma H + q)$$

Where: σ_h = lateral earth pressure on the shoring, kilopascals;

γ = Unit weight of retained soil;

H = Height of shoring (i.e., depth of excavation), in metres;

q= Surcharge at ground surface to account for traffic and equipment.

For earth pressure distribution on other soil types, please refer to Canadian Foundation Engineering Manual.

The parameter values in the following table may be used for shoring design calculations:

Table 18: Recommended Parameters for Temporary Shoring Design

Soil Description	Bulk Unit Weight (kN/m^3)	Effective Friction Angle, (°)	Cohesion Intercept, C′, kPa	Ka	Kp
Non-Cohesive Granular Fill	21	35	0	0.27	3.69
Cohesive Fill	19	28	0	0.36	2.77
Very stiff to hard Clayey Silt to Silty Clayey Till	21	34	5-15	0.28	3.54
Very dense Sand and Silt Till	22	34-36	0	0.28-0.26	3.54-3.85

For shoring design, consider a groundwater elevation range between 151 m and 154 m.

Some unavoidable inward horizontal deformation and vertical settlement of the adjacent ground will occur as a result of excavation, installation of shoring, and deflection of the ground support system as well as deformation of the soil in which the toe of the shoring is embedded. As a preliminary guideline, typical settlements behind sheet pile shoring can be estimated to be about 0.2 percent of the excavation depth, provided good construction practices are used. This is only a preliminary assessment of the potential settlements and is provided only to assist the shoring designer with evaluating the potential impacts of the expected settlements.

The Highway 409 ramp retaining wall is in close proximity to the east side of the proposed shaft excavation. Based on available data, it is understood that the Highway 409 ramp is supported on pile foundations and therefore impacts by the ground movements resulting from the excavation should be minimal and not likely to have significant impacts on this retaining wall structure. Monitoring of the retaining wall have been considered in the Geotechnical Instrumentation and Monitoring Plan, and instrumentation has been installed to monitor the movement of the retaining wall.

The presence of cobbles and/or boulders was inferred during the borehole drilling program near the vicinity of the proposed shaft. The presence of cobbles and/or boulders can result in damage to sheet piles and could obstruct installation of sheet piles. This could require, for example pre-drilling staggered holes to facilitate sheet pile penetration.

The temporary shoring of excavations and horizontal movement of ground surface should conform to OPSS 539, Construction Specification for Temporary Protection Systems and monitored throughout the construction process. The design of the temporary sheet pile shoring system should meet the requirements of Performance Level 2 in accordance with OPSS 539. Performance level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. A monitoring plan consisting of survey targets and tilt meters installed on the interior facings of the temporary shoring will be developed to monitor the movement of the shoring system during its service life.

7.3 Other Considerations

Several ground monitoring instruments and cables are located near the proposed shaft area. The Contractor should be made aware of the excavation challenges associated with nearby instruments and cables. The Contractor will be required to stop the excavation and notify TTP if any instruments or cables are compromised during the shaft construction.

8.0 Design Recommendations for Retaining Walls Constructed as Part of New Metrolinx Infrastructure

Retaining walls will be required adjacent to the tunnel portals and north of the proposed track alignment. The wall is a combination of a secant pile wall with concrete caissons and H-piles with precast concrete panels.

Based on the 90% Structural Design Report (submission S104A-STR-3383-001 R1 dated April 18, 2019), it is understood that for the northwest wall, where the wall heights are up to 1.5 m, the wall construction will consist of H-piles with precast concrete panels. For wall heights between 1.5 m and 3.5 m, a secant pile wall will be utilized. A cast-in-place concrete wall supported on H-piles is proposed for the remainder of the wall.

For the northeast wall, a secant pile wall with concrete caissons attached to the tunnel portals is proposed.

At the southwest and southeast corner of the tunnel a caisson wall is proposed to permanently support the existing and future tracks passing through the existing tunnel. The south walls extend to allow for 2H:1V grading from the invert of the new tunnels to the existing grade.

Figure 6 shows the proposed types of retaining walls. The proposed retaining walls include a combination of:

a. Secant pile wall with concrete caissons attached with the tunnel portal up to 3.5 m high,

b. Caisson wall to permanently support the existing and future tracks passing through the existing tunnel, and

c. H-piles with lagging and precast concrete panel up to 1.5 m.

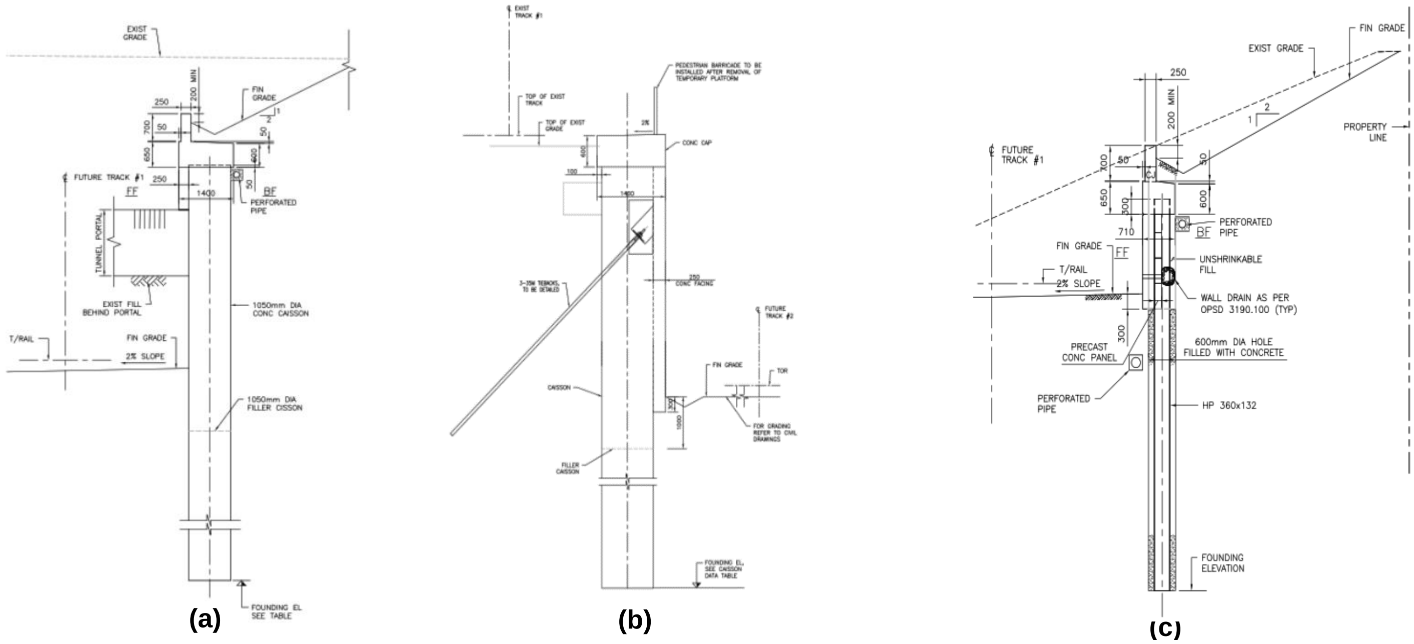

Figure 6 – Types of Retaining Walls: (a) Secant pile wall, (b)Caisson Wall with tieback, (c) H-pile with lagging and concrete panel

8.1 Foundation Design Recommendations for Retaining Walls

8.1.1 Caisson Foundations

The design for the retaining wall foundation utilizes caissons as a deep foundation alternative. Caissons will be founded in the very dense sand and silt glacial till at or below approximate Elev. 150.0 m. The design for caissons founded at or below this elevation can use a factored axial geotechnical resistance at Ultimate Limit State (ULS) of 800 kPa and a geotechnical reaction at Serviceability Limit State (SLS) of 600 kPa for 25 mm of settlement. Temporary casing will be required when drilling through the fills above the glacial tills to prevent sloughing of the caisson hole and to control groundwater infiltration.

It is understood that pre-bored steel H-piles grouted with concrete are being considered for the northwest retaining wall. The pre-bore diameter should be selected according to the pile size. The pre-bore diameter should not exceed the pile size. Given the glacial till soils that dominate the subsurface condition, difficulties should be anticipated during the borehole augering due to presence of cobbles/boulder within the glacial till stratum. Pile tip protection should be provided for driven H-piles at this site to minimize damage while driving within very dense till containing cobbles and boulders. Further comments regarding obstructions are provided in Section 9.1. In areas where the groundwater level is above the pile tip elevation, tremie placement of concrete will be required to maintain stability of the pile base.

Lateral loads acting on the retaining walls are discussed below. Sliding resistance should be calculated in accordance with Section 6.10.5 of the CHBDC 2014. A coefficient of friction (tan δ) of 0.45 may be used in the sliding assessment between concrete and compacted granular fill, and a coefficient of friction (tan φ') of 0.3 may be used in the sliding assessment between concrete and cohesive fill. The above coefficients of friction are un-factored and a resistance factor of 0.8 should be applied in accordance with Table 6.2 of CHBDC 2014 based on the available subsurface conditions. Overturning analysis of the retaining walls will be carried out by the structural designer, following common design methods for retaining wall design.

8.1.2 Tie-Back Anchors

It is understood that permanent tieback anchors will be used to reduce the lateral movement of the wall and to reduce the settlement of the existing tracks. All permanent tieback anchors should be designed in accordance with the Metrolinx General Guidelines for Design of Railway Bridges and Structures (March 2018) and applicable Manual for Railway Engineering (AREMA 2008 and 2012). Permanent tiebacks should be designed for a service life of 100 year and have triple corrosion protection as outlined in Metrolinx General Guidelines for Design of Railway Bridges and Structures (March, 2018).

Usually tremie-grouted soil anchors cannot develop higher capacities. Therefore, small diameter, pressure-grouted soil anchors are recommended which can develop higher capacities than tremie-grouted anchors.

Suitable bond zones for soil anchors are considered to be present in the native, undisturbed cohesive to non-cohesive tills. However, these soils may contain pockets/layers of water-bearing zones of granular soils. There is a risk during drilling and anchor installation of disturbing or loosening these soils if they are encountered around the bond zone; disturbance would significantly reduce the load-carrying capacity of the anchors. The ultimate soil-to-grout bond stress achieved for any soil anchor will depend on the specific composition and state of the soils along the bond length. For these reasons, it is recommended that all soil anchors be drilled using controlled density drilling fluids and that hollow-stem auger type anchor drilling equipment be prohibited from use. Otherwise, significant loss and/or disturbance of ground may occur when drilling through the saturated granular soils. Temporary casing or drilling fluid may be required to install the tiebacks where the native soils do not have adequate cohesion to prevent caving or raveling and where groundwater is present.

For preliminary design purposes it can be assumed that straight-sided, pressure-grouted soil anchors may achieve the ultimate bond stresses given in Table 19 below, provided that the installation methods are suitable for the soil and groundwater conditions.

A suitable method for improving soil anchor capacity and performance is considered to be the use of secondary/tertiary grouting of the bond zone. In addition to increasing the apparent bond stress, this technique allows for individual anchors to be re-grouted to improve the soil-to-grout bond stress if proof-testing during construction demonstrates a particular anchor to be deficient. To be effective in increasing anchor capacity, the grout pressure must be sufficient to fracture the primary grout and the surrounding soil mass. It is estimated that secondary grouting may increase these ultimate values listed in Table 19 by approximately 25% though the increase in bond strength is dependent on the contractor's equipment, methods, and workmanship, as well as soil properties.

Within the table below, ultimate bond stress values are provided for the fill and native materials whether above or below the groundwater level. These values are considered preliminary for initial evaluation of support options and will need to be verified by field testing prior to installation of production anchors. Furthermore, it is noted that the fill materials as encountered in the boreholes were variable in composition and density/consistency. Although higher capacities may be achievable in the fill materials, reliance upon higher values at this stage of design is not recommended.

Table 19: Preliminary Soil Anchor Bond Stresses for Design

Soil Type	Ultimate Bond Stress Grouted Anchors
Clayey Silt Fill	30 kN/m
Sandy Fill	60 kN/m
Clayey Silt Till	80 kN/m
Sandy Silt Till	130 kN/m

The above bond stresses are based on the following assumptions:

- The tie-backs have a nominal outside diameter between 150 and 200 millimetres;
- The grout is injected using a minimum positive pressure of about 1 MPa;
- The effective bond length of the tie-backs is not more than 8 metres;
- The minimum spacing of the anchors is more than 1.6 m; and
- Ultimate bond stresses can be increased by approximately 25% if secondary grouting is carried out.

A factor of safety of at least 2 should be applied to the calculated ultimate capacity of the soil anchors.

In general, to limit the vertical load component, while still permitting reasonable control of drilling and grouting fluids, it is recommended that the soil anchors be installed with an inclination of about 15° from horizontal. Further, the bond zone should be located a distance of 0.15H behind a line drawn from the point of zero net earth pressure to the ground surface at an angle of about 30° measured from the plane of the wall, where H is the wall height. It will be critical that the free length of the anchors, between the bond zone and the retaining wall, be filled with a low-strength, compressible filler grout. Otherwise, the tested anchor capacity may be misleading with the anchor developing initial tensile capacity within the active ground mass behind the wall, but not actually providing the full resistance to wall loading that the test results indicate.

The tieback anchors should be performance-and proof tested to confirm that the tiebacks have adequate pullout capacity. Performance and proof tests should be completed as described in Metrolinx General Guidelines for Design of Railway Bridges and Structures (March 2018) Article 5.1 and 5.2 of Part 6 and OPSS 942 (Construction Specification for Prestressed Soil and Rock Anchors).

Proof testing of the anchor unit capacity should be conducted by incrementally loading and unloading selected test anchors in cycles as described in Metrolinx General Guidelines for Design of Railway Bridges and Structures (March 2018) Article 5.1.1 through 5.1.4 of Part 6.

The acceptance of the tieback anchors should be based on the criteria described in Metrolinx General Guidelines for Design of Railway Bridges and Structures (March 2018) Article 6 of Part 6.

It is recommended that the structural engineer review the contents of this report and adjust the current tie-back design utilizing the comments provided above.

8.2 Lateral Earth Pressure Design for Permanent Retaining Walls

The soil parameters provided in Table 20 may be used for the estimation of earth pressures:

Table 20: Lateral Earth Pressure Coefficients

Soil Description	Bulk Unit Weight (kN/m^3)	Effective Friction Angle, (°)	Undrained Shear Strength, su (kPa)	Cohesion Intercept, C′, kPa
Non-Cohesive Granular Fill	21	35	0	0
Cohesive Fill	19	28	25-50 (Note 1)	0
Very stiff to hard Clayey Silt to Silty Clayey Till	21	32-34	100-200	5-15 (Note 2)
Very dense Sand and Silt Till	22	34-36	0-5	0

Notes:

1. For the purpose of undrained analyses, minimum undrained strength of the silty clayey till, based on Figure 1 above is 30 kPa. For the design purposes, a range of 25 kPa to 50 kPa is applicable.
2. For the effective strength based design purposes, a cohesion intercept range of 5 kPa to 15 kPa is applicable.

The following recommendations are made concerning the design of the retaining walls. It should be noted that these design recommendations and parameters are related to level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope. Further these recommendations assume conventional backfill material is placed behind the retaining walls. Metrolinx General Design Guidelines require that all structures supporting rail infrastructure shall be designed for at rest conditions.

The minimum earth pressure distribution acting on the walls through the overburden may be calculated on the basis of the following equation:

$$\sigma_z = [d\gamma + \gamma'(z-d) + q] \times K$$

Where:

σ_z	= effective lateral earth pressure acting at depth z
K	= earth pressure coefficient, provided below
γ'	= effective unit weight of retained soil, provided below
γ	= total unit weight of retained soil, provided below
d	= depth to water table below ground surface (consider groundwater elevation range between 152 m and 154 m)
q	= uniform surcharge at ground surface behind the wall (including the loads incurred by existing structure and traffic loading)

Where drainage is not provided, full hydrostatic groundwater pressure should be included in the design ($\sigma_w = \gamma_w h_w$, where γw = 10 kN/m3).

The lateral earth pressures acting on the permanent retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction and traffic loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls.

Seismic (earthquake) loading should be taken into account in the design. These estimates are based on the Monobe-Okabe (M-O) pseudo-static method of analysis. The M-O method produces seismic loads that are more critical than the static loads that act prior to an earthquake. It should be noted that in computation of seismic earth pressure coefficients, the wall back-face geometry, backfill slope and wall friction need to be addressed.

The horizontal seismic coefficient, K_h used in the calculation of the seismic active earth pressure coefficient, is defined as a ratio of the peak ground acceleration in the horizontal direction to the gravity acceleration, g. The seismic coefficient in the vertical direction K_v is defined similarly.

For design purposes, the following unfactored seismic lateral earth pressure parameters can be used (assuming wall friction is neglected, the back wall is vertical, and the ground surface is horizontal in front of the toe):

Table 21: Unfactored Seismic Lateral Earth Pressure Coefficients

At Wall Movement Condition (Lateral Yielding)	Compacted Granular 'A' and Granular 'B' Type II		Non-Cohesive Granular Fill		Cohesive Fill	
	Angle of Internal Friction, 35°		Angle of Internal Friction, 35°		Angle of Internal Friction, 28°	
	Unit Weight = 22 kN/m^3 (Wall friction neglected)		Unit Weight = 22 kN/m^3 (Wall friction neglected)		Unit Weight = 19 kN/m^3 (Wall friction neglected)	
	Top Ground Surface Angle		Top Ground Surface Angle		Top Ground Surface Angle	
	Horizontal	2H:1V	Horizontal	2H:1V	Horizontal	2H:1V
Seismic Active Earth Pressure (K_{AE})	0.3	0.48	0.3	0.48	0.4	0.8
Seismic Passive Earth Pressure (K_{PE})	NA	NA	NA	NA	NA	NA

It should be noted that pressures from backfill compaction equipment are either also included in the design or controlled (with respect to distance and size of equipment) to avoid exceeding the wall design pressures. Surcharge pressures from traffic should also be included in the design for the permanent retaining walls.

A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem. Compaction equipment should be used in accordance with OPSS 501.06. A rail live load of Copper E90 will be considered in the design. Other surcharge loadings should be accounted for in the design, as required.

The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I) in or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II).

For Case I, the pressures are based on the existing overburden soil materials and the unfactored soil parameters provided below in Table 22 may be used.

Table 22: Soil Parameters for Case I (Existing Fill)

Parameters		Granular Fill	Cohesive Fill
Total Soil Unit Weight (γ), kN/m^3		21	19
Effective Soil Unit Weight (γ), kN/m^3		11	9
Angle of Internal Friction, degrees		32-35	28
Coefficient of Lateral Earth Pressure	"active" (K_a)	0.31-0.27	0.36
	"at-rest" (K_o)	0.50	0.53

For Case II, the pressures are based on the granular fill as placed and the following unfactored parameters (Table 23) may be assumed:

Table 23: Soil Parameters for Case II (New Fill)

Soil Unit Weight:	Granular 'A' or Granular 'B' Type II 22 kN/m^3	Granular 'B' Type I 21 kN/m^3
Coefficients of static lateral earth pressure: Active, K_a	0.27	0.31
At rest. K_o	0.50	0.50

Passive pressure in the upper 1.2 m below ground surface should be ignored.

For retaining structures not supporting rail infrastructure, If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the wall support does not allow lateral yielding, at rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:

- Rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
- Horizontal translation of 0.001 times the height of the wall for active earth pressure to activate; or a combination of both

The resistance to lateral loading versus deflection in front of a single pile may be calculated from the coefficient of horizontal subgrade reaction (k_h in kPa/m). k_h is determined based on the equations given below (CFEM 1996, CHBDC, 2014):

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

Where:

k_h = the coefficient of horizontal subgrade reaction (kPa/m);

n_h = the constant of subgrade reaction (kPa/m);

z = the depth (m); and

B = the pile diameter or width (m).

For cohesive soils:

$$k_h = \frac{67 S_u}{B}$$

Where:

k_h = the coefficient of horizontal subgrade reaction (kPa/m);

S_u = the undrained shear strength of the soil (kPa); and

B = the pile diameter or width (m).

The following values of n_h and S_u in Table 24 may be assumed in the structural analyses.

Table 24: Values of n_h and S_u

Soil Unit	n_h (kPa/m)	S_u (kPa)
Compact sandy fill	8,000	
Very stiff clayey silt till		150-200
Hard sandy clayey silt till		200-250
Compact sand (Northeast end)	5,000	

8.3 Global Slope Stability Analysis

Slope stability analyses have been completed for representative critical slope sections selected for their height and proximity to the rail corridor and Highway 401. The sections are shown on the Civil Grading Plan 1 and 2 drawings (Drawing Nos. C-101 and C-102) in Appendix O. The sections analyzed are listed in Tables 25 and 26 below. Target minimum factors of safety of 1.3 and 1.5 have been used for the design of the embankment slopes against deep-seated global failures for temporary and permanent conditions, respectively. Target minimum factor of safety of 1.1 has been used for design seismic event. The design long-term phreatic surface was assumed to correspond to the initial groundwater level based on information from boreholes and piezometers and follow the excavation and subgrade near surfaces. Two (2) scenarios of groundwater level were considered in the analyses when slope contained retaining walls: a) groundwater with a drainage system behind the retaining wall; and b) groundwater with no drainage system behind the retaining wall. The analyses results considering groundwater with a drainage system behind the retaining wall are present in Table 25 below. The results considering groundwater with no drainage system behind the retaining wall are present in Table 26.

In consideration of the potential for ground acceleration to be generated during design earthquake at this site, the seismic performance of the slopes was assessed using a pseudo-static slope stability analysis. For the seismic stability analysis, horizontal seismic coefficient, K_h was taken as half of the seismic horizontal acceleration coefficient. In addition, the seismic vertical acceleration coefficient K_v was taken as 10% of the seismic horizontal acceleration coefficient. Based on NBCC 2015 seismic hazards map, the design seismic horizontal acceleration coefficient is 0.12 for this site.

The SLOPE/W computer program developed by GeoSlope International was employed for computation of the factor of safety, using the Morgensten-Price method to illustrate the static slope stability analysis, developed on the basis of limit equilibrium.

The results of these analyses indicate that permanent embankment side slopes no steeper than 2H:1V will have the Factors of Safety against global stability summarized in Tables 25 and 26 provided the bottom of retaining wall/filler caissons used for retaining wall where applicable is established at the minimum elevation indicated in Tables 25 and Table 26 below and shown on the slope models attached in Appendix O for the sections analyzed. Analyses results for the relevant cases are also provided in Appendix O.

Table 25: Summary of Global Slope Stability Analysis (Considering a Drainage System behind the Retaining Wall)

Slope Section	Factors of Safety (FS) for Global Slope Stability			Minimum Required Bottom Elevation* of Retaining Wall/Caissons to Meet Minimum FS for Global Stability (m)
	Temporary	Permanent	Seismic	
W1 South	1.3	1.6	1.2	148.8
W1 North	1.3	1.5	1.2	148.8
W2 South	1.5	1.5	1.3	149.5
W2 North	1.4	1.6	1.4	149.5
Between E1 and E2 South	1.3	1.5	1.1	146.5
Between E1 and E2 North	1.4	1.5	1.2	146.5
E2 South	1.3	1.5	1.1	146.5
E2 North	1.3	1.5	1.1	147.0
E5 North	1.3	1.5	1.2	No Retaining Wall on North Side

Table 26: Summary of Global Slope Stability Analysis (Considering No Drainage System behind the Retaining Wall)

Slope Section	Factors of Safety (FS) for Global Slope Stability			Minimum Required Bottom Elevation* of Retaining Wall/Caissons to Meet Minimum FS for Global Stability (m)
	Temporary	Permanent	Seismic	
W1 South	1.3	1.6	1.2	145.5
W1 North	1.3	1.6	1.2	146.0
W2 South	1.3	1.5	1.1	148.0
W2 North	1.5	1.5	1.4	149.2
Between E1 and E2 South	1.3	1.5	1.1	145.0
Between E1 and E2 North	1.4	1.5	1.3	146.0
E2 South	1.3	1.5	1.1	146.0
E2 North	1.3	1.6	1.1	147.0
E5 North	1.3	1.5	1.2	No Retaining Wall on North Side

Embankment reconstruction, where implemented, should be constructed with in accordance with OPSD 200.010 (Earth / Shale Grading, Undivided Rural) or OPSD 200.020 (Earth / Shale Grading, Divided Rural), OPSS 206 (Construction Specification for Grading). If required, the existing slopes should be benched in accordance with OPSD 208.010 (Benching of Earth Slopes).

Erosion and drainage control measures are recommended for all permanent slopes. Erosion protection measures may include, but not limited to vegetative mats, hydro-seed layers etc. Surface drainage on the crest of the embankment should be directed away from the slopes.

8.4 Temporary Excavation Slopes and Temporary Protection Systems

Where space and construction activities permit the construction of unsupported open-cut excavations, these excavations should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. Based on the OHSA classification system, the soils to be excavated on site would be classified as follows:

- **Fill Materials** **Type 3**
- **Very stiff to hard clayey silt till** **Type 2**

Temporary unsupported excavations for utility construction (i.e., those that are open for a relatively short time period) will be made with side slopes no steeper than 1H:1V. Stockpiles of excavated materials and heavy construction equipment should be kept at least the same horizontal distance from the edge of excavation as the depth of the excavation to prevent local instabilities. If steeper temporary excavations are required due to space limitations, then engineered support methods, such as a trench box, will be employed.

Temporary support of the highway embankments and rail line will be required during the removal of the existing retaining walls and the construction of the new retaining walls. The retaining wall excavations are expected to encounter the existing fill and the very stiff to hard native clayey silt till. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act and Regulations for Construction Projects, CHBDC and AREMA, as applicable.

The temporary systems supporting the highway embankments and rail lines should be designed and constructed in accordance with OPSS 539 (Construction Specification for Temporary Protection Systems) and Chapter 8 – Part 28 of Metrolinx General Guidelines for Design of Railway Bridges and Structures. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that any adjacent utilities/structures can tolerate this magnitude of deformation.

The design of the temporary work supports must consider the temporary works design as part of the assessment of ground movements and their impact on existing structures and underground utilities at the site. The criteria for the design and performance of the temporary works must also consider such analysis.

8.5 Caisson Construction Concerns

Caisson construction should generally be carried out in accordance with Special Provision No. 903S01.

Groundwater seepage into the caisson excavations is anticipated at the site. Depending on the period of the year, "perched" groundwater may also be encountered within the fill soils above the clayey silt till layer. Soil sloughing and water seepage may also occur in the unsupported hole. Therefore, temporary liners should be available to support the caisson sidewall to control ground loss and mitigate settlement of the existing rail tracks, and provide seepage cut-off where required. Localized zones of non-cohesive soils were observed to exhibit heaving conditions during the advance of some of the supplementary boreholes. The caisson founding elevations are expected to be sufficiently above the pressured non-cohesive zones to prevent heave of the caisson base. However, should basal heave of the caissons occur during construction resulting in disturbance of the caisson base, the caisson should be advanced to encounter undisturbed founding soil, below the zone of basal heave.

Concerns regarding cobbles or boulders handling and removal will be discussed in the following section. Recommendations on how to address the issue will also be outlined.

8.6 Lateral and Vertical Deformation Study for the Retaining Walls

A purpose of Wood's Deformation study is to validate the zone of influence (defined as per PA Output Specification, Schedule 15, part 3-7) induced by the construction of new retaining walls. It is understood that it is the responsibility of the retaining wall designer (i.e. WSP) to ensure the actual deformation of the new retaining wall satisfies the design criteria.

The deformation study was performed and reported under a separate cover (submission S104B-INST-3383-002, R0). The detailed design basis of the retaining walls itself is available in the submission titled "Structural Design Report (Retaining Walls + Portals / Headwalls), RER Highway 401 Rail Tunnel on the Kitchener Corridor, 90% DD-Structural Design Package, S104A-STR-3383-001 R1 dated April 18, 2019 (WSP,2019). The analyses considered geotechnical investigation and in-situ/laboratory test results available from the previous investigations and data gathered from current investigations conducted by Wood.

To accommodate the design track elevations for the new rail corridor east and west of the proposed tunnel, cut slopes with benches and permanent retaining walls are proposed at the east and west ends of the proposed tunnels. The new track profile will eventually meet the existing track elevations. The retaining walls will be either caisson walls or H-pile walls, depending on the retained height of the soil. It is understood that the south retaining walls are rail-carrying structures and hence appropriately sized tie-back anchors are proposed for the southeast and southwest retaining walls to ensure the wall deformations are within the limits of movements as described in RER 401 Tunnel PA Schedule 15 Part 3-7, Article 2.4 Part 7 and 10. It is understood that these tie-back anchors will be designed as permanent structures.

8.6.1 Retaining Wall Deformation Analyses

For the purpose of structural design, WSP performed a 3-D finite element analyses for each wall using SAP 2000 software considering the assigned soil – structure interaction. The purpose of the structural modeling was to identify the location and magnitude of maximum internal stresses (axial, shear, and bending moment) of the structure under design. Detailed description of the design analyses, magnitude of lateral and vertical soil displacements including material parameters and boundary conditions are available in WSP report (submission S104A-STR-3383-001 R1 dated April 18, 2019).

8.6.2 Geotechnical Soil – Structure Interaction Analyses

Based on the geotechnical investigations, the base of the fill at the East and West end retaining walls are around elevations of 150.5 m and 154.5 m, respectively. The soil layers beneath the fill is identified to be clayey silt till and sandy silt till.

Some pertinent consideration for the deformation analyses are:

- Retaining wall deformation: per PA requirement, the maximum allowable vertical and horizontal deformation is 20 mm. Maximum allowable wall rotation is 0.3 % of the retained height. Also, as per PA, these limits of movements are not applicable if such displacements/rotations:

  - Occurs only during the initial loading of such retaining walls;
  - Is incidental to the construction of such retaining walls;
  - Is within an acceptable range for such deformations established in accordance with the design for the retaining walls completed by TTP and good industry practice.

- Train service load [for railway tracks only, applicable to South walls: as per Metrolinx design guideline, and AREMA standard], Cooper E80 load was considered at service, and assumed to be taken by the closest rail track to the retaining wall. Based on Wood's discussions with TTP and Structural designers (WSP) it is understood that in addition to the Retaining Wall Limits of Movement as given in PA Schedule 15, Part 3-7 Article 2.4.10, TTP has also considered the Rail Infrastructure Limits of Movement as given in PA Schedule 15, Part 3-7 Article 2.4.7 to ensure the Rail Infrastructure Limits of Movement are not exceeded during the completion of works.
- Construction sequence; Refer to retaining wall design (submission S-104B-STR-3383-001, R1 dated June 18, 2019) for the excavation sequence, existing rail tracks re-location, and proposed rail tracks ballast fill placement.
- Tie-back anchors: Tie-back anchors, five on southeast and three on southwest caisson retaining walls, are proposed to limit the wall deformation.

Due to the close proximity of the Metrolinx rail infrastructure, a 3-dimensional soil – structure Finite Element Modelling (FEM) was carried to validate the zone of influence and analyze the deformation of the north retaining walls proposed for the RER 401 rail tunnel project. The analyses were carried out using PLAXIS3D software.

8.6.3 Description of Modelling

The PLAXIS3D embedded pile model was used to model the lateral and vertical deflections. The embedded pile model consists of beam elements connecting to the surrounding soil by means of special interfaces, a particular elastic region around the pile whose dimension is equivalent to the pile diameter is assumed in which plastic behaviour is neglected, and the installation effects of the pile are not taken into account. Therefore, the embedded pile model may be applied effectively in modelling the piles in which installation process results in low disturbance. The embedded pile model is also influenced by pile properties (pile length, pile diameter).

To simplify the model, one section from each side are constructed over an extended length. In other words, a 2D analysis was performed in a 3D environment. The results from the modelling and subsequent analysis are reported in submission S104B-INST-3383-002, R0. The vertical and horizontal deformation values obtained from PLAXIS3D analyses (Wood, 2019) are the maximum deflections anticipated at this extended length, taking into account the boundary effect of the tunnel (tunnel is assumed to have zero normal movement) and changing geometry of the caisson wall. Lower deflections of the caisson wall are reported in the 3D SAP model completed by WSP. For detailed values on the anticipated vertical and horizontal deformation of the new retaining walls, reference should be made to report titled "Structural Design Report" completed by WSP with submission number S-102B-STR-3383-001.

9.0 Construction Considerations

Major excavation concerns related to retaining wall design include control of the excavation faces, control of ground deformations and the presence of unknown obstructions. Other excavation concerns include the presence of cobbles/boulders and temporary groundwater control.

9.1 Cobble/Boulder Obstructions

If cobbles/boulders are encountered during the tunnel, retaining wall and/or high mast light pole excavations they can be easily removed, as the size of the excavation is expected to be larger than cobble/boulder diameter. If necessary, the cobble/boulder can be pulverised to facilitate removal. Further, the upper soils below the highway pavement are comprised of granular and cohesive fills, therefore it is not anticipated that cobbles/boulders would be encountered during the installation of the pipe arch, which will be installed in the fill materials. Voids in the tunnel sidewalls created by boulder removal should be filled with shotcrete, where needed. If other obstructions, such as waste concrete or construction debris are encountered within the fills, alternative drilling tools are available to advance the drill hole. Mitigative measures, in the event of cobble or boulder obstructions encountered in the fills above the tunnel will be addressed in the Tunnel Design Report.

9.2 Temporary Groundwater Control

Groundwater levels were measured in boreholes and monitoring wells during the supplementary investigation and during each previous geotechnical investigation between 2015 and 2018. These groundwater levels have ranged between 3.4 mbgs and 7.8 mbgs (elevations of 154.9 and 150.3 m).

During the supplementary geotechnical investigation, groundwater was encountered during and upon completion of drilling at boreholes BH2017-02, BH2017-06 and BH2017-07. The groundwater level in these three boreholes was approximately 20.9 m, 9.9 m and 12.2 m respectively. Boreholes BH2017-03, BH2017-05 and BH2017-08 were equipped with monitoring wells. The groundwater levels measured in the monitoring wells on April 19, 2018 were 3.40 m in BH2017-03, 7.53 m in BH2017-05 and 3.83 m in BH2017-08. Groundwater seepage into the excavations is anticipated at the site. Borehole drilling as well as CPT investigation results indicate the presence of permeable layer with perched ground water. Care should be taken in advancing the excavation through perched ground water zone.

A Dewatering Management Plan has been prepared by Wood and is presented under separate cover, as Document Number: S121-ENV-3383-011 dated July 26, 2018.

9.3 Bedding and Backfill Recommendations

The native till deposits encountered at the majority of the borehole locations will provide adequate subgrade support for the potential services. Where very loose to loose soils are encountered at the trench base, or where disturbance of the trench base has occurred such as due to groundwater seepage or construction traffic, such materials should be sub-excavated and replaced with selected granular fill compacted to at least 98% of Standard Proctor Maximum Dry

Density (SPMDD). The support of pipes in these areas can also be achieved with placement of Controlled Density Fill after the unsuitable, deleterious materials are removed.

Bedding and cover for the sewers must be constructed in accordance with the Ontario Provincial Standard Specifications 410 and 514. The bedding and cover requirements for the sewers shall be a Class "B" bedding consisting of a Granular "A" material compacted to 100% SPMDD, and in accordance with the OPSS specifications for pipe bedding. Embedment material consisting of Granular "A" should be used to backfill around the pipe to at least 300 mm above the top of the pipe. This backfill should be placed in thin layers and each layer compacted to at least 100% SPMDD.

Granular “A” material can be used as bedding for the sewers where the subgrade conditions in the trench are dry and stable. Recycled asphalt will not be allowed for use in Granular “A” bedding material. In the case of saturated subgrade, the use of clear stone gravel or HL4 coarse aggregate, properly wrapped in a geotextile filter fabric, may be considered. If this is done, all joints should be overlapped and stitched and there should be no breaks or rips in the filter fabric as these would permit the entrance of fine minerals resulting in lost ground and potential settlement of the ground surface and/or the pipe. It should be noted that this is very difficult to achieve in practice, particularly if a trench box is used for side support. The bedding materials must be placed in lifts not exceeding 150 mm thick and be compacted to a minimum of 100% of SPMDD or compacted to a dense state by vibration in the case of clear stone bedding material.

Backfill to the retaining walls will consist of granular fill meeting the requirements of OPSS 1010 Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill and bedding will be placed and compacted in accordance with OPSS 501 (Compacting) placed in maximum 300 mm thick loose lifts and compacted to at least 95 percent of the material’s standard proctor maximum dry density. Light compaction equipment will be used immediately adjacent to the wall; otherwise compaction stresses on the wall may be greater than that imposed by the backfill material.

9.4 Frost Protection

A minimum depth of earth cover of 1.2 m (or equivalent synthetic insulation), should be provided for frost protection, as per OPSD 3090.101, Foundation Frost Penetration Depths in Southern Ontario.

9.5 Erosion Protection

Project Issued for Construction (IFC) drawings generally have erosion and sediment control measures identified, however, these measures represent a single point in time. Erosion and Sediment Control (ESC) on any project is a dynamic process wherein adjustments are required to fit the construction sequence, which can also be a dynamic process. Therefore, where adjustments to the ESC plan are necessary to prevent the initiation of erosion, TTP shall implement the adjustments accordingly.

Prior to the commencement of any work on the RER 401 Tunnel that creates conditions where erosion and/or sediment transport might be initiated, the Construction Manager and/or the Environmental Manager will review the erosion and sediment control procedures that the Construction Team proposes to use. These procedures shall be in accordance with OPSS 805 and the Erosion & Sediment Control Guidelines for Urban Construction, published by the TRCA, December 2006 (Erosion Guidelines).

10.0 Recommendations for Monitoring

10.1 Monitoring of Ground Losses During Tunneling

Soil excavation volumes will be estimated throughout the construction period as a means to infer potential excessive loss of ground by comparing measured to theoretical excavation volumes, however performance monitoring of the surface, sub-surface and in-tunnel monitoring systems will provide more reliable indications of the ground behaviour during the tunneling operations. During grouting, grout volumes will be measured and also compared to theoretical volumes to identify if larger than expected volumes have been used, indicating the possible presence of a void space in the grout zone. The real-time monitoring program will provide evidence of ground loss by measuring deformations during and after the tunnel construction. In addition to the displacement monitoring, noise and vibration monitoring will be carried out in compliance with Schedule 15 Part 5-7 of the PA.

10.2 Instrumentation and Monitoring of Displacements of Ground and Existing Tunnel

Schedule 15, Part 3-7 of the Project Agreement provides the requirements for Instrumentation and Monitoring. A detailed discussion of the Instrumentation and Monitoring plan is provided under separate cover in the Geotechnical Instrumentation and Monitoring Plan, as Work Submittal: S-104A (100% Design Development Package). The Geotechnical Instrumentation and Monitoring Plan will be developed to maintain a safe work area for the construction personnel and the public, measure actual displacements in real time, compare the measured displacements with the modelled results, modify the design and construction methods to address discrepancies between the measured and modelled displacements, and confirm that the Limits of Movement included in Part 3-7 have not been exceeded.

The instrumentation and monitoring program will utilize instruments incorporating the latest technology available in the tunneling practice. Non-intrusive monitoring such as reflectorless Robotic Total Station based monitoring (complemented with surface targets) and micro electro mechanical system (MEMS) based monitoring instruments (MEMS in-place inclinometers and/or Shape Accel Arrays (SAA), MEMS crack-meter, etc.) will be installed in the following critical infrastructures:

- Highway 401 lanes and adjacent structures;
- Highway 409 lanes and associated retaining wall system;
- The existing rail tunnel; and
- The existing rail corridor;

All automated instruments will be connected to a secure internet service to provide a secure web-based Automated Data Acquisition and Management System (ADAMS). The ADAMS will be GIS based using a secure internet connection capable of receiving, processing, and presenting real-time instrument data. Relevant instrument calibration parameters and data reduction algorithms will be inputted to the ADAMS so the users can visualize the end result. Robotic survey system and MEMS based monitoring can easily be incorporated into the real-time monitoring and interpretation as the tunneling progresses.

The monitoring program will consist of three primary phases: pre-construction, during construction and post-construction. The pre-construction phase is to be completed 6 months prior to the start of construction and includes visual condition surveys of the pavements, existing tunnel and existing infrastructure. The visual pavement surveys will be supplemented by ground penetrating radar (GPR) measurements. Instrumentation installation will be carried out at the start of the 6 months pre-construction period and baseline readings will be established during this period. The construction period is estimated to extend for approximately 24 months. During construction, all instrumentation will be continuously monitored by automated and manual surveys. Data will be uploaded and stored on secure server accessible to all approved parties and triggers will be established to automatically inform pre- selected parties of measurements reaching review, alert and stop levels. A 12-month duration post-construction period will follow the completion of construction. Instrumentation monitoring will continue during this period, and a post-construction survey will be completed, including GPR measurements of the highway pavement structure.

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11.0 Foundation Design Recommendations for the Proposed High Mast Lighting Pole (HMLP)

11.1 Introduction

This section of the report provides geotechnical parameters and recommendations for the detailed design of a proposed HMLP located north of the existing rail tunnel beneath Highway 401 between Islington Avenue and Kipling Avenue, Kitchener Rail Corridor, Toronto, Ontario, and approximately 5 m North of the existing HMLP.

Three geotechnical investigations were done previously for this project between 2010 and 2016. In addition, Wood performed a supplementary investigation in 2018.

11.2 Foundation Design Soil Parameters

Table 27 provides the recommended design soil parameters based on interpretation from the investigations referenced in the report.

Table 27: Design Soil Parameters for HMLP

Design Soil Parameters				
Soil Description	Bulk Unit Weight (kN/m^3)	Internal Friction Angle, (°)	Undrained Shear Strength, S_u (kPa)	Rankine Passive Earth Pressure Coefficient (2)
Non-Cohesive Granular Fill	21	30-35	-	1.0 - 3.7
Cohesive Fill	19	-	25-50	-
Very stiff to hard Clayey Silt to Silty Clay Till	21	-	100-200	-
Very Dense Sand and Silt Till	22	34-36	-	1.0 - 3.8

Notes: (1) Where unconfined compressive strength equals 2 times the undrained shear strength for a cohesive soil (Cohesive Fill and Clayey Silt Till).
(2) The passive earth pressure coefficient is calculated from: $K_p = \frac{1+\sin\varphi'}{1-\sin\varphi'}$ (where φ' is the effective friction angle) for the full mobilization of the passive resistance. A soil – structure analyses along with acceptable deformation should be performed by the design engineer in consultation with the geotechnical engineer. If no soil – structure analyses is performed, Kp = 1.0 should be used in the design to limit the movement.

11.3 Lateral Loads and Resistances on HMLP Design Considerations

The Guidelines for the Design of High Mast Pole Foundations (MTO, 2004) include standard design procedures for the design of HMP foundations subject to wind loads. It is understood that the foundations for HMLP are made of reinforced, cast in place concrete and are classified as caisson type piles. As indicated, Table 27 provides the recommended foundation design parameters for the proposed HMLP foundation.

According to Guidelines for the Design of High Mast Pole Foundations, the soil resistance within the larger of the entire frost depth zone, very weak top layer of soil, or 1.5 times the diameter of caisson should be ignored for rotation, ultimate resistance and bending moment calculations of the caisson. Table 28 presents a summary of recommended depth of soil resistance to be neglected in this project.

Table 28: Depth of Soil Resistance to be Neglected for HMLP Design

Application	Caisson in Cohesive Soil	Caisson in Cohesionless Soil
Rotation	1.2 m	1.2 m
Ultimate Resistance	Larger of 1.2 m or 1.5D ([1])	1.2 m
Bending Movement	Larger of 1.2 m or 1.5D ([1])	1.2 m

Note: (1) D is the diameter of caisson, m

At Serviceability Limit State (SLS) loads, the caisson should be designed to meet a foundation rotation limit of 0.005 radians. While at Ultimate Limit State (ULS), ULS wind load on the foundation shall be calculated and checked against the ultimate lateral passive resistance. The calculated caisson lengths under both SLS and ULS should be checked against the "short pile" limit as defined by Broms in order for the caisson to remain rigid at the SLS load. An approximate solution proposed in Guidelines for the Design of High Mast Pole Foundations needs to be adopted to solve this layered soil case. It is a modified version of Wong's "Percentage Contribution" method as indicated in the Guidelines, which states that the total percent contribution should be 120% as opposed to the 100% given by Wong. This equation becomes conservative for a larger ranger of layered conditions.

$$\sum \frac{T_i}{L_{REQ_i}} = 1.20$$

Where:

T_i = thickness of layer i

L_{REQ_i} = governing length obtained from the rotation and ultimate lateral capacity calculations for the particular soil type in layer i

The proportions (length and diameter) of the caisson are based on applied load, relative stiffness of caisson to soil and minimum size restrictions to accommodate the pole anchorage. Reinforcement in the caisson is based on applied moment on the foundation at ULS. In order to minimize interference between the anchorage and the reinforcing cage, maximum size of reinforcement bars is also recommended in Guidelines for the Design of High Mast Pole Foundations.

Based on the recent discussions between the CA, and TTP, it is understood that a 45 m high mast light pole is being considered. In this report, geotechnical parameters for four different heights of 25 m, 30 m, 35 m and 45 m for the HMLP are discussed. According to Guidelines for the Design of High Mast Pole Foundations (MTO 2004), the minimum length of the drilled shaft (caisson) as the foundation supporting the HMLPs is suggested and factored serviceability and ultimate limit states wind loads for design in Toronto area is calculated from CHBDC 2014 and the ultimate lateral resistance related to the minimum length of the caisson is listed in Table 29.

The design is based on the 50-year reference wind pressure (q_{50}) in accordance with Table A.3.1.1 in CHBDC CSA S6-14.

Table 29: Factored ULS and SLS, Minimum Embedment and Lateral Resistance of the Caisson Foundation for HMLP

HMLP Height	SLS Wind Load on the Foundation P_{sls} (kN)(1)	ULS Wind Load on the Foundation P_{uls} (kN)(1)	Condition	Minimum Estimated Embedment Depth Based on Lateral Loads (m)(2)	Diameter of Caisson Considered (m)(2)	Total Estimated Length of the Caisson based on Lateral Loads (m)(2)	ULS Lateral Resistance P_{uls} (kN)(2)	SLS Lateral Resistance P_{sls} (kN)(2)
25 m	18	33	On the ground	8.0	1.22	8.3	44	18
			In Median	8.1	1.22	9.3		
30 m	23	43	On the ground	8.9	1.37	9.2	55	23
			In Median	9.1	1.37	10.3		
35 m	31	57	On the ground	10.0	1.37	10.3	69	31
			In Median	10.1	1.37	11.3		
45 m	34	64	On the ground	10.5	1.52	10.8	80	34
			In Median	10.6	1.52	11.8		

(1) Based on CHBDC 2014 CSA S6-14
(2) Based on Guidelines for design of High Mast Pole Foundations

Discussion on resistance to axial loads for the caisson is provided in Section 11.4 below. The total depth of caisson should be verified with the available axial resistances from the soils based on design axial loads.

11.4 Resistance to Axial Loads for Caisson Foundations for HMLP

The applied axial loads on the caissons supporting structures should be checked against the nominal bearing resistances. If insufficient, the diameter and/or the length of caisson should be increased accordingly. The detailed load information of structures is not available at this time.

However, for preliminary design purposes, the minimum diameter and maximum embedment length of the caisson of 1.2 m and 13 m below the grade, respectively, are considered in this report.

In accordance with CHBDC 2014 and CFEM 2006, the nominal axial resistances of the caissons are determined. Accordingly, based on standards, the caisson factored geotechnical axial resistances, in ULS, are calculated and listed in Table 30 below.

Table 30: Summary of Axial Resistances of Caissons for HMLP

Embedment Depth of Caisson (1) (m)	Factored Compression Unit Shaft Resistance at ULS (kPa)	Factored Compression Unit Tip Resistance at ULS (kPa)	Factored Unit Uplift Resistance at ULS (kPa)
1.2 to 7	12	54	9
7 to 9	9	54	7
9 to 13	14	216	11

Note:

(1) Shaft resistance of the top 1.2 m was neglected

11.5 Adfreeze

The foundation design for HMLP should account for groundwater levels reaching the ground surface (finished grade) periodically throughout the year. Soil in contact with the foundation can freeze to the foundation within the frost depth, developing a substantial adfreeze bond. The uplift forces due to adfreeze should be considered in the design. The Canadian Foundation Engineering Manual (CFEM, 4th Edition) indicates that the unit adfreeze stress could reach 65 kPa for fine-grained soils frozen to concrete within the zone of frost penetration. If required, as a measure to reduce the risk of adfreezing the upper portion of the caissons within the frost zone may be backfilled with non-frost susceptible granular materials. Provision should be made for drainage around the foundation perimeter, below the maximum depth of frost penetration. A positive surface grade should be provided to shed runoff before it enters the backfill.

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12.0 References

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Wood Environment & Infrastructure (2019) Deformation Study, Regional Express Rail, Highway 401 Rail Tunnel on the Kitchener Corridor, S104B-INST-3383-002, R0.

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification for Grading
OPSS 501.PROV	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 572	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS 1801	Material Specifications for Corrugated Steel Pipe (CSP) Products

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010 Benching of Earth Slopes

OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3101.150	Walls Abutment, Backfill, Minimum Granular Requirement

Design Build Special Provision (DBSP)

DBSP 0539	Construction Specification for Temporary Protection System
DBSP 0903	Construction Specification for Deep Foundations

List of Special Provisions Referenced in this Report

SP 903S01

13.0 Closure

This 100% Foundation Design Report was prepared by Vishu Vasisht, B.Sc., Geotechnical Project Coordinator, Nazmur Rahman, M.A.Sc., P.Eng. (for Sections 8.1.2, 8.3, and 11), and Mathi Shan, M.Sc., P.Eng. Senior Geotechnical Engineer. Mr. Ty Garde, M.Eng., P.Eng., Principal Geotechnical Engineer and a Designated Foundation Contact for Wood, conducted an independent review of the report.

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