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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Bloor Street Underpass Structure Site No. 22-181 Highway 401 Improvements from Brock Road to Courtice Road Regional Municipality of Durham W.O. 10-20011

Submitted to:

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REPORT



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**PRELIMINARY FOUNDATION REPORT
BLOOR STREET UNDERPASS**

PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
BLOOR STREET UNDERPASS
STRUCTURE SITE NO. 22-181
HIGHWAY 401 IMPROVEMENTS FROM BROCK ROAD TO
COURTICE ROAD, REGIONAL MUNICIPALITY OF DURHAM
W.O. 10-20011**



PRELIMINARY FOUNDATION REPORT BLOOR STREET UNDERPASS

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 results of the foundation investigation carried out for the proposed replacement of the Highway 401-Bloor 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 Bloor Street underpass (MTO Structure Site No. 22-181) is located approximately 600 m east of the Wilson Street overpass and approximately 5.7 km west of the Courtice Road interchange, in the City of Oshawa. The existing Bloor Street underpass is a two-span structure with span lengths of approximately 22 m between the abutments and pier.

The overall surface topography in the vicinity of the site is generally flat-lying to gently sloping, with the natural ground surface at approximately Elevation 84 m to 86 m, rising from the east to west. The existing grade of Highway 401 is similar to the natural ground surface in this area, rising from about Elevation 84 m to 86 m from east to west in the immediate vicinity of the Bloor street underpass; the pavement grade is at about Elevation 85 m under the existing structure. Bloor Street has been constructed on embankments that are up to about 6 m in height, with the Bloor Street grade at approximately Elevation 90 m to 91 m at the structure site.

3.0 INVESTIGATION PROCEDURES

The field investigation was carried out in March and April 2015, during which time two boreholes (Boreholes B1 and B2) were drilled at the locations shown on Drawing 1.

The boreholes were advanced with a CME-75 truck-mounted drill rig, supplied and operated by Strong Soil Search Inc. of Claremont, Ontario, using 150 mm diameter continuous flight solid stem augers. The boreholes extended to depths of 13.8 m and 14.2 m below ground surface. Soil samples were obtained at 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 Soil).

The groundwater conditions were observed in the open boreholes throughout the drilling operations and a standpipe piezometer was installed in one borehole (Borehole B2) to permit monitoring of the groundwater level at the site. The piezometer consists of 50 mm diameter PVC pipe, with a slotted screen sealed at a select depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was sealed and backfilled to the ground surface with bentonite pellets. The piezometer installation



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details and water level readings are indicated on the borehole record contained in Appendix A. Borehole B1 was backfilled to the ground surface with bentonite pellets in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's engineering staff who located the boreholes in the field, supervised the drilling, sampling and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and testing. Index and classification tests consisting of water contents, Atterberg limits, and grain size distributions were carried out on selected soil samples. All of the geotechnical laboratory tests were carried out to MTO and/or ASTM Standards as applicable.

The borehole locations were established in the field by Golder personnel using a hand-held GPS unit. The ground surface elevation at each borehole was estimated from the digital terrain model for the site as provided by AECOM. The borehole locations (referenced to the MTM NAD83 co-ordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

| Borehole Number | MTM NAD83 Northing (m) | MTM NAD83 Easting (m) | Ground Surface Elevation (m) | Borehole Depth (m) |
|-----------------|------------------------|-----------------------|------------------------------|--------------------|
| B1 | 4,861,073.7 | 358,718.0 | 89.6 | 13.8 |
| B2 | 4,860,928.9 | 358,606.5 | 88.1 | 14.2 |

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* (Brennand, 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 B1 and B2 were advanced in the vicinity of the proposed north and south abutments, respectively, for the replacement structure. The detailed subsurface soil and groundwater conditions encountered in the boreholes, together with 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 interpreted stratigraphic profile is shown on Drawing 1. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile drawing are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.



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In general, the subsurface conditions at the site consist of asphalt and fill underlain by a stiff to hard clayey silt to silty clay deposit, which is in turn underlain by a silty sand to silt and sand to gravelly sand till deposit. A layer of stiff to very stiff silty clay till is also present within the non-cohesive till deposit. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

Approximately 100 mm and 200 mm of asphalt was encountered immediately below the existing ground surface in Boreholes B1 and B2, respectively.

4.2.2 Fill

Approximately 3.3 m and 2.4 m of fill material was encountered beneath the asphalt in Boreholes B1 and B2, respectively. The base of the fill was encountered at about Elevation 86.2 in Borehole B1 on the north side of the highway, and at about Elevation 85.5 m in Borehole B2 on the south side of the highway.

The fill ranges in composition from sand to silty sand containing trace gravel, to sandy clayey silt or clayey silt containing some sand and trace gravel. The result of a grain size distribution test performed on a selected sample of the clayey silt portion of the fill deposit is shown on Figure B1 contained in Appendix B.

An Atterberg limits test carried out on a selected sample of the clayey silt portion of the fill measured a liquid limit of about 32 per cent, a plastic limit of about 15 per cent, and a plasticity index of about 17 per cent. This result, which is plotted on a plasticity chart on Figure B2 in Appendix B, indicates that the cohesive portion of the fill can be classified as a clayey silt of low plasticity. Measured water contents of samples of the cohesive portion of the fill ranged from about 18 to 25 per cent.

One SPT 'N'-value of 14 blows per 0.3 m of penetration was measured within the non-cohesive portion of the fill, indicating a compact relative density. The SPT 'N'-values measured within the cohesive portion of the fill range from 10 to 17 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

4.2.3 Clayey Silt to Silty Clay

A deposit of clayey silt to silty clay was encountered underlying the fill material in both of the boreholes advanced at this site. The thickness of this clayey silt to silty clay deposit ranges from approximately 2.1 m to 5.1 m, thickening and extending deeper on the north side of the highway based on the results from the two boreholes. The base of the clayey silt to silty clay deposit was encountered between Elevations 83.4 m and 81.1 m.

The results of two grain size distribution tests performed on selected samples of the clayey silt to silty clay deposit are shown on Figure B3 contained in Appendix B.

Atterberg limits testing was carried out on selected samples of this deposit and measured liquid limits between about 20 and 41 per cent, plastic limits between about 10 and 15 per cent, and plasticity indices between about 10 and 26 per cent. These results, which are plotted on a plasticity chart on Figure B4 in Appendix B, indicate that the material is classified as clayey silt to silty clay of low to intermediate plasticity. The measured water content of samples of the clayey silt to silty clay deposit range between 9 and 26 per cent, near or above the plastic limit of the material.

The SPT 'N'-values measured within the clayey silt to silty clay range from 15 blows to 33 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency.



4.2.4 Silt and Sand to Gravelly Sand Till

A deposit of silty sand to silt and sand till was encountered below the clayey silt to silty clay in both of the boreholes advanced at this site; at depth in Borehole B2, the till deposit grades to a gravelly sand. Borehole B1 was terminated within this deposit after penetrating the deposit to a thickness of 5.3 m, corresponding to Elevation 75.8 m at the bottom of the borehole. In Borehole B2 the thickness of the upper portion of this deposit (above the silty clay till layer as described in Section 4.2.5) was about 2.3 m; below the silty clay till layer, Borehole B2 terminated in the lower portion of gravelly sand till after penetrating it for a thickness of 4 m, corresponding to Elevation 73.9 m.

This deposit varies in composition from silty sand to silt and sand till containing trace to some gravel and trace to some clay, grading at depth to gravelly sand containing some silt and trace clay. The results of grain size distribution tests performed on selected samples of the till deposit are shown on Figure B5 contained in Appendix B. The measured water content of samples of the till deposit range between 8 per cent and 17 per cent.

The measured SPT 'N'-values recorded within the silty sand to silt and sand deposit range from 5 blows per 0.3 m of penetration to 100 blows per 0.08 m of penetration. However, the SPT 'N'-value of 5 blows recorded within this deposit in Borehole B2 is interpreted to result from disturbance of the soil due to water inflow during sampling activities; as such, the relative density for this deposit is considered to be compact to very dense.

4.2.5 Silty Clay Till

A layer of silty clay till was encountered within the non-cohesive till deposit in Borehole B2. The thickness of this silty clay till layer was about 3.2 m and the base of the cohesive till layer was encountered at about Elevation 77.9 m. The deposit contains some sand and trace gravel.

A measured water content of a sample of the silty clay till deposit is about 8 per cent. The SPT 'N'-values measured within the silty clay till are 12 blows and 22 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency.

4.3 Groundwater Conditions

The groundwater levels were monitored in the open boreholes upon completion of drilling operations, and a piezometer was installed in Borehole B2 to monitor the groundwater level at the site. The water level measurements on both the open boreholes and piezometer are shown on the borehole records contained in Appendix A and summarized as follows:

| Borehole No. | Ground Surface Elevation (m) | Depth to Water Level (m) | Groundwater Elevation (m) | Date |
|---------------------|-------------------------------------|---------------------------------|----------------------------------|--|
| B1 | 89.6 | 9.5 | 80.1 | March 17, 2015 (Completion of drilling) |
| B2 | 88.1 | 6.1 | 82.0 | April 12, 2015 (Completion of drilling) |
| | | 5.8 | 82.3 | June 7, 2016 (Piezometer) |



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The water levels observed in the open boreholes on completion of drilling may not represent long-term stabilized groundwater levels. The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., and was reviewed by Ms. Nikol Kochmanová, P.Eng. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

**PRELIMINARY FOUNDATION DESIGN REPORT
BLOOR STREET UNDERPASS
STRUCTURE SITE NO. 22-181
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-Bloor Street underpass (MTO Structure Site 22-183). 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 the construction or design-build contractor. 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 Bloor Street underpass will be demolished and a new structure will be built to replace it. No design or as-built drawings are available for the existing Bloor 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 length of approximately 132 m. Highway 401 will be widened from the existing six lanes to ten lanes, including an additional two eastbound lanes to accommodate the new eastbound off-ramp. Additionally, the Bloor Street grade is proposed to be raised to approximately Elevation 94 m with approach embankments that are up to approximately 8 m in height at the abutments, while the existing Highway 401 grade will be maintained.

Both shallow and deep foundations have been considered for support of the abutments and piers for the proposed Bloor Street underpass replacement structure. It should be noted that subsurface conditions below the piers are uncertain at this stage and have been assessed based on interpolation between the boreholes; therefore, these recommendations should be taken as preliminary and further investigation will be required for detail design. For all options, protection systems are expected to be required along Highway 401 and Bloor Street to facilitate removal of the existing structure, although the replacement structure alignment should minimize conflict with existing foundation elements and hence may allow for existing footings to remain in place; protection systems are also expected to be required along the Bloor Street approach embankment for construction of the north and south abutments and south pier. 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.



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- **Strip footings founded on the very stiff to hard clayey silt to silty clay deposit at the piers:** Shallow footings are feasible for support of the piers. This option will require excavation to a minimum depth of 1.2 m below the existing/proposed grade at Highway 401, to achieve the required frost protection depth. This foundation type may not be feasible at the abutments for the assessed geotechnical resistances, in part due to the loading and resulting settlement under the new/widened approximately 8 m high approach embankments. If this option is adopted, temporary protection systems are expected to be required along Highway 401 and Bloor Street to facilitate the construction of the new footings. Based on subsurface conditions, groundwater control requirements should be limited to controlling seepage from perched water within non-cohesive fill materials, on top of the relatively lower permeability clayey silt to silty clay deposit at the north pier. Footing excavations for the south pier may extend below the groundwater table in the silt and sand till deposit (to be confirmed based on additional investigation in detailed design), and greater dewatering effort may be required to minimize disturbance of the foundation subgrade.
- **Footings “perched” on a compacted granular pad within the approach embankments:** Settlement will occur in the stiff portions of the silty clay layer (particularly in the vicinity of the south abutment) under the new approximately 8 m high approach embankment loading. Additionally, this foundation type would require a longer structure span, and would preclude the use of integral abutments. Because this option is less desirable from a geotechnical perspective than either strip footings founded on native soils or driven steel piles, it is not discussed further in this report.
- **Driven steel H-piles or pipe piles founded within the very dense silt and sand to gravelly sand till deposit:** Driven steel H-piles or pipe 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). It is understood that due to the skew of Bloor Street over Highway 401, conventional abutments are preferred over integral abutments for this structure. In either case, the abutments may be constructed with a pile cap perched within the approach embankments above the Highway 401 grade, ensuring that minimum depth for frost protection is achieved, to minimize excavation and protection system requirements; the perched pile cap could be constructed with either an open configuration (for conventional abutments) or in a false abutment configuration with retained soil system walls (for integral abutments). Driven steel piles are also feasible at the pier locations, but more investigation is required to determine the pile tip depth and geotechnical resistances; the pile cap for a deep foundation solution may occupy less space than a footing for a shallow foundation option. Pile driving shoes are recommended to protect the pile tips from damage during driving into the very dense/hard, glacially derived deposits.
- **Caissons founded within the very dense silt and sand to gravelly sand till deposit:** Caissons are considered feasible for the support of the abutments, and a perched pile cap would minimize excavation and protection system requirements at the new abutments. 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 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, deep foundations are feasible for the support of piers and abutments. Typically at the abutments, pile foundations would be preferred with a perched pile cap on the approach embankments, to



PRELIMINARY FOUNDATION REPORT BLOOR STREET UNDERPASS

permit for a conventional pile supported abutment configuration and minimize excavation and groundwater control requirements. At the piers, shallow foundations are preferred from a geotechnical/foundations perspective due to the presence of a suitable bearing stratum at a relatively shallow depth (subject to borehole investigation at the piers in detailed design). 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 piers for the new replacement structure, spread/strip footings should be founded on the very stiff to hard clayey silt to silty clay deposit for the north pier, and potentially the compact to dense silt and sand till deposit at the south pier. 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 Ontario Provincial Standard Drawing (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 maintained at approximately Elevation 85 m below the underpass structure, with a swale between the eastbound lanes and off-ramp, where the south pier is to be located. As such, the maximum (highest) founding elevation recommended for the preliminary design of the footings is approximately Elevation 83.8 m at the north pier, and approximately Elevation 81.8 m at the south pier. The footing excavation for the south pier is expected to extend below the water table in the silt and sand to silty sand till deposit, and groundwater control is anticipated to be required at this location to minimize disturbance to the foundation subgrade.

Due to the variability encountered between Boreholes B1 and B2, additional boreholes should be advanced in the vicinity of the piers during detail design 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 subgrade at the preliminary 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) |
|------------------------|---|--|
| North pier | 400 | 350 |
| South pier | 350 | 275 |

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

These preliminary geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. These geotechnical resistances are given for loads will that be applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC 2006)* and its *Commentary*.



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

6.4.1 Founding Elevation

The abutments and associated wingwalls for the replacement structure may be supported on steel piles driven to found within the very dense ("100-blow") silt and sand to gravelly sand till deposit.

For preliminary design purposes, if pile-supported abutments were to be adopted for the design of the replacement structure, 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; it is noted that these tip elevations extend below the base of the boreholes advanced as part of the preliminary investigation, and additional investigation will be required during detailed design to confirm these recommendations:

| Foundation Element | Estimated Pile Cap Elevation (m) | Approximate Surface Elevation of "100-Blow" Soil (m) | Preliminary Pile Tip Elevation (m) |
|---------------------------|---|---|---|
| North Abutment | 86 | 76 | 74 |
| North Pier | 83.8 | 76 | 74 |
| South Pier | 81.8 | 74.5 | 72.5 |
| South Abutment | 85 | 74.5 | 72.5 |

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 potentially 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 and axial geotechnical resistance at SLS (for 25 mm of settlement) may be taken as follows:

| Foundation Element | Factored Geotechnical Resistance at ULS (kN) | Geotechnical Resistance at SLS (kN) |
|---------------------------|---|--|
| Abutments | 1,400 | 1,200 |
| Piers | 1,150 | 1,000 |



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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.). 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.

Pile installation should be in accordance with Ontario Provincial Standard Specification (OPSS.PROV) 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.

6.5 Caisson Foundations

6.5.1 Founding Elevation

Caissons founded within the very dense silt and sand to gravelly sand till deposit may be considered for support of the abutments and piers for the proposed replacement structure. If adopted, the following caisson founding elevations may be used for preliminary design purposes, assuming about 2 m penetration into "100-blow" soil; however, these elevations should be taken as preliminary, and further investigation will be required at the abutments and piers during detailed design:

| Foundation Element | Estimated Pile Cap Elevation (m) | Approximate Surface Elevation of "100-Blow" Soil (m) | Estimated Design Tip Elevation (m) |
|--------------------|----------------------------------|--|------------------------------------|
| North Abutment | 86 | 76 | 74 |
| North Pier | 83 | 76 | 74 |
| South Pier | 83 | 74.5 | 72.5 |
| South Abutment | 85 | 74.5 | 72.5 |

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.



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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 | 4,200 | 3,400 |
| 1.5 | 6,000 | 5,000 |

The preliminary resistances/reactions provided above will need to be re-evaluated and modified, as necessary, during detail design in consideration of 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.

6.6 Approach Embankments

Approach embankments up to approximately 8 m in height will be required along the alignment of the new Bloor Street underpass replacement structure.

6.6.1 Embankment Slope Construction

In accordance with MTO's standard practice and OPSD 208.010 (*Benching of Earth Slopes*), a minimum 2 m wide bench should be provided where the embankment slopes are equal to or greater than 8 m in height. To reduce erosion of the 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. Consideration may also be given to the use of a curb and gutter at the road edge, an interceptor drain along the crest of the embankment slope parallel to Bloor Street, and/or armoured drainage channels on the slope face to direct surface water flow from the Bloor Street grade down to original grade.

6.6.2 Global Slope Stability

Preliminary static slope analysis has been completed for the Bloor Street underpass approach embankments, using the commercially available program *Slide 6.0* from Rocscience, to check that the target minimum factor of safety is achieved. A target minimum factor of safety of 1.3 is normally used in the design of slopes under static conditions. This minimum factor of safety is considered appropriate for the proposed widened/new approach embankments on this project, considering the design requirements and the available field and laboratory data.

Preliminary stability analyses were completed for an 8 m high slope in short-term (undrained) and long-term (effective stress) conditions, using the parameters shown in the following table.



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| Deposit | Unit Weight | Undrained Shear Strength (kPa) | Effective Friction Angle (Degrees) |
|--|-------------|--------------------------------|------------------------------------|
| Embankment fill | 21 | - | 32 |
| Stiff to hard clayey silt to silty clay | 20 | 75 | 32 |
| Compact to dense silt and sand till | 21 | - | 32 |
| Stiff to very stiff silty clay till | 21 | 75 | 32 |
| Compact to very dense gravelly sand till | 21 | - | 35 |

The results of the static global stability analysis indicate that a minimum factor of safety of 1.3 is achieved for 8 m high side slopes oriented no steeper than 2H:1V, as shown on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed during detail design based on the refined geometry, including mid-height benching if utilized, and additional subsurface information as may be available.

6.6.3 Settlement under Approach Embankment Loading

Preliminary settlement assessments have been completed for the approach embankment construction using the commercially available computer program *Settle-3D* from Roscience, using the estimated elastic deformation moduli given in the table below, based on correlation with the SPT 'N'-values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

| Soil Deposit | Bulk Unit Weight (kN/m ³) | Elastic Modulus (MPa) |
|--|---------------------------------------|-----------------------|
| Embankment fill | 21 | - |
| Stiff to hard clayey silt to silty clay | 20 | 45 |
| Compact to dense silt and sand till | 21 | 45 |
| Stiff to very stiff silty clay till | 21 | 35 |
| Compact to very dense gravelly sand to silty sand till | 21 | 100 |

Based on this preliminary assessment, the settlement of the foundation soils under an 8 m high approach embankment is estimated to be approximately 40 mm to 50 mm. The majority of this settlement is associated with the stiff/compact layers of soil encountered during the preliminary investigation, and this settlement is anticipated to occur during and immediately following completion of the embankment construction.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. Where granular fill is used for embankment construction, settlement of the fill itself is expected to occur during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.



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

At the piers, the construction of new strip footings or pile caps for the replacement structure will require excavations up to about 1.2 m below the existing Highway 401 grade or the future swale grade. Some excavation will be required into the existing Bloor Street embankment side slopes, particularly in the vicinity of the north abutment, as well as for removal of the existing abutments. The existing fill and stiff portion of the clayey silt to silty clay deposit are classified as Type 3 soils, while the native very stiff to hard and 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 for shallow foundations excavations and removals within Highway 401 at the pier locations, and along the Bloor Street embankments for construction of the new north abutment and removal of the existing abutments, depending on construction staging requirements. 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

The soils encountered within the expected excavation depths at this site are generally cohesive, with relatively low permeability; however, the excavation for a footing or pile cap at the south pier may extend into the water-bearing silt and sand to silty sand till deposit. Some groundwater seepage is anticipated from water perched within granular fills on top of the cohesive deposits, from seams/interlayers of non-cohesive soil within the clayey silt to silty clay deposit, and from the silt and sand till to silty sand till deposit where encountered. Based on the available subsurface information and the potential depth of excavation, it is anticipated that the seepage at most excavation locations will be relatively minor, such that it can be handled by pumping from properly filtered sumps within the excavation. However, more significant dewatering effort may be required at the south pier to lower the groundwater level to at least 0.5 m to 1 m below the foundation subgrade level during footing construction. Further assessment of this aspect will be required as part of the detailed design, based on the results of further investigation.

6.7.3 Subgrade Protection

The clayey and 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 and protection system installation.

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 (for both shallow and deep foundations);
- the subsurface conditions at greater depth below deep foundation elements at the abutment areas, to facilitate deep foundation design;
- the presence, thickness and strength/compressibility properties of soils within the footprint of the approach embankments, to confirm subgrade preparation/stripping requirements, global stability and settlement; and
- the groundwater level and permeability of the non-cohesive till deposit for more detailed assessment of groundwater control requirements during construction.

In addition, decommissioning of the piezometer installed as part of this preliminary foundation investigation should be completed at or following the detailed design stage.



PRELIMINARY FOUNDATION REPORT BLOOR STREET UNDERPASS

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Maridalia Guerrero Pena, M.Sc., and was reviewed by Ms. Nikol Kochmanová, P.Eng. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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MGP/NK/LCC/sm

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PRELIMINARY FOUNDATION REPORT BLOOR STREET UNDERPASS

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ASTM International:

- ASTM D1586-11 Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils

Ontario Provincial Standard Specifications (OPSS)

- OPSS.PROV 539 Construction Specification for Temporary Protection Systems
OPSS.PROV 903 Construction Specification for Deep Foundations

Ontario Provincial Standard Drawings (OPSD)

- 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

Other

- Ontario Regulation 213 Construction Projects (as amended)
Ontario Regulation 903 Wells (as amended)



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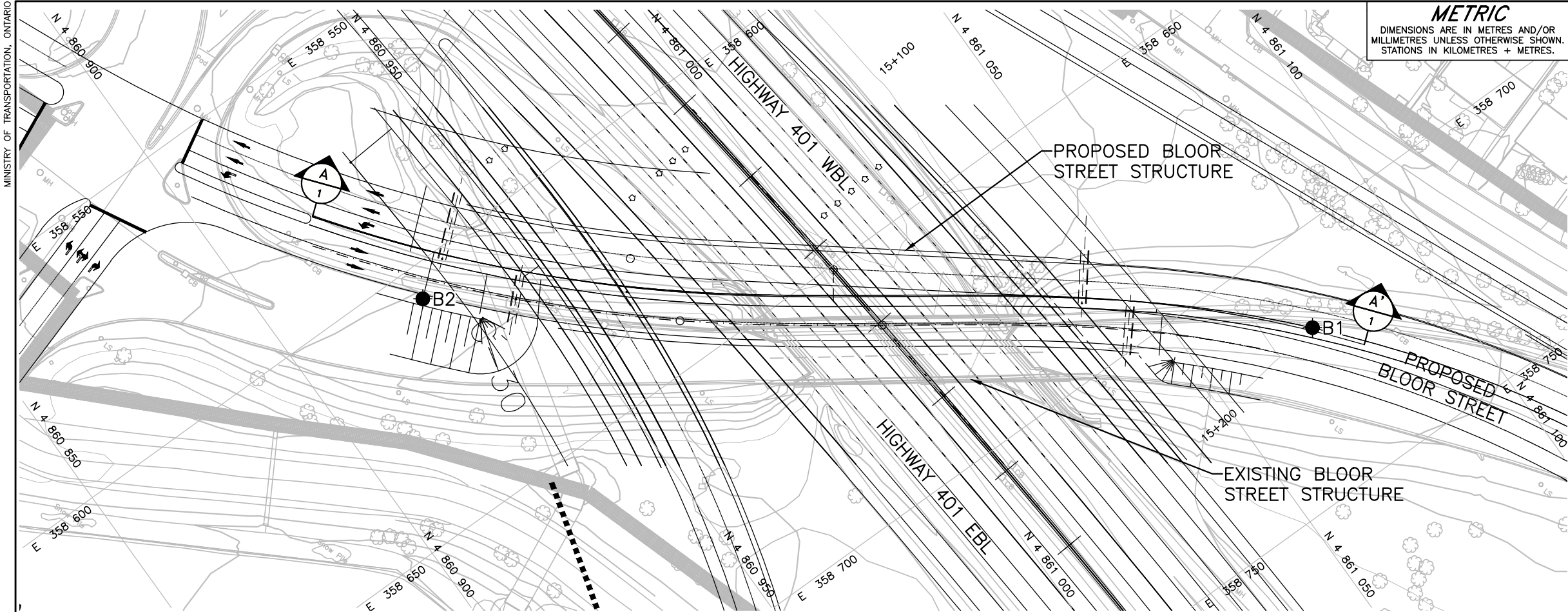
TABLE 1 – COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES

| Foundation Option | Feasibility | Advantages | Disadvantages | Estimated Costs | Risk / Consequences |
|--|--|--|---|--|---|
| Strip footings founded on native soils | <ul style="list-style-type: none"> • Feasible for support of the piers • May not be feasible for support of the abutments, based on assessed geotechnical resistances and settlements under the loading from the approach embankments behind the abutments | <ul style="list-style-type: none"> • Conventional excavation and construction techniques • Very stiff and dense soils (with SPT 'N'-values greater than 30 blows per 0.3 m of penetration) present at a reasonable depth below Highway 401 grade, with good geotechnical resistance and settlement performance • Excavation depths are reasonable and generally above groundwater level but further investigation will be necessary at detailed design • Minimal groundwater seepage and limited dewatering required at north pier | <ul style="list-style-type: none"> • Excavations at the pier location will require protection systems along Highway 401 • Potential dewatering required at south pier, pending finalization of swale grade and confirmation of subsurface conditions and groundwater level at this location | <ul style="list-style-type: none"> • Estimated cost is approximately \$600/m³ for construction of shallow foundations | <ul style="list-style-type: none"> • Some groundwater control may be required to maintain water level 0.5 m to 1 m below footing subgrade level at south pier |
| Steel H-piles or pipe piles | <ul style="list-style-type: none"> • Feasible for support for abutments and piers | <ul style="list-style-type: none"> • Conventional excavation and construction methods • Abutment pile caps could be maintained higher than spread footings, potentially reducing depth of excavation and protection system and groundwater control requirements • Excavations would be maintained above the groundwater level at the site • Steel H-piles allow for conventional pile supported | <ul style="list-style-type: none"> • Temporary excavation support still required to facilitate removal of existing abutments and widening of Highway 401 • Large working area required for pile driving within Highway 401, which may make this option less attractive • 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 • Risk of variability in pile tip elevations based on limited data from |

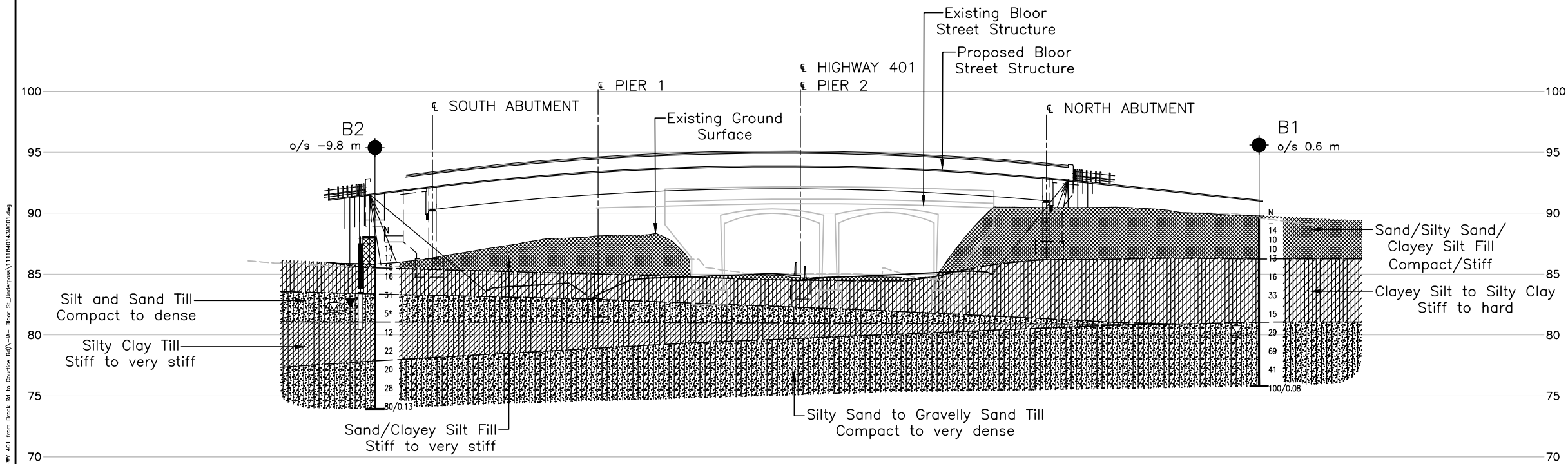
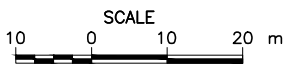


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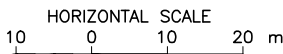
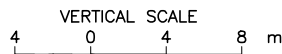
| Foundation Option | Feasibility | Advantages | Disadvantages | Estimated Costs | Risk / Consequences |
|-------------------|---|---|---|---|--|
| | | abutments as well as integral abutments, if acceptable for structure skew | | | preliminary investigation; further investigation required during detailed design |
| Caissons | <ul style="list-style-type: none"> Feasible for support of abutments and piers | <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 | <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, although this can be mitigated with the use of appropriate drilling methods and equipment Risk of variability in caisson base elevations and capacities based on limited data from preliminary investigation; further investigation required during detailed design |



PLAN



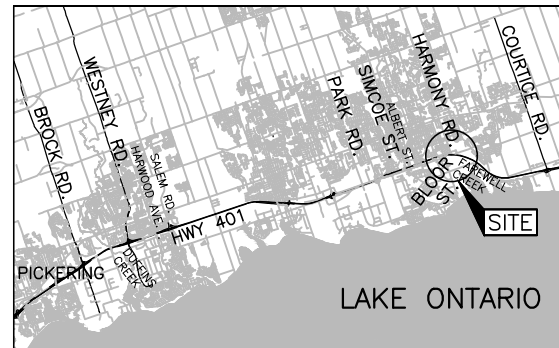
A-A' PROFILE A-A'



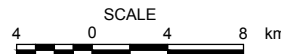
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 30M15-294
WO No. 10-20011

BLOOR STREET UNDERPASS
HIGHWAY 401 IMPROVEMENTS
BOREHOLE LOCATIONS AND
SOIL STRATA



KEY PLAN



LEGEND

- Borehole - Current Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- ≡ WL upon completion of drilling
- ≡ WL in piezometer, measured on June 7, 2016
- * SPT "N" value affected by disturbance during sampling

BOREHOLE CO-ORDINATES

| No. | ELEVATION | NORTHING | EASTING |
|-----|-----------|-----------|----------|
| B1 | 89.6 | 4861073.7 | 358718.0 |
| B2 | 88.1 | 4860928.9 | 358606.5 |

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 AECOM, 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. Design general arrangement plan obtained from drawing file ACAD-x-design.dwg received April 28, 2014. Design general arrangement profile obtained from drawing file 01_GA_BloorSt_Underpass.dwg received June 6, 2015.

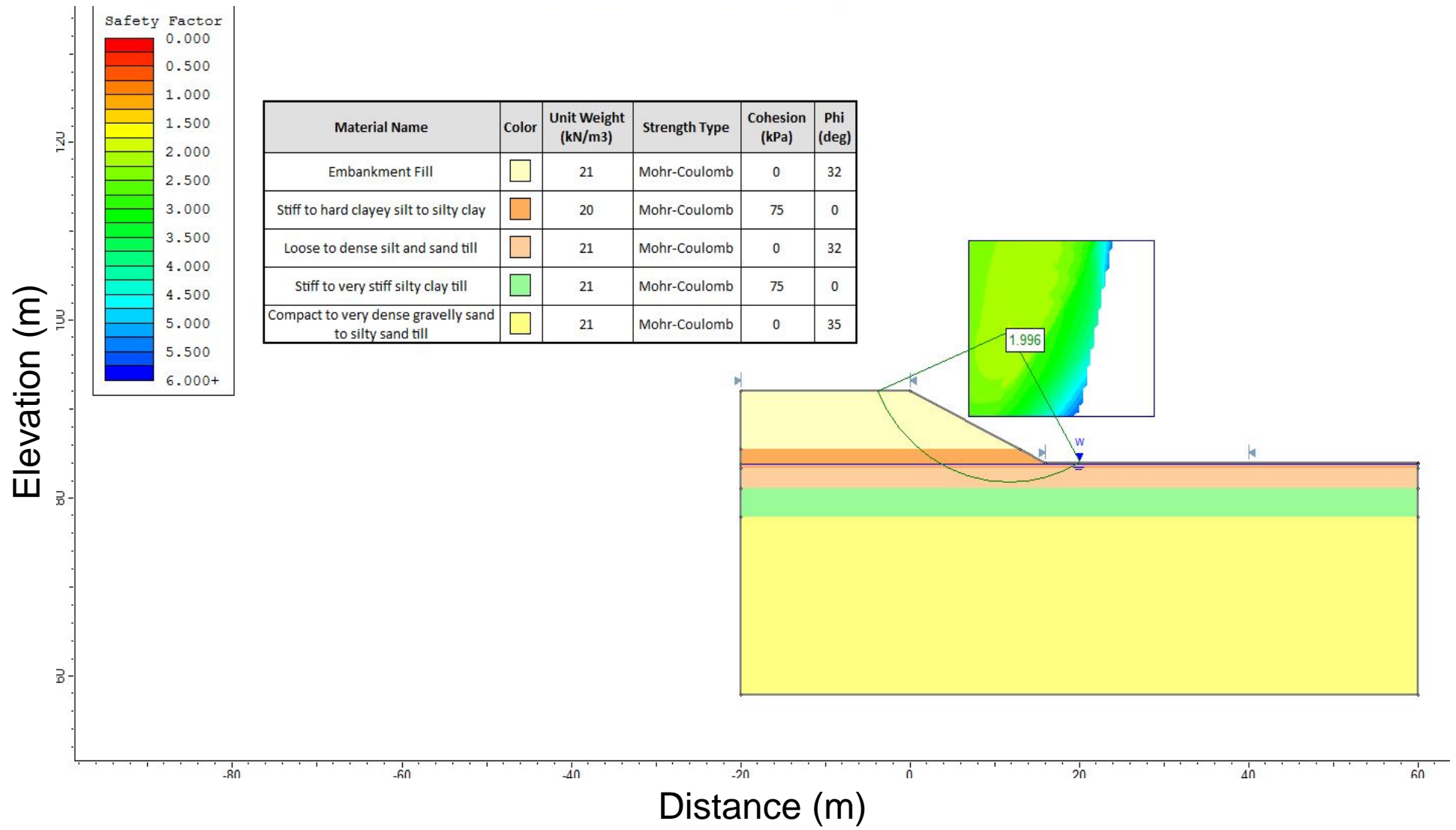


| NO. | DATE | BY | REVISION |
|-----------------------|-----------|--------------------------|---------------|
| 1 | 4/5/2017 | JFC/MR | 1 |
| Geocres No. 30M15-294 | | | |
| HWY. 401 | | PROJECT NO. 11-1184-0143 | DIST. CENTRAL |
| SUBM'D. MWK | CHKD. MGP | DATE: 4/5/2017 | SITE: 22-181 |
| DRAWN: JFC/MR | CHKD. LCC | APPD. NK | DWG. 1 |



STATIC GLOBAL STABILITY BLOOR STREET UNDERPASS – APPROACH EMBANKMENT

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 |

GTA-MTO 001 T:\PROJECTS\2011\11-1184-0143 (HWY 401 FROM BROCK RD TO COURTYERD)\LOG\11-1184-0143.GPJ GAL-GTA.GDT 10/12/16 KD

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

| PROJECT <u>11-1184-0143</u> | | RECORD OF BOREHOLE No B1 | | | | SHEET 2 OF 2 | | METRIC | | | | | | | | | | | | |
|---|-------------|--|--------|------|----------------------------|----------------------------|--|--------------------|--|--|--|---|-------------------------------------|-----------------------------------|---|--|--|--|--|--|
| W.O. <u>10-20011</u> | | LOCATION <u>N 4861073.7 ; E 358718.0</u> | | | | ORIGINATED BY <u>TD</u> | | | | | | | | | | | | | | |
| DIST <u>Central</u> HWY <u>401</u> | | BOREHOLE TYPE <u>150 mm O.D. Continuous Flight Solid Stem Augers</u> | | | | COMPILED BY <u>PKS/MGP</u> | | | | | | | | | | | | | | |
| DATUM <u>Geodetic</u> | | DATE <u>March 17, 2015</u> | | | | CHECKED BY <u>LCC</u> | | | | | | | | | | | | | | |
| 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 kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | | | | | | | | | |
| --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | <div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div> | | | | | <div style="display: flex; justify-content: space-between;"> 10 20 30 </div> | | | | | | | | |
| NOTES: 1. Borehole caved to a depth of 13.4 m below ground surface (Elev. 76.2 m) upon completion of drilling, March 17, 2015 2. Water level measured in borehole at a depth of 9.5 m below ground surface (Elev. 80.1 m) upon completion of drilling, March 17, 2015 | | | | | | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2011\11-1184-0143 (HWY 401 FROM BROCK RD TO COURTYCE RD)\LOG\11-1184-0143.GPJ GAL-GTA.GDT 10/12/16 KD

| PROJECT | | 11-1184-0143 | | RECORD OF BOREHOLE No B2 | | SHEET 1 OF 2 | | METRIC | | | | | | | | | | | | | | | |
|--------------|-------|---|------------|---------------------------------|------|---|-------------------------|-----------------|---|------------------------------|----------------|-------------|----------------|-------------------|---------------------------------------|--|---|----|----|----|----|--|--|
| W.O. | | 10-20011 | | LOCATION | | N 4860928.9 ; E 358606.5 | | ORIGINATED BY | | | | | | | | | | | | | | | |
| DIST | | Central HWY 401 | | BOREHOLE TYPE | | 150 mm O.D. Continuous Flight Solid Stem Augers | | COMPILED BY | | | | | | | | | | | | | | | |
| DATUM | | Geodetic | | DATE | | April 12, 2015 | | CHECKED BY | | | | | | | | | | | | | | | |
| | | | | | | | | LCC | | | | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | 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 | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | γ | GR | SA | SI | CL | | |
| | | | | | | | | 20 40 60 80 100 | ○ UNCONFINED + FIELD VANE | ● QUICK TRIAXIAL × REMOULDED | W _p | W | W _L | 10 20 30 | | | | | | | | | |
| 88.1 | 0.0 | GROUND SURFACE | | | | | | | | | | | | | | | | | | | | | |
| 87.5 | 0.2 | ASPHALT | | | | | | | | | | | | | | | | | | | | | |
| 87.5 | 0.6 | Sand, trace gravel, trace silt (FILL) Brown Moist | | 1 | AS | - | | | | | | | | | | | | | | | | | |
| | | Clayey silt, some sand, trace gravel (FILL) Stiff to very stiff Brown to grey Moist | | 2 | SS | 14 | | | | | | | | | | | | | | | | | |
| | | | | 3 | SS | 17 | | | | | | | | | | | | | | | | | |
| | | | | 4A | SS | 18 | | | | | | | | | | | | | | | | | |
| | | | | 4B | SS | 18 | | | | | | | | | | | | | | | | | |
| 85.5 | 2.6 | Sandy CLAYEY SILT, trace to some gravel Very stiff to hard Grey Moist | | 5 | SS | 16 | | | | | | | | | | | | | | | | | |
| | | | | 6A | SS | 31 | | | | | | | | | | | | | | | | | |
| | | | | 6B | SS | 31 | | | | | | | | | | | | | | | | | |
| 83.4 | 4.7 | SILT and SAND, trace gravel, trace clay (TILL) Compact to dense Grey Moist to wet | | 7 | SS | 5* | | | | | | | | | | | | | | | | | |
| | | | | 8 | SS | 12 | | | | | | | | | | | | | | | | | |
| | | | | 9 | SS | 22 | | | | | | | | | | | | | | | | | |
| 81.1 | 7.0 | SILTY CLAY, some sand, trace gravel (TILL) Stiff to very stiff Grey Moist | | 10 | SS | 20 | | | | | | | | | | | | | | | | | |
| | | | | 11 | SS | 28 | | | | | | | | | | | | | | | | | |
| 77.9 | 10.2 | Gravelly SAND, some silt, trace clay (TILL) Compact, to very dense Grey Moist | | 12 | SS | 80/0.13 | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | | | | |
| 73.9 | 14.2 | END OF BOREHOLE | | | | | | | | | | | | | | | | | | | | | |

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2011\11-1184-0143 (HWY 401 FROM BROCK RD TO COURTYCE RD)\LOG\11-1184-0143.GPJ GAL-GTA.GDT 10/12/16 KD

| PROJECT 11-1184-0143 | | RECORD OF BOREHOLE No B2 | | | | SHEET 2 OF 2 | | METRIC | | | | | | | | |
|--------------------------------------|--|---|--------|------|----------------------------|--------------------|---|--------------------|--|--|--|---|---------------------------------|--|--|--|
| W.O. 10-20011 | | LOCATION N 4860928.9 ; E 358606.5 | | | | ORIGINATED BY TD | | | | | | | | | | |
| DIST Central HWY 401 | | BOREHOLE TYPE 150 mm O.D. Continuous Flight Solid Stem Augers | | | | COMPILED BY BM/MGP | | | | | | | | | | |
| DATUM Geodetic | | DATE April 12, 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 γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | | W _p W W _L | | | |
| --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 | | | | | 10 20 30 WATER CONTENT (%) | | | | |
| | NOTES: * SPT 'N' value interpreted to be affected by disturbance due to groundwater inflow during sampling. 1. Borehole caved to a depth of 6.1 m below ground surface (Elev. 82.0 m) upon completion of drilling, April 12, 2015. 2. Water level in borehole measured at a depth of 6.1 m below ground surface (Elev. 82.0 m) upon completion of drilling, April 12, 2015. 3. Water level in piezometer measured at a depth of 5.8 m below ground surface (Elev. 82.3 m) on June 7, 2016. | | | | | | | | | | | | | | | |

GTA-MTO 001 T:\PROJECTS\2011\11-1184-0143 (HWY 401 FROM BROCK RD TO COURTYARD RD)\LOG11-1184-0143.GPJ GAL-GTA.GDT 10/12/16 KD



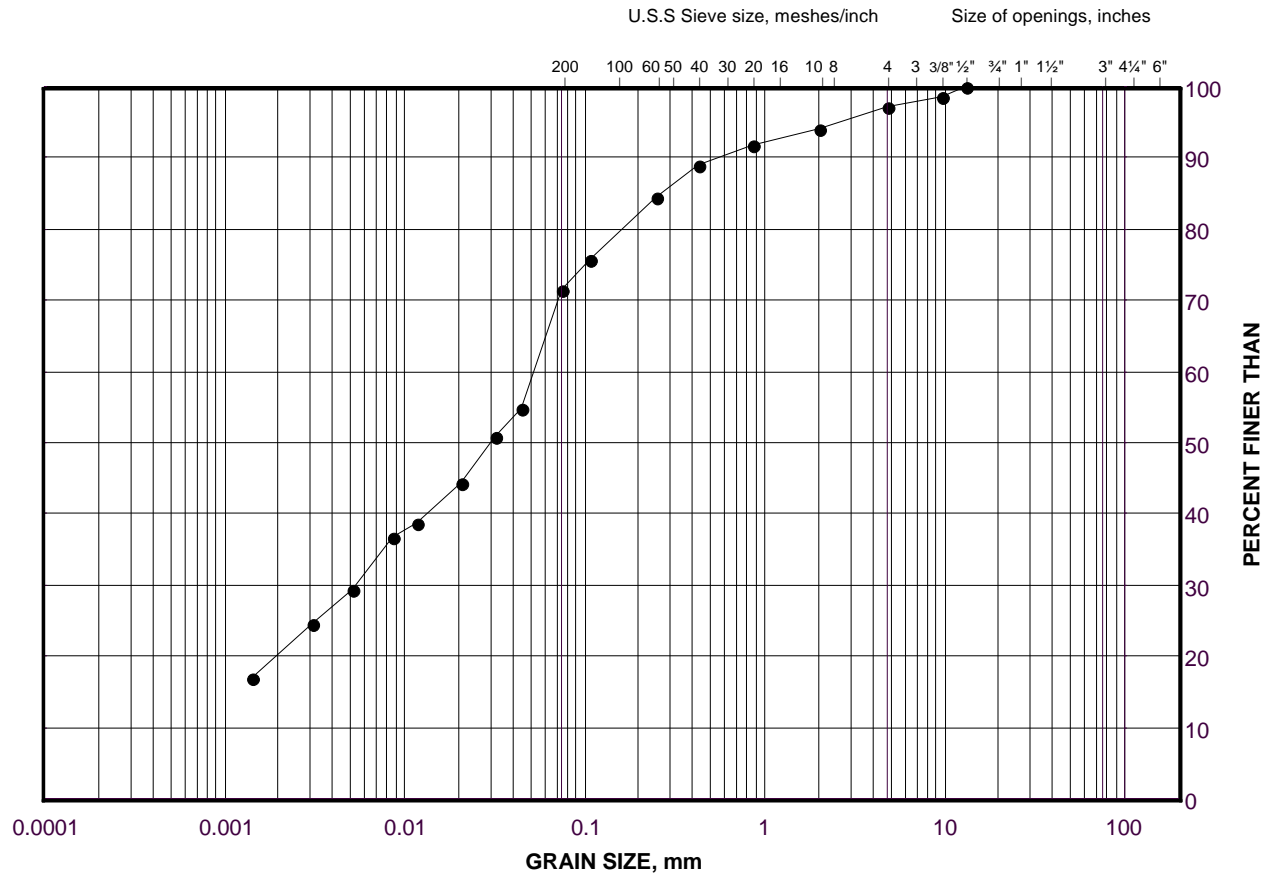
APPENDIX B

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS

Clayey Silt Fill

FIGURE B1



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

LEGEND

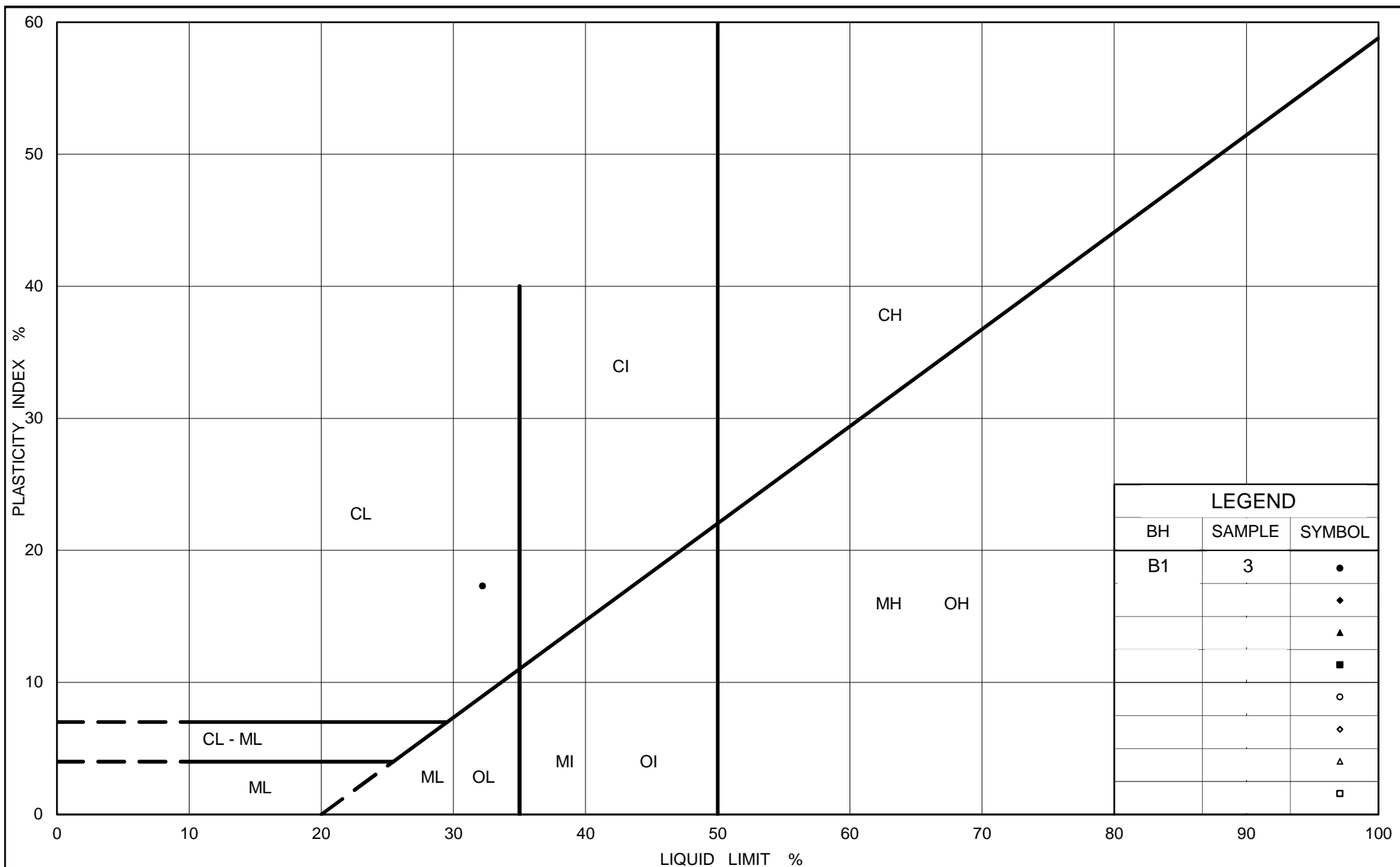
| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| • | B1 | 4 | 87.1 |

Project Number: 11-1184-0143

Checked By: NK

Golder Associates

Date: 23-Jul-15



PLASTICITY CHART

Clayey Silt Fill

Figure No. B2

Project No. 11-1184-0143

Checked By: MWK

Clayey Silt to Silty Clay

U.S.S Sieve size, meshes/inch

Size of openings, inches

PERCENT FINER THAN

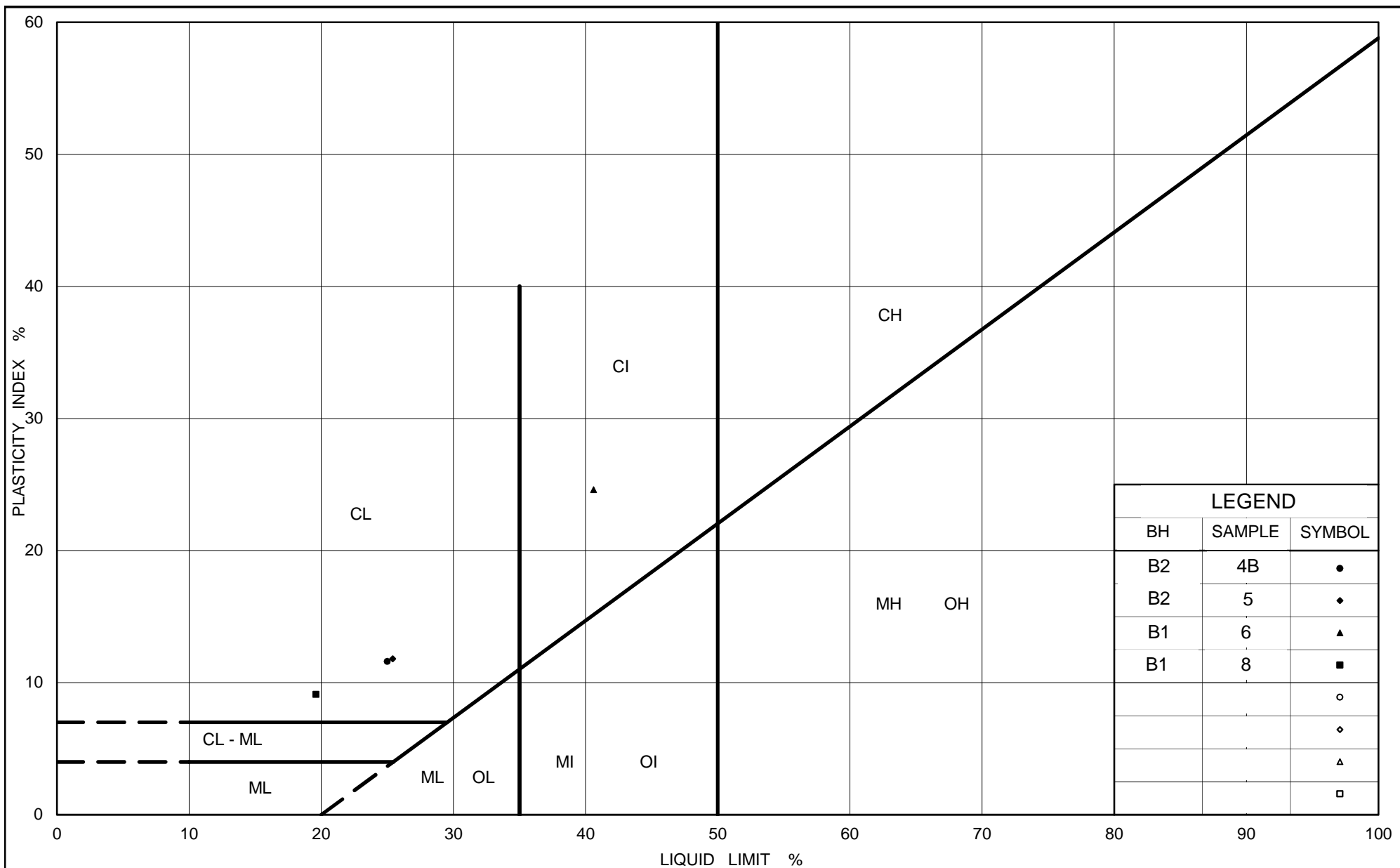
GRAIN SIZE, mm

| Grain Size (mm) | Percent Finer Than (%) - Square Markers | Percent Finer Than (%) - Circular Markers |
|-----------------|---|---|
| 0.0075 | 25 | 15 |
| 0.015 | 45 | 20 |
| 0.03 | 60 | 25 |
| 0.06 | 75 | 30 |
| 0.12 | 85 | 35 |
| 0.25 | 95 | 40 |
| 0.5 | 100 | 50 |
| 1.0 | 100 | 60 |
| 2.0 | 100 | 70 |
| 4.0 | 100 | 80 |
| 8.0 | 100 | 85 |
| 16.0 | 100 | 90 |
| 32.0 | 100 | 92 |
| 64.0 | 100 | 95 |
| 128.0 | 100 | 98 |
| 256.0 | 100 | 100 |

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

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| ● | B2 | 5 | 84.8 |
| ■ | B1 | 6 | 84.7 |

Date: 23-Jul-15



PLASTICITY CHART Clayey Silt to Silty Clay

Figure No. B4

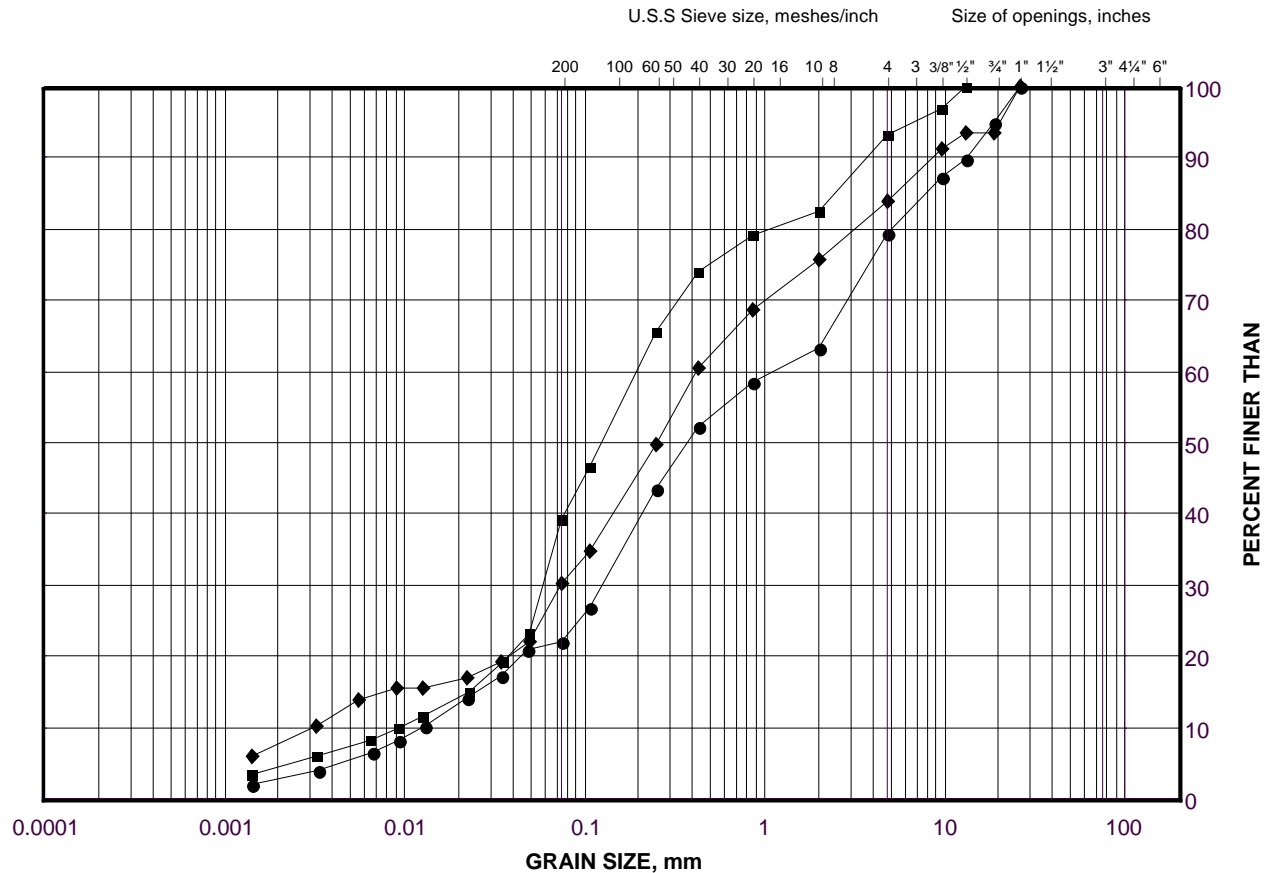
Project No. 11-1184-0143

Checked By: MWK

GRAIN SIZE DISTRIBUTION RESULTS

Silt and Sand to Gravelly Sand Till

FIGURE 5



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| ● | B2 | 10 | 77.2 |
| ■ | B2 | 7 | 81.7 |
| ◆ | B1 | 9 | 80.2 |

Project Number: 11-1184-0143

Checked By: NK

Golder Associates

Date: 12-Oct-16

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