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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Albert Street Underpass Structure Site No. 22-177 Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

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REPORT



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**PRELIMINARY FOUNDATION REPORT
ALBERT STREET UNDERPASS, W.O. 10-20011**

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
ALBERT STREET UNDERPASS
STRUCTURE SITE NO. 22-177
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, Ontario. This report addresses the proposed replacement of the existing Albert Street underpass.

The terms of reference for the preliminary foundation engineering services are outlined in MTO's Request for Proposals (RFP) for Assignment No. 2010-E-0062, dated June 2011. The scope of work for the preliminary foundation engineering services is presented in Section 5.8 of AECOM's *Technical Proposal* for this assignment, as well as Golder's Scope Change for Foundations Engineering Services letter dated December 8, 2014.

2.0 SITE DESCRIPTION

The Albert Street underpass is located in the City of Oshawa, in the Regional Municipality of Durham. The existing Albert Street underpass is a two-span structure supported on spread footings with a span of about 30 m.

The natural ground surface at this site is at approximately Elevation 100 m to 101 m. Highway 401 has been constructed in a cut, with its grade at approximately Elevation 96 m in the vicinity of the underpass. The Albert Street grade varies from about Elevation 102 m to 103 m, rising northward. A church and heritage centre are located in the northeast and northwest quadrants of the structure site, respectively. Residential areas are present in the southeast and southwest quadrants of the structure site.

3.0 INVESTIGATION PROCEDURES

The field investigation was carried out in March 2015, at which time two boreholes, designated as Boreholes A1 and A2, were advanced to a depth of approximately 9.5 m and 8.0 m below the existing ground surface, respectively, within the vicinity of the Albert Street Underpass.

The boreholes were advanced using a CME-55 truck-mounted drill rig supplied and operated by DBW Drilling Ltd. of North York, Ontario. The boreholes were advanced using 150 mm diameter solid stem augers, with soil samples obtained at approximately 0.75 m and 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers driven by an automatic hammer, in accordance with the Standard Penetration Test (SPT) procedure. (ASTM D1586-11 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the Soil).

The groundwater conditions were observed in the open borehole during and immediately following the drilling operations. The boreholes were backfilled with bentonite on completion of drilling, in accordance with Ontario Regulation 903 (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and geotechnical laboratory testing. All of the



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laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Index and classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected samples. The results of the geotechnical laboratory testing are included in Appendix B.

The borehole locations, including MTM NAD 83 northing and easting coordinates and the ground surface elevations referenced to geodetic datum, are summarized in the following table, presented on the Record of Borehole sheets in Appendix A and are summarized on Drawing 1.

Borehole Number	UTM NAD83 Northing (m)	UTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
A1	4,860,447.6	356,680.2	102.1	9.5
A2	4,860,377.6	356,679.4	102.0	8.0

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984) and *Urban Geology of Canadian Cities* (Brennard, 1998). The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. More recent alluvial deposits of gravel, sand, silt and/or clay are present in the creek valleys.

4.2 Subsurface Conditions

Boreholes A1 and A2 were advanced in the vicinity of the proposed north and south abutments, respectively, at the Albert Street underpass site. The detailed subsurface soil and groundwater conditions encountered in the boreholes, and the results of in situ and laboratory testing, are presented on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented in Appendix B.

The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The interpreted stratigraphic profile at the structure site, shown on Drawing 1, is a simplification of the subsurface conditions. Variation in the stratigraphic boundaries between and beyond boreholes will exist and is to be expected.

In general, thin layers of topsoil and fill are underlain by a thin, very stiff to hard clayey silt layer, then a compact to very dense upper silt and sand till deposit. This upper till deposit is underlain by a dense to very dense non-cohesive deposit that varies in composition from silt and sand to sand and gravel, which is in turn underlain by a lower till deposit that varies in composition from clayey silt till to silty sand till. The surface of very dense soils having Standard Penetration Test “N”-values of greater than 100 blows per 0.3 m of penetration was encountered at depths of 6 m to 7.5 m, at approximately Elevation 94.5 m to 96 m. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.



4.2.1 Topsoil

Approximately 500 mm of topsoil was encountered immediately below the existing ground surface in Boreholes A1 and A2.

4.2.2 Fill

Fill material was encountered in Boreholes A1 and A2 immediately below the topsoil. The fill was encountered at a depth of about 0.5 m, corresponding to Elevation 101.6 m and 101.5 m with a thickness of about 0.4 m and 0.9 m in Boreholes A1 and A2, respectively. The fill consists of silty clay containing trace sand, trace gravel and trace to some organics. Natural water contents of 23 and 24 per cent were measured on two samples of the fill.

One Standard Penetration Test (SPT) “N”-value of 15 blows per 0.3 m of penetration was measured within the fill, suggesting that the fill has a stiff to very stiff consistency.

4.2.3 Clayey Silt

A thin deposit of clayey silt was encountered underlying the fill at a depth of about 0.9 m and 1.4 m in Boreholes A1 and A2, respectively. The clayey silt was encountered at about Elevation 101.2 m and 100.6 m with a thickness of about 0.5 m and 0.4 m in Boreholes A1 and A2, respectively.

The deposit consists of brown clayey silt containing trace sand and trace gravel. Atterberg limits testing was conducted on one sample of the deposit and measured a plastic limit of 13 per cent, a liquid limit of 25 per cent and a plasticity index of 12 per cent. The test result, which is plotted on a plasticity chart on Figure B1 in Appendix B, confirms that the deposit consists of clayey silt of low plasticity. The natural water content measured on one selected sample of the silty clay was 14 per cent, near the plastic limit of the material.

The measured SPT “N”-values within the silty clay are 18 blows and 35 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.

4.2.4 Upper Silt and Sand Till

A 3.1 m to 3.2 m thick deposit of silt and sand till was encountered underlying the silty clay deposit at a depth of about 1.4 m and 1.8 m in Boreholes A1 and A2, respectively. The silty sand till was encountered at about Elevation 100.7 m and 100.2 m in these boreholes.

The till deposit consists of silt and sand containing trace to some clay and trace gravel. The silt and sand is brown to grey, contains oxidation staining and is moist to wet. The results of grain size distribution tests completed on two samples of the deposit are presented on Figure B2 in Appendix B. Natural water contents of 6 and 8 per cent were measured on two samples of the deposit.

The measured SPT “N”-values within the silt and sand till range from 26 blows to 135 blows per 0.3 m of penetration, indicating a compact to very dense relative density.

4.2.5 Silt and Sand to Sand and Gravel

A non-cohesive deposit was encountered underlying the upper silt and sand till deposit at a depth of about 4.6 m and 4.9 m (Elevation 97.5 m and 97.1 m) in Boreholes A1 and A2, respectively. The deposit is about 4 m thick where it was fully penetrated in Borehole A1; it is at least 3.1 m in thickness on the south side of the highway, where Borehole A2 terminated within the deposit.



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The deposit varies in composition from silt and sand, to silty sand, to sand containing trace silt and trace gravel, to sand and gravel. The presence of cobbles was inferred from difficulties advancing augers (auger grinding) in Borehole A1 at depths ranging from about 4.6 m to 6.1 m, corresponding to Elevations 97.5 m and 96.0 m, during the drilling operations. The results of grain size distribution tests completed on two selected samples of the deposit are presented on Figure B3 in Appendix B. Natural water contents of 13 and 16 per cent were measured on two samples of the deposit.

The measured SPT "N"-values within the sand ranged from 43 blows to greater than 100 blows per 0.3 m of penetration, indicating a dense to very dense relative density.

4.2.6 Lower Clayey Silt Till to Silty Sand Till

A lower till deposit was encountered underlying the silt and sand to sand and gravel deposit, at a depth of about 8.6 m (Elevation 93.5 m) in Borehole A1. The borehole terminated within the till deposit at about Elevation 92.6 m, penetrating the deposit for a thickness of about 0.9 m.

The deposit consists of clayey silt till containing some sand and trace gravel that transitions into silty sand till containing trace to some clay and trace gravel. The natural water content measured on one sample of the silty sand till was 9 per cent.

One measured SPT "N"-value within the silty sand till was 100 blows per 0.08 m of penetration, indicating a very dense relative density.

4.3 Groundwater Conditions

The groundwater levels were measured in the open boreholes upon completion of drilling operations. Details of the measured groundwater levels are shown on the borehole records in Appendix A. The groundwater levels recorded in the open boreholes are summarized below:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Interpreted Groundwater Elevation (m)	Date	Comments
A1	102.1	5.8	96.3	March 9, 2015	Open Borehole
A2	102.0	Caved to 7.3*	94.7*	March 10, 2015	Open Borehole

* Note that wet soils were observed at a depth of 4.6 m (Elevation 97.4 m) during sampling in Borehole A2.

The groundwater level at this site will be subject to seasonal fluctuations and precipitation events. The water level should be expected to be higher during the spring season or following periods of heavy precipitation or snow melt.



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5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ms. Haley Schafer and was reviewed by Ms. Nikol Kochmanová, P.Eng., a geotechnical engineer with Golder. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
ALBERT STREET UNDERPASS
STRUCTURE SITE NO. 22-177
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO COURTICE ROAD
REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



6.0 DISCUSSION AND PRELIMINARY ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation recommendations in support of the proposed replacement of the existing Highway 401-Albert Street underpass (MTO Structure Site 22-177). These preliminary recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the preliminary subsurface investigation at this site. This Preliminary Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of MTO to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. This Preliminary Foundation Design Report shall not be used or relied upon for any other purpose or by any other parties, including contractors. Further investigation and design will be required during the detailed design stage.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the contract documents. Contractors must make their own interpretation of the factual information provided in the Preliminary Foundation Investigation Report, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

It is understood that as part of the future improvements and widening of Highway 401 from Brock Road to Courtice Road in the Regional Municipality of Durham, the existing Albert Street underpass will be replaced. The existing underpass structure was constructed in 1939. No design or as-built drawings are available for the existing Albert Street underpass.

Based on the preliminary General Arrangement drawing provided by AECOM, the proposed replacement structure is to consist of a three-span structure with a total span length of approximately 73.1 m. Highway 401 will be widened from the existing six lanes to ten lanes, including an additional two westbound lanes to accommodate the new eastbound off-ramp and one lane for the new westbound off-ramp. Additionally, the Highway 401 grade is proposed to be lowered to approximately Elevation 95.5 m, while the Albert Street grade will be maintained.

Both shallow and deep foundation options have been considered for support of the abutments and piers for the proposed Albert Street underpass replacement structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and relative costs is provided in Table 1 following the text of this report.

- **Strip footings founded on the very dense silt and sand to sand and gravel deposit:** Shallow footings are feasible at this site due to the generally very dense nature of the native soils below the Highway 401 grade, although this foundation type will preclude the use of integral abutments. This option would require excavation to a depth of about 1.2 m below the proposed Highway 401 grade at the pier locations, and about 5.5 m to 7 m of excavation below the natural ground surface/Albert Street grade at the abutments, to provide adequate protection against frost penetration. Temporary protection systems are expected to be required along Highway 401 to facilitate the removal of the existing structure, widening of the highway and



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the construction of the new structure; depending on construction staging, protection systems may also be required along Albert Street. The proposed founding level will be near or below the groundwater level at the site, and groundwater control is expected to be required to enable shallow foundations to be constructed in “dry” conditions.

- **Footings “perched” on a compacted granular pad in the approach embankment:** Footings “perched” in the approach embankments above the Highway 401 grade are geotechnically feasible for support of the new abutments. However, an open structure is not geometrically feasible given the proximity of the local road to the north of the highway, and buildings to the south of the highway; a closed structure type is required. Therefore, this alternative is not discussed further in this report.
- **Driven steel H-piles or pipe piles founded in the very dense silt and sand to sand and gravel deposit at the south abutment, or the hard/very dense lower clayey silt till to silty sand till deposit at the north abutment:** Driven steel H-piles or steel pipe (tube) piles are feasible for support of the abutments, and would permit design of conventional abutments, semi-integral abutments (for tube piles) or integral abutments (for H-piles). The abutments may be constructed with a pile cap perched above the Highway 401 grade in a false abutment configuration, with reinforced soil system (RSS) walls in front of the pile caps. Driven steel piles are feasible but not recommended at the pier locations due to the shallow depth to “100-blow” material, which would require pre-augering prior to pile installation. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense/hard deposits at the abutments.
- **Caissons founded on the very dense silt and sand to sand and gravel or hard/very dense lower clayey silt till to silty sand till deposit:** Caissons are considered feasible for the support of the abutments; however this option would preclude integral abutment design. A perched pile cap would minimize excavation and groundwater control requirements at the new abutments; however, it is understood that a closed abutment configuration is required for this structure, so this advantage likely cannot be realized at this site. Caissons are also feasible at the piers, and may be geometrically desirable as they may be constructed in a smaller footprint than a spread footing or pile cap. This option will be more expensive than either shallow foundations or pile foundations, although fewer caisson elements would be required in comparison to the number of steel piles that would be required. If caissons are adopted for support of the abutments or piers, they would extend into water-bearing non-cohesive soil deposits; temporary liners would be required during construction to control potential ground losses and/or disturbance at the caisson base.

Based on the above considerations, both shallow and deep foundation options are considered feasible for the support of the new abutments. Typically at the abutments, pile foundations would be preferred with a perched pile cap in a false abutment configuration, to permit integral abutments and minimize excavation and groundwater control requirements. However, for this site, a closed structure type is required due to the proximity of the local road on the north side of the highway, and buildings on the south side, together with the requirement for widening of the Highway 401 cut. Therefore, shallow foundations are preferred given the shallow depth to “100-blow” soil below the Highway 401 grade. At the piers, shallow foundations are preferred from a geotechnical/foundations perspective due to the presence of a suitable bearing stratum at shallow depth. However, short caisson foundations may also be adopted at the piers; although more expensive, they may occupy a smaller footprint than other foundation types.



6.3 Shallow Foundations

6.3.1 Founding Elevation and Frost Protection Requirements

For support of the abutments, piers and associated wing walls for the new proposed bridge structure, spread/strip footings should be founded on the very dense silt and sand to sand and gravel deposit. Strip or spread footings should be founded at a minimum depth of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*). If adequate soil cover cannot be provided for the footing, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

The Highway 401 grade is proposed to be lowered to approximately Elevation 95.5 m below the underpass structure; as such the maximum (highest) founding elevation recommended for the preliminary design of the footings is Elevation 94.3 m.

It is noted that the preliminary investigation comprised one borehole near each of the north and south abutment areas. While the surface elevation and properties of the silt and sand to sand and gravel deposit are reasonably consistent between Boreholes A1 and A2, additional boreholes should be advanced in the vicinity of the piers to confirm the founding levels and geotechnical resistances.

6.3.2 Geotechnical Axial Resistance and Reaction

The following factored geotechnical axial resistance at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS, for 25 mm of settlement) may be used for preliminary design of strip footings founded on the properly prepared silt and sand to sand and gravel deposit at the design elevations given in the preceding section:

Foundation Alternative	Factored Geotechnical Axial Resistance at ULS (kPa)	Geotechnical Reaction at SLS for 25 mm of Settlement (kPa)
Footing on properly prepared very dense silt and sand to sand and gravel	600	400

NOTE: The geotechnical resistance/reaction values given above are estimated for a 3 m wide spread/strip footing.

The geotechnical resistances provided herein are given for loads will that 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 the Table 10.2 in *CFEM* (2006).

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

6.4.1 Founding Elevation

If geometrically suitable, the abutments and associated retaining walls for the replacement structure may be supported on steel piles driven to found within the very dense (“100-blow”) silt and sand to sand and gravel deposit at the south abutment, and the hard/very dense clayey silt till to silty sand till at the north abutment. Typically, it would be recommended that the abutment pile caps be perched above the Highway 401 grade to



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minimize excavation and groundwater control requirements, and to start pile driving at a sufficient height above the surface of the “100-blow” soil to minimize requirements to pre-auger to install piles.

For preliminary design purposes, if integral abutments were to be adopted for the design of the structure replacement, it has been assumed that the pile caps would be “perched” within the embankments. The following pile tip elevations are recommended for preliminary design, assuming about 2 m penetration into “100-blow” soil:

Foundation Element	Estimated Pile Cap Elevation (m)	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)	Approximate Pile Length (m)	Founding Soil at Tip Elevation
North Abutment	100	94.5	92.5	7.5	Clayey silt till
South Abutment	99	96	94	5	Silt and sand to sand and gravel

Based on the above elevations, the proposed piles are estimated to be approximately 5 m to 7.5 m long. Given the relatively shallow depth to the “100-blow” soil, pre-augering may be required, with the piles subsequently installed within the CSP and pre-augered hole using centralizers.

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the glacially-derived soils at this site, as well as the potential for damage to the pile tips during seating on the bedrock. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of experiencing refusal on boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. Piles should be reinforced at the tip with driving shoes and/or flange plates in accordance with OPSD 3000.100 (*Steel H-Pile Driving Shoe*) or OPSD 3001.100 (*Steel Tube Pile Driving Shoe*) Type II, as appropriate, to reduce the potential for damage to the piles during driving. In very dense strata containing cobbles and/or boulders, as encountered at this site, driving shoes (such as Titus Standard ‘H’ Bearing Pile Points) are preferred over flange plates.

6.4.2 Geotechnical Axial Resistance/Reaction

For HP 310x110 piles driven to the design tip elevations given above, the factored axial geotechnical resistance at ULS may be taken as 1,400 kN. The axial geotechnical reaction at SLS may be taken as 1,200 kN for 25 mm of settlement. The same axial resistances may be used in the preliminary design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.). Additional, deeper boreholes will be required during detail design if this foundation option is selected, in order to confirm these geotechnical resistances.

Pile installation should be in accordance with OPSS 903 (*Deep Foundations*). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO’s Standard Drawing SS103-11, *Pile Driving Control*) during the final stages of driving to verify that the required ultimate capacity has been achieved.



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If pile foundations are adopted, the preliminary geotechnical resistances/reactions provided above will have to be re-evaluated and modified, as necessary, during detail design in consideration of the structure geometry and additional subsurface investigation at the foundation elements.

6.5 Caisson Foundations

6.5.1 Founding Elevation

Caissons founded within the very dense silt and sand to sand and gravel or lower clayey silt to silty sand till deposits may be considered for support of the abutments and piers for the proposed replacement structure; however, as the “100-blow” soils are present at a shallow depth below the Highway 401 cut grade, this foundation option may not be as cost-effective as shallow foundations for the structure geometry at this site. If adopted, the following caisson founding elevations may be used for preliminary design purposes, assuming about 2 m penetration into “100-blow” soil:

Foundation Element	Estimated Pile Cap Elevation (m)	Approximate Surface Elevation of “100-Blow” Soil (m)	Estimated Design Tip Elevation (m)	Founding Soil at Tip Elevation
North Abutment	100	94.5	92.5	Silt and sand to sand and gravel
South Abutment	99	96	94	Clayey silt to silty sand till

If caisson foundations are adopted, a temporary liner and a head of water/drilling slurry will be required to support the overburden soils during construction and balance groundwater pressures, to minimize disturbance to the side walls and to control base disturbance/basal heave. In addition, placement of concrete by tremie methods would be required.

6.5.2 Geotechnical Axial Resistance/Reaction

The following factored geotechnical axial resistance at ULS and geotechnical reaction at SLS (for 25 mm of settlement) may be used for preliminary design of caisson foundations:

Caisson Diameter (m)	Factored Geotechnical Axial Resistance at ULS (kN)	Geotechnical Reaction at SLS for 25 mm of Settlement (kN)
1.2	4,000	3,000
1.5	6,000	5,000

If caisson foundations are adopted, the preliminary geotechnical resistances/reactions provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of the structure geometry and additional subsurface investigation at the foundation elements. Lower geotechnical resistance values may



apply at the piers, depending on the depth to the 100-blow soil layer and the groundwater conditions within the highway cut.

6.6 Retained Soil System (RSS) Walls

6.6.1 Founding Elevations

If perched pile caps are used in a false abutment configuration, and for retaining walls adjacent to the abutments and wing walls at this site, retained soil system (RSS) walls are a suitable and feasible alternative to conventional concrete retaining walls supported on shallow foundations. Such walls would be constructed parallel to the Highway 401 cut. Retaining walls beyond the structure replacement are addressed in a separate Foundation Investigation and Design Report for all retaining walls associated with this assignment.

The front facing panels and the reinforced soil mass of the RSS wall should be founded below any existing topsoil or unsuitable fill soils. Typically, the front facing panels are supported on a footing and/or granular levelling pad at a shallow depth below the ground surface in front of the wall. It is recommended that the facing panels be founded at a minimum depth of 0.5 m below the lowest surrounding grade, in accordance with MTO's *RSS Design Guidelines*. The levelling pad should consist of a minimum thickness of 0.3 m of compacted OPSS.PROV 1010 Granular A, which should extend at least 0.5 m beyond the outside edge of both sides of the facing footing, then outward/downward at 1H:1V.

6.6.2 Geotechnical Resistance/Reaction

For the RSS facing panels founded on compacted granular fill as described above, preliminary design may be completed based on a factored geotechnical resistance at ULS of 150 kPa, and a geotechnical reaction at SLS (for 25 mm of settlement) of 100 kPa.

Assuming that the RSS wall (up to approximately 6 m high if constructed in a false abutment configuration in front of a perched pile cap) acts as a unit and uses the full width of the reinforced soil mass (which can be taken as approximately 0.8 times the wall height for preliminary design), a factored geotechnical resistance at ULS of 600 kPa and a geotechnical reaction at SLS of 400 kPa (for 25 mm of settlement) may be used for preliminary design. The preliminary geotechnical resistance/reaction values should be reviewed and revised if necessary during detail design, after the RSS wall configuration and any "step" elevations are confirmed, taking into account any additional subsurface information at that time.

6.6.3 Global Stability of RSS Walls

Preliminary slope stability analyses have been performed for conceptual RSS walls adjacent to the abutments, using the commercially available program *Slide 6.0*, produced by Rocscience Inc., to check that a minimum factor of safety of 1.5 is achieved for the proposed maximum retaining wall heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed walls on this site, considering the design requirements and the available field and laboratory testing data.

The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations (Bowles, 1984 and Kulhawy and Mayne, 1990):



PRELIMINARY FOUNDATION REPORT ALBERT STREET UNDERPASS, W.O. 10-20011

Soil Deposit	Bulk Unit Weight (kN/m ³)	Effective Friction Angle	Undrained Shear Strength (kPa)
Granular backfill in RSS wall	21	35°	-
Very dense silt and sand to sand and gravel	21	35°	-
Hard/very dense lower clayey silt to silty sand till	21	35°	-

The results of the static global stability analysis indicate that a minimum factor of safety of 1.5 is achieved for RSS walls up to approximately 6 m in retained soil height, assuming level ground in front of and behind the wall, as shown on Figure 1. This preliminary assessment of the global stability of RSS walls should be reviewed and confirmed as part of the detail design.

It should be noted that the internal stability of a reinforced earth structure is to be assessed by the proprietary product designer to ensure the internal and external stability of the wall is adequate.

6.7 Construction Considerations

The following sections identify future construction considerations that may impact the future detail design and construction, and for which provision may be required in the contract documents produced as part of detail design.

6.7.1 Open-Cut Excavation and Temporary Protection Systems

The construction of the new strip footings for the replacement structure will require excavations up to about 1.2 m below the existing Highway 401 grade at the pier locations. Widening of the Highway 401 cut and construction of the abutments will require excavations of up to about 5.5 m to 7 m below the Albert Street grade. The existing fill is classified as a Type 3 soil, while the native very stiff to hard and dense to very dense soils are classified as Type 2 soils, according to the Occupational Health and Safety Act (OHSA). As such, temporary open-cut excavations above the groundwater level should be made with side slopes no steeper than 1H:1V. All excavations must be carried out in accordance with Ontario Regulation 213 (Ontario Occupational Health and Safety Act for Construction Projects) (as amended).

It is anticipated that due to space constraints and construction staging requirements, temporary protection systems will be required along the existing Highway 401 eastbound and westbound lanes to facilitate the removal of the existing bridge foundations and walls, and excavation for the widened Highway 401 and new abutments. Temporary protection systems are also likely to be required for the shallow foundation excavations within Highway 401 at the pier locations. Temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the protection system should meet Performance Level 2 as specified in OPSS.PROV 539, provided any adjacent utilities can tolerate this magnitude of deformation.

The selection and design of the protection system will be the responsibility of the Contractor.



6.7.2 Groundwater Control

Groundwater was encountered at about Elevation 96.3 m in the open Borehole A1, on the north side of the highway cut, on completion of drilling. Borehole A2 (on the south side of the highway cut) caved to approximately Elevation 94.7 m on completion of drilling, and this is considered indicative of the approximate groundwater level in this area, although wet soils were also noted at approximately Elevation 97.4 m during sampling operations. The excavations for shallow foundations at the abutments and piers (to about Elevation 94.3 m), together with excavations for the highway cut widening (to about Elevation 95.5 m), will extend near or below the groundwater level at the site. Further investigation and confirmation of the groundwater level and its seasonal variations is recommended as part of detail design.

At this preliminary stage, it is anticipated that an active dewatering system (such as the use of a system of well points) will be required to lower the groundwater in the silt and sand to sand and gravel deposit to at least 0.5 m to 1 m below the proposed foundation level during excavation and footing construction. An accurate prediction of the groundwater pumping volumes cannot be made at this stage. However, it is considered likely that groundwater pumping volumes could exceed 50 m³/day during initial drawdown stages and/or if multiple excavation areas are being dewatered at one time.

At this preliminary stage, it is anticipated that the zone of influence for the dewatering operations would be relatively localized at the structure site. Assuming the dewatering system is properly constructed and operated such that there is no loss of fine soil particles, the dewatering operations are not expected to cause excessive settlement in the dense to very dense silt and sand to sand and gravel deposit, or the overlying soils which are above the groundwater level. However, the potential for dewatering/settlement impacts on the existing or new structure foundations and any adjacent utilities should be re-assessed at the detailed design phase.

6.7.3 Subgrade Protection

The silty native soils that will be exposed within the excavations at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or precipitation and ponded water. To limit the effects of this disturbance, a concrete working slab should be placed on the subgrade within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a minimum 28-day compressive strength of 20 MPa.

6.7.4 Obstructions

The soils at this site are glacially derived and as such should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in any future investigation at this site, to further assess the presence of cobbles and boulders and permit the contractor to assess the impact on foundation construction.

6.7.5 Vibration Monitoring During Pile or Caisson Installation

A maximum peak particle velocity (PPV) of 100 mm/s is generally considered applicable for bridge structures in good condition; lower thresholds are applicable for nearby residential and commercial facilities (between 25 mm/s and 50 mm/s). If pile driving is adopted at the abutments, or if caissons are adopted for the piers, then vibration monitoring is recommended adjacent to the bridge site to demonstrate/confirm that vibration levels do not exceed the threshold levels.



6.8 Recommendations for Future Work During Detail Design

During the detail design phase, additional investigation is recommended to confirm or assess the following:

- the subsurface conditions and geotechnical/foundation recommendations at the pier locations;
- the subsurface conditions at greater depth below foundation elements at the abutment areas, to facilitate deep foundation design if required;
- the groundwater level for more detailed assessment of groundwater control requirements during construction.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Nikol Kochmanová, P.Eng.. Ms. Lisa Coyne, P.Eng., a Designated MTO Foundations Contact, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.



Nikol Kochmanová, P.Eng.
Geotechnical Engineer



Lisa Coyne, P.Eng.
Designated MTO Foundations Contact

HS/NK/LCC/sm

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PRELIMINARY FOUNDATION REPORT ALBERT STREET UNDERPASS, W.O. 10-20011

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- Canadian Geotechnical Society, 1992. *Canadian Foundation Engineering Manual*, 3rd Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.
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- NAVFAC, 1982. *Design Manual DM 7.2: Soil Mechanics, Foundation and Earth Structures*. U.S. Navy. Alexandria, Virginia.

ASTM International:

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

Ministry of Transportation Ontario:

Drawing SS103-11 Pile Driving Control

Ontario Occupational Health and Safety Act:

Ontario Regulation 213 Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)



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OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile, Driving Shoe
OPSD 3001.100	Foundation, Piles, Steel Tube Piles, Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

Ontario Water Resources Act:

Ontario Regulation 903 Wells (as amended)



PRELIMINARY FOUNDATION REPORT ALBERT STREET UNDERPASS, W.O. 10-20011

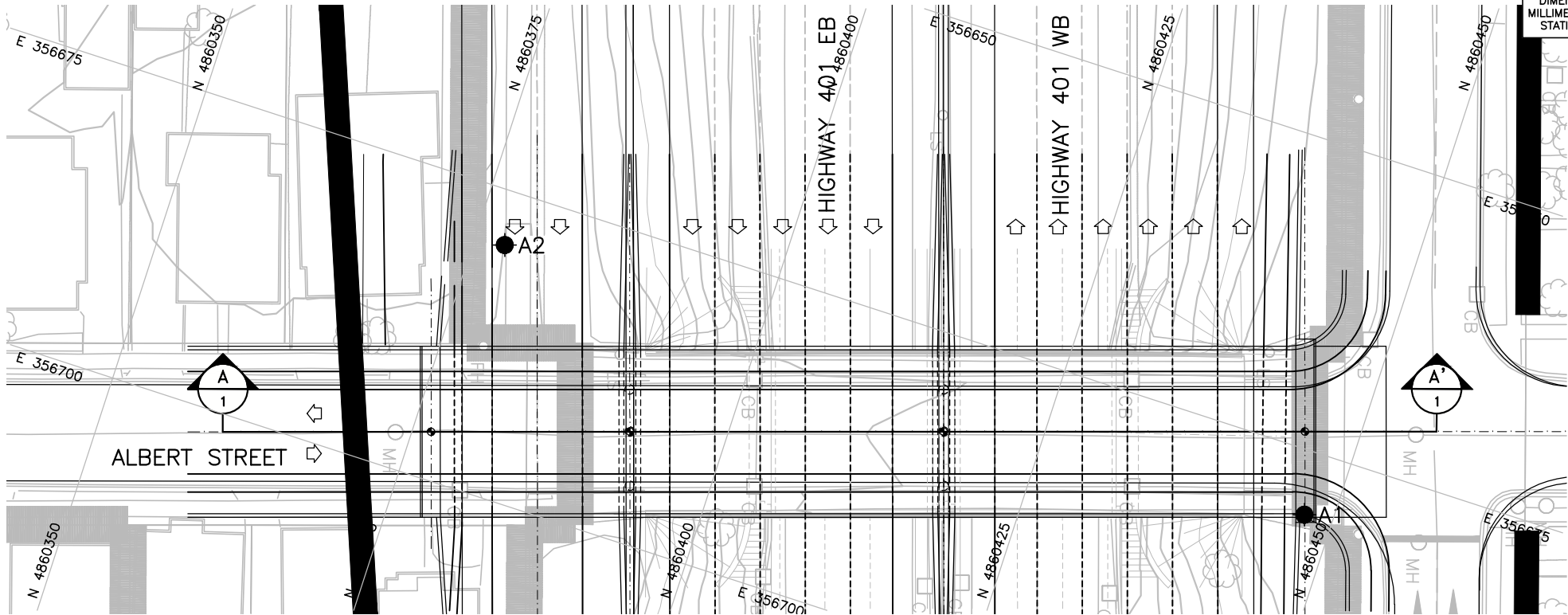
TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Strip footings	<ul style="list-style-type: none"> Feasible for support of the new abutments and piers 	<ul style="list-style-type: none"> Conventional excavation and construction techniques Very dense soils (with SPT “N” values greater than 100 blows per 0.3 m of penetration) present at shallow depth below Highway 401 cut grade, with good geotechnical resistance and settlement performance 	<ul style="list-style-type: none"> Excavation will be required to extend up to 5.5 m to 7 m below Albert Street grade; however, this is required at any rate for Highway 401 cut widening Protection systems required at abutments and piers Excavations will extend near/below the groundwater level and groundwater control will be required 	<ul style="list-style-type: none"> Estimated cost is approximately \$600/m³ for construction of shallow foundations 	<ul style="list-style-type: none"> Some risk of challenging groundwater control in what may be a relatively thin layer containing fine-grained silt as well as sand and gravel
Steel H-piles or pipe piles	<ul style="list-style-type: none"> Feasible for support of abutments if abutment pile caps can be “perched” above Highway 401 grade in false abutment configuration Not recommended for the support of the piers due to shallow depth to 100-blow soil 	<ul style="list-style-type: none"> Conventional construction methods Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system and groundwater control requirements Steel H-piles allow for integral abutment configuration, and pipe piles for semi-integral abutment configuration 	<ul style="list-style-type: none"> Temporary excavation support still required to facilitate removal of existing abutments and widening of Highway 401 Design geotechnical resistance may not be achieved if piles refuse on cobble and boulder layers 	<ul style="list-style-type: none"> Estimated cost is approximately \$200/m length for pile installation and \$600/m³ for pile cap construction 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving

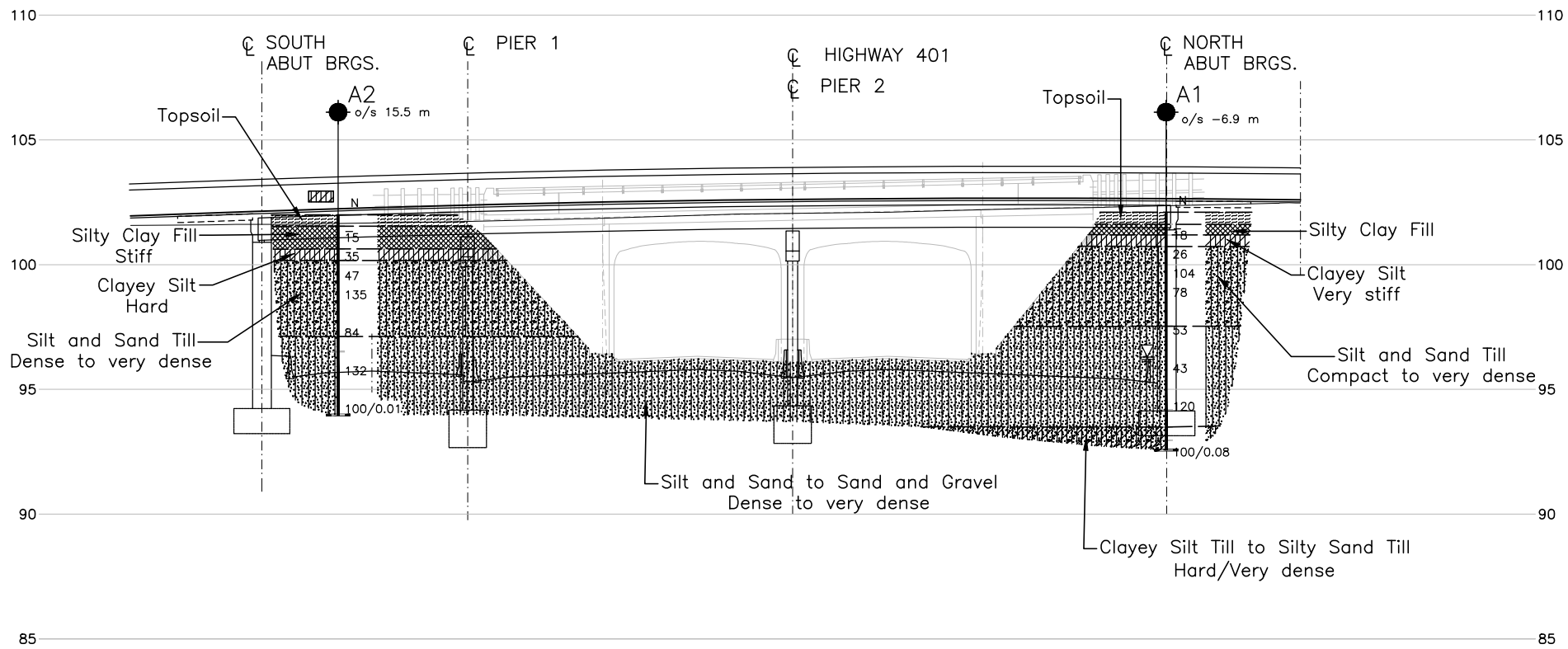


PRELIMINARY FOUNDATION REPORT ALBERT STREET UNDERPASS, W.O. 10-20011

Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
Caissons	<ul style="list-style-type: none">• Feasible but not recommended for support of abutments or piers, due to shallow depth to 100-blow soils and feasibility of shallow foundations, plus requirement for highway cut widening in the vicinity of the abutments	<ul style="list-style-type: none">• Abutment pile caps could be maintained higher than spread footings in an open structure configuration, potentially reducing depth of excavation, protection system and groundwater control requirements• Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles	<ul style="list-style-type: none">• Caissons would extend below the groundwater level at the site into water-bearing non-cohesive soils, with potential for loss of ground or base disturbance• Temporary liners would be required, plus special measures such as use of drilling mud and tremie placement of concrete; likely not possible to inspect caisson base• Precludes use of integral abutments	<ul style="list-style-type: none">• Estimated cost is approximately \$1,000/m length for caisson installation and \$600/m³ for pile cap construction; the cost may be higher to account for temporary liners	<ul style="list-style-type: none">• Risk of loosening or disturbing founding soils at base of caissons



PLAN
SCALE
0 5 10 m

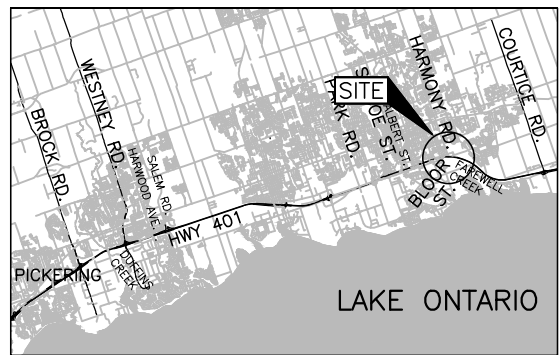


A-A' 1
CENTRELINE OF ALBERT STREET
VERTICAL SCALE
0 2.5 5 m
HORIZONTAL SCALE
0 5 10 m

METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
WO No. 10-20011

ALBERT STREET UNDERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS AND
SOIL STRATA



KEY PLAN
SCALE
0 4 8 km

LEGEND

- Borehole - Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

No.	ELEVATION	NORTHING	EASTING
A1	102.1	4860447.6	356680.2
A2	102.0	4860377.6	356679.4

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Preliminary Design Report.

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

REFERENCE

Base plans provided in digital format by URS, drawing file nos. X-Base.dwg, X-Property.dwg and Street Names.dwg, and the Proposed Design obtained from drawing file x-design_130625.dwg, all dated July 05, 2013, received April 11, 2014. General Arrangement provided in digital format by AECOM, drawing file 01_GA_Albert St_Underpass.dgn, received June 25, 2015.

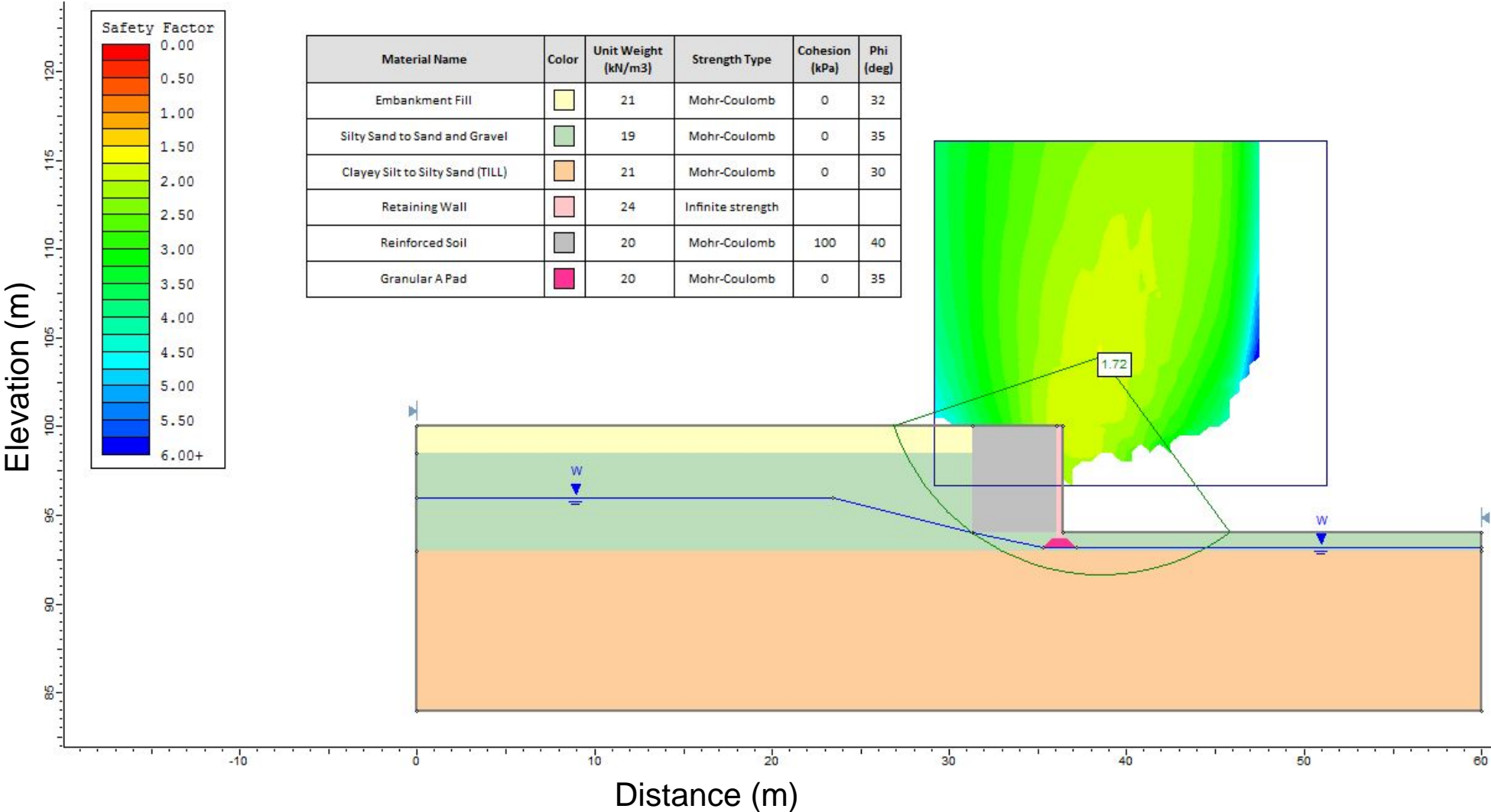


NO.	DATE	BY	REVISION
Geocres No. 30M15-452			
HWY. 401	PROJECT NO. 11-1184-0143		DIST. CENTRAL
SUBM'D. MWK	CHKD. PKS	DATE: 4/5/2017	SITE: 22-177
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 1



STATIC GLOBAL STABILITY
 ALBERT STREET UNDERPASS – RETAINED SOIL SYSTEM WALL

Figure 1





APPENDIX A

Borehole Records



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

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

Notes: 1
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils Consistency

	c_u, s_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT		11-1184-0143		RECORD OF BOREHOLE No A2		SHEET 1 OF 1		METRIC									
G.W.P.		10-20011		LOCATION		N 4860377.6 ; E 356679.4		ORIGINATED BY		TD							
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY		PKS							
DATUM		Geodetic		DATE		March 10, 2015		CHECKED BY		LCC							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
102.0	GROUND SURFACE																
0.0	TOPSOIL																
101.5																	
0.5	Silty clay, trace to some organics, trace gravel, trace sand (FILL) Stiff Dark to light brown Moist		1	AS	-												
100.6			2A	SS	15												
1.4	CLAYEY SILT, trace gravel, trace sand Hard Brown Moist		3A	SS	35												
100.2			3B	SS													
1.8	SILT and SAND, trace to some clay, trace gravel (TILL) Dense to very dense Brown to grey, oxidation staining Moist		4	SS	47												
			5	SS	135												
97.1	- becoming grey and wet below a depth of 4.6 m		6A	SS	84												
4.9	SAND, trace gravel, trace silt Very dense Brown Moist		6B	SS													
96.5																	
5.5	SILT and SAND, trace gravel, trace clay Very dense Grey Wet		7	SS	132												
94.0			8	SS	100/0.0												
8.0	END OF BOREHOLE																
NOTES:																	
1. Borehole caved to a depth of 7.3 m below ground surface (Elev. 94.7 m) upon completion of drilling, March 10, 2015. Borehole dry above this depth on completion of drilling.																	

GTA-MTO 001 T:\PROJECTS\201111-1184-0143 (HWY 401 FROM BROCK RD TO COURTYCE RD)\LOG11-1184-0143.GPJ GAL-GTA.GDT 9/6/16 KD

PROJECT		11-1184-0143		RECORD OF BOREHOLE No S1		SHEET 1 OF 1		METRIC								
G.W.P.		10-20011		LOCATION		N 4860398.8 ; E 356528.6		ORIGINATED BY								
DIST		Central HWY 401		BOREHOLE TYPE		150 mm O.D. Continuous Flight Solid Stem Augers		COMPILED BY								
DATUM		Geodetic		DATE		March 9, 2015		CHECKED BY								
								LCC								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
101.3	GROUND SURFACE															
0.0	TOPSOIL															
100.9																
0.4	SILTY CLAY, trace gravel, trace sand, trace organics, containing rootlets Very stiff Brown Moist		1	AS	-											
			2	SS	21											
99.8																
1.5	SILT and SAND, trace to some clay, trace gravel (TILL) Compact to very dense Brown, oxidation staining Moist		3	SS	24											
			4	SS	55											
			5	SS	57											
	- auger grinding on inferred cobble or boulder between a depth of 3.1 m and 6.1 m - becoming grey below a depth of 3.4 m															
			6	SS	100/0.13											
95.8																
5.5	SAND and GRAVEL, trace to some silt, trace clay Very dense Grey, oxidation staining Moist		7	SS	100/0.1											
94.7																
6.8	Silty SAND, trace clay, trace gravel (TILL) Very dense Brown Moist		8	SS	100/0.1											
	END OF BOREHOLE															
	NOTES:															
	1. Borehole dry upon completion of drilling, March 9, 2015															
	2. Borehole caved to a depth of 5.5 m below ground surface (Elev. 95.8 m) upon completion of drilling, March 9, 2015															

PROJECT <u>11-1184-0143</u>		RECORD OF BOREHOLE No S2		SHEET 1 OF 2		METRIC	
G.W.P. <u>10-20011</u>		LOCATION <u>N 4860328.0 ; E 356527.1</u>		ORIGINATED BY <u>TD</u>			
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>Solid Stem Augers</u>		COMPILED BY <u>PKS</u>			
DATUM <u>Geodetic</u>		DATE <u>March 11, 2015</u>		CHECKED BY <u>LCC</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
100.4	GROUND SURFACE							20 40 60 80 100		W _P	W	W _L					
0.0	TOPSOIL							20 40 60 80 100									
99.9																	
0.5	Silty clay, trace to some sand, trace organics (FILL) Stiff Brown Moist Hydrocarbon odour		1	AS	-		100										
99.0			2	SS	12												
1.4	SILTY CLAY to CLAY, trace sand Stiff to very stiff Brown Moist		3	SS	14		99										
			4	SS	15		98										
			5	SS	15		97										
			6	SS	10		96										
94.3	CLAYEY SILT, some sand, trace gravel (TILL) Hard Grey Moist		7	SS	38		94										
93.4	SILT and SAND, trace to some gravel Dense Grey Wet		8	SS	32		93										
91.9	SAND and GRAVEL, trace silt Very dense Grey Moist		9	SS	136		92										
8.5			10	SS	90		91										
							90										
							89										
88.2	CLAYEY SILT, some sand, trace to some gravel (TILL) Hard Grey Moist		11	SS	145		88										
12.2			12	SS	89		87										
							86										

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 T:\PROJECTS\201111-1184-0143 (HWY 401 FROM BROCK RD TO COURTYARD RD)\LOG11-1184-0143.GPJ GAL-GTA.GDT 9/6/16 KD

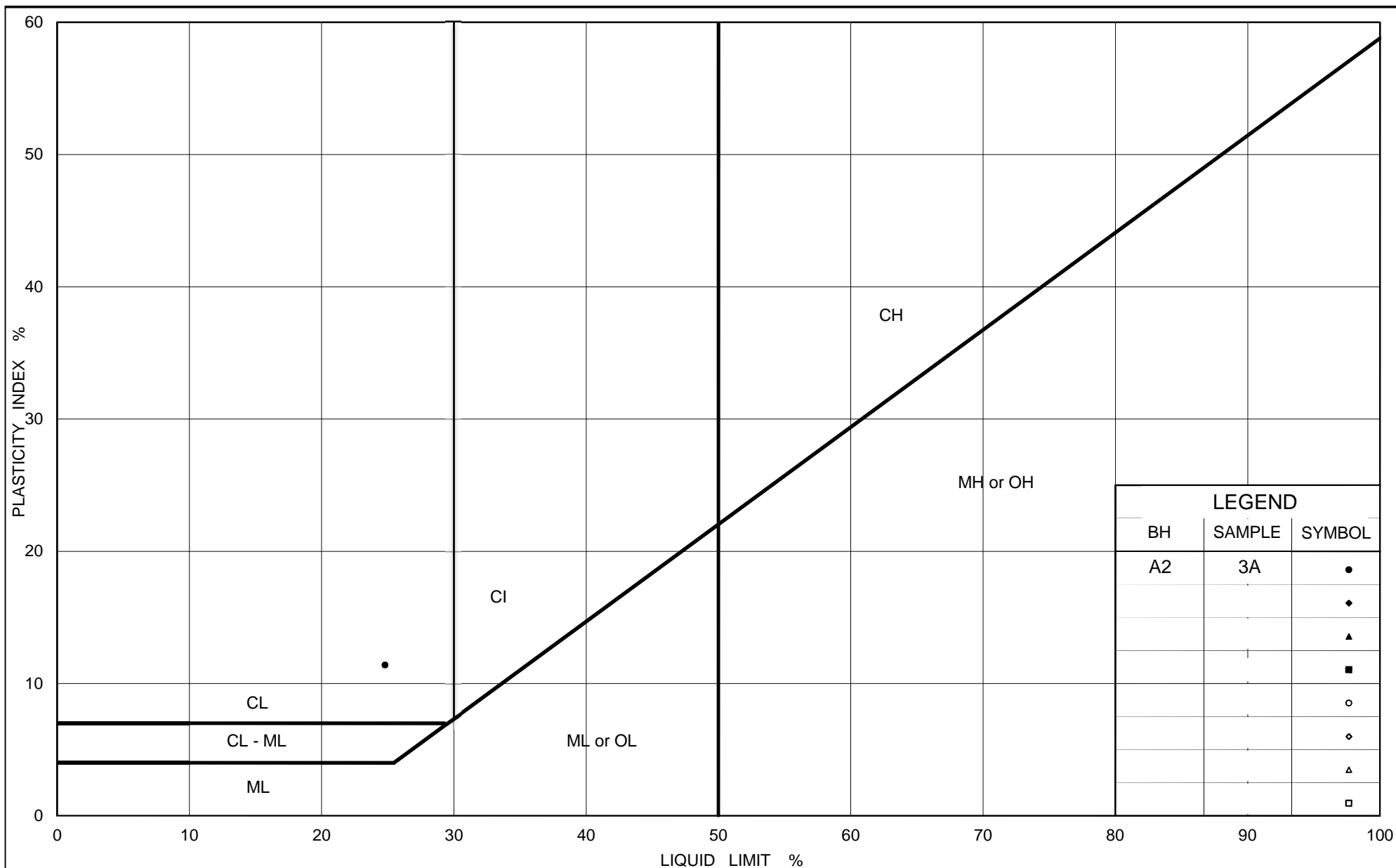
PROJECT		11-1184-0143		RECORD OF BOREHOLE No S2		SHEET 2 OF 2		METRIC							
G.W.P.		10-20011		LOCATION		N 4860328.0 ; E 356527.1		ORIGINATED BY							
DIST		Central HWY 401		BOREHOLE TYPE		Solid Stem Augers		COMPILED BY							
DATUM		Geodetic		DATE		March 11, 2015		CHECKED BY							
LCC															
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS		ELEVATION SCALE		DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	γ	GR SA SI CL		
84.7	END OF BOREHOLE		13	SS	100/0.13		85								
15.7	NOTES: 1. Water level measured in borehole at a depth of 7.6 m below ground surface (Elev. 92.8 m) upon completion of drilling, March 11, 2015 2. Borehole caved to a depth of 8.2 m below ground surface (Elev. 92.2 m) upon completion of drilling, March 11, 2015 3. Water level in piezometer measured as follows: Date Depth (m) Elev. (m) Jun 7/16 6.8 93.6														

GTA-MTO 001 T:\PROJECTS\2011\11-1184-0143 (HWY 401 FROM BROCK RD TO COURTYARD RD)\LOG11-1184-0143.GPJ GAL-GTA.GDT 9/6/16 KD



APPENDIX B

Geotechnical Laboratory Test Results



PLASTICITY CHART Clayey Silt

Figure No. B1

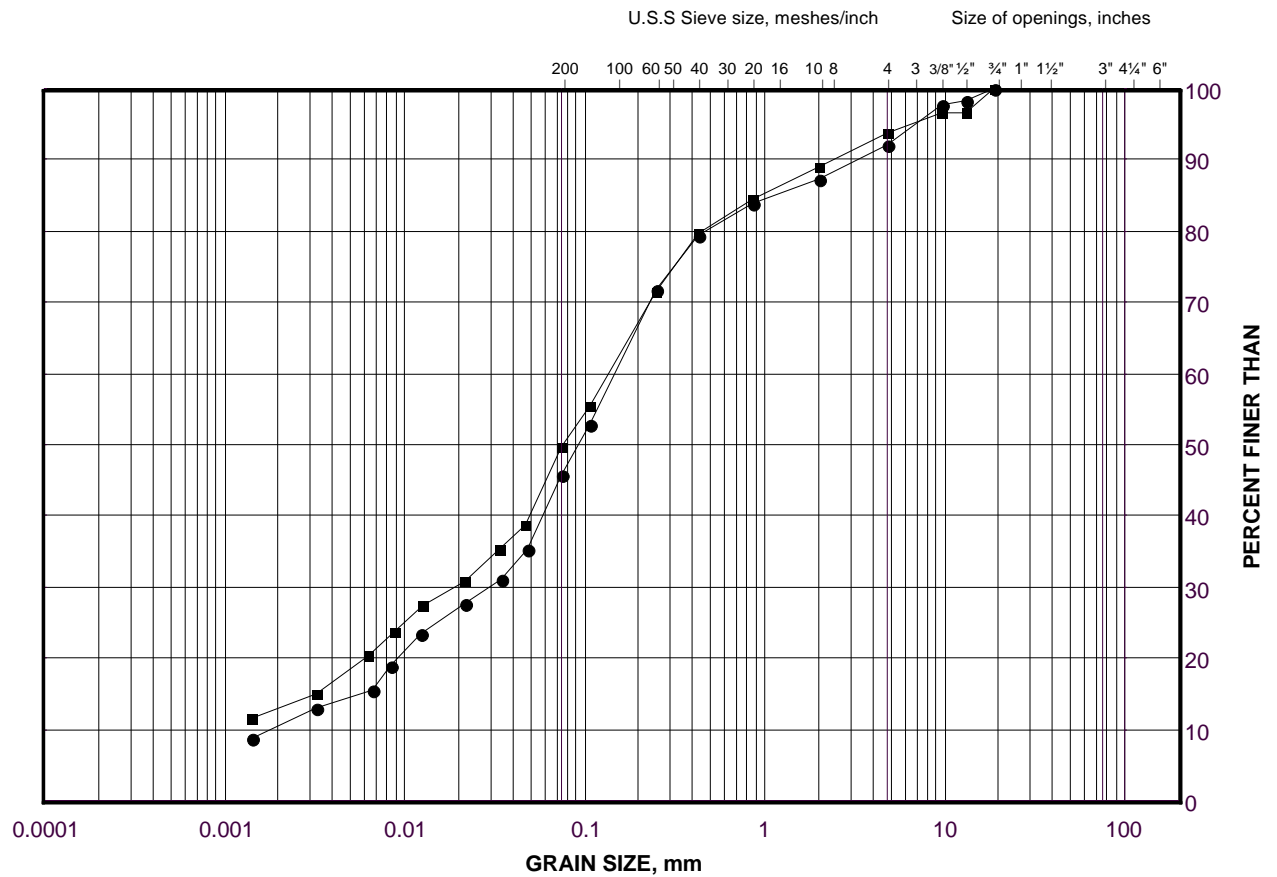
Project No. 11-1184-0143

Checked By: NK

GRAIN SIZE DISTRIBUTION

Upper Silt and Sand Till

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	A1	4	99.6
■	A2	5	98.7

Project Number: 11-1184-0143

Checked By: NK

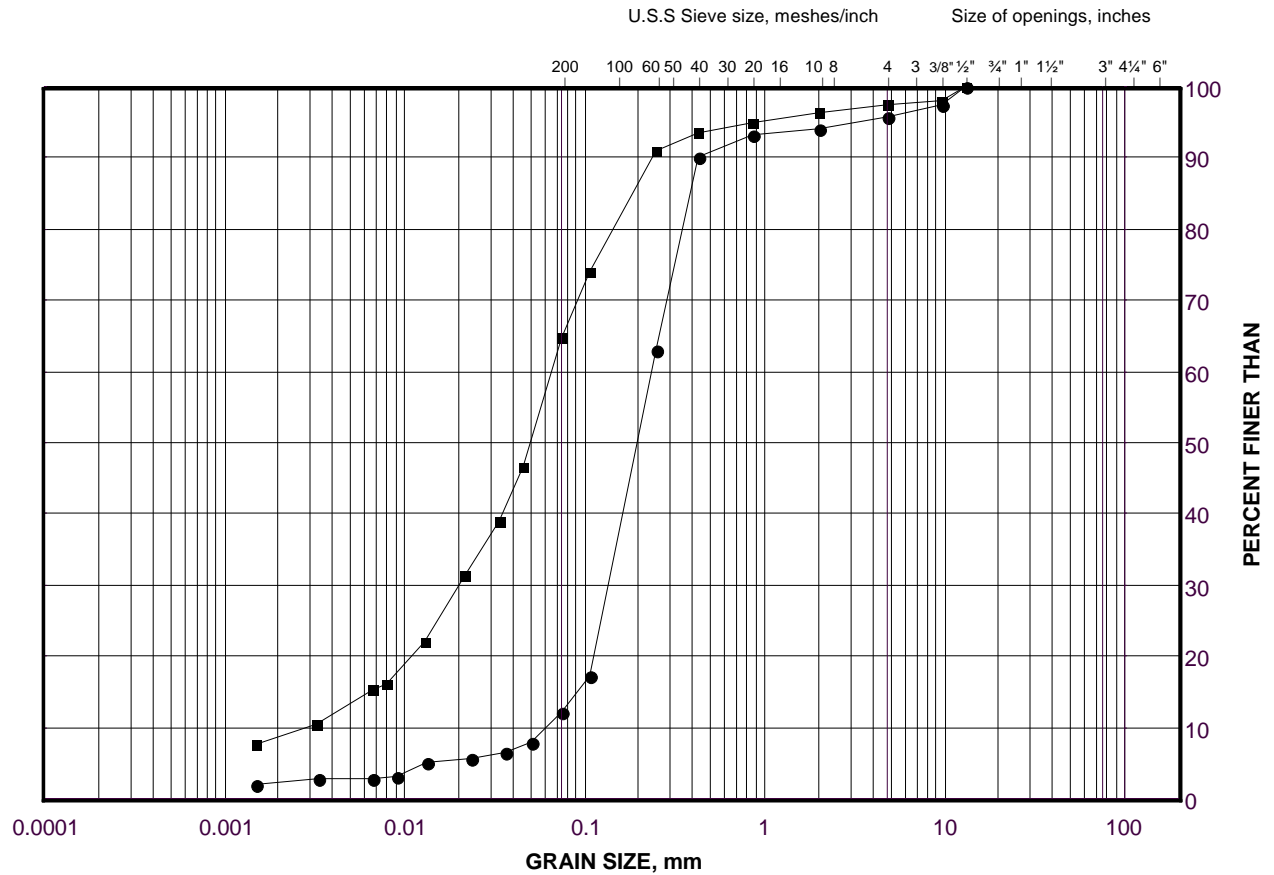
Golder Associates

Date: 14-Jun-16

GRAIN SIZE DISTRIBUTION

Silt and Sand to Sand and Gravel

FIGURE B3



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	A1	7	95.8
■	A2	8	94.2

Project Number: 11-1184-0143

Checked By: NK

Golder Associates

Date: 14-Jun-16

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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