ADDENDUM NO. 1

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

HIGHWAY 427 EXPANSION – EXTENSION SECTION
HIGHWAY 427 NBL AND SBL OVERPASS AT STREET ‘A’
CITY OF VAUGHAN, ONTARIO
ASSIGNMENT NO.: 2014-E-0056
WO 2016-11005
WORK ORDER NO. 18A

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PRELIMINARY FOUNDATION INVESTIGATION REPORT (GENERAL)

HIGHWAY 427 EXPANSION – EXTENSION SECTION
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1.0 INTRODUCTION

Peto MacCallum Limited (PML) has been retained by AECOM Canada Ltd. (AECOM) under the Ministry of Transportation, Ontario (MTO) Work Order No. 18 (WO No. 18), as specialist sub-consultant to provide preliminary Foundation Engineering services for the proposed expansion of Highway 427.

This preliminary Foundation Investigation and Design Report provides information for planning purposes at the locations of nine (9) structures and five (5) high fill areas within the extension section of the project as described in Section 2.0 of this report. A separate report will address the widening section of the project. The project limits and general location of each structure and high fill area are shown on the Site Location Plan on Drawing 1.

The Terms of Reference and Scope of Work for the Foundation Engineering services are outlined in the MTO WO No. 18 under Agreement No. 2014-E-0056, issued on June 22, 2015 and the PML revised proposal, dated August 21, 2015.

In addition, preliminary Foundation Engineering services have been provided for one (1) structure at Street 'A' under the MTO WO No. 18A issued as an addendum to WO No. 18 on January 19, 2016. The information pertaining to the location of the Street 'A' structure is included in Sheet K of this revised preliminary Foundation Investigation and Design Report.

This report is based on a desktop study of available GEOCRES reports and supplemental boreholes advanced by PML. The Design-Builder shall satisfy himself as to the sufficiency of the subsurface information and supplement the information as needed to meet the detail design requirements. The existing subsurface investigations must be reviewed at the time of detail design to determine if they meet the then-current MTO requirements for the structure type and span configuration.

2.0 PROJECT DESCRIPTION

The Highway 427 expansion consists of widening a 4 km long section of the existing freeway from the Canadian National Railway (CNR) corridor south of Albion Road to Highway 7 and a 6.6 km northerly extension of the existing freeway from Highway 7 to Major Mackenzie Drive in the City of Toronto and the City of Vaughan. The freeway will be widened by 2 lanes in each direction from the CNR corridor south of Albion Road to Highway 7. The northerly extension of the freeway includes the construction of 8 lanes from Highway 7 to Rutherford Road and 6 lanes from Rutherford Road to Major Mackenzie Drive.

The extension section includes ten (10) structures and five (5) high fill areas. As part of Highway 427 northerly extension, the existing Langstaff Road and Major Mackenzie Drive grade will be modified and new bridge structures are proposed to carry the roadways over Rainbow Creek and West Robinson Creek, respectively.

The overall Highway 427 alignment is oriented in a south-north direction. In general, the surface topography along the Highway 427 alignment is relatively flat to gently sloping toward the south, with sparsely to densely treed areas in the vicinity of Rainbow and West Robinson creeks. Commercial, residential, and industrial developments exist on both sides of the Highway 427 alignment from the southern limit of the project to north of Zenway Boulevard. Farm lands are present within the northern section of the Highway 427 alignment, from north of Zenway Boulevard to the northern limit of the project. A Canadian Pacific Railway (CPR) corridor traverses the northern section of the Highway 427 extension alignment.

3.0 INVESTIGATION PROCEDURES

3.1 Previous Foundation Investigations

Ten (10) GEOCRES reports were available for the structures and high fill areas within the Highway 427 extension section. As part of the previous investigations, sixty-four (64) boreholes were advanced for the proposed nine (9) structures and five (5) high fill areas between February and April, 2009. Eleven (11) piezometers were installed in selected boreholes for the structures and high fill areas. The details of these investigations are summarized in the existing GEOCRES reports.

3.2 Current Foundation Investigation

The existing GEOCRES reports were reviewed and new boreholes were advanced to supplement the existing subsurface information. The level of investigative effort for the current investigation was assigned by MTO in the WO No. 18. Three (3) contingency boreholes were added to the originally assigned investigative effort assigned in the WO No. 18.

The investigation at the location of the Street 'A' structure was assigned by MTO in the WO No. 18A.

The fieldwork for nine (9) structures and five (5) high fill areas was carried out between September 28 and December 8, 2015, during which time a total of twenty-one (21) boreholes were advanced for the structures to depths ranging from 9.6 m to 58.0 m. The fieldwork for the Street 'A' structure was carried out on July 11 to 15, 2016 and comprised four (4) boreholes put down to depths of 23.2 to 24.7 m. The Record of Borehole sheets are contained in site-specific appendices following the preliminary Foundation Investigation and Design Report (FIDR) sheets. The locations of these boreholes together with the boreholes from previous investigations are shown in plan on FIDR sheets for each structure.

The boreholes were laid out by J.D. Barnes Ltd., Ontario Land Surveyors contracted by PML or by PML and surveyed in MTM NAD 83 northing and easting coordinates. Where borehole locations had to be moved, the as-drilled locations were surveyed by PML in reference to the laid out locations.

The field investigation was carried out using truck-mounted and track-mounted drill rigs supplied and operated by Davis Drilling Ltd. of Milton, Ontario and Tri-Phase Group of Mississauga, Ontario. The boreholes were advanced using hollow stem augers or tri-cone using mud rotary drilling techniques. Generally, soil samples were obtained at ground surface and then at depth intervals of 0.75 m to 3.0 m, using a nominal 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures. All boreholes, except boreholes MMD-2 and MMD-3, WRB-1 and WRB-2 were advanced at least 3 m into the "refusal" stratum, defined as a material for which SPT 'N'-values exceed 100 blows per 0.3 m of penetration. Boreholes MMD-2 and MMD-3 were drilled to a depth of 30.0 m and Dynamic Cone Penetration Tests (DCPT) were advanced further from the bottom of boreholes to refusal at 37.2 m and 35.7 m depths, respectively. Although the refusal criteria had been reached at higher levels, boreholes WRB-1 and WRB-2 were further advanced to depths of 23.3 m and 14.2 m and DCPTs were advanced from the bottom of boreholes to refusal at depths of 27.5 m and 15.7 m, respectively, to verify the compactness conditions at the sites.

Where possible, the groundwater conditions in the open boreholes (or inside the augers) were observed during and upon completion of drilling. Piezometers were installed in boreholes MMD-2, MMRC-1 and 427S-1 to permit monitoring of the groundwater level at these locations. The piezometers consisted of nominal 50 mm diameter PVC pipes with slotted screens, surrounded with filter sand and seals placed at selected depths within the boreholes. The boreholes and annulus surrounding the riser pipes above the screen were backfilled to the ground surface with bentonite pellets. All other open boreholes were backfilled upon completion of drilling in accordance with Ontario Regulation 903, as amended by O.Reg 331/B.

Full-time supervision of the fieldwork was conducted by PML engineering staff members who monitored the sampling and in situ testing operations, tied in borehole locations to existing site features and logged the boreholes. PML engineering staff also arranged for the clearance of underground services and appropriate permit applications.

3.3 Laboratory Analysis

The soil samples were identified in the field in accordance with the MTO Soil Classification procedures and transported to the Toronto PML laboratory for further visual classification and testing. Classification testing [water content determination (392), grain size distributions (111) and Atterberg limits (87)] was carried out on selected soil samples. Only index property testing of the soils was conducted and no complex testing (consolidation tests, triaxial tests) was carried out.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geology

The Highway 427 alignment within the project limits lies within the physiographic region known as the Peel Plain, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). A surficial till sheet, which generally follows the surface topography, is present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with scatter silt to sand zones. Shallow, localized deposits of loose sand, silt and/or soft clay scatteredly overlie the till sheet, and represent relatively recent deposits, formed in small glacial meltwater ponds throughout the Peel Plain and often near river valleys. The glacial till sheet is underlain by discontinuous seams of gravel, sand and silt. The site is underlain by grey shale bedrock of the Georgian Bay Formation, which is generally highly weathered in its upper portion.

4.2 Site Specific Descriptions and Subsurface Conditions

Each structure category (underpass, overpass, bridge) and location, site complexity rating (level of investigative effort), and relevant GEOCRES Report with specific boreholes advanced as part of the previous and/or current investigations along with the information for high fill areas are summarized in Table A1 following the text of this report.

A summary of the soil and groundwater conditions encountered at each site, together with site-specific drawings showing the borehole locations and stratigraphic profile are presented on the individual FIR sheets contained in Part C of this report. The detailed subsurface and groundwater conditions as encountered in the boreholes advanced during the current investigation and the results of geotechnical laboratory tests carried out on selected soil samples are given on the Record of Borehole sheets and laboratory test results figures included in the relevant appendices for each structure. A copy of the Record of Borehole sheets and laboratory test results figures from the previous investigations are also included in the relevant appendices.

Occurrence of sloughing of the borehole sidewalls upon completion of drilling was noted and recorded in the Record of Borehole sheets. Where cave-in was noted during drilling, the boreholes remained open by filling the borehole with water. At some locations, with deep boreholes where sloughing was encountered mud rotary drilling was implemented.

It should be noted that the stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic sections are inferred from non-continuous sampling and represent transitions between soil types rather than exact planes of geologic change. The subsoil conditions will vary between and beyond the borehole locations.

Till deposits in southern Ontario typically contain cobbles and/or boulders. Auger grinding, hard drilling and split-spoon sampler bouncing are noted on the Record of Borehole sheets and may suggest the presence of cobbles and/or boulders within the till deposit.

4.3 Groundwater Conditions

Where the drilling techniques allowed, the groundwater level was observed in open boreholes during and upon completion of drilling. The groundwater level measurements in boreholes and piezometers are contained in Table A2 of this report.

It should also be noted that the groundwater level is subject to seasonal fluctuations in response to precipitation events and snow melt and is generally expected to be higher during the spring season and thereafter periods of heavy rainfall.

It should be noted that the sub-artesian conditions were encountered at specific sites and typically occur where a cohesionless soil deposit at depth is overlain by impervious cohesive clayey silt/silty clay till. The details of the artesian conditions are included in the individual FIR sheets contained in Part C of this report, where applicable.

5.0 CLOSURE

This preliminary Foundation Investigation Report was prepared by Mr. Al Varshoi, MESc, P.Eng, and Ms. Marzieh Kamranzadeh, MSc, EIT and reviewed by Mr. Brian R. Gray, MEng, P.Eng. Principal Consultant. The report was independently reviewed by Mr. Carlos M. P. Nascimento, P.Eng., MTO Designated Principal Contact.

Sincerely

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MK/BRG/CN:jk-mi

ADDENDUM NO. 1

PART B
PRELIMINARY FOUNDATION DESIGN REPORT (GENERAL)

HIGHWAY 427 EXPANSION – EXTENSION SECTION
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6.0 ENGINEERING RECOMMENDATIONS FOR PRELIMINARY DESIGN

6.1 General

This part of the report provides preliminary project wide foundation recommendations to assist selection and preliminary design of foundation systems for the proposed ten (10) structures and five (5) high fill areas along the proposed Highway 427 extension section from Highway 7 to Major Mackenzie Drive.

The preliminary recommendations provided herein are based on the interpretation of the factual data obtained from the boreholes advanced during the previous and current investigations at each structure site. The interpretation and recommendations are intended to provide the designers with preliminary information to assess feasible foundation alternatives for the preliminary design of the proposed structure foundations and high fill areas. Further foundation investigation and design will be required during detail design.

Preliminary recommendations for structure foundation are provided in Section 6.2 of this report. For high fill areas identified along the Highway 427 extension section, no new boreholes have been advanced and recommendations are based on the boreholes advanced during the previous investigation and relevant structure boreholes. Preliminary recommendations for high fills are included in Section 6.8 of this report

Where comments are made on construction, they are intended to highlight those aspects that could affect the design of the project, and for which special provisions may be required during construction. Those requiring information on aspects of construction should make their own interpretation of the provided factual information.

For the integral abutment design, the H-piles should be driven to refusal in the very dense sand and silt till or bedrock anticipated at the depths/elevations and designed to reference axial resistances that were provided in the Preliminary Foundation Design Report (FDR) sheets.

Typically, to accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I.

The sites are generally adequate for the use of integral abutments in the Highway 427 extension section.

6.2 Structure Foundation Recommendations

It is understood that nine (9) structures were initially proposed within the extension section of Highway 427 from Highway 7 northerly to Major Mackenzie Drive. One (1) structure at Street ‘A’ was added at a later stage.

It is noted that the current investigation was generally limited to the number of boreholes identified in the MTO WO No. 18, with four additional boreholes advanced at the Street ‘A’ structure under the MTO WO No. 18A. Boreholes were strategically located at selected foundation elements to supplement previous investigations and obtain representative subsurface information. No boreholes were advanced at the approach embankment locations. Further investigations at the final locations of the structure abutments and piers will be required during detail design to obtain subsurface information specific to the foundation locations and to confirm that the subsurface conditions and the geotechnical parameters and resistance values provided in this preliminary design phase are valid for the detail design of the foundations and meet the then-current MTO requirements.

The foundation design for all highway structures must be carried out in accordance with the requirements in the Canadian Highway Bridge Design Code, 2014 (*CHBDC, 2014*). Design of railway grade separations must be carried out in accordance with the local railway authority requirements and American Railway Engineering and Maintenance-of-Way (AREMA) manual.

The following sub-sections provide general and project-wide recommendations applicable to all structure sites and high fill areas, including design assumptions and limitations associated with the recommendations provided in the preliminary Foundation Design Report (FDR) sheets.

Reference to Design-Build standard specifications such as DB 902, DB 903 and DB 539 were included for each site in Part C of this report. Selected Non Standard Special Provisions (NSSP) were provided in the preliminary Foundation Design Report (FDR) sheets, where applicable. Due to preliminary nature of the report and the Design-Build project delivery mode, the contractor was alerted in Part C of this report to potential problems related to cobbles and boulders and vibration monitoring.

6.2.1 Spread Footings

Preliminary foundation recommendations for spread footings on native undisturbed soil (free of topsoil, organics loosened/softened and deleterious materials) or on a granular pad are provided where subsoil conditions are suitable for shallow foundations, as indicated on the individual FDR sheets for each structure.

The granular pad for support of the abutments (and/or piers as designed by the Project Co.) should be designed for site-specific conditions and be at least 2.0 m thick and be comprised of Granular A in conformance with OPSS.PROV 1010 (Aggregates). The granular pad should extend at least 1.0 m beyond the outside edge of the footings in all directions, and then downward at a 1 Horizontal to 1 Vertical (1H:1V) gradient to native soils free of organics and deleterious materials in accordance with MTO guidelines (see Figure 1). The granular pad should be placed in maximum 150 mm loose lifts and uniformly compacted to 100% of ASTM D-689 (Standard Proctor) Maximum Dry Density (SPMDD) in accordance with OPSS.PROV 501 (Compacting).

The preliminary geotechnical resistance values at factored Ultimate Limit States (ULS) and Serviceability Limit States (SLS) for 25 mm of settlement provided in FDR sheets assume a 3.0 m wide footing. These preliminary design values are provided for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Clauses 6.10.3 and 6.10.4 of the *CHBDC* and its *Commentary (2014)*.

The preliminary geotechnical resistance values will have to be re-evaluated and modified if necessary during detail design based on any additional subsurface investigation at the locations of the foundation elements and final arrangement of the footings.

The geotechnical horizontal resistance/sliding resistance between concrete footings and the subsoils (or the granular pad) should be calculated in accordance with Clause 6.10.5 of the *CHBDC (2014)*.

The footings should be provided with a minimum 1.4 m of soil cover for frost protection as per OPSD 3090.101 (Frost Penetration for Southern Ontario), as measured vertically and perpendicular from the face of the abutment slope to the edge of the underside of the footing.

If adequate soil cover cannot be provided for the footing, an equivalent thickness of extruded closed cell insulation (e.g. Styrofoam) should be used to compensate for the lack of soil cover. For preliminary design purposes, an equivalency of 25 mm of insulation for every 0.3 m reduction in soil cover may be used. The insulation sheets should extend laterally at least 1.4 m beyond the edge of the footings. The surface of the insulation sheets should be sloped such that groundwater contacting the impervious sheets is directed away into a ditch.

6.2.2 Driven Steel H-Piles / Steel Pipe Piles

Preliminary recommendations for driven steel H-piles (HP 310x110) are provided where considered practical for foundation design of abutments and piers. Alternatively, consideration was also given to driven steel pipe piles 324 mm (12 ¾ in) outer diameter and 6 mm (1/4 in) thickness. Pipe piles are not preferred at this project site due to presence of boulders.

Based on the subsurface conditions encountered at each foundation element of each structure, the factored geotechnical axial resistance at ULS and the geotechnical axial resistance at SLS for 25 mm of settlement for driven steel H-piles founded at the anticipated elevation are provided on the individual FDR sheets. The preliminary ULS and SLS resistance values should be re-evaluated and modified, if necessary, during detail design stage, in consideration of any additional subsurface information at each foundation element.

The ULS resistance values should be verified in the field by the use of the Hiley Formula (MTO Standard Drawing SS103-11, Pile Driving Control). Alternatively PDA testing should be included. For complex sites, if warranted during detail design stage, the ultimate load capacity and/or load-settlement behaviour (serviceability) should be verified by full-scale pile load tests.

Pile installation should be in accordance with OPSS 903 and DB 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile size and length of pile and as such should be defined during detail design stage. In the extension section of this project, pile installation should be performed using fixed leads. The contractor should make an appropriate assessment of the effect for the potential for pile driving alignment and tolerance, pile damage and surface subsidence if submitting a proposal for the use of swinging leads.

The soils at some structure locations are typically very dense/hard glacial tills and to provide adequate length of pile at these locations, pre-augering (soil left in place) may be required to penetrate the very dense/hard glacial till soils to provide a minimum pile length of 5 m below the pile cap for integral abutments and 3 m for conventional abutments (refer to individual FDR sheets). These pre-auguring dimensions should be designed by the structural engineer to permit adequate distribution of loads over the pile group. For the installation of steel H-piles, consideration will have to be given to the possible presence of cobbles and/or boulders within the till deposits. Where applicable, the piles should be reinforced with driving shoes such as Titus Standard or flange plates as per OPSD 3000.100 (Steel H-Pile Driving Shoe) for protection during driving.

The resistance of piles against lateral loads should take into account the batter of the pile (if any), the relative rigidity of the pile to surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile, and group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. In case of a vertical pile, the resistance to lateral loading will be derived solely from the soil in front of the pile, whereas a battered pile derive lateral resistance from the soil in front of the pile as well as the horizontal component of the axial load present in the inclined pile.

In the estimation of resistance to lateral loading, pile group action should be accounted for, if the pile spacing in the direction of loading is less than six to eight pile diameters.

For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. The coefficients of horizontal subgrade reaction should be generated for detail design purposes.

The structural design of the piles should be based on full downdrag load, where applicable and as indicated on the FDR sheets, unless measures to significantly reduce anticipated post-construction settlements are undertaken. In this case the downdrag loads can be eliminated. For preliminary design, downdrag should be designed in accordance with *CHBDC*.

All pile caps should be provided with a minimum of 1.4 m of soil cover or equivalent thickness of insulation for frost protection purposes as per OPSD 3090.101 (Foundation Frost Depths for Southern Ontario).

6.2.3 Caissons

Preliminary foundation recommendations for caissons founded within competent soils or shale bedrock were provided, where caissons considered to be practical for foundation design as indicated on the individual FDR sheets.

Based on the subsurface conditions encountered at each foundation element of each structure, the factored geotechnical axial resistance at ULS and the geotechnical axial resistance at SLS for 25 mm of displacement are provided for caisson diameters equal to 1.2 m and 1.5 m on the individual FDR sheets. The geotechnical resistance values are associated with a recommended caisson base elevation. The factored ULS and SLS resistance values provided will have to be re-evaluated and modified, if necessary, during detail design in consideration of the additional subsurface investigations at the locations of each bridge foundation element.

For complex sites, if warranted during detail design stage, the ultimate load capacity and/or load-settlement behaviour (serviceability) should be verified by full-scale caisson load tests.

Caisson installation should be in accordance with OPSS 903 and DB 903 (Deep Foundations).

It should be noted that “running” or “flowing” of water-bearing cohesionless strata, where encountered, could pose difficulties during drilling of caisson foundations. Therefore, where caisson foundations are considered, temporary or permanent caisson liners may be required to support these type of soils during construction and allow for cleaning and inspection of the caisson base. OHSA prohibits man entry into the caisson, the inspections shall be carried out with a downhole camera. At some locations (as indicated on the FDR sheets), it is recommended caissons be drilled while using slurry methods such as maintaining a constant head of an appropriated fluid, such as Bentonite slurry, inside the caisson liners to counterbalance high groundwater pressure followed by tremie concrete placement. Where caissons are relatively long, temporary liners may be difficult to withdraw due to the length of the liners and the typically hard/very dense nature of the “100-blow” soils in which the caissons are installed. In such cases and to avoid “necking” of the caissons, permanent liners would be preferred for the construction of the caissons and the reduced shaft resistance (i.e. due to the smooth liner/soil interface) has been considered in the preliminary geotechnical resistance values provided in the FDR sheets for the full length of the caissons. The use of permanent liners should be re-assessed and geotechnical resistance values revised, if necessary, when the caisson installation method has been determined during detail design. Consideration will also have to be given to the possible presence of cobbles and/or boulders within the till deposits of these sites. Caisson drilling equipment must be capable of penetrating such obstacles, where applicable (see Section 6.7.4).

The resistance to lateral loading developed by the soils in front of the caissons (assuming vertical caissons) and the reductions due to group effects should be accounted for and assessed during detail design. The coefficients of horizontal subgrade reaction should be generated for detail design purposes.

The structural design of the caissons should be based on full downdrag load where applicable and should be considered during detail design, unless measures to significantly reduce anticipated post-construction settlements are undertaken in which case the downdrag loads can be eliminated. For preliminary design, downdrag loads should be designed in accordance with *CHBDC*. Further analysis of downdrag loads is required during detail design.

Caisson caps, as applicable, should be provided with a minimum of 1.4 m of soil cover or equivalent thickness of insulation for frost protection.

6.3 Structure Retaining Walls / Wing Walls

The proposed structures may require the construction of retaining walls and/or wing walls depending on the proposed crossing configuration, available space and surrounding ground elevations. Feasible retaining wall/wing wall options may include:

- Retained Soil System (RSS) walls: RSS walls are considered to be a feasible wall option for most of the structure abutment / approach locations provided differential settlements are within tolerable limits and an adequate Factor of Safety against global instability is achieved. The performance of an RSS wall during foundation settlement depends primarily on the characteristics of its front facing system. Construction of RSS walls should be in conformance with the MTO RSS Design Guidelines and Special Provision 599S22. Sub-excavation of surficial loosened/softened materials, where encountered, and replacing with compacted granular material, will be required to construct the reinforced soil mass. The front facing of RSS walls is typically supported on a granular pad. The granular pad must be founded on competent native soils or approved engineered fill, after sub-excavation and backfilling the areas where topsoil, fill, loosened/softened, organics and deleterious native soils exist. The factored geotechnical axial resistance at ULS and the geotechnical axial resistance at SLS for the tolerable displacement should be provided for the front panel of the wall and reinforced earth mass during detail design. It should be noted that the limiting displacement value for SLS design that should be assessed and confirmed during detail design will be dependent on the actual facing type or possibly the serviceability limit of the supporting roadway or foundation (typically less than 25 mm). The internal stability of a reinforced earth wall should be assessed by the proprietary product supplier/designer. The global stability of the RSS wall should be confirmed by the foundation consultant at detail design stage taking into account the final geometry and configuration of the RSS walls.

- Conventional retaining walls: Retaining walls supported on spread footings or on deep foundations (often cantilevered beyond the abutment foundation) depending on the site-specific subsoil conditions are considered to be feasible. The preliminary foundation recommendations for this type of retaining wall can be considered to be similar to the recommendations provided for the preliminary design of the structure foundations elements.

For settlement sensitive sites, retaining walls will be affected by the post-construction settlement of the wall backfill materials, depending on the height/thickness of the backfill. The selection of the wall option for such sites will thus be dependent on the predicted settlement and should be assessed during detail design. Measures to reduce settlement could be achieved by incorporating site improvement techniques, such as using light weight fill materials (slag or expanded polystyrene (EPS)), preloading or surcharging, installing wick drains, and staged construction as discussed in the individual FDR sheets, where applicable. The preferred settlement mitigation option is site-specific and should be confirmed when additional soil information and project scheduling is known during detail design.

6.4 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated retaining walls/wing 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 loadings, on the freedom of lateral movement of the structure, as well as on the drainage conditions behind the walls. The following general recommendations are made concerning the design of the stems/wing walls.

These recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope in accordance with Clause C6.12.2.2 of the *CHBDC Commentary (2014).*

- Backfill to the abutment and retaining walls should be in conformance with OPSS 902 and DB 902 (Excavating and Backfilling-Structures) and should consist of Granular A or Granular B Type II material. This material should be compacted in accordance with OPSS.PROV 501 (Compacting), OPSD 3101.150 (Walls Abutment, Backfill) and OPSD 3121.150 (Walls Retaining, Backfill).

- Where applicable, longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with the standards noted above.

- The granular fill may be placed either in a zone with width equal to at least 1.4 m behind the back of the wall stem (Case I on Figure C6.20(a) of the *Commentary to the CHBDC* (2014)) 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 on Figure C6.20(b) of the C*HBDC Commentary* (2014)).

- For the case where the pressures are based on granular fill behind the wall, the following parameters may be assumed.

	GRANULAR A	**GRANULAR B TYPE II**
Soil Unit Weight:	22.5 kN/m^3	21 kN/m^3
Coefficients of Static Lateral Earth Pressure:		
Active, K_a	0.27	0.27
At Rest, K_o	0.43	0.43

- If the wall support and superstructure allow lateral yielding of the abutment stem and retaining walls, active earth pressures (K_a) should be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures (K_o) 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 presented in Clause C6.12 and Table C6.6 of the *CHBDC Commentary* (2014). The earth pressure conditions for design of an integral abutment should be in accordance with MTO Report SO-96-01.

For the case where the pressures are based on existing materials behind the wall, the required parameters for design should be assessed on a site-by site basis during detail design.

- The design of lateral earth pressure should also include the effect of compaction pressure and local surcharge pressure in accordance with Clause 6.12.2.3 and Table 6.3 of the *CHBDC (2014).*

6.5 Approach Embankments

The configuration of the structure approaches varies from site to site and includes approach embankment construction with fills depending on the design grades and ground elevations for each crossing. Based on the available information provided at each structure site, recommendations associated with the approaches stability and settlement are provided on the individual FDR sheets. The following sub-sections provide project-wide recommendations associated with the preliminary design and construction of the approach embankments.

6.5.1 Subgrade Preparation and Embankment Construction

It is recommended that, where encountered, topsoil, organics and/or loosened/softened material, and deleterious soils be stripped from the proposed embankment footprint. The depth and extent of stripped material should be determined during detail design when additional subsurface information is available. Particular attention will be required in low valley areas where thicker layers of organic/alluvial soils may be present. After stripping, the exposed subgrade should be proof-rolled to identify any loosened/softened areas requiring sub-excavation or additional compaction prior to fill placement.

Embankment fill should be excavated, placed and compacted in accordance with OPSS 206 (Grading) and OPSS.PROV 501 (Compacting).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments in accordance with OPSS.PROV 804 (Seed and Cover) and OPSS 802 (Topsoil).

6.5.2 Approach Embankment Stability

The preliminary assessment for the stability of the approaches at each structure site was evaluated using the commercially available program Slide (Version 3.0) produced by Rocscience Inc. and is provided on the respective FDR sheets for each structure site. The assessments assume approach embankment side slopes at a gradient of 2 Horizontal to 1 Vertical (2H:1V) associated with a maximum approach height as indicated on the Preliminary General Arrangement drawings provided at the time of this report. Where designated as safe or adequate against deep-seated slope instability, a target Factor of Safety of 1.3 under static conditions is implied, assuming appropriate subgrade preparation and proper placement and compaction of embankment fill materials. The safety factor for seismic stability analyses should be in accordance with Clause C4.6.7 of the C*HBDC Commentary* (2014). Assessment of the overall stability of the embankment side slopes under seismic conditions is discussed in more details in Section 6.6.

Approaches equal to or greater than 8.0 m in height, where deemed feasible, should be constructed with a 2 m wide berm to control surficial erosion in accordance with general MTO guidelines so that no uninterrupted 2H:1V slope is greater than 8.0 m in height.

The preliminary assessment of stability of the approach slopes should be reviewed and confirmed based on the actual subsoil conditions encountered within the proposed approach/embankment footprint during detail design. Mitigation measures to improve slope stability for greater embankment heights may include slope flattening, utilizing light weight fill materials, use of geogrid reinforcement, ground improvement techniques, constructing stability berms, staged construction, or a combination of these options.

6.5.3 Approach Embankment Settlement

Settlement of the approach embankments will occur due to compression of the embankment fill itself, as well as compression and consolidation of the foundation soils. The total settlement within the founding soils has been estimated based on the existing site-specific subsoil conditions for preliminary design using hand/spreadsheet calculations and the results are reported on the individual FDR sheets for each site. These preliminary estimates do not include compression of the fill itself, which would typically occur during and shortly after the construction of embankment. The magnitude of fill compression is usually about 1% to 2% of the embankment height. Where granular fill is used for embankment construction, settlement of the fill itself is expected to occur during or immediately after completion of embankment construction, whereas non-granular earth fill or rock fill materials will exhibit additional consolidation settlement over time.

Embankment and platform width design should allow for the anticipated settlements and future padding of the pavement structure.

Where estimated post-construction consolidation settlement within the foundation soils exceeds acceptable limits (defined in the Embankment Settlement Criteria for Design specified in the MTO memorandum, July 2, 2010) measures to reduce such settlement to acceptable values have been proposed. For preliminary design, acceptable settlement values are assumed to be less than 25 mm at or near structure locations. Measures to mitigate embankment settlements may include utilizing light weight fill materials, ground improvement techniques, pre-loading and surcharging with staged construction, or a combination of these options. Comprehensive investigation, in situ and laboratory testing and analyses should be carried out during detail design to further estimate the anticipated amount and time rate of post-construction settlements and to develop the final design and construction requirements of the approach embankments in such site conditions, as well as develop mitigation measures to reduce anticipated settlements to acceptable levels.

6.6 Seismic Considerations

The Peak Ground Acceleration (PGA) for the project site is 0.081 for the City of the Vaughan, Ontario (National Building Code of Canada, 2015). The soil classification at each site for seismic design should be in accordance with Clause 4.4.3.2 of the CHBDC *(2014).*

Seismic loading must be taken into account in accordance with Clause 4.5.3 of the *CHBDC* (2014), as it can result in increased lateral earth pressures acting on the abutment stem and any associated wing walls/retaining walls.

Abutment stem and retaining/wing walls should be designed to withstand the combined loading for the appropriate static pressure conditions plus the earthquake-induced dynamic earth pressure in accordance with Clause 3.5 of the *CHBDC* (2014). The earthquake-induced pressure distribution is assumed to be linear with maximum pressure at the top of the wall and minimum pressure at its toe (an inverted triangular pressure distribution). The static and seismic active earth pressure coefficients can be determined in accordance with Clauses 6.12 and 4.6.5 of the *CHBDC* (2014) and its *Commentary.*

Approach Embankment design, liquefaction susceptibility of the soil deposits underlying the proposed embankments (and foundations) and the consequent stability of the embankments under seismic loading conditions should be assessed during detail design stage in accordance with Clauses C4.6.6 and C4.6.7 of the *CHBDC Commentary* (2014), respectively.

6.7 Construction Considerations

6.7.1 Excavation and Backfill

Preliminary recommendations for open-cut excavations are provided on a site-specific basis on the FDR sheets for each site and include the type of soils anticipated to be within the foundation excavations according to the Occupational Health and Safety Act (OHSA), as well as the recommended maximum side slope inclination for temporary excavations. All backfill is to be placed and compacted in accordance with OPSS.PROV 501 (Compacting).

6.7.2 Temporary Protection Systems

Temporary protection systems will be required where excavation geometries are steeper than those recommended for safe excavation and adjacent to structures or roads carrying traffic. Where required, the temporary excavation support system should be designed and constructed in accordance with OPSS.PROV 539 and DB 539 (Temporary Protection Systems). In general, the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539 (Temporary Protection Systems). Performance Level 1 may be required adjacent to railways.

6.7.3 Surface Water / Groundwater Control

Surface water run-off should be diverted away from the excavations at all times.

Anticipated groundwater levels within the foundation excavations at each structure site and anticipated groundwater and surface water control measures are included on the individual FIDR sheets.

At locations where near surface granular (non-cohesive) soils are present with a high water table, groundwater infiltration should be anticipated during excavation in such deposits, particularly during wet periods of the year. Dewatering at these sites will be required to allow for construction of foundation elements in a dry condition. Dewatering will be required before any excavation within floodplains with high groundwater table. Alternatively, the excavation should be carried out within a properly designed cofferdam.

6.7.4 Pile Installation / Caisson Construction

Till deposits have been encountered at the structure sites along the proposed Highway 427 extension section. The presence of cobbles and/or boulders was inferred during drilling within the till deposits, as noted on the Record of Borehole sheets, and may affect the driving of steel H-piles or construction of caissons.

It is noted that to ensure stability of caisson sidewall and base, provisions for liner installation, mud drilling techniques and depressurization methods should be made as appropriate for site specific groundwater/artesian conditions. Preliminary recommendations regarding potential obstructions during pile driving and caisson installation have been provided on the site-specific Preliminary FDR sheets.

6.7.5 Subgrade Preparation

The soils exposed at the footing subgrade will be susceptible to disturbance from construction traffic. Consideration should be given to pouring a concrete working slab (mud slab) on the subgrade within four hours after preparation, inspection and approval of the footing subgrade.

6.8 High Fills Recommendations

6.8.1 Slope Stability

Preliminary assessment of the stability of the fill embankment slopes was included in the previous report by others (Golder, 2009) for a typical high fill embankment and the results were summarized on FIDR Sheet J. A commercially available program such as Slide, produced by Rocscience Inc., should be used for slope stability analysis during detail design, when embankment cross-section geometry and provisions for stability and settlement mitigation measures are known. The safety factor for seismic stability analyses should be in accordance with Clause C4.6.7 of the C*HBDC Commentary* (2014). Assessment of the stability of the embankment side slopes under seismic conditions should be carried out during detail design.

The preliminary assessment of stability of the embankment slopes should be reviewed and confirmed based on the actual subsoil conditions encountered within the proposed embankment footprint during the detail design. Mitigation measures to improve slope stability, if required, may include slope flattening, utilizing light weight fill materials, constructing stability berms, staged construction, ground improvement techniques or a combination of these options.

6.8.2 Settlement Assessment

Preliminary assessment of the magnitude of settlement of the fill embankment is provided on the FIDR Sheet J. The preliminary assessment of settlement magnitude should be reviewed and confirmed based on the actual subsoil conditions encountered within the proposed embankment footprint during the detail design.

Settlement of the fill embankments will occur due to compression and consolidation of the foundation soils under the weight of the overlying fill material as well as from compression of the embankment fill itself. The preliminary estimates do not include compression of the embankment fill itself, which would occur during and after the construction of embankment depending on the type of materials used. The magnitude of fill compression is usually about 1% to 2% of the height of embankment. Where granular fill is used for embankment construction, settlement of the fill itself is expected to occur during or immediately after completion of embankment construction. Non-granular earth fill or rock fill materials may exhibit additional consolidation settlement over time.

The settlement tolerance for embankments range from 25 mm to 100 mm depending on the distance from a structure in accordance to the Embankment Settlement Criteria for design in MTO memorandum dates July 2, 2010. The highway design criteria will be site-specific and based on maintenance considerations at the detail design stage.

Embankment and platform width design should allow for the anticipated settlements and future padding of the pavement structure.

Further investigation, in situ and laboratory testing and analyses should be carried out during detail design to confirm the anticipated magnitude of settlement, assess the time rate of post-construction settlement, and where required develop mitigation measures such as preloading, surcharging, wick drains, utilizing ground improvement techniques, light weight fill, or combination of these options to reduce anticipated settlements to acceptable levels.

6.8.3 Embankment Construction Considerations

Topsoil, fill, loosened/softened, organics and deleterious soils should be stripped from the proposed embankment footprint. The depth and extent of stripped material shall be determined during detail design when additional subsurface information is available. Particular attention will be required in low valley areas where thicker layers of organic/alluvial soils may be present.

After stripping, the exposed subgrade should be proof-rolled to identify any loosened/softened areas requiring sub-excavation or additional compaction prior to fill placement.

Embankment fill should be placed and compacted in accordance with OPSS 206 (Grading) and OPSS.PROV 501 (Compacting). New embankment fill placed against existing embankment slopes or on a sloping ground surface should be benched into the existing slope in accordance with OPSD 208.010 (Benching).

In accordance with MTO standard practice, a minimum 2 m wide berm should be provided where the embankment side slopes are equal to or greater 8.0 m in height such that the uninterrupted slope height does not exceed 8.0 m. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments in accordance with OPSS.PROV 804 (Seed and Cover) and OPSS 802 (Topsoil).

Trafficability of construction equipment may be problematic in low floodplain areas where loosened/softened and organic alluvial material may be encountered and where environmental constraints may be imposed on site access. Further, drainage in these areas is likely to be poor, with groundwater levels varying subject to seasonal fluctuations. The contractor must be prepared to supply equipment capable of working on this terrain and/or provide alternative measures to improve trafficability such as placement of geo-synthetics with granular/rock roadways in working area.

Potential environmental impacts will need to be minimized during construction access into sensitive floodplain or valley areas. Specific access preparation procedures such as the use of temporary work bridges, winter construction and/or gravel roadways underlain by geo-synthetics should be considered. Further, sediment control measures such as silt fences, straw bales and/or granular check-dams will need to be installed downgradient of the works to reduce sediments impacts to surface water bodies, in accordance with OPSS 805 (Temporary Erosion and Sediment Control Measures).

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Mr. Al Varshoi, P.Eng. and Ms. Marzieh Kamranzadeh, MSc, EIT and reviewed by Mr. Brian R. Gray, MEng, P.Eng. Principal Consultant. Mr. Carlos M. P. Nascimento, P.Eng., MTO Designated Principal Contact conducted an independent review of the report.

Sincerely

Peto MacCallum Ltd.

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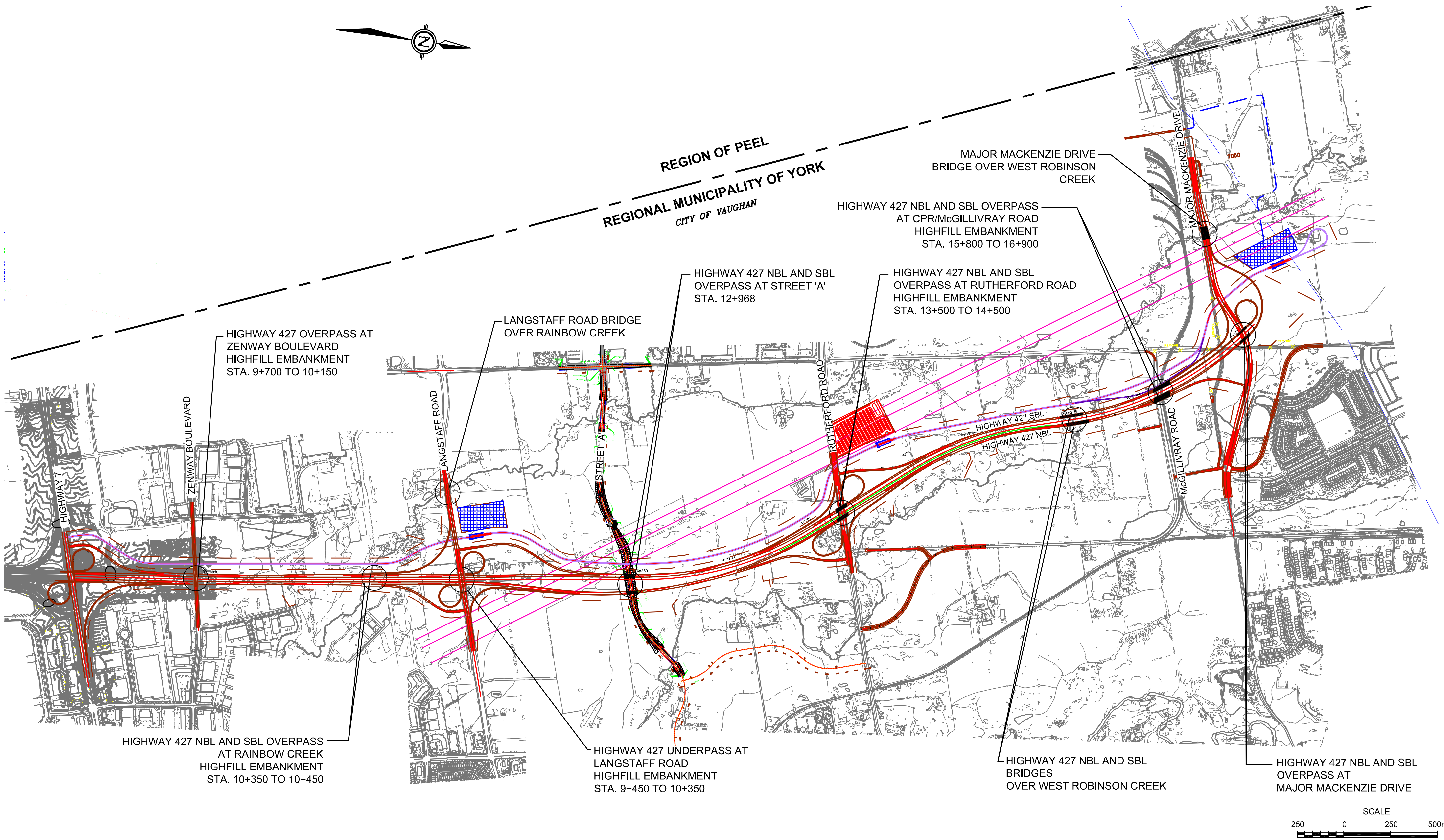


METRIC

HIGHWAY 427 EXPANSION

EXTENSION FROM HIGHWAY 7 TO MAJOR MACKENZIE DRIVE

STRUCTURES AND HIGHFILL EMBANKMENTS

Ontario

Peto MacCallum Ltd.
CONSULTING ENGINEERS

ASSIGNMENT No.: 2014-E-0056

WORK ORDER No.: 18 AND 18A

SITE LOCATION PLAN

DWG. 1

Table A1
Summary of Structures and High Fill Areas

Structure Number	Structure Category	Site Name (Location)	Complexity Rating	Existing GEOCRES Report	Boreholes		Preliminary FIDR Sheet
					Previous	Current	
Not Assigned	Underpass	Zenway Boulevard at Highway 427 NBL and SBL	Medium	30M13-167	S1 to S3	ZB-1, ZB-2	Sheet A
Not Assigned	Bridges	Highway 427 NBL and SBL Bridges over Rainbow Creek	Medium	30M13-168	S4 to S9	RC-1, RC-2	Sheet B
Not Assigned	Bridge	Langstaff Road Bridge over Rainbow Creek	Medium	30M13-170	S10 and S11	LRC-1 to LRC-3	Sheet C
Not Assigned	Underpass	Langstaff Road Underpass at Highway 427 NBL and SBL	Medium	30M13-169	S12 to S14, S14A	LR-1, LR-2	Sheet D
Not Assigned	Overpasses	Highway 427 NBL and SBL over Rutherford Road	Medium	30M13-171	S15 to S18	RR-1, RR-2	Sheet E
Not Assigned	Bridges	Highway 427 NBL and SBL over West Robinson Creek	Medium	30M13-172	S19 to S24, S19A	WRB-1, WRB-2	Sheet F
Not Assigned	Overpasses	Highway 427 NBL and SBL at CPR/ McGillivray Road	Medium	30M13-173	S25 to S30	MRG-1, MRG-2	Sheet G
Not Assigned	Overpasses	Highway 427 NBL and SBL at Major Mackenzie Drive	Medium	30M13-174	S34 and S36	MMD-1 to MMD-4	Sheet H
Not Assigned	Bridge	Major Mackenzie Drive EBL and WBL over West Robinson Creek	Medium	30M13-175	S31 and S32	MMRC-1, MMRC-2	Sheet I
Not Applicable	High Fill	Zenway Boulevard – STA. 9+700 to 10+150	Medium	30M13-177	E2 to E5, S1 to S3	ZB-1, ZB-2	Sheet J
Not Applicable	High Fill	South of Rainbow Creek – STA. 11+350 to 11+450	Medium		E6, E7, S4 and S5	-	
Not Applicable	High Fill	Langstaff Road – STA. 9+450 to 10+125	Medium		E8 to E13, S10 to S14, S14A	LR-1, LR-2, LRC-1 to LRC-3	
Not Applicable	High Fill	Rutherford Road – STA. 13+500 to 14+550	Medium		C9, C10, C13, C14, E14 to E16, E18, E19, S16, S18	RR-1, RR-2	
Not Applicable	High Fill	CPR/ McGillivray Road – STA. 15+800 to 16+900	Medium		E21 to E27, S26, S28 to S30, S36	MRG-1, MRG-2, MMD-1 to MMD-4	
Not Assigned	Overpass	Highway 427 NBL and SBL at Street 'A'	Medium	N/A	–	427N-1, 427N-2, 427S-1, 427S-2	Sheet K

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
Zenway Boulevard Underpass	S1	182.2	11.8	170.4	April 27, 2009	On Completion of Drilling
	S2	181.4	> 15.7	< 165.7	April 17, 2009	On Completion of Drilling
	S3	181.1	10.7	170.4	April 16, 2009	On Completion of Drilling
			1.3	179.8	May 13, 2009	In Piezometer
			0.9	180.2	June 15, 2009	In Piezometer
			0.9	180.2	July 09, 2009	In Piezometer
	ZB-1	183.0	9.7	173.3	September 30, 2015	On Completion of Drilling
	ZB-2	181.5	18.3	163.2	September 30, 2015	On Completion of Drilling
Rainbow Creek Bridges	S4	182.5	6.0	176.5	February 27, 2009	On Completion of Drilling
	S5	181.6	6.0	175.6	February 26, 2009	On Completion of Drilling
			4.4	177.2	April 24, 2009	In Piezometer
			4.4	177.2	May 13, 2009	In Piezometer
			4.6	177.0	May 21, 2009	In Piezometer
			4.7	176.9	June 15, 2009	In Piezometer
			4.9	176.7	July 9, 2009	In a Piezometer
	S6	177.6	3.0	174.7	March 13, 2009	On Completion of Drilling
	S7	175.8	0.9	174.9	March 13, 2009	On Completion of Drilling
	S8	175.8	0.9	176.7	March 12, 2009	On Completion of Drilling
	S9	176.0	1.2	174.8	March 16, 2009	On Completion of Drilling
			3.6	172.4	April 24, 2009	In Piezometer
			1.2	174.8	May 13, 2009	In Piezometer
			0.9	175.1	May 21, 2009	In Piezometer
			0.5	175.5	June 15, 2009	In Piezometer
			0.5	175.5	July 9, 2009	In Piezometer
	RC-1	175.7	4.6	171.1	November 6, 2015	During Drilling
	RC-2	177.3	3.4	173.9	October 15, 2015	On Completion of Drilling

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
Langstaff Road Bridge over Rainbow Creek	S10	183.4	> 8.1	< 175.3	March 20, 2009	On Completion of Drilling
	S11	180.9	7.8	173.1	March 20, 2009	On Completion of Drilling
			0.0	180.9	April 24, 2009	In Piezometer
			0.0	180.9	May 13, 2009	In Piezometer
			0.0	180.9	June 15, 2009	In Piezometer
			0.0	180.9	July 9, 2009	In Piezometer
	LRC-1	181.9	7.9	174.0	October 14, 2015	On Completion of Drilling
	LRC-2	180.6	10.5	170.1	October 15, 2015	On Completion of Drilling
	LRC-3	179.9	14.6	165.3	October 16, 2015	On Completion of Drilling
Langstaff Road Underpass	S12	187.5	5.2	182.3	March 26, 2009	On Completion of Drilling
			6.9	180.6	May 13, 2009	In Piezometer
			6.7	180.8	June 15, 2009	In Piezometer
			6.3	181.2	July 9, 2009	In Piezometer
	S13	187.7	7.8	179.9	March 31, 2009	On Completion of Drilling
	S14	187.7	17.7	170.0	April 2, 2009	On Completion of Drilling
	S14A	187.7	21.8	165.9	April 13, 2009	On Completion of Drilling
	LR-1	187.9	9.1	178.8	September 28, 2015	On Completion of Drilling
	LR-2	188.1	18.3	169.8	September 29, 2015	During Drilling
Rutherford Road Overpasses	S15	194.0	7.6	186.4	March 25, 2009	On Completion of Drilling
	S16	194.6	6.0	188.6	March 20, 2009	On Completion of Drilling
	S17	194.6	11.2	183.4	March 25, 2009	On Completion of Drilling
			4.0	190.6	April 24, 2009	In Piezometer
			4.1	190.5	May 25, 2009	In Piezometer
			4.0	190.6	June 15, 2009	In Piezometer
			3.8	190.8	July 9, 2009	In Piezometer
	S18	194.3	7.6	186.7	March 23, 2009	On Completion of Drilling
	RR-1	194.6	N/R	N/R	N/R	N/R – Not recorded due to use of mud rotary drilling
	RR-2	193.6	N/R	N/R	N/R	

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
West Robinson Creek Bridges	S19	193.8	6.1	187.7	March 2, 2009	On Completion of Drilling
	S19A	193.8	2.1	191.7	March 10, 2009	On Completion of Drilling
	S20	193.9	6.1	187.8	March 3, 2009	On Completion of Drilling
	S21	194.0	6.1	187.9	March 5, 2009	On Completion of Drilling
	S22	193.7	6.0	187.7	March 6, 2009	On Completion of Drilling
	S23	197.2	8.5	185.7	March 9, 2009	On Completion of Drilling
			3.8	193.4	April 24, 2009	In Piezometer
			3.8	193.4	May 21, 2009	In Piezometer
			4.0	193.2	June 15, 2009	In Piezometer
			4.1	193.1	July 9, 2009	In Piezometer
	S24	199.2	8.0	191.2	March 3, 2009	On Completion of Drilling
	WRB-1	200.0	10.2	189.8	December 7, 2015	During Drilling
	WRB-2	195.0	7.9	187.1	December 1, 2015	On Completion of Drilling
CPR / McGillivray Road Overpasses	S25	201.8	16.6	185.2	March 16, 2009	On Completion of Drilling
	S26	201.5	11.5	190.0	March 12, 2009	On Completion of Drilling
	S27	201.1	> 38.4	< 162.7	March 13, 2009	On Completion of Drilling
	S28	200.8	12.8	188.0	March 17, 2009	On Completion of Drilling
			8.5	192.3	April 27, 2009	In Piezometer
			8.5	192.3	May 13, 2009	In Piezometer
			8.6	192.2	May 25, 2009	In Piezometer
			9.1	191.7	June 15, 2009	In Piezometer
			9.1	191.7	July 9, 2009	In Piezometer
	S29	202.0	15.2	186.8	April 27, 2009	On Completion of Drilling
	S30	202.3	10.7	191.6	April 27, 2009	On Completion of Drilling
Major Mackenzie Drive Overpasses	S34	205.2	21.8	183.4	March 10, 2009	During Drilling
	S36	205.2	7.3	197.9	April 27, 2009	In Piezometer
			6.4	198.8	May 25, 2009	In Piezometer
			6.2	199.0	June 15, 2009	In Piezometer
			6.2	199.0	July 9, 2009	In Piezometer
	MMD-2	204.5	10.2	194.3	November 16, 2015	In Piezometer
			9.3	195.2	December 23, 2015	In Piezometer

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
Major Mackenzie Drive Bridge over West Robinson Creek	S31	201.3	3.4	197.9	March 19, 2009	On Completion of Drilling
	S32	201.8	4.9	196.9	March 18, 2009	On Completion of Drilling
			3.1	198.7	April 24, 2009	In Piezometer
			3.4	198.4	May 13, 2009	In Piezometer
			3.4	198.4	May 25, 2009	In Piezometer
	MMRC-1	200.7	3.7	197.0	October 13, 2015	On Completion of Drilling
			2.9	197.8	December 23, 2015	In Piezometer
	MMRC-2	202.1	3.7	198.4	October 9, 2015	On Completion of Drilling
High Fill at Zenway Boulevard Underpass	E2	188.3	> 9.6	< 178.7	April 17, 2009	On Completion of Drilling
	E3	181.6	> 8.2	< 173.4	April 14, 2009	On Completion of Drilling
	E4	183.0	> 6.7	< 176.3	April 7, 2009	On Completion of Drilling
	E5	183.2	> 6.7	< 176.5	April 7, 2009	On Completion of Drilling
	S1	182.2	11.8	170.4	April 27, 2009	On Completion of Drilling
	S2	181.4	> 15.7	<165.7	April 17, 2009	On Completion of Drilling
	S3	181.1	10.7	170.4	April 16, 2009	On Completion of Drilling
			1.3	179.8	May 13, 2009	In Piezometer
			0.9	180.2	June 15, 2009	In Piezometer
			0.9	180.2	July 09, 2009	In Piezometer
	ZB-1	183.0	9.7	173.3	September 30, 2015	On Completion of Drilling
	ZB-2	181.5	18.3	163.2	September 30, 2015	On Completion of Drilling
High Fill at South of Rainbow Creek	E6	179.1	> 5.2	< 173.9	February 27, 2009	On Completion of Drilling
	E7	178.3	> 5.2	< 173.2	March 2, 2009	On Completion of Drilling
	S4	182.5	6.0	176.5	February 27, 2009	On Completion of Drilling
	S5	181.6	6.0	175.6	February 26, 2009	On Completion of Drilling
			4.4	177.2	April 24, 2009	In Piezometer
			4.4	177.2	May 13, 2009	In Piezometer
			4.6	177.0	May 21, 2009	In Piezometer
			4.7	176.9	June 15, 2009	In Piezometer
			4.9	176.7	July 9, 2009	In Piezometer

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
High Fill at Langstaff Road Underpass	E8	186.7	> 6.7	< 180.0	April 1, 2009	On Completion of Drilling
	E9	181.4	3.0	178.4	April 14, 2009	On Completion of Drilling
	E10	185.6	> 8.2	< 177.4	April 14, 2009	On Completion of Drilling
	E11	186.9	> 8.2	< 178.7	April 14, 2009	On Completion of Drilling
	E12	187.4	> 8.2	< 179.2	April 14, 2009	On Completion of Drilling
	E13	187.3	> 8.2	< 179.1	April 13, 2009	On Completion of Drilling
	S10	183.4	> 8.1	< 175.3	March 20, 2009	On Completion of Drilling
	S11	180.9	7.8	173.1	March 20, 2009	On Completion of Drilling
			0.0	180.9	April 24, 2009	In Piezometer
			0.0	180.9	May 13, 2009	In Piezometer
			0.0	180.9	June 15, 2009	In Piezometer
			0.0	180.9	July 9, 2009	In Piezometer
	S12	187.5	5.2	182.3	March 26, 2009	On Completion of Drilling
			6.9	180.6	May 13, 2009	In Piezometer
			6.7	180.8	June 15, 2009	In Piezometer
			6.3	181.2	July 9, 2009	In Piezometer
	S13	187.7	7.8	179.9	March 31, 2009	On Completion of Drilling
	S14	187.7	17.7	170.0	April 2, 2009	On Completion of Drilling
	S14A	187.7	21.8	165.9	April 13, 2009	On Completion of Drilling
	LR-1	187.9	9.1	178.8	September 28, 2015	On Completion of Drilling
	LR-2	188.1	18.3	169.8	September 29, 2015	During Drilling
	LRC-1	181.9	7.9	174.0	October 14, 2015	On Completion of Drilling
	LRC-2	180.6	10.5	170.1	October 15, 2015	On Completion of Drilling
	LRC-3	179.9	14.6	165.3	October 16, 2015	On Completion of Drilling
High Fill at Rutherford Road Overpasses	C9	188.3	> 9.8	< 178.5	March 27, 2009	On Completion of Drilling
	C10	188.6	> 9.8	< 178.8	March 30, 2009	On Completion of Drilling
			7.6	181.0	April 24, 2009	In Piezometer
			8.0	180.6	May 21, 2009	In Piezometer
			7.9	180.7	May 21, 2009	In Piezometer
			7.9	180.7	June 15, 2009	In Piezometer
			7.6	181.0	July 9, 2009	In Piezometer
	C13	193.8	> 9.8	< 184.0	April 6, 2009	On Completion of Drilling

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
(Cont'd) High Fill at Rutherford Road Overpasses **(Note 1)**	C14	194.5	> 11.3	< 183.2	April 3, 2009	On Completion of Drilling
	E14	191.5	> 8.2	< 183.3	March 25, 2009	On Completion of Drilling
	E15	192.2	2.7	189.5	March 26, 2009	On Completion of Drilling
	E16	193.2	> 9.8	< 183.4	March 20, 2009	On Completion of Drilling
	E18	193.2	12.5	180.7	March 25, 2009	On Completion of Drilling
	E19	195.3	7.9	187.4	April 1, 2009	On Completion of Drilling
	S16	194.6	6.0	188.6	March 20, 2009	On Completion of Drilling
	S18	194.3	7.6	186.7	March 23, 2009	On Completion of Drilling
High Fill at CPR / McGillivray Road Overpasses	E21	202.2	> 5.2	< 197.0	March 11, 2009	On Completion of Drilling
	E22	202.4	> 11.3	< 191.1	April 29, 2009	On Completion of Drilling
	E23	203.0	> 11.3	< 191.7	April 29, 2009	On Completion of Drilling
	E24	203.8	11.9	191.9	March 17, 2009	On Completion of Drilling
	E25	203.8	12.5	191.3	March 17, 2009	On Completion of Drilling
	E26	204.3	> 12.8	< 191.5	March 18, 2009	On Completion of Drilling
	E27	203.8	9.8	194.0	March 18, 2009	On Completion of Drilling
	S26	201.5	11.5	190.0	March 12, 2009	On Completion of Drilling
	S28	200.8	12.8	188.0	March 17, 2009	On Completion of Drilling
			8.5	192.3	April 27, 2009	In Piezometer
			8.5	192.3	May 13, 2009	In Piezometer
			8.6	192.2	May 25, 2009	In Piezometer
			9.1	191.7	June 15, 2009	In Piezometer
			9.1	191.7	July 9, 2009	In Piezometer
	S29	202.0	15.2	186.8	April 27, 2009	On Completion of Drilling
	S30	202.3	10.7	191.6	April 27, 2009	On Completion of Drilling
	S36	205.2	7.3	197.9	April 27, 2009	In Piezometer
			6.4	198.8	May 25, 2009	In Piezometer
			6.2	199.0	June 15, 2009	In Piezometer
			6.2	199.0	July 9, 2009	In Piezometer
	MMD-2	204.5	10.3	194.3	November 16, 2015	In Piezometer
			9.3	195.2	December 23, 2015	In Piezometer

Table A2
Summary of Groundwater Level Measurements

Site Name	Borehole No.	Ground Surface Elevation at Borehole Location (m)	Depth to Groundwater Level Below Ground Surface (m)	Groundwater Elevation (m)	Date of Measurement	Measurement Detail
Highway 427 / Street ‘A’ Overpass	427N-1	189.1	N/R	N/R	N/R	N/R – Not recorded due to use of mud rotary drilling
	427N-2	188.9	N/R	N/R	N/R	N/R – Not recorded due to use of mud rotary drilling
	427S-1	189.3	15.2	174.1	July 12, 2016	During Drilling
			4.4	184.9	July 21, 2016	In Piezometer
			3.1	186.2	August 2, 2016	In Piezometer
	427S-2	189.4	18.3	171.1	July 11, 2016	During Drilling

Note 1: Water levels were not obtained in boreholes RR-1, RR-2, 427N-1 and 427N-2 because these boreholes were drilled by mud rotary methods.

Figure 1: Typical Granular A Pad

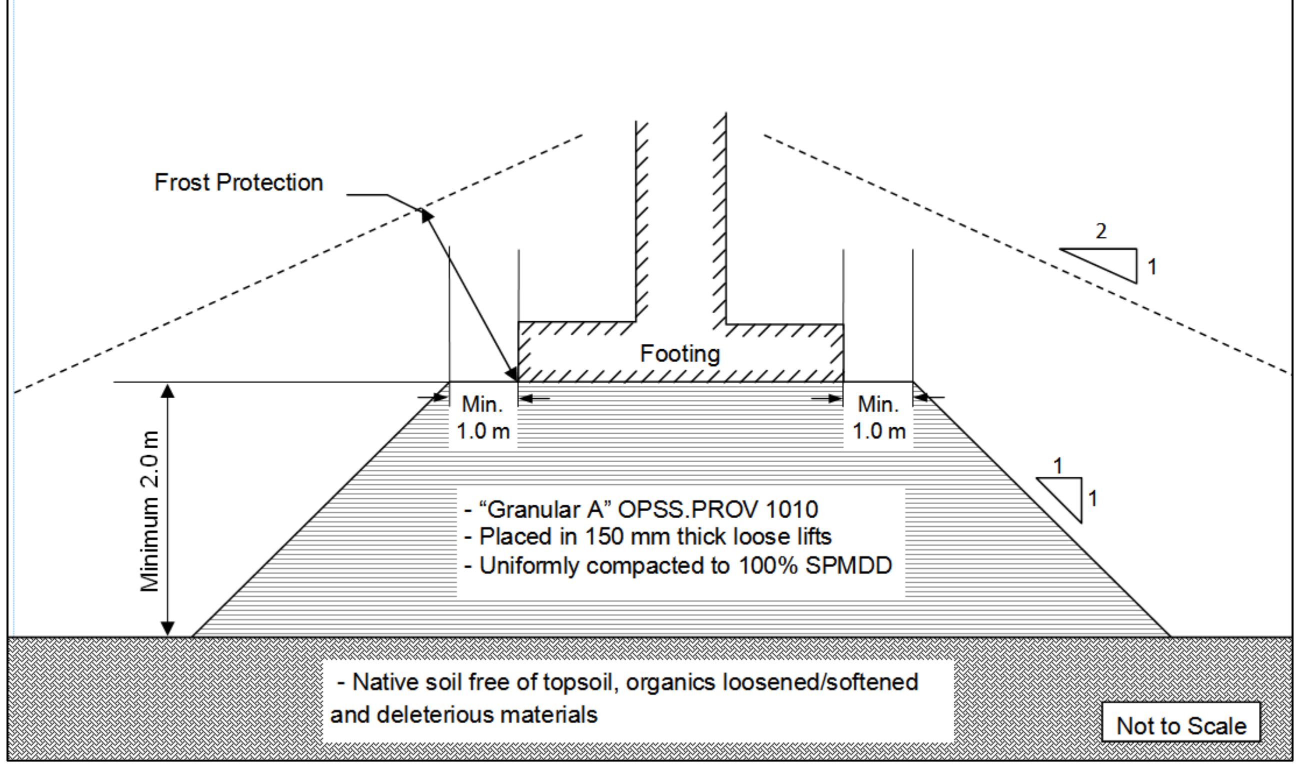
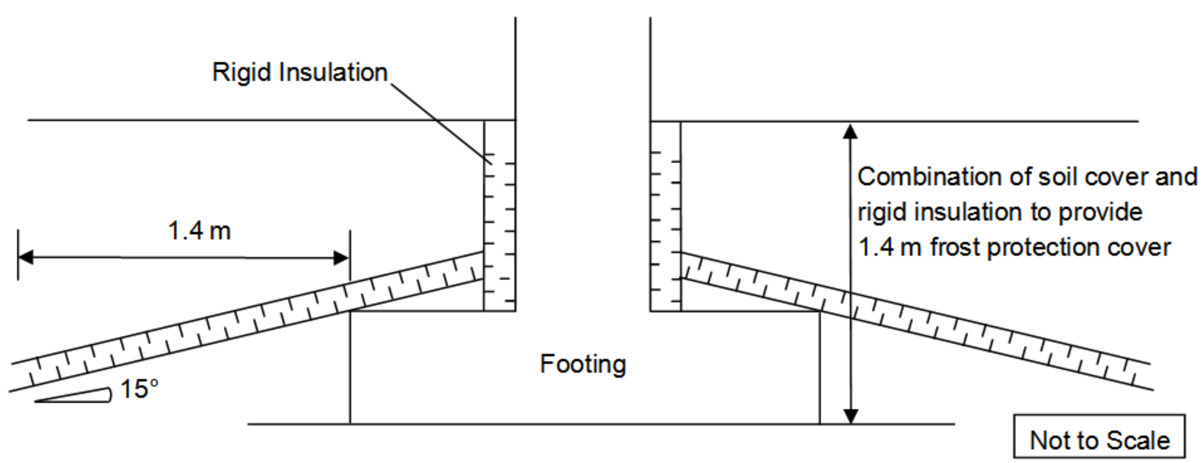


Figure 2: Typical Rigid Insulation

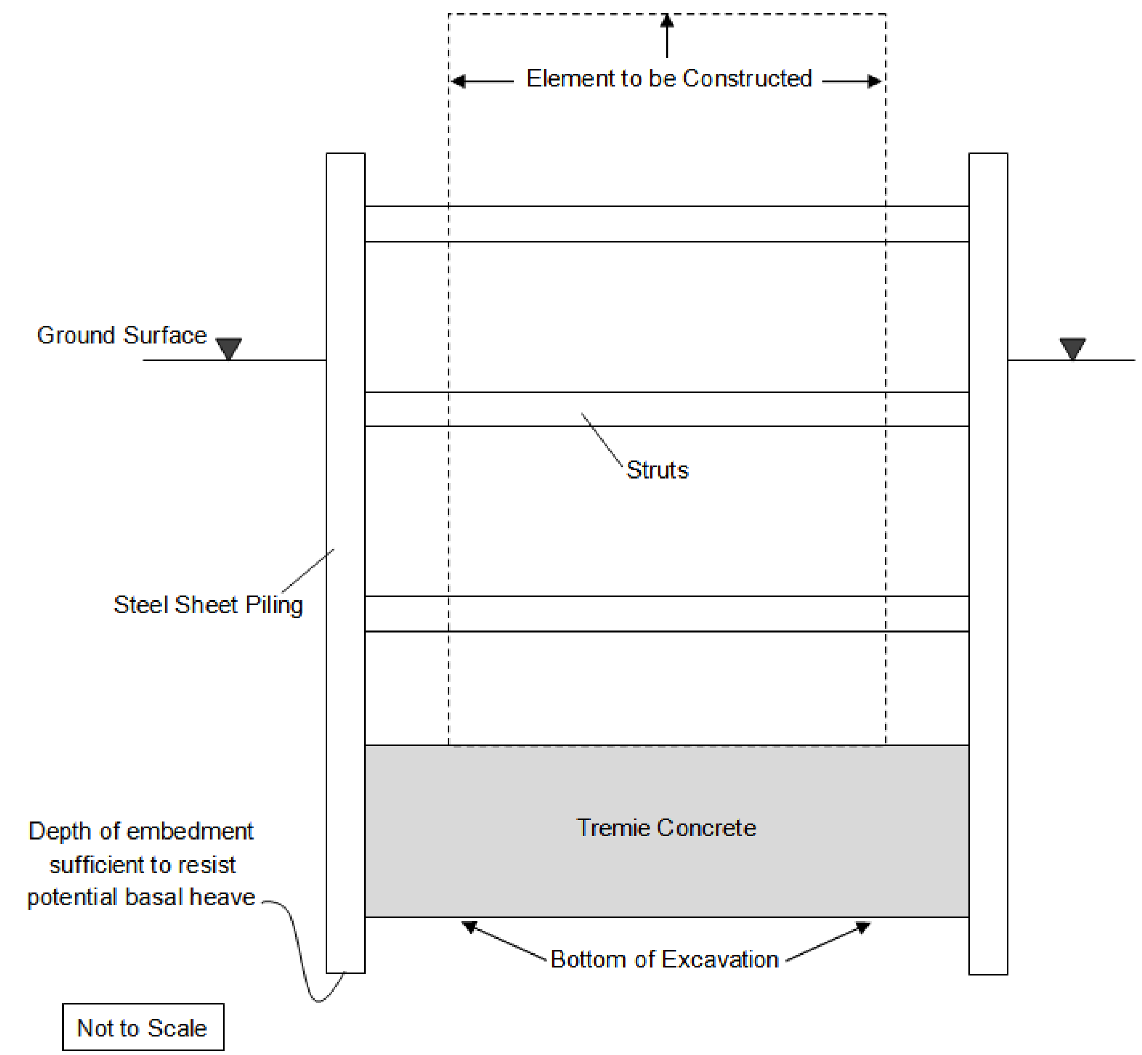


Note 1: Insulation to provide frost protection where sufficient soil cover can not be provided (25 mm extruded closed cell insulation thermally equivalent to 0.3 m soil cover)

Note 2: Refer to the following web sites for explicit information on rigid insulation application:

- http://msdssearch.dow.com/PublishedLiteratureDOWCOM/dh_0031/0901b803800311e8.pdf?filepath=styrofoam/pdfs/noreg/178-00542.pdf&fromPage=GetDoc
- http://insulation.owenscorning.ca/assets/0/188/1c2c8d2d-4fe5-4038-970c-9028b3af7414.pdf

Figure 3: Conceptual Cofferdam Construction



Note 1: Only a conceptual sketch is depicted. The contractor shall design a site specific cofferdam system.

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 – 10	10 – 20	20 – 30	30 – 40	> 40
	TRACE	*SOME*	*WITH*	*ADJECTIVE ('SILTY')*	*AND ('AND SILT')*

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	n	1,%	POROSITY	e_{max}	1,%	VOID RATIO IN LOOSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	w	1,%	WATER CONTENT	e_{min}	1,%	VOID RATIO IN DENSEST STATE
ρ_w	kg/m^3	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_P	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_S	%	SHRINKAGE LIMIT	C_U	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m^3	DENSITY OF DRY SOIL	I_P	%	PLASTICITY INDEX $= w_L - w_P$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$	q	m^3/s	RATE OF DISCHARGE
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL				v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$	i	1	HYDRAULIC GRADIENT
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	j	kN/m^3	SEEPAGE FORCE
e	1,%	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

PART C
PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT SHEETS

HIGHWAY 427 EXPANSION – EXTENSION SECTION
HIGHWAY 427 NBL AND SBL OVERPASS AT STREET 'A'
CITY OF VAUGHAN, ONTARIO
ASSIGNMENT NO.: 2014-E-0056
WO 2016-11005
WORK ORDER NO. 18A

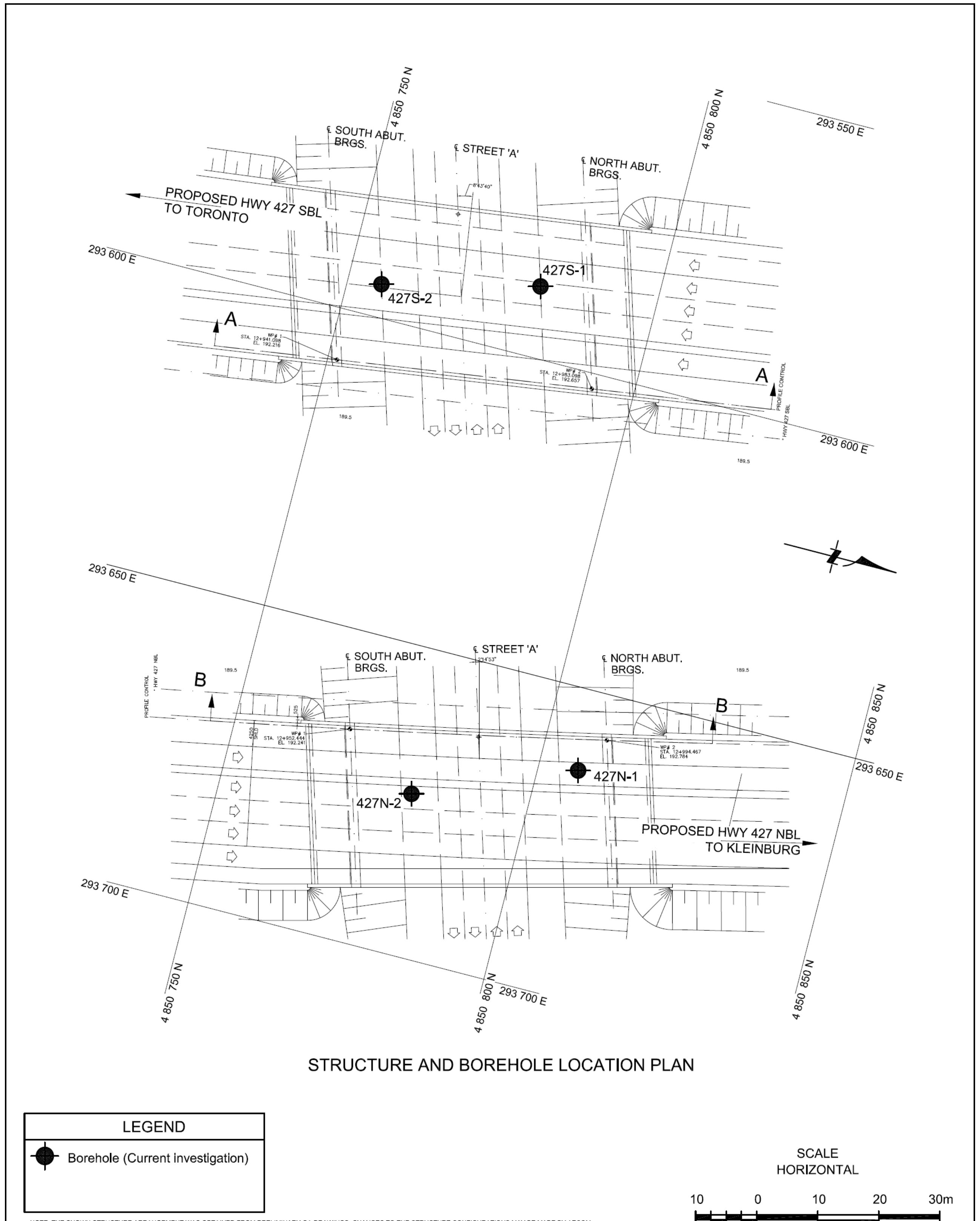
Structure Description: Highway 427 NBL and SBL Overpass at Street ‘A’

Highway 427 Proposed Grade: 192.2 – 192.8 m

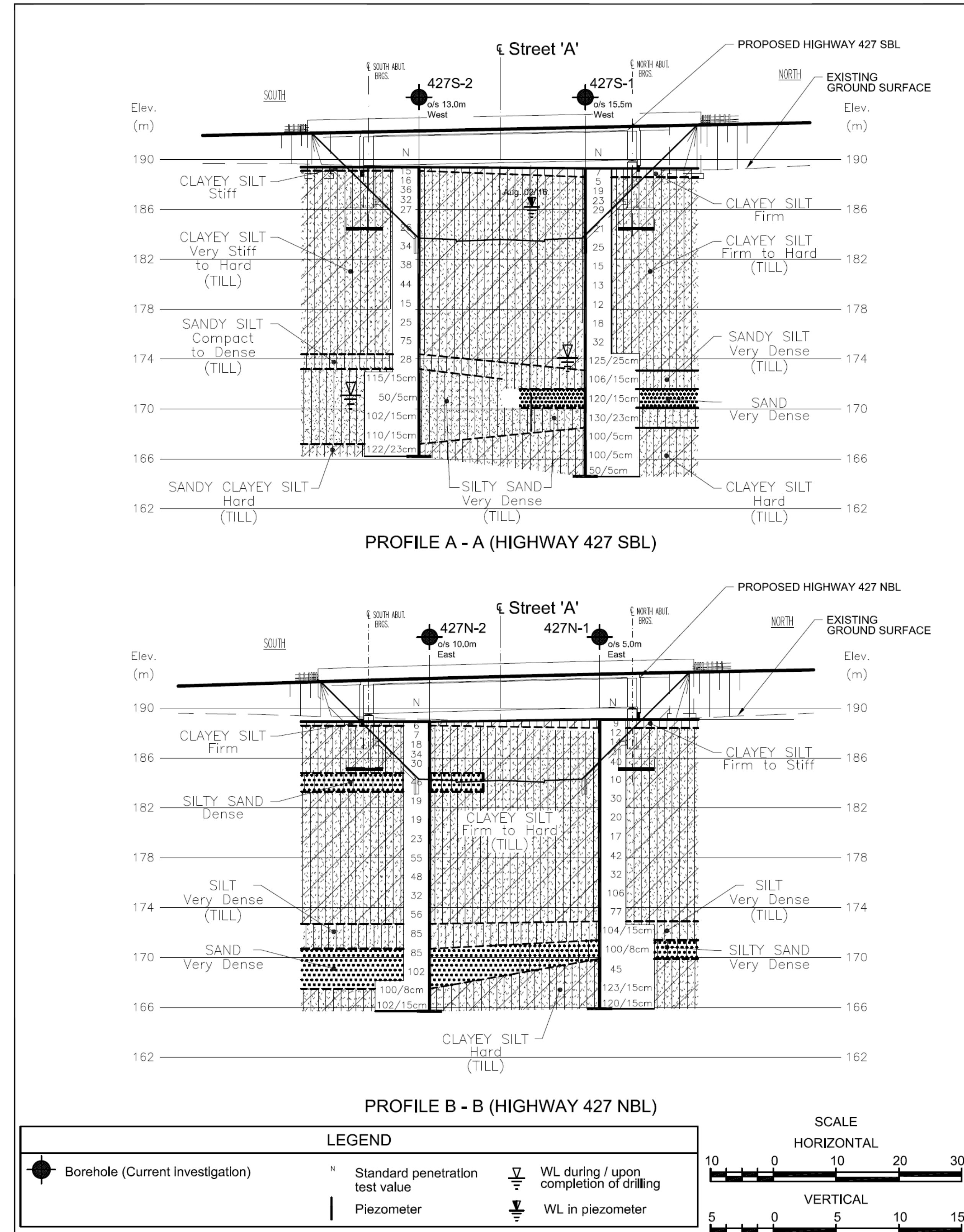
Existing Ground Elevation: 188.9 – 189.4 m

Complexity Rating: Middle

Station: 12+968



STRUCTURE AND BOREHOLE LOCATION PLAN



FOUNDATION INVESTIGATIONS

Site Description:

The proposed overpass is located at Street 'A' between Langstaff Road and Rutherford Road some 900 m west of Highway 27 in Vaughan, Ontario. The site topography is relatively flat, wooded and surrounded by farmland or vacant land.

Borehole Information:

Borehole No	Borehole Location	MTM NAD 83 – Northing	MTM NAD 83 – Easting	Borehole Elevation (m)	Borehole Depth (m)
427N-1	North Abutment (NBL)	4 850 806.5	293 663.2	189.1	23.2
427N-2	South Abutment (NBL)	4 850 781.0	293 673.7	188.9	23.2
427S-1	North Abutment (SBL)	4 850 780.8	293 588.5	189.3	24.7
427S-2	South Abutment (SBL)	4 850 755.5	293 594.6	189.4	23.2

Subsurface Conditions:

- **Clayey Silt:** Surficial clayey silt was present in all the boreholes. Containing topsoil inclusions, this unit was firm to stiff in consistency and 20 to 25% in moisture content. The clayey silt was 300 to 700 mm thick and penetrated at elevation 188.4 to 189.1.
- **Clayey Silt Till:** Directly beneath the clayey silt at depths of 0.3 to 0.7 m (elevation 188.4 to 189.1) in all the boreholes was a cohesive deposit of clayey silt till. This deposit was interlayered with cohesionless soils (described below) and extended to the termination depths of 23.2 to 24.7 m (elevation 164.6 to 166.2). The clayey silt till was firm to hard in consistency and had a moisture content of 9 to 32%, typically 12 to 18%. The results of Atterberg limits testing and grain size distribution analyses conducted on 11 samples of the deposit are presented in Figures SA-PC-1 and SA-GS-1 respectively. It is noteworthy that shale fragments were encountered in the lower portion of the clayey silt till.
- **Sand / Silty Sand:** A layer of silty sand was revealed within the clayey silt till at 4.1 m depth (elevation 184.8) in borehole 427N-2. This layer was dense (SPT-'N' value of 46) and 13% in moisture content. The silty sand was 1.5 m in thickness and penetrated at a depth of 5.6 m (elevation 183.3). Lower strata of sand / silty sand were overlain by silt till / sandy silt till at depths of 17.7 to 18.2 m (elevation 170.7 to 171.6) in boreholes 427N-1, 427N-2 and 427S-1. These strata were very dense (SPT-'N' values of 85 to over 120) and had a moisture content of 8 to 11%. The sand / silty sand was 1.5 to 3.2 m thick and penetrated at depths of 19.2 to 21.4 m (elevation 167.5 to 170.1). The results of grain size distribution analyses performed on 2 samples of the strata are presented in Figure SA-GS-2.
- **Cohesionless Till:** Cohesionless till of various granulometric composition (silt, sandy silt, silty sand) was identified at depths of 15.0 to 16.2 m (elevation 172.7 to 174.4) in all the boreholes. The till was very dense, locally compact (SPT-'N' values of 28 to over 130) and 6 to 17% in moisture content. The till had a thickness of 1.5 to 7.2 m and was penetrated at depths of 17.7 to 22.2 m (elevation 167.2 to 171.4). The results of grain size distribution analyses conducted on 4 samples of the cohesionless till are presented in Figures SA-GS-3 and SA-GS-4. It is worth noting that the sandy silt till / silty sand till contained shale fragments.

Groundwater Conditions:

- **Boreholes 427N-1 and 427N-2:** No groundwater was observed during or upon completion of drilling.
- **Borehole 427S-1:** In the process of augering, water was detected at a depth of 15.2 m depth (elevation 174.1). The piezometric water level was at 4.4 m depth (elevation 184.9) on July 21 and a depth of 3.1 m (elevation 186.2) on August 2, 2016.
- **Borehole 427S-2:** Water was detected at 18.3 m depth (elevation 171.1) during drilling. No groundwater was observed upon completion of drilling.

FOUNDATION RECOMMENDATIONS

Note: The site-specific foundation recommendations are for planning purposes only. Refer to Section 6.0 of the Foundation Design Report for the project-wide foundation recommendations, design assumptions and limitations.

General: Based on General Arrangement drawings for the NBL and SBL structures received from AECOM in July 2016, the proposed overpass will carry the Highway 427 traffic over Street 'A'. The overpass consists of two single 42.0 m span structures (for NBL and SBL) with approach embankments approximately 3 and 4 m high at the south and north abutments, respectively. Based on the existing subsurface information, the feasible foundation options for the proposed overpass abutments are listed below with advantages and disadvantages associated with each option.

Foundation Option	*Advantages*	*Disadvantages*
Spread footings founded on very stiff to hard clayey silt till / dense silty sand	• Lower cost than deep foundations • Conventional construction	• Some post-construction settlement due to consolidation of underlying soils
Steel H-Piles driven into "100-blow" clayey silt till	• Allows for integral abutment design	• May require reinforcement to facilitate driving through the very dense / hard till containing shale fragments and possible cobbles / boulders
Caissons bored to found within "100-blow" clayey silt till	• Higher bearing resistance than steel H-Piles	• Drilling must be advanced through the very dense / hard till containing shale fragments and possible cobbles / boulders • Requires temporary or permanent liner to prevent seepage inflow and softening of the caisson base

A – Spread Footings: Spread footings may be founded on the very stiff to hard clayey silt till / dense silty sand at or below elevation 185.0 at the north and south abutments of both structures. It is recommended, however, that less competent soils at the north abutment of the NBL structure be subexcavated to approximate elevation 183.5 and replaced with engineered fill. All footings should be placed at a minimum depth of 1.4 m below the lowest surrounding grade for frost protection.

Founding Stratum	*Geotechnical Resistance*	
	Factored ULS	**SLS**
Very Stiff to Hard Clayey Silt Till / Dense Silty Sand	450 kPa	300 kPa

B – Steel H-Piles: Steel HP 310x110 piles driven to found within the "100-blow" clayey silt till or cohesionless soils at or below elevation 167.0 for the NBL structure and elevation 171.0 for the SBL structure are feasible for support of the south and north abutments. Pile lengths will be about 18 m for the NBL structure and 14 m for the SBL structure.

Location	*Pile*	*Geotechnical Axial Resistance*	
		Factored ULS	**SLS**
Abutment	HP 310x110	1,600 kN	1,400 kN

<u>C – Caissons</u>: Caissons should be founded a minimum 2 m within the “100-blow” clayey silt till or cohesionless soils at or below elevation 166.0 for the NBL structure and elevation 170.0 for the SBL structure. Caissons would be approximately 19 m long for the NBL structure and 15 m long for the SBL structure.

Location	***Caisson Diameter***	***Geotechnical Axial Resistance***	
		Factored ULS	**SLS**
Abutments	1.2 m	4,500 kN	3,500 kN
	1.5 m	6,500 kN	5,500 kN

Recommended Foundation Alternative: Spread footings founded on very stiff to hard clayey silt till / dense silty sand or steel HP 310x110 piles driven to found within the “100-blow” clayey silt till or cohesionless soils are recommended from a foundation engineering perspective.

• ABUTMENT TYPE

The site soils are suitable for construction of conventional, integral or semi-integral abutments.

• APPROACHES

<u>Height</u>: Based on the GA drawings, the south and north approach embankments will be approximately 3 to 4 m high. Based on the subsoil conditions encountered at the site, approach embankments consisting of up to 4 m high earth fill can be constructed. However, sub-excavation of 0.3 to 0.7 m of clayey silt with organics would be required.

<u>Stability</u>: Approach embankments up to 4 m high, constructed of select subgrade materials or granular fill, with side slopes no steeper than 2 horizontal to 1 vertical (2H:1V) will have an adequate factor of safety against deep-seated slope instability. Measures to stabilise the embankment slope face due to potential surface water flow / seepage at the slope surface may have to be implemented.

<u>Settlement</u>: Assuming the use of conventional earth or granular embankment fills, where applicable, the total settlement at the south and north approach embankments is assessed to be in the order of 50 and 70 mm respectively. About 20 per cent of the total settlement is expected to take place during and immediately after completion of construction (i.e. elastic settlement); the remaining consolidation settlement is anticipated to occur over a period of 6 to 9 months. Measures to reduce post-construction settlement to acceptable values may be undertaken (preloading with a surcharge, construction staging). Further geotechnical analyses need to be carried out during detail design to assess the construction requirements of the new embankment fills, including appropriate settlement monitoring instrumentation.

• CONSTRUCTION CONSIDERATIONS

<u>Excavation</u>: The firm to stiff clayey deposits above the water table are classified as Type 3 soils according to OHSA. Temporary excavations (i.e. open for a relatively short time period) should be made with side slopes no steeper than 1H:1V in Type 3 soils assuming dewatering is provided. For saturated granular soils below the water table, temporary shoring may be required.

<u>Groundwater / Surface Water Control</u>: It is anticipated that conventional sump pumping techniques will be sufficient to adequately control groundwater within the foundation excavations. If artesian conditions are present, basal heave will need to be assessed and more elaborate dewatering measures may be required. Artesian groundwater conditions should be expected when advancing deep foundations such as piles through the silty/sandy soils.

<u>Obstructions During Pile Driving</u>: Pile tip reinforcement for steel H-Piles should be used to facilitate driving into or through the very dense / hard till containing shale fragments and possible cobbles / boulders (though not encountered in the current boreholes). Caisson drilling equipment must be capable of penetrating obstructions when cobbles / boulders are present in the till deposits.

• RECOMMENDATIONS FOR ADDITIONAL WORK

- Further subsurface investigation should be carried out during detail design to confirm the subsoil and groundwater conditions at the location of the bridge foundation elements.

Ontario

Peto MacCallum Ltd.
CONSULTING ENGINEERS
Foundation Design

RECORD OF BOREHOLE No 427N-1

METRIC

G.W.P. ______ LOCATION Highway 427 Co-ords: 4 850 806.5 N; 293 663.2 E ORIGINATED BY A.H.

DIST Central HWY 427 BOREHOLE TYPE Continuous Flight Hollow Stem Augers and Mud Rotary COMPILED BY G.D.

DATUM Geodetic DATE July 15, 2016 CHECKED BY B.R.G.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
189.1	Ground Surface				
0.0	Clayey silt, organics Firm to stiff, Dark brown, Moist	1	SS	9	
188.4 0.7	Clayey silt trace to some sand trace gravel Stiff to hard, Light brown, Moist	2	SS	12	
		3	SS	14	
	(TILL)	4	SS	31	2 10 55 33
	Grey	5	SS	40	
		6	SS	10	
		7	SS	30	
		8	SS	20	
		9	SS	17	1 14 59 26
		10	SS	42	
		11	SS	32	
	a thin layer of sand	12	SS	106	
174.1	Cont'd				

+7, ×5: Numbers refer to Sensitivity; 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

Ontario

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CONSULTING ENGINEERS
Foundation Design

RECORD OF BOREHOLE No 427N-1

METRIC

G.W.P. ______ LOCATION Highway 427 Co-ords: 4 850 806.5 N; 293 663.2 E ORIGINATED BY A.H.

DIST Central HWY 427 BOREHOLE TYPE Continuous Flight Hollow Stem Augers and Mud Rotary COMPILED BY G.D.

DATUM Geodetic DATE July 15, 2016 CHECKED BY B.R.G.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
174.1 15.0	Clayey silt some sand, trace gravel Hard, Grey, Moist (TILL) (Cont'd.)	13	SS	77	
172.9 16.2	Silt, with sand some gravel, trace clay Very dense, Grey, Moist (TILL)	14	SS	104/15cm	15 27 50 8
171.4 17.7	Silty sand, trace gravel Very dense, Grey, Moist	15	SS	100/8cm	
169.9 19.2	Clayey silt, trace sand Hard, Grey, Moist	16	SS	45	0 1 62 37
	trace gravel shale fragments (TILL)	17	SS	123/15cm	
		18	SS	120/15cm	
165.9 23.2	End of borehole				

* Borehole charged with drilling water

+7, ×5: Numbers refer to Sensitivity; 20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

Ontario

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RECORD OF BOREHOLE No 427N-2 1 of 2 METRIC

G.W.P. ____ LOCATION Highway 427 Co-ords: 4 850 781.0 N; 293 673.7 E ORIGINATED BY A.H.

DIST Central HWY 427 BOREHOLE TYPE Continuous Flight Hollow Stem Augers and Mud Rotary COMPILED BY G.D.

DATUM Geodetic DATE July 14, 2016 CHECKED BY B.R.G.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
188.9	Ground Surface				
0.0 / 188.6 / 0.3	Clayey silt, organics Firm Dark brown Moist	1	SS	6	
	Clayey silt some sand, trace gravel Firm to hard Light brown Moist (TILL)	2	SS	7	
		3	SS	18	1 12 60 27
		4	SS	34	
	Grey	5	SS	30	
184.8 / 4.1	Silty sand, trace clay Dense Grey Moist	6	SS	46	0 59 34 7
183.3 / 5.6	Clayey silt some sand, trace gravel Very stiff to hard Grey Moist (TILL)	7	SS	19	
		8	SS	19	
		9	SS	23	
		10	SS	55	2 15 61 22
		11	SS	48	
		12	SS	32	
173.9	Cont'd				

ON MTO_NEW LOGO HWY 427 15TF013A.GPJ ON_MOT.GDT 24/08/2016 3:53:43 PM

$+^{7}, \times^{5}$: Numbers refer to Sensitivity (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 427N-2 2 of 2 METRIC

G.W.P. ____ LOCATION Highway 427 Co-ords: 4 850 781.0 N; 293 673.7 E ORIGINATED BY A.H.

DIST Central HWY 427 BOREHOLE TYPE Continuous Flight Hollow Stem Augers and Mud Rotary COMPILED BY G.D.

DATUM Geodetic DATE July 14, 2016 CHECKED BY B.R.G.

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
173.9 / 15.0	Clayey silt some sand, trace gravel Hard Grey Moist (TILL) (Cont'd.)	13	SS	56	
172.7 / 16.2	Silt, some sand trace clay, trace gravel Very dense Grey Moist (TILL)	14	SS	85	6 19 66 9
170.7 / 18.2	Sand with silt, trace gravel Very dense Grey Moist	15	SS	85	
	with gravel, trace silt Wet	16	SS	102	
	trace gravel				
167.5 / 21.4	Clayey silt trace sand, trace gravel shale fragments Hard Grey Moist (TILL)	17	SS	100/8cm	
		18	SS	102/15cm	
165.7 / 23.2	End of borehole				

* Borehole charged with drilling water

ON MTO_NEW LOGO HWY 427 15TF013A.GPJ ON_MOT.GDT 24/08/2016 3:53:43 PM

$+^{7}, \times^{5}$: Numbers refer to Sensitivity (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 427S-1

 METRIC

G.W.P. LOCATION Highway 427 Co-ords: 4 850 780.8 N; 293 588.5 E ORIGINATED BY A.H.

DIST Central HWY 427 BOREHOLE TYPE Continuous Flight Hollow Stem Augers and Mud Rotary COMPILED BY G.D.

DATUM Geodetic DATE July 12 and 13, 2016 CHECKED BY B.R.G.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 | SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100 | PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR SA SI CL
189.3	Ground Surface				
0.0	Clayey silt, trace sand Firm Light brown Moist	1	SS	7	
188.6 0.7	Clayey silt trace to some sand trace gravel Firm to hard Light brown Moist (TILL)	2	SS	5	
		3	SS	19	
		4	SS	23	
		5	SS	29	2 19 50 29
	Grey	6	SS	21	
		7	SS	25	
		8	SS	15	1 15 62 22
		9	SS	13	
		10	SS	12	
		11	SS	18	3 8 51 38
		12	SS	32	
174.3	Cont'd				

Elevation scale: 189, 188, 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175

ON MTO_NEW LOGO HWY 427 15TF013A.GPJ ON_MOT.GDT 24/08/2016 3:53:43 PM

$+^3, \times^5$: Numbers refer to Sensitivity — 20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 427S-1

 METRIC

G.W.P. LOCATION Highway 427 Co-ords: 4 850 780.8 N; 293 588.5 E ORIGINATED BY A.H.

DIST Central HWY 427 BOREHOLE TYPE Continuous Flight Hollow Stem Augers and Mud Rotary COMPILED BY G.D.

DATUM Geodetic DATE July 12 and 13, 2016 CHECKED BY B.R.G.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR SA SI CL
174.3 15.0	Clayey silt some sand, trace gravel Hard Grey Moist (TILL) (Cont'd.)	13	SS	125/25cm	
173.1 16.2	Sandy silt trace clay, trace gravel shale fragments Very dense Grey Moist to wet (TILL)	14	SS	106/15cm	
171.6 17.7	Sand, trace silt Very dense Grey Wet	15	SS	120/15cm	0 92 (8)
170.1 19.2	Silty sand some clay, trace gravel shale fragments Very dense Grey Moist (TILL)	16	SS	130/23cm	
168.5 20.8	Clayey silt some sand, trace gravel shale fragments Hard Grey Moist (TILL)	17	SS	100/5cm	
		18	SS	100/5cm	
		19	SS	50/5cm	
164.6 24.7	End of borehole				

Elevation scale: 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

* 2016 07 12

▽ Water level observed during drilling

▼ Water level measured in piezometer

Water Level Readings:

Date	Depth	Elev.
July 21, 2016	4.4	184.9
Aug. 02, 2016	3.1	186.2

Monitoring Well Legend:

- Flush mount cover and concrete
- Bentonite seal
- Filter sand
- Screen

ON MTO_NEW LOGO HWY 427 15TF013A.GPJ ON_MOT.GDT 24/08/2016 3:53:44 PM

$+^3, \times^5$: Numbers refer to Sensitivity — 20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No 427S-2

METRIC

G.W.P.	
LOCATION	Highway 427 Co-ords: 4 850 755.5 N; 293 594.6 E
ORIGINATED BY	A.H.
DIST	Central
HWY	427
BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
COMPILED BY	G.D.
DATUM	Geodetic
DATE	July 11, 2016
CHECKED BY	B.R.G.

SOIL PROFILE: ELEV / DEPTH, DESCRIPTION, STRAT PLOT. SAMPLES: NUMBER, TYPE, "N" VALUES. GROUND WATER CONDITIONS. ELEVATION SCALE. DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100. SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30. UNIT WEIGHT γ kN/m³. REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL.

ELEV / DEPTH	DESCRIPTION
189.4	Ground Surface
189.1 / 0.0	Clayey silt, trace sand. Stiff, Dark brown, Moist
0.3	Clayey silt, trace to some sand, trace gravel. Very stiff to hard, Light brown, Moist (TILL) — Grey
174.4	Cont'd

NUMBER	TYPE	"N" VALUES	GR SA SI CL
1	SS	15	
2	SS	16	
3	SS	36	
4	SS	32	1 12 57 30
5	SS	27	
6	SS	26	
7	SS	34	
8	SS	38	
9	SS	44	1 18 63 18
10	SS	15	
11	SS	25	
12	SS	75	

Elevation scale: 189, 188, 187, 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175

+³, ×³: Numbers refer to Sensitivity. 20 / 15 – 5 / 10 (%) STRAIN AT FAILURE

Ontario

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RECORD OF BOREHOLE No 427S-2

METRIC

G.W.P.	
LOCATION	Highway 427 Co-ords: 4 850 755.5 N; 293 594.6 E
ORIGINATED BY	A.H.
DIST	Central
HWY	427
BOREHOLE TYPE	Continuous Flight Hollow Stem Augers
COMPILED BY	G.D.
DATUM	Geodetic
DATE	July 11, 2016
CHECKED BY	B.R.G.

ELEV / DEPTH	DESCRIPTION
174.4 / 15.0	Sandy silt, some clay, trace gravel. Compact to dense, Grey, Moist (TILL)
173.2 / 16.2	Silty sand, trace to some gravel, trace clay, shale fragments. Very dense, Grey, Moist to wet (TILL)
167.2 / 22.2	Clayey silt, sandy, trace gravel. Hard, Grey, Moist (TILL)
166.2 / 23.2	End of borehole

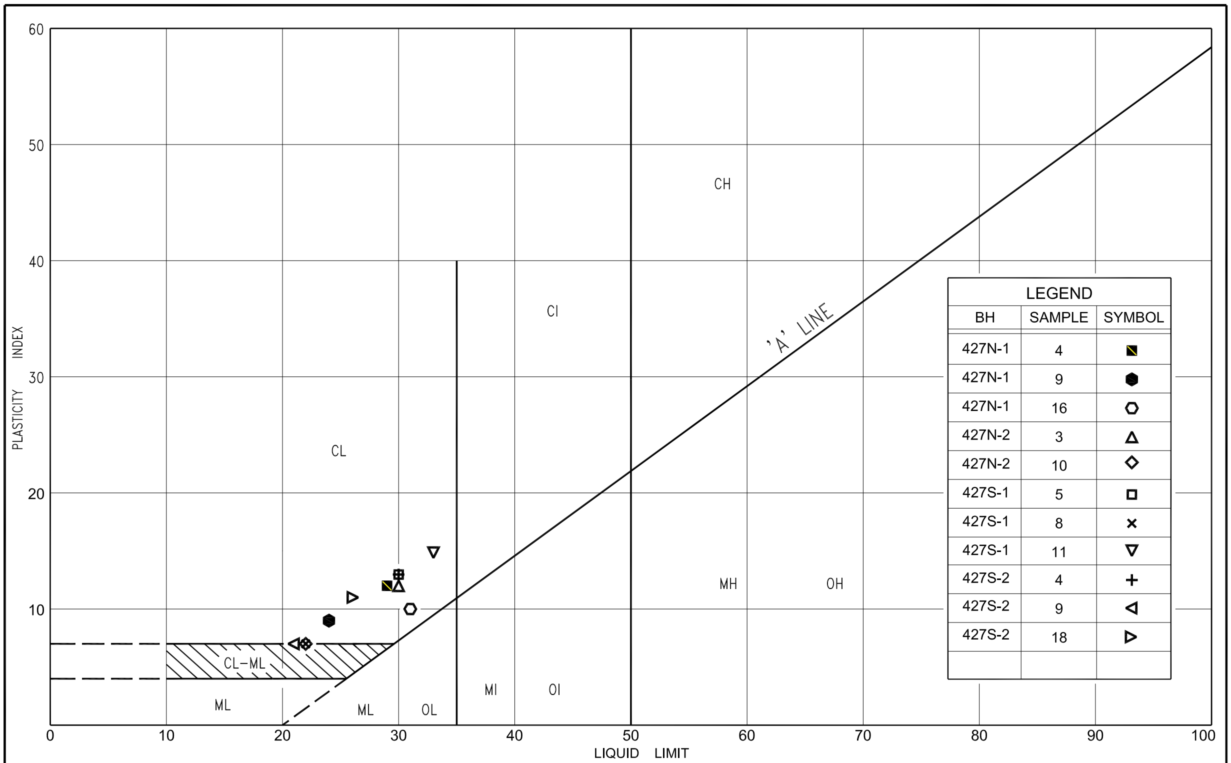
NUMBER	TYPE	"N" VALUES	GR SA SI CL
13	SS	28	7 31 47 15
14	SS	115/15cm	
15	SS	50/5cm	
16	SS	102/15cm	11 47 36 6
17	SS	110/15cm	
18	SS	122/23cm	6 32 40 22

Elevation scale: 174, 173, 172, 171, 170, 169, 168, 167

* 2016 07 11

∇ Water level observed during drilling

+³, ×³: Numbers refer to Sensitivity. 20 / 15 – 5 / 10 (%) STRAIN AT FAILURE



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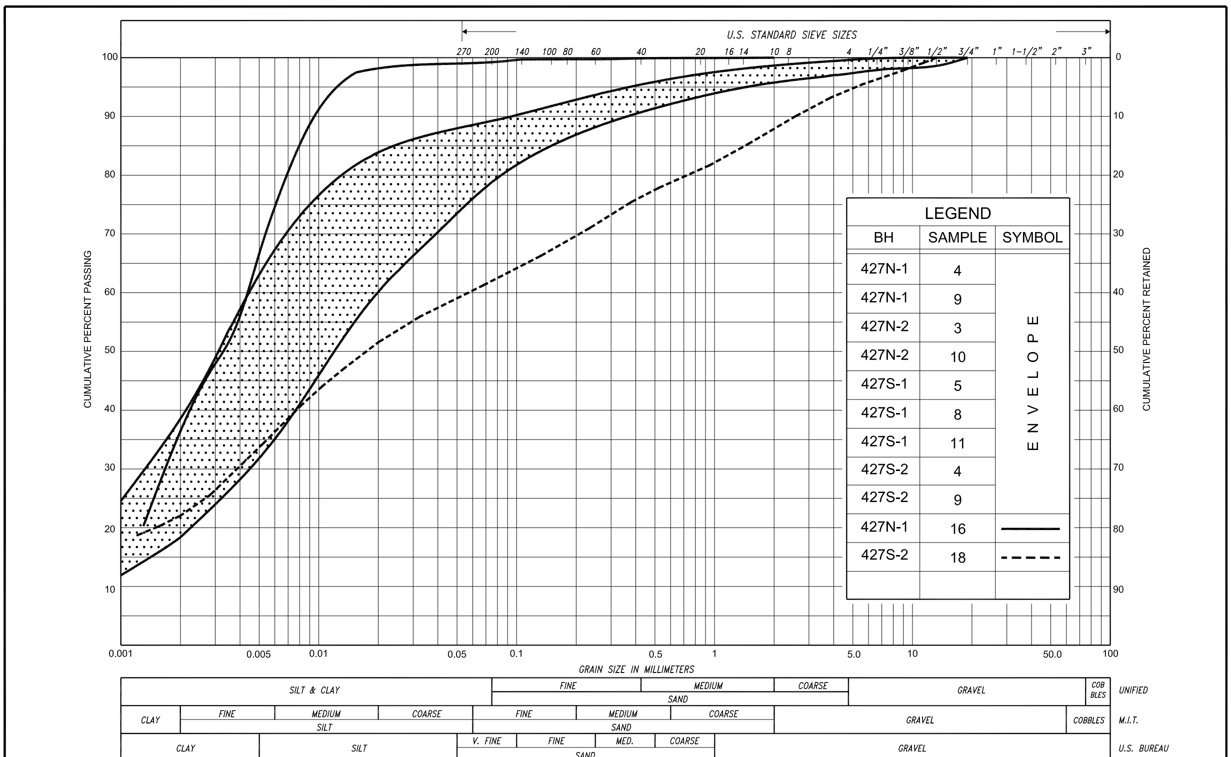
PLASTICITY CHART

CLAYEY SILT, trace sand to sandy, trace gravel (CL)
(TILL)

FIG No. SA-PC-1

HWY: 427



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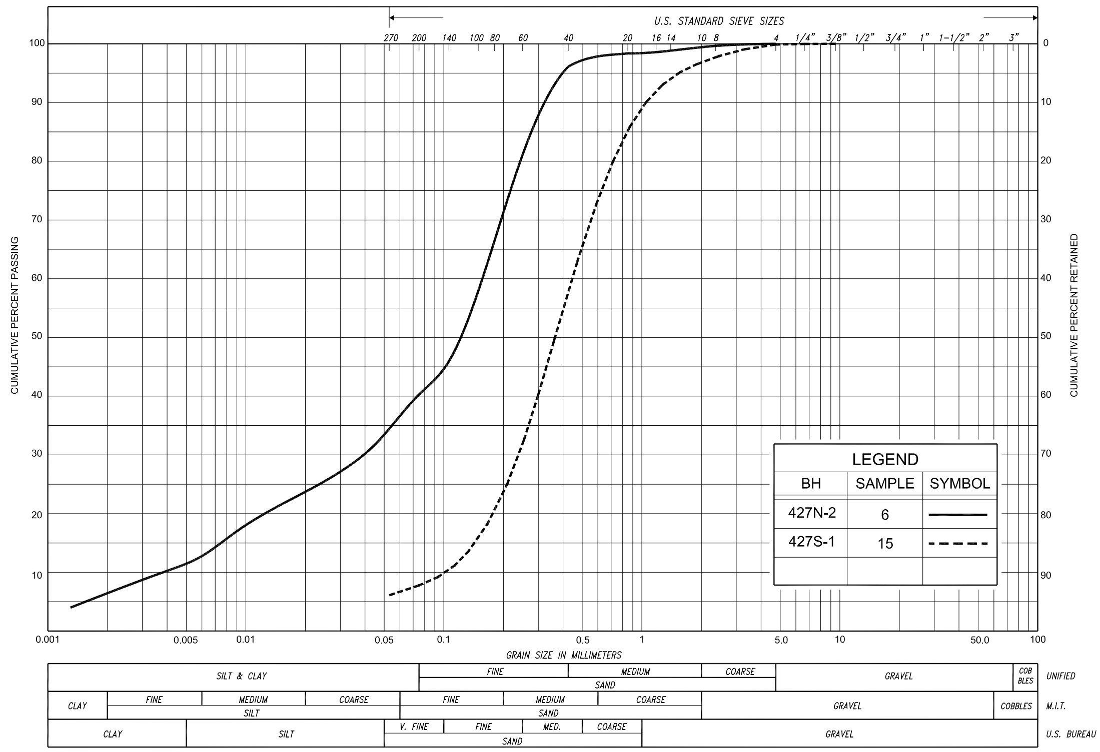
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace sand to sandy, trace gravel
(TILL)

FIG No. SA-GS-1

HWY: 427



LEGEND		
BH	SAMPLE	SYMBOL
427N-2	6	———
427S-1	15	- - - -

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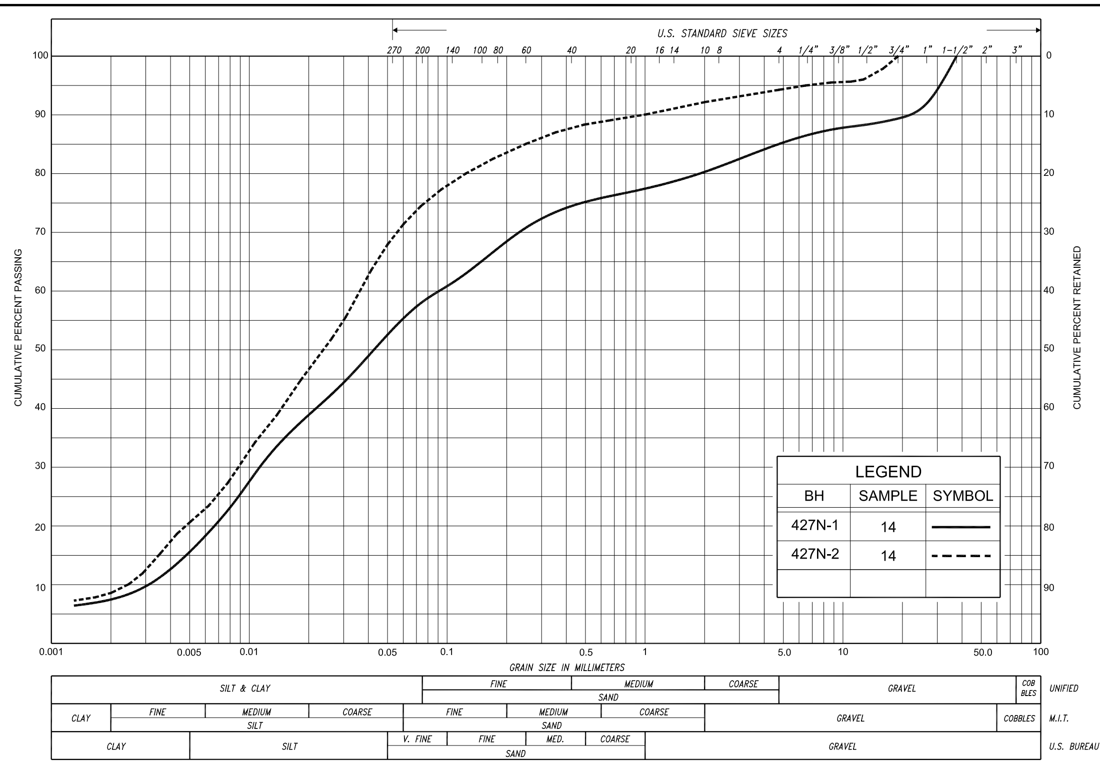
GRAIN SIZE DISTRIBUTION

SILTY SAND, trace clay / SAND, trace silt

FIG No.	SA-GS-2
HWY:	427



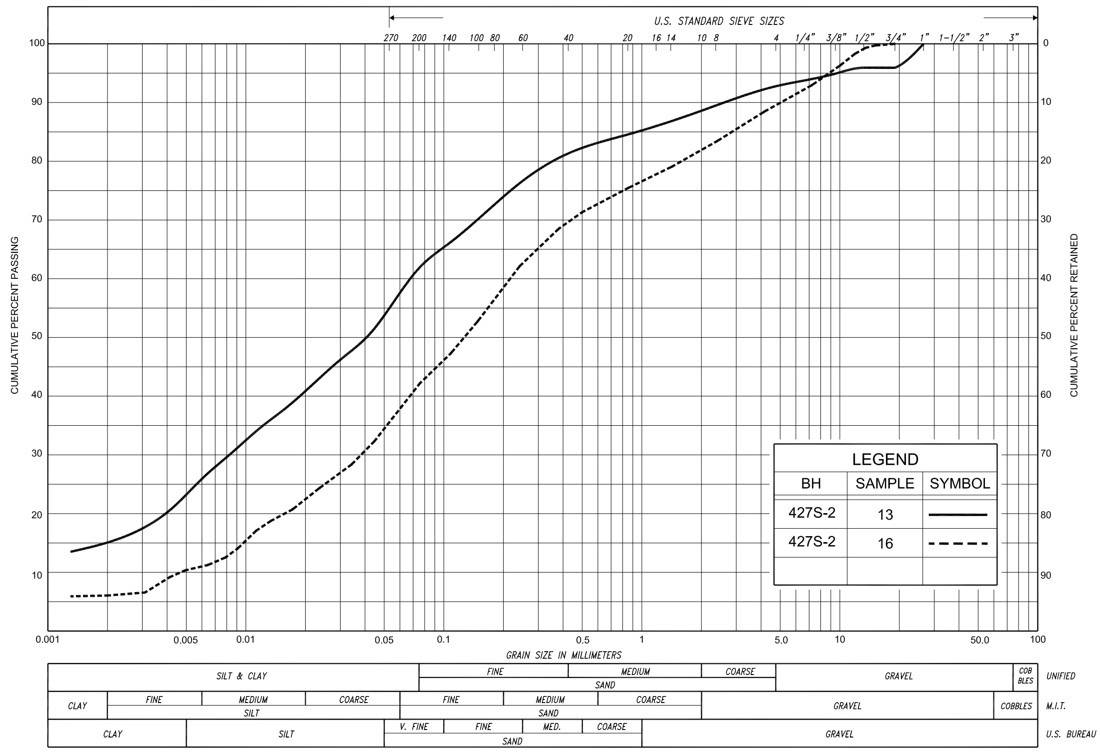
LEGEND		
BH	SAMPLE	SYMBOL
427N-1	14	———
427N-2	14	- - - -

Ontario

GRAIN SIZE DISTRIBUTION

SILT, some to with sand, trace to some gravel, trace clay (TILL)

FIG No.	SA-GS-3
HWY:	427



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GRAIN SIZE DISTRIBUTION

SANDY SILT / SILTY SAND, trace to some clay, trace to some gravel
(TILL)

FIG No.	SA-GS-4
HWY:	427