



June 2011

## FOUNDATION INVESTIGATION AND DESIGN REPORT

### Keele Street Underpass Highway 401 Eastbound Collector Rehabilitation from Jane Street to Avenue Road Toronto, Ontario G.W.P. 2368-09-00

**Submitted to:**  
URS Canada Inc.  
75 Commerce Valley Drive East  
Markham, Ontario  
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REPORT

GEOCREs No. 30M11-237

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**Distribution:**

- 1 Copy – MTO Foundations Section
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**FOUNDATION REPORT - KEELE STREET UNDERPASS,  
HIGHWAY 401 EBC REHABILITATION**

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# **PART A**

**FOUNDATION INVESTIGATION REPORT  
KEELE STREET UNDERPASS  
HIGHWAY 401 EBC REHABILITATION FROM  
JANE STREET TO AVENUE ROAD  
TORONTO, ONTARIO  
G.W.P. 2368-09-00**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. on behalf of Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the rehabilitation of the Highway 401 eastbound collector lanes (EBC) between Jane Street and Avenue Road in Toronto, Ontario. Foundation engineering services are required under two phases:

- Phase 1: Foundation Engineering Assessment of existing foundations at six (6) structure locations, namely Keele Street Underpass; Dufferin Street Overpass; Ramp 401W – Dufferin Street N/S over Bridgeland Avenue; Ramp 401W – Yorkdale & Dufferin NB over Dufferin Street; Bathurst Street Overpass; and Avenue Road Underpass. This phase of the work has been completed and the results have been reported in a Technical Memorandum, dated July 21, 2010.
- Phase 2: Foundation Investigation, Design and Analyses under two components as follows:
  - Foundation Investigation and Design for:
    - Overhead Sign support structures; and
    - Temporary bridge deck support structure for Rapid Bridge Replacement (RBR) of Ramp 401W-Dufferin Street N/S bridge over Bridgeland Avenue; and
  - Foundation Investigation and/or Analysis and Design at six bridge structures: Keele Street Underpass; Dufferin Street Overpass; Ramp 401W – Dufferin Street N/S over Bridgeland Avenue; Ramp 401W – Yorkdale & Dufferin NB over Dufferin Street; Bathurst Street Overpass; and Avenue Road Underpass.

This report addresses the widening and rehabilitation of the Keele Street Underpass associated with the Phase 2 Foundation Investigation.

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal (RFP) for Agreement No. 2009-E-0011, issued on December 16, 2009 and MTO's Addendum No. 1, dated June 14, 2010. The scope of work for the foundation engineering services is presented in Golder's scope change letter, dated September 16, 2010.

### 2.0 SITE DESCRIPTION

The Keele Street Underpass is a 3-span steel-girder structure with span lengths of 40.9 m, 42.0 m, and 40.3 m crossing Highway 401 in Toronto, Ontario. The Highway 401 grade is at about Elevation 172.5 m under the bridge structure and the Keele Street grade rises from about Elevation 178.9 m at the south approach to about Elevation 180.1 m at the north approach.

Based on the available drawings, the abutments and piers are supported on spread footings. Highway 401 is constructed in a cut at the Keele Street Underpass location and the Keele Street approach embankments consist of the existing native ground forming the topographic high ground in the area.



### **3.0 INVESTIGATION PROCEDURES**

The field work for this subsurface investigation was carried out in September and October 2010, at which time four boreholes (Boreholes 2010-1 to 2010-4) were advanced using a Diedrich D-50 turbo, a D-90 track-mounted and a D-120 track-mounted drill rigs, supplied and operated by Walker Drilling of Utopia, Ontario. Boreholes 2010-1 to 2010-4 were advanced at the locations shown on Drawing 1.

Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. A standpipe piezometer was installed in Boreholes 2010-1 and 2010-4 to monitor the groundwater level at the site. The piezometers consist of a 1.5 m long slotted screen installed within a filter sand pack, above which the borehole annulus is backfilled to ground surface with bentonite pellets; the details of the piezometer installation are shown on the Record of Borehole 2010-01 and 2010-04. The remaining boreholes were backfilled immediately below ground surface with bentonite pellets upon completion, in accordance with Ontario Regulation 903 (as amended by Ontario Regulation 372). The upper approximately 0.4 m was backfilled with cold patch asphalt in Boreholes 2010-2 and 2010-3.

The field work was monitored on a full-time basis by a member of Golder's technical staff who located the boreholes in the field, directed 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 laboratory testing. Index and classification tests (water contents, Atterberg limits and grain size distributions) were carried out on selected soil samples. All geotechnical laboratory testing was completed to ASTM and MTO LS standards, as applicable.

The borehole locations were measured on-site relative to the existing bridge and site features and the ground surface elevations were obtained from the Digital Terrain Model for the site, provided by URS. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized below and are shown on Drawing 1. Also shown on Drawing 1 are the locations of two boreholes advanced as part of the previous investigation at the site (Geocres No 30M11-084).

<b>Borehole No.</b>	<b>MTM NAD83 Northing</b>	<b>MTM NAD83 Easting</b>	<b>Ground Surface Elevation</b>	<b>Borehole Depth</b>
2010-1	4,842,588.7	306,315.7	179.5 m	43.1 m
2010-2	4,842,539.0	306,289.7	172.2 m	33.4 m
2010-3	4,842,502.5	306,327.7	172.5 m	28.3 m
2010-4	4,842,462.1	306,338.6	178.0 m	38.4 m
1*	4,842,588.8	306,281.9	178.8 m	15.7 m
2*	4,842,459.7	306,354.5	177.8 m	15.7 m

\* Borehole locations obtained from the Digital Terrain Model as plotted relative to centerline of Highway 401 stations and off-sets.





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

### **4.1 Regional Geology**

This section of Highway 401 is located within the physiographic region known as the Peel Plain, according to *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)<sup>1</sup>.

A surficial till sheet, which generally follows the surface topography, is generally present throughout much of this area. The till is typically comprised of clayey silt to silty clay, with occasional sand to silt zones; it is mapped in this area as the Halton Till. Shallow, localized deposits of loose sand and silt and/or soft clay can overlie this uppermost till sheet, and these represent relatively recent deposits, formed in small glacial melt water ponds scattered throughout the Peel Plain and concentrated near river valleys, such as the West Don River valley. The recent sand, silt and clay and uppermost till deposits in this area overlie and are interbedded with stratified deposits of sand, silt and clay.

### **4.2 Subsurface Conditions**

As part of the subsurface investigation, four boreholes (Boreholes 2010-1 to 2010-4) were advanced at the Highway 401-Keele Street Underpass bridge structure. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A; the results of geotechnical laboratory testing are also presented on Figures B1 to B5 contained in Appendix B. The Record of Borehole No. 1 and No. 2 from the previous (MTO) investigation are presented in Appendix C.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profile on Drawing 1 are inferred from 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.

In general, the soils encountered at the site consist of a relatively thin layer of fill overlying a deposit of stiff to hard clayey silt till, which is underlain by a deposit of silt and sand to sandy silt to silt. In Boreholes 2010-1 and 2010-3, a deposit of clayey silt to silty clay was encountered underlying the cohesionless deposit.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

#### **4.2.1 Asphalt and Topsoil**

Approximately 200 mm to 300 mm of topsoil was encountered immediately below the existing ground surface in Boreholes 2010-1 and 2010-4, which were advanced on the embankments adjacent to the abutments of the existing structure.

<sup>1</sup> Chapman, L.J. and Putman, D.F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Society, Special Volume 2, Third Edition. Accompanied by Map p. 2715, Scale 1:600,000.





Approximately 0.4 m of asphalt associated with the existing road structure was encountered immediately below the existing ground surface in Boreholes 2010-2 and 2010-3, which were advanced from the Highway 401 grade at the pier locations.

### 4.2.2 Fill

Fill materials were encountered below the topsoil and asphalt in all boreholes. The fill layer is between about 0.2 m and 2.1 m thick, with the base of the fill encountered between Elevations 171.1 m and 177.2 m depending on the location of the boreholes relative to the Highway 401 grade or top of the embankments at the abutments.

The fill consists of clayey silt containing trace to some sand and gravel where encountered beneath the topsoil, and silty sand to sand containing trace gravel where encountered beneath the asphalt. The fill also contains organic materials. Atterberg limits testing carried out on one recovered sample of cohesive fill measured a plastic limit of 13 percent, a liquid limit of 24 percent, and a corresponding plasticity index of 11 percent. These results, which are plotted on a plasticity chart on Figure B1 in Appendix B, indicate that the deposit consists of clayey silt of low plasticity. The natural water content measured on selected samples of the fill ranges from 6 to 13 percent.

The Standard Penetration Test (SPT) "N" values measured within the fill range from 13 to 17 blows per 0.3 m of penetration, indicating a compact relative density within the cohesionless fill, and suggesting a stiff to very stiff consistency within the cohesive fill.

### 4.2.3 Clayey Silt Till

A deposit of clayey silt till was encountered directly below the fill in all boreholes at a depth between 0.6 m and 2.3 m below ground / pavement surface. The surface of the deposit ranges between Elevations 171.1 m and 177.2 m and the thickness of this deposit ranges from 23.3 m to 32.9 m.

The till deposit consists of clayey silt, with a plasticity of silty clay in places, and contains varying quantity of sand, and trace to some gravel. Silty sand to sand seams were also encountered within the deposit at various depths. The results of grain size distribution tests carried out on eight (8) selected samples of the till are provided on Figure B2 in Appendix B. Atterberg limits testing was carried out on fourteen (14) selected samples of this deposit and measured plastic limits varying from 10 to 18 percent, liquid limits varying from 17 to 37 percent, and plasticity indices varying from 6 to 19 percent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, indicate that the till deposit generally consists of clayey silt of low plasticity. The natural water content measured on selected samples of the clayey silt till ranges from 8 to 23 percent.

The measured SPT "N" values within the clayey silt to silty clay till range from 9 to 135 blows per 0.3 m of penetration, suggesting a stiff to hard consistency.

### 4.2.4 Sand and Silt to Sandy Silt to Silt

A deposit of sand and silt to sandy silt to silt was encountered underlying the clayey silt till deposit in all boreholes. The surface of the deposit was encountered at depths between 23.9 m and 34.4 m below ground



surface, corresponding to between Elevations 148.6 m and 143.6 m. Boreholes 2010-2 and 2010-4 terminated within this deposit, penetrating into it for a thickness of 3.4 m and 4.0 m, respectively. The deposit is 10.7 m and 3.1 m thick in Boreholes 2010-1 and 2010-3, respectively, where it was fully penetrated.

The sand and silt to sandy silt to silt deposit contains trace to some clay and trace gravel. The results of grain size distribution tests carried out on six (6) samples of the cohesionless deposit are provided on Figure B4 in Appendix B. The natural water content measured on the recovered samples of the sand and silt deposit range from 12 to 26 percent.

The measured SPT “N” values range from 36 blows to 140 blows per 0.3 m of penetration and as high as 100 blows per 0.1 m of penetration, indicating that the cohesionless deposit has a dense to very dense relative density.

A dynamic cone penetration test (DCPT) was driven from the bottom of Borehole 2010-2 from 27.8 m to 33.4 m below ground surface and terminated on a stratum exhibiting 200 blows per 0.2 m of penetration, at Elevation 138.8 m.

### 4.2.5 Clayey Silt to Clay

A clayey silt to clay deposit was encountered underlying the sand and silt to sandy silt to silt deposit in Boreholes 2010-1 and 2010-3, at a depth of 42.7 m (Elevation 136.8 m) and 27.0 m (Elevation 145.5 m) below ground surface, respectively. Both boreholes terminated within this deposit, penetrating it for a thickness of 0.4 m and 1.0 m in Boreholes 2010-1 and 2010-3, respectively.

This deposit consists of clayey silt containing some sand and occasional silty sand seams as encountered in Borehole 2010-1, and clay containing some gravel, trace sand and silt pockets as encountered in Borehole 2010-3. Atterberg limits testing was carried out on two selected samples of this deposit and measured plastic limits of 17 and 20 percent, liquid limits of 28 and 51 percent, and corresponding plasticity indices of 11 and 31 percent. These results, which are plotted on a plasticity chart on Figure B5 in Appendix B, indicate that the deposit in Borehole 2010-1 consists of clayey silt of low plasticity, and in Borehole 2010-3 it consists of clay of high plasticity. The natural water content measured on selected samples of the clayey silt and clay are 23 and 22 percent, respectively.

The measured SPT “N” values within the clayey silt and clay strata are 112 and 46 blows per 0.3 m of penetration, suggesting a hard consistency.

A dynamic cone penetration test (DCPT) was driven from the bottom of boreholes 2010-3 from 28.0 m to 28.3 m below ground surface, and terminated on a stratum exhibiting 300 blows per 0.23 m of penetration, at Elevation 144.2 m.

### 4.2.6 Groundwater Conditions

Details of the water levels observed in the open boreholes at the time of drilling are summarized on the Record of Borehole sheets following the text of this report. Two standpipe piezometers were installed in Boreholes



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2010-1 and 2010-4 to monitor the groundwater level(s) at the site. The water levels measured within the open boreholes upon completion of drilling and in the piezometers are summarized below:

Borehole Number	Ground Surface Elevation (m)	Depth to Water Level (m)	Depth to Water Elevation (m)	Date
2010-1	179.5	25.9	153.6	June 3, 2011 in piezometer
2010-2	172.2	5.6	166.6	September 30, 2010 in open borehole
2010-3	172.5	26.4	147.9	October 1, 2010 in open borehole
2010-4	178.0	27.2	150.8	June 3, 2011 in piezometer

The water levels in Boreholes 2010-2 and 2010-3 were measured upon completion of drilling and are not representative of the stabilized groundwater level at this site which ranges from about Elevation 151 m to 154 m in June 2011. The higher water levels measured during or shortly after drilling operations suggest that perched groundwater conditions exist above and within the upper portion of the clayey silt till deposit which contains sand seams / interlayers.

The water level(s) at the bridge site should be expected to fluctuate seasonally in response to changes in precipitation and snow melt, and should be expected to be higher during the spring season or during any period of heavy precipitation.



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### 5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Nikol Kochmanová, EIT, and reviewed by Mr. Kevin Bentley, P.Eng., an Associate and Senior Geotechnical Engineer at Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact and Principal with Golder, conducted an independent review and quality control audit of this report.

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**FOUNDATION REPORT - KEELE STREET UNDERPASS,  
HIGHWAY 401 EBC REHABILITATION**

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# **PART B**

**FOUNDATION DESIGN REPORT  
KEELE STREET UNDERPASS  
HIGHWAY 401 EBC REHABILITATION FROM  
JANE STREET TO AVENUE ROAD  
TORONTO, ONTARIO  
G.W.P. 2368-09-00**



## **6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS**

### **6.1 General**

This section of the report provides foundation recommendations for the design of the proposed rehabilitation or replacement of the existing Highway 401 – Keele Street Underpass. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation at this site. The interpretation and recommendations contained in this report are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out detail design of the foundations for the proposed structure rehabilitation or replacement. Where comments are made on construction, they are provided in order to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

It is understood that the proposed bridge rehabilitation option will consist of replacing the superstructure (girders and beams), while the initial intent was to maintain the existing substructure (abutments and piers). The proposed rehabilitation also includes widening of the existing bridge on both the east and west sides to add an extra lane on the northbound direction of Keele Street, which will result in a wider bridge deck by approximately 2.5 m. In addition, the Keele Street grade will be raised by about 600 mm. This widening condition in addition to the proposed grade raise will induce further loads on the existing bridge foundations. Based on the existing drawings, the bridge abutments and piers are founded on spread footings. As a result, the abutment footings would require widening on both sides of the existing bridge, whereas the pier footings would not need to be extended as the existing pier caps are long enough to support the proposed bridge deck widening. Further to a preliminary assessment of foundation alternatives (Golder's Technical Memorandum, dated November 15, 2010) and structural assessments made by URS for the rehabilitation option, a decision was made in consultation with the MTO, to maintain the abutments and replace the existing piers in order to support the additional loads resulting from the proposed rehabilitation (and widening) option for the bridge structure.

However, based on subsequent conversations with URS and the Minutes of Meeting from a progress meeting held on February 9, 2011 between MTO and URS, we understand that a full bridge replacement option is now being considered. Based on the General Arrangement drawing provided to us on April 8, 2011 titled "Hwy 401 and Keele St. Interchange Bridge Replacement", dated February 2011, the existing bridge is to be removed and a new bridge constructed consisting of three spans, with span lengths of 35 m between the north abutment and north pier, 42 m between piers, and 35 m between the south pier and south abutment. The proposed new bridge is shown to consist of integral abutments and is about 11 m shorter than the existing bridge as a result of relocating the new abutments approximately 5 m to 6 m closer to the piers from the existing location (i.e. moving the abutments closer to the travelled collector EBL and WBL). As a result, the existing ground surface behind the proposed new abutment foundations (which is currently the existing front slope of existing abutments) will need to be raised by backfilling up to 3 m at the approach slabs.

The following sections of this Foundation Design Report present an assessment of the conditions of the founding soils at the abutment and pier locations based on the results of the current investigation as well as an assessment of the feasible foundation alternatives for the proposed bridge rehabilitation and replacement options.



### 6.2 Assessment of Existing Structure Foundations

The existing Highway 401 – Keele Street underpass was constructed in 1964, and was first rehabilitated in 1989. The Keele Street bridge structure consists of three spans, with span lengths of 40.9 m between the north abutment and north pier, 42.0 m between the piers, and 40.3 m between the south pier and south abutment. Based on the available drawings, the bridge abutments and piers are supported on spread footings with footing dimensions and founding elevations as summarized below.

Foundation Element	Footing Dimension (m) B x L x t	Founding Elevation
North Abutment	2.75 x 15.5 x 0.6	176.3 m
North Pier	4.6 x 9.2 m x 1.2	170.2 m
South Pier	4.6 x 9.2 m x 1.2	169.6 m
South Abutment	2.75 x 15.5 x 0.6	174.8 m

The design geotechnical axial resistance for the Keele Street Bridge footings as reported in the original Foundation Investigation report, dated August, 1963 was provided in terms of an allowable bearing capacity of 288 kPa (3 tsf). This bearing capacity value translates into a factored geotechnical resistance of 432 kPa at ULS as assessed during the Phase 1 Desktop Study for this Project (Golder's Technical Memorandum, dated July 21, 2010).

The results of the current subsurface investigation indicate that the Keele Street bridge approaches were constructed in cut and thus the abutment footings, as well as the pier footings, are founded on the native clayey silt to silty clay till. The results of the subsurface investigation also indicate that the strength of the upper till deposit is less than that indicated by the previous boreholes (i.e. Boreholes 1 and 2, Geocres No. 30M11-084). Current SPT 'N'-values measured within the till at the founding levels of the existing abutment and pier footings typically range from 11 to 18 blows per 0.3 m of penetration, suggesting a stiff to very stiff consistency. SPT 'N'-values measured within the till deposit at the same founding levels in the previous investigation range from 26 to 56 blows per 0.3 m of penetration, suggesting a very stiff to hard consistency. These differences may be partially due to the use of an automated hammer with higher efficiency in the current investigation as compared to a manually operated hammer (i.e. rope cathead) likely used in the previous investigation. To better correlate the current SPT 'N'- values with those obtained in the previous investigation, the current 'N'-values were corrected to 60% efficiency of hammer energy transfer. The resulting 'N<sub>60</sub>'-values at the abutment and pier founding levels generally range from 16 to 27 blows per 0.3 m of penetration. Based on the results of the current investigation and corrected SPT 'N'-values to 'N<sub>60</sub>' values, the estimated factored geotechnical axial resistance at Ultimate Limit States (ULS) and the geotechnical resistance at Serviceability Limit States (SLS) for the existing abutment and pier foundations, using the existing foundation geometry and reported founding elevations, are presented below.





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Foundation Element	Existing Footing Width (m)	Founding Elevation (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa) (for 25 mm settlement)
North Abutment	2.75	176.3	400	300
North Pier	4.6	170.2	400	265
South Pier	4.6	169.6	400	265
South Abutment	2.75	174.8	400	300

### 6.2.1 Required Foundation Resistance for Rehabilitation Option

Based on the foundation evaluations carried out by URS in accordance with Section 6 of the CHBDC, CAN/CSA S6-06, the required foundation resistances for the original, existing and proposed future bridge loads have been estimated by URS as summarized below.

Foundation Element	Factored Geotechnical Resistance at ULS (kPa)			Geotechnical Resistance at SLS (kPa)		
	Original <sup>1</sup>	Existing <sup>2</sup>	New	Original	Existing	New
North Abutment	200	211	229	145	153	171
North Pier	411	440	491	298	321	352
South Pier	408	437	487	296	319	349
South Abutment	204	214	231	151	159	170

<sup>1</sup> Based on original bridge loads upon completion of the bridge construction in the 1960's

<sup>2</sup> Current loads further to bridge rehabilitation in the 1980's.

Based on the above requirements, the geotechnical resistance values provided in Section 6.2 are adequate for the support of the existing and additional loads at the existing abutment locations and can be used for design of the widened abutment footings assuming that the widened footings will be founded at the same levels as the existing abutment footings. Foundation options for the replacement option are given in Section 6.3.

At the existing pier locations however, the design geotechnical resistances are less than those required for the existing and future (increased) bridge loads. Detailed assessment of the pier foundations condition and recommendations for rehabilitation or replacement are provided in Section 6.4.

## 6.3 Abutment Foundations

The proposed rehabilitation option for the Highway 401 – Keele Street Underpass includes the raising of the bridge grade by 600 mm and widening of the bridge structure on both the east and west side of the existing bridge by a total of 2.5 m. Based on the assessment of the existing abutment foundations and required geotechnical resistances as discussed in Sections 6.2 and 6.2.1, the existing abutment spread footings may be widened to support the proposed bridge deck widening. Alternatively, should full replacement of the structure be chosen, the new abutments may be founded on shallow or deep foundations.



From a geotechnical perspective, it is recommended that the widened footings for the rehabilitation option be founded on shallow foundations to be compatible with the existing abutment foundations. If the option to shorten the bridge span lengths and completely re-design and replace the bridge structure (including abutments and foundations) is chosen, steel H-piles are considered the preferred option to accommodate the proposed integral abutment design at the new locations. Steel tube (pipe) piles could also be considered as a pile option; however, steel tubes are considered to pose a higher risk of “hanging up” or being deflected away from vertical for integral abutment design due to the presence of cobbles and boulders within the till material. Shallow foundations are still considered to be the most practical option for the replacement option, however, the shortened spans result in the foundations being located along the inclined front slope of the existing cut approach to the Highway 401 underpass resulting in inclined loading conditions and are not suitable for integral abutment design.

### 6.3.1 Spread Footings

#### 6.3.1.1 Founding Elevations

For support of the abutments for the widened portions of the structure for the rehabilitation option, strip or spread footings should be founded at the same level as the existing footings, with 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*).

The recommended founding elevations for the abutment widening and estimated required excavation depths are as provided below. These elevations are compatible with the existing foundation elevations. New spread /strip footings for the replacement option would be founded below these elevations (with a minimum depth of 1.2 m below the lowest surrounding grade for frost protection) given that the shortened end spans result in the new abutment foundations being located at lower elevation on the front slope of the existing cut slope leading to Highway 401.

Foundation Element	Founding Stratum	Strip/Spread Footing Founding Elevation*	Approximate Maximum Excavation Depth
North Abutment	Stiff to very stiff clayey silt to silty clay till	176.3 m	3.2 m
South Abutment	Stiff to very stiff clayey silt to silty clay till	174.8 m	3.2 m

\* For widened footings at the existing abutment locations; and at lower elevations for new footings at the relocated abutments.

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to verify that all existing fill and other unsuitable material have been removed. The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation within four hours to protect the integrity of the subgrade. This requirement can either be added as a note on the Contract Drawings or a Non-Standard Special Provision can be included in the contract package. A sample NSSP is included for this item in Appendix D.



### 6.3.1.2 Geotechnical Resistance

Strip or spread footings for the bridge abutment widening or replacement, placed on the properly prepared, stiff to very stiff clayey silt to silty clay till, at or below the design elevations given in Section 6.3.1.1, should be designed based on the factored geotechnical axial resistance at ULS equal to 400 kPa and geotechnical resistance at SLS equal to 300 kPa, assuming a 2.75 m wide footing to match the existing abutment footings.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above. The resistance values should also be reviewed if the location of the new abutment foundations for the replacement option change in longitudinal direction as the existing ground surface is sloped downward toward Highway 401 which could lead to a reduction in the geotechnical resistance values.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.4 and C6.7.4 in the *Canadian Highway Bridge Design Code (CHBDC)*.

### 6.3.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the stiff to very stiff clayey silt to silty clay till, the coefficient of friction,  $\tan \phi'$ , can be taken as 0.45.

### 6.3.2 Steel H-Pile / Steel Tube Foundations

Steel H-Piles or steel tube piles may be used to support the new (relocated) abutments, especially if integral abutments are being considered. For the installation of either friction or end-bearing piles, consideration must be given to the potential presence of cobbles and boulders usually present within clayey silt to silty clay till deposits, although boulder/rock fragments were only noted at one borehole location at this site. Steel H-piles are preferred over steel tube piles given that H-piles are more conventional for integral abutment design and the fact that steel tubes are considered to pose a higher risk of “hanging up” or being deflected away from vertical during driving. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard “H” Bearing Pile Point design for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

If corrugated steel pipes (CSPs) are installed as part of the integral abutment design, the CSPs should be backfilled with loose, fine to medium sand. A Non Standard Special Provision (NSSP) detailing the installation



method and gradation of this sand should be included in the Contract Documents; an example is included in Appendix D.

The pile caps for the new abutments should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (as per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*).

### 6.3.2.1 Friction and End-Bearing Piles

For HP 310 x 110 piles driven to design tip Elevation 152 m (i.e. piles about 23 m long), the factored geotechnical axial resistance at ULS may be taken as 900 kN. The geotechnical resistance at SLS (for 25 mm of settlement) may be taken as 700 kN. The following note, or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the Structural Manual Section 3.3.3 (MTO, 2008)):

*“Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of 1,800 kN per pile, but must be driven below tip Elevation 153.5 m”*

Alternatively, the steel H-piles could be driven at least 1.5 m into the “100-blow” material to or below a design tip Elevation 140 m for the north abutment and Elevation 142.5 m for the south abutment. The design factored geotechnical axial resistance at ULS may be taken as 1,600 kN, and the geotechnical resistance at SLS (for 25 mm of settlement) may be taken as 1,400 kN. The following note, or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the Structural Manual Section 3.3.3 (MTO, 2008)):

*“Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 3,200 kN per pile, but must be driven below the following tip elevations:*

- *North Abutment: Elevation 141.5 m*
- *South Abutment: Elevation 144.0 m*

In addition, the Contract Documents should make allowances for varying pile lengths in the event that piles either achieve the required set at higher elevations or have to be driven to a lower elevation.

Similar axial resistances and drawing notes may be used in the design for 324 mm (12 ¾ in) diameter, 6.4 mm (¼ in) wall thickness tube (pipe) piles.

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. As indicated in the Contract Drawing Notes above, the pile capacity should be checked in the field by the use of the Hiley formula (MTO Standard Drawing SS 103-11) during the final stages of driving starting about 1.5 m above the design pile tip elevation to verify that the ultimate capacity noted above has been achieved.



### **6.3.2.2 Resistance to Lateral Loads**

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction ( $k_h$  in kPa/m) is determined based on the equations given below (as noted in Section C6.8.7.1 (Table C6.5 and CFEM 1992) and in Section C6.8.7.3 of the *Commentary to CHBDC*).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{Where}$$

$k_h$  is the coefficient of horizontal subgrade reaction (kPa/m);  
 $n_h$  is the constant of subgrade reaction (kPa/m);  
 $z$  is the depth (m); and  
 $B$  is the pile diameter / width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{Where}$$

$k_h$  is the coefficient of horizontal subgrade reaction (kPa/m);  
 $s_u$  is the undrained shear strength of the soil (kPa); and  
 $B$  is the pile diameter / width (m).

It is understood that an integral abutment foundation design is being considered. Where the integral design includes the installation of 3 m long CSP liners (with the annular space between the pile and the liner filled with uniform grained, uncompacted sand), the upper portion of the H-pile will be generally free to flex and move laterally within the limits of the CSP. With this design, the passive lateral resistance over the length of the pile within the CSP liner should be based on the resistance provided by loose sand. The passive lateral resistance on the exterior of the CSP should be based on the resistance provided by the surrounding soil conditions.

The following values of  $n_h$  and  $s_u$  may be assumed in the structural analyses, using the interpreted stratigraphic conditions as shown on the stratigraphic profile in the area of each foundation element (refer to Drawing 1):

Soil Unit	$n_h$ (kPa/m)	$s_u$ (kPa)
Loose sand within CSP	2,200	-
Stiff to hard clayey silt till	-	100
Dense sand and silt to silty sand	11,000	-
Very dense silt	15,000	-
Hard clayey silt	-	200

A factored geotechnical lateral resistance of 110 kN at ULS, and a geotechnical lateral resistance of 35 kN at SLS (for 10 mm of horizontal deflection at pile cap level) was calculated for a vertical free-headed HP 310x110 pile based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc. The structural capacity of the pile should be checked and verified by the structural engineer.



Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.02, 1986) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided above.

### 6.3.3 Caisson Foundations

Consideration could be given to the use of caissons socketted into the hard clayey silt to silty clay till for support of the foundation elements for the new abutments. Consideration could also be given to extending the caissons into the “100-blow” material at the abutment locations.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling the caisson. If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner would be required to support the soils during construction, to prevent the movement of the cohesionless soils from coming up into the liners, and to permit inspection and cleaning of the caisson base.

The caisson caps for the new piers should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*) unless the caps are positioned at the top of the columns.

#### 6.3.3.1 Founding Elevations

Caissons may be founded within the clayey silt to silty clay till deposit, or about 2 m into the “100-blow” material at greater depths if higher capacities are required. The estimated caisson tip elevations for new abutment foundations are summarized below.

Founding Stratum	Design Caisson Tip Elevation
Stiff to hard clayey silt to silty clay till	152.0 m
Dense sand and silt to silty sand	147.0 m
2 m into “100-blow” material	139.5 m



### 6.3.3.2 Geotechnical Resistances

The recommended values for factored geotechnical axial resistance at ULS and geotechnical resistance at SLS (for 25 mm of settlement) for caissons founded at the elevations given in Section 6.3.3.1 are provided below.

Founding Stratum	Design Caisson Tip Elevation	Caisson Diameter	Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm settlement)
Stiff to hard clayey silt till	152.0 m	0.6 m	1,100 kN	950 kN
		1.2 m	3,000 kN	2,500 kN
		1.5 m	4,000 kN	3,350 kN
Dense sand and silt to silty sand	147.0 m	0.6 m	1,500 kN	1,200 kN
		1.2 m	4,000 kN	3,000 kN
		1.5 m	5,350 kN	4,000 kN
2 m into "100-blow" material	139.5 m	0.6 m	2,500 kN	2,000 kN
		1.2 m	7,000 kN	5,000 kN
		1.5 m	9,500 kN	7,000 kN

### 6.3.3.3 Resistances to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.3.2.2 may be used for design.

A factored geotechnical lateral resistance of 500 kN at ULS, and a geotechnical lateral resistance of 225 kN at SLS (for 10 mm of horizontal deflection at the caisson pile cap level) was calculated assuming a 1.2 m diameter free-headed caisson based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc and checked for validity using closed form solutions for single piles in a homogenous soil, as suggested by Poulos and Davis (1984) and Brom's method (CFEM 1992 in CHBDC 2006). A structural rigidity (EI value) of 178 MN·m<sup>2</sup> was assumed for the concrete caisson in the analysis and the structural capacity of the caisson should be checked and verified by the structural engineer.

## 6.4 Pier Foundations

Based on the geotechnical resistance assessments at the pier locations for the rehabilitation option as discussed in Sections 6.2 and 6.2.1, the design geotechnical resistances for the existing bridge piers are less than those required for the existing and future (increased) bridge loads. Under the proposed future loads (i.e. required factored geotechnical axial resistance at ULS of 490 kPa and geotechnical resistance at SLS of 350 kPa, as indicated by URS), the existing pier footings would undergo a total estimated additional settlement from the original loading conditions ranging from about 5 mm to 15 mm. It is anticipated that about 75 percent of this additional settlement would have occurred under the existing loads (i.e. for a geotechnical resistance at SLS of about 320 kPa). Further, the applied resistance factor for assessment of the factored geotechnical axial resistance at ULS would be increased under future (increased) bridge loads by about 16 percent (i.e. a factor of





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0.57 instead of 0.5) which translates into a reduced factor of safety for Ultimate Limit States design from 2.0 to 1.7. Similarly, the factor of safety applied for calculation of the allowable bearing capacity (based on current investigation results) would be reduced from 3.0 (assumed to be used in the original design) to approximately 2.6. These data are summarized below for the existing bridge north pier.

Loading Condition at the North Pier	Factored Geotechnical Resistance at ULS (kPa)	Resistance Factor (ULS)	Estimated FOS (Allowable Bearing Capacity)	Geotechnical Resistance at SLS (kPa)	Max. Total Settlement (mm)
Original Design	432 <sup>1</sup>	0.5	3.0	265 <sup>2</sup>	25
Original Load	411	0.48	3.15	298	29
Existing	440	0.51	2.9	321	31-37
Future	491	0.57	2.6	352	34-40

Notes: 1 Back-calculated based on original design allowable bearing capacity of 288 kPa (3tsf)

2 Calculated geotechnical resistance at SLS for the existing subsoil conditions. At the time of the bridge design in the 1960's, SLS values were not calculated.

Considering the above foundation conditions at the pier locations, alternative measures for the rehabilitation of the existing structure have been assessed. A summary of the advantages and disadvantages associated with each measure is provided below, and a comparison of the alternative measures based on advantages, disadvantages, risks, consequences and relative costs is provided in Table 1 following the text of this report.

- 1) Replacement of the existing bridge piers and pier foundations considering that the existing superstructure is to be replaced and that at least some foundation improvement is required at the existing pier locations to adhere to the CHBDC foundation design requirements (i.e. factor of safety for design). This option would likely result in significant traffic disruption and high capital costs;
- 2) Implementing measures to supplement the existing pier foundation resistances to support the proposed future (increased) bridge loads with appropriate design factors of safety and maximum tolerable settlement per the CHBDC. From a foundations perspective, such measures may include:
  - a. Adding a new footing section between the two existing pier footings at each pier location to support a new column that would extend to the underside of the bridge deck. This design will provide for a larger footing area and therefore support increased loads. In this case, the new footing sections should be keyed into the existing footings using appropriate dowels to allow for the potential differential settlement of the new footing sections relative to the existing footings. For this configuration, the factored ULS and SLS geotechnical resistance values of 400 kPa and 265 kPa, respectively, (as provided in Section 6.2 for the pier locations) should be used in the design. If differential settlements (up to 25 mm) between the new and existing footings can not be accommodated, then consideration should be given to installing caissons within the new foundation sections to support the columns extending to the underside of the bridge deck. The caissons would be advanced to a sufficient depth to provide the required geotechnical resistances.



- b. Installing caissons to support the existing footings on grade beams or caisson caps. The new caissons would be installed outside of the existing footings and could be extended a sufficient depth to provide the required resistances. The caisson caps may alternatively be tied to the sides of the existing footings.
  - c. Retrofitting the support system beneath the existing pier footings by installing micropiles to a depth sufficient to provide the required foundation resistances. This alternative is normally costly as it requires a specialist contractor and equipment. Further, a site-specific design and construction specification for the installation of micropiles would be required; or
- 3) To avoid increased traffic disruption that would be caused by foundation improvement works during construction (which we understand to be undesired at this site), another option that may be examined is to “do nothing” to improve the geotechnical resistances at the pier locations under the proposed additional bridge loads and accept a reduced design factor of safety as described above. This option however, requires an assessment of risk and can only be considered at the bridge owner’s discretion. If selected, it is recommended that this option be considered in conjunction with risk mitigation measures that should include:
- a. Minimizing the proposed additional loads on the pier foundations by using lightweight materials for the construction of the new bridge deck and/or other means that may be appropriate from a structural design perspective; and
  - b. Implementing a monitoring program with appropriate review/alert levels and response plan during construction and for a period of time post-construction to assess foundation movements under the new bridge loads.

For options 1 and 2, new foundations are required to either replace or supplement the existing pier foundations. Further discussion and design recommendations for each of the new foundation options is provided below. A summary of the advantages, disadvantages, risks and relative costs associated with each of the foundation options is provided in Table 1 following the text of this report.

Based on the above considerations for the bridge rehabilitation option, and in particular considering the required geotechnical resistance for the new piers, the preferred option from a geotechnical/foundations perspective is to support the piers for the structure widening on deep foundations. We understand that the recent consideration to shorten the span lengths and completely re-design and replace the bridge structure (including piers and foundations) would result in shallow foundations being the preferred option from a foundations perspective.

### 6.4.1 Spread Footings

#### 6.4.1.1 Founding Elevations

For support of the new pier foundations, strip or spread footings should be founded below the existing fill on the stiff to very stiff clayey silt to silty clay till. 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*).



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The following founding elevations are recommended for strip or spread footings for support of the piers. These elevations are compatible with the existing foundation elevations, however, the new footings may be founded at or below these elevations.

Foundation Element	Founding Stratum	Strip/Spread Footing Founding Elevation	Approximate Maximum Excavation Depth
North Pier	Stiff to very stiff clayey silt to silty clay till	170.2 m	2.0 m
South Pier	Stiff to very stiff clayey silt to silty clay till	169.6 m	2.9 m

The footing subgrade should be inspected by a Quality Verification Engineer following excavation, in accordance with OPSS 902 (*Excavating and Backfilling Structures*) to verify that all existing fill or other unsuitable material have been removed.

The founding soils will be susceptible to disturbance. If the concrete for the footings cannot be poured immediately after excavation and inspection, it is recommended that a concrete working slab (100 mm thick of 20 MPa compressive strength concrete) be placed in the excavation within four hours to protect the integrity of the subgrade. This requirement can either be added as a note on the Contract Drawings or a Non-Standard Special Provision can be included in the contract package. A sample NSSP is included for this item in Appendix D.

### 6.4.1.2 Geotechnical Resistance

Strip or spread footings for the pier placed on the properly prepared, stiff to very stiff clayey silt to silty clay till, at or below the design elevations given in the Section 6.4.1.1, should be designed based on a factored geotechnical axial resistance at ULS of 400 kPa and geotechnical resistance at SLS of 265 kPa for the bridge piers assuming a 4.6 m wide footing.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads. Therefore, the geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given for loads applied perpendicular to the surface of the footing. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.4 and C6.7.4 in the *CHBDC*.

### 6.4.1.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footing and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. For cast-in-place concrete footings constructed on the stiff to very stiff clayey silt to silty clay till, the coefficient of friction,  $\tan \phi'$ , can be taken as 0.45.



### 6.4.2 Steel H-Pile / Steel Tube Foundations

Steel H-Piles or steel tube piles may be used to support the new piers. For the installation of either friction or end-bearing piles, consideration must be given to the potential presence of cobbles and boulders usually present within clayey silt to silty clay till deposits, although boulder/rock fragments were only noted at one borehole location at this site. Steel H-piles are preferred over steel tube piles given that H-piles are more conventional for integral abutment design and the fact that steel tubes are considered to pose a higher risk of “hanging up” or being deflected away from vertical during driving. The piles should be reinforced at the tip for protection during driving to reduce the potential for damage to the piles in the event that cobbles/boulders and dense layers are encountered within the till deposits. The steel H-piles should be reinforced with flange plates as per OPSD 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or driving shoes such as Titus Standard “H” Bearing Pile Point design for protection during driving as per OPSS 903 (Deep Foundations). Similarly, if steel tube piles are being considered, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe). The requirement for driving shoes should be included in the Contract Drawings.

The pile caps for the new piers should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*).

#### 6.4.2.1 Friction and End-Bearing Piles

For HP 310 x 110 piles driven to a design tip Elevation 152 m (i.e. piles about 20 m long), the factored geotechnical axial resistance at ULS may be taken as 900 kN. The geotechnical resistance at SLS (for 25 mm of settlement) may be taken as 700 kN. The following note, or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 2 from the Structural Manual Section 3.3.3 (MTO, 2008)):

*“Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of 1,800 kN per pile, but must be driven below tip Elevation 153.5 m”*

Alternatively, the steel H-piles could be driven at least 1.5 m into the “100-blow” material to or below a design tip Elevation 140 m for the north pier and Elevation 142.5 m for the south pier. The design factored geotechnical axial resistance at ULS may be taken as 1,600 kN, and the geotechnical resistance at SLS (for 25 mm of settlement) may be taken as 1,400 kN. The following note, or similar, should be shown on the Contract Drawing assuming that a resistance factor of 0.5 is applied to the use of the Hiley calculation based on MTO experience in the Southern Ontario region (Note 1 from the Structural Manual Section 3.3.3 (MTO, 2008)):

*“Piles to be driven in accordance with Standard SS-103-11 using an ultimate geotechnical resistance of 3,200 kN per pile, but must be driven below the following tip elevations:*

- North Pier: Elevation 141.5 m
- South Pier: Elevation 144.0 m

In addition, the Contract Document should make allowances for varying pile lengths in the event that piles either achieve the required set at higher elevations or have to be driven to a lower elevation.

Similar axial resistances and drawing notes may be used in the design for 324 mm (12 ¾ in) diameter, 6.4 mm (¼ in) wall thickness tube (pipe) piles.



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 be checked in the field by the use of the Hiley formula (MTO Standard Drawing SS 103-11) during the final stages of driving starting about 1.5 m above the design pile tip elevation to verify that the ultimate capacity noted above has been achieved.

### 6.4.2.2 Resistance to Lateral Loads

Resistance to lateral loading can be derived using vertical piles, with enhanced support offered by battered piles, if required. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles. The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction ( $k_h$  in kPa/m) is determined based on the equations given below (as noted in Section C6.8.7.1 (Table C6.5 and CFEM 1992) and in Section C6.8.7.3 of the *Commentary to CHBDC*).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{Where}$$

$k_h$  is the coefficient of horizontal subgrade reaction (kPa/m);  
 $n_h$  is the constant of subgrade reaction (kPa/m);  
 $z$  is the depth (m); and  
 $B$  is the pile diameter / width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B} \quad \text{Where}$$

$k_h$  is the coefficient of horizontal subgrade reaction (kPa/m);  
 $s_u$  is the undrained shear strength of the soil (kPa); and  
 $B$  is the pile diameter / width (m).

The following values of  $n_h$  and  $s_u$  may be assumed in the structural analyses, using the interpreted stratigraphic conditions as shown on the stratigraphic profile in the area of each foundation element (refer to Drawing 1):

Soil Unit	$n_h$ (kPa/m)	$s_u$ (kPa)
Stiff to hard clayey silt till	-	100
Dense sand and silt to silty sand	11,000	-
Very dense silt	15,000	-
Hard clayey silt	-	200

A factored geotechnical lateral resistance of 150 kN at ULS, and a geotechnical lateral resistance of 60 kN at SLS (for 10 mm of horizontal deflection at pile cap level) was calculated for a vertical free-headed HP 310x110 pile based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc. The structural capacity of the pile should be checked and verified by the structural engineer.



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Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.02, 1986) as follows:

Pile Spacing in direction of Loading (d = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
8d	1.00
6d	0.70
4d	0.40
3d	0.25

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided above.

### 6.4.3 Caisson Foundations

Consideration could be given to the use of caissons socketted into the hard clayey silt to silty clay till for support of the foundation elements for the new piers. Consideration could also be given to extending the caissons into the “100-blow” material at the pier locations.

Running or flowing of water-bearing cohesionless soil strata could occur during or after drilling the caisson. If caisson foundations are adopted for support of any of the foundation elements, a temporary or permanent liner would be required to support the soils during construction, to prevent the movement of the cohesionless soils from coming up into the liners, and to permit inspection and cleaning of the caisson base.

The caisson caps for the new piers should be provided with a minimum of 1.2 m of soil cover to provide adequate protection against frost penetration (per OPSD 3090.101 – *Foundation Frost Depths for Southern Ontario*) unless the caps are positioned at the top of the columns.

#### 6.4.3.1 Founding Elevations

Caissons may be founded within the clayey silt to silty clay till deposit, or about 2 m into the “100-blow” material at greater depths if higher capacities are required. The estimated caisson tip elevations for new pier foundations are summarized below.

Founding Stratum	Design Caisson Tip Elevation
Stiff to hard clayey silt to silty clay till	152.0 m
Dense sand and silt to silty sand	147.0 m
2 m into “100-blow” material	139.5 m



### 6.4.3.2 Geotechnical Resistances

The recommended factored geotechnical axial resistance at ULS and geotechnical resistance at SLS (for 25 mm of settlement) for caissons founded at the elevations given in Section 6.4.3.1 are provided below.

Founding Stratum	Design Caisson Tip Elevation	Caisson Diameter	Factored Geotechnical Axial Resistance at ULS	Geotechnical Resistance at SLS (for 25 mm settlement)
Stiff to hard clayey silt till	152.0 m	0.6 m	1,100 kN	950 kN
		1.2 m	3,000 kN	2,500 kN
		1.5 m	4,000 kN	3,350 kN
Dense sand and silt to silty sand	147.0 m	0.6 m	1,500 kN	1,200 kN
		1.2 m	4,000 kN	3,000 kN
		1.5 m	5,350 kN	4,000 kN
2 m into "100-blow" material	139.5 m	0.6 m	2,500 kN	2,000 kN
		1.2 m	7,000 kN	5,000 kN
		1.5 m	9,500 kN	7,000 kN

### 6.4.3.3 Resistances to Lateral Loads

Resistance to lateral loading will be derived from the soil in front of the caissons. The resistance to lateral loading in front of the caisson may be calculated using subgrade reaction theory and the equations and soil parameters provided in Section 6.4.2.2 may be used for design.

A factored geotechnical lateral resistance of 500 kN at ULS, and a geotechnical lateral resistance of 225 kN at SLS (for 10 mm of horizontal deflection at the caisson pile cap level) was calculated assuming a 1.2 m diameter free-headed caisson based on an analysis using the commercially available program *LPILE Plus* (Version 5.0) produced by Ensoft Inc. A structural rigidity (EI value) of 178 MN·m<sup>2</sup> was assumed for the concrete caisson in the analysis and the structural capacity of the caisson should be checked and verified by the structural engineer.

## 6.5 Seismic Site Coefficient

According to Table C4.2 of the *Commentary* to the *CHBDC (2006)*, this site is located in Seismic Zone 1, and the site-specific zonal acceleration ratio (A) for the Toronto area is 0.05. The Site Coefficient (S) may be taken as 1.2, consistent with Soil Profile Type II in accordance with Section 4.4.6 and Table 4.4 of the *CHBDC (2006)*.

## 6.6 Bridge Approaches

The proposed rehabilitation / replacement of the Highway 401 – Keele Street Underpass structure includes the raising of the bridge grade by 600 mm. The raising of the bridge grade will require transitional raising and widening of the approach embankments behind the abutments. It is considered that this grade raise will not detrimentally impact the stability of the approach embankment nor result in significant settlements. The existing





embankment side-slopes are sloped at about 2H:1V and it is assumed the new widened side-slopes will be maintained at 2H:1V or shallower.

The proposed bridge and foundation re-design and replacement option involves shortening the end spans by about 5 m to 6 mm at both the south and north abutment locations. As a result, the existing ground surface (currently the existing front slopes for the existing bridge that will need to be properly stripped) behind the proposed new abutment foundations will require a grade raise up to 3 m high to match the level of the new approach slab and Keele St. road grade. Considering that the existing front slopes leading down to the Highway 401 level are constructed predominantly in cut through the stiff to hard clayey silt till, the total settlement of the foundation soil is expected to be less than 25 mm as a result of the expected grade raise (up to 3 m high). Granular fill should be used to raise the grade in the areas behind the abutments and below the approach slab and approach embankment to minimize the potential for additional settlement as described in Section 6.6.1.

Recommendations pertinent to the widening of the approach embankments, subgrade preparation and embankment construction / grade raise are provided below.

### 6.6.1 Subgrade Preparation and Embankment Construction

The existing native soils are considered to be an appropriate subgrade for the widening / new construction of the Keele Street approach embankments. However, to improve performance and minimize the differential settlement between the widening / new construction and the existing approach embankments, it is recommended that prior to the placement of any fill, all topsoil, organic matter, and any softened/loosened native soils be stripped from below the approach embankment areas, including the side slopes at the transitions immediately adjacent to the abutments. For the replacement option where abutment foundations are to be replaced / relocated, any existing erosion / surface protection material should be removed and the native clayey silt till exposed along the front slopes prior to placing embankment fill / backfill as part of the grade raise.

The use of granular fill is recommended rather than the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. The new embankment fills should be benched into the existing embankments in accordance with OPSS 208.010 (*Benching of Earth Slopes*). The fill for the widened / new embankment should be placed and compacted in accordance with OPSS 501 (*Compacting*), with inspection and field density testing by qualified personnel during placement operations to confirm that appropriate materials are used and that adequate levels of compaction are achieved.

To reduce the potential for erosion of the embankment side slopes due to surface water runoff and to establish vegetation on the slopes, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. Topsoil should be placed on granular fill side-slopes in accordance with OPSS 802 (*Topsoil*) and covered with erosion protection in accordance with OPSS 804 (*Seed and Cover*) or pegged sod in accordance with OPSS 803 (*Sodding*). Topsoil and erosion protection should be placed in early summer to avoid wet periods of the year which may cause surficial sloughing of the topsoil material along the side-slopes and to establish vegetation prior to the Fall / Winter months.



## **6.6.2 Lateral Earth Pressures for Design**

The lateral earth pressures acting on the widened / new abutment stems and any associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the stems/walls.

The following recommendations are made concerning the design of the / widened portion of the abutment walls. These design recommendations and parameters are applicable to level backfill and ground surface behind the abutment stems. Where there is sloping ground behind the stems or walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of SP 110S13 (*Aggregates*) Granular 'A' or Granular 'B' Type II but containing less than 5 percent passing the No. 200 sieve size should be used as backfill behind the abutment walls. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS 501 (*Compacting*). Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill*) and 3121.150 (*Walls, Retaining, Backfill*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressure for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- For restrained structures, the granular fill should be placed in a zone with width equal to at least 1.2 m behind the back of the walls ( in accordance with Figure C6.20(a) of the Commentary to the CHBDC). For unrestrained structures, granular fill should be placed within the wedge shaped zone defined by a line drawn at no steeper than 1.5H:1V extending up and back from the rear face of the base of the footing (in accordance with Figure C6.20(b) of the Commentary to the CHBDC).
- For restrained structures, the pressures are based on the existing overburden soils and the following parameters (unfactored) may be used assuming native clayey silt fill:

	Earth Fill
Soil unit weight:	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure:	
Active, $K_a$	0.33
At rest, $K_o$	0.50

- For unrestrained structures, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:



	<b>Granular A</b>	<b>Granular 'B' Type II</b>
Soil unit weight	22 kN/m <sup>3</sup>	21 kN/m <sup>3</sup>
Coefficients of static lateral earth pressure		
Active, $K_a$	0.27	0.27
At rest, $K_o$	0.43	0.43

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the Commentary to the CHBDC.

A restrained structure is typically a concrete box culvert or a rigid frame bridge where the rotational and/or horizontal movement is not sufficient to mobilize the active earth pressure condition. For this condition, an at-rest pressure plus any compaction surcharge should be included in the design of the structure.

## **6.7 Construction Considerations**

### **6.7.1 Excavation and Groundwater Control**

The foundation excavations at the abutments will extend to depths of about 3 m to 4 m below present grade, through the existing fill into the stiff to hard clayey silt till. At the pier locations, the depth of this excavation will extend to depths of about 2 m to 3 m through the existing fill into the stiff to hard clayey silt till. If space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill is classified as Type 3 soil and the till is classified as Type 2 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical through the fill materials and through the till to within 1.2 m of the bottom of the excavation.

Excavations for widened / new abutment foundations and for new pier foundations will be maintained above the groundwater level, and groundwater inflow is expected to be relatively minor, especially during drier periods of the year. Some groundwater may be “perched” within the existing granular fill at the site, and therefore some water inflow should be expected into the foundation excavations, particularly during wet months; however, it is anticipated that water inflow can be handled by pumping from filtered sump pumps placed at the base of the excavations.

### **6.7.2 Temporary Excavation Support**

Temporary excavation support may be required at the abutments and piers to facilitate the construction of the widened / new abutments and new pier foundations at the Keele Street Underpass. The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 (*Temporary Protection Systems*). The lateral movement of the temporary shoring system should meet Performance Level 2 as



specified in OPSS 539, provided that the existing structure, as well as any adjacent utilities, can tolerate this magnitude of deformation.

### **6.7.3 Subgrade Protection**

The clayey silt till that will be exposed at the foundation subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the drawings and/or with an NSSP. An example NSSP for the concrete working slab is included in Appendix D.

### **6.7.4 Vibration Monitoring During Pile Installation**

Depending on the chosen rehabilitation / replacement option and construction sequence, vibration monitoring may be necessary at the existing structure during driven pile installation, if piles are selected, to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A maximum peak particle velocity (PPV) of 50 mm/s would be appropriate at the existing structure for the rehabilitation option (i.e. if portions of the existing structure and foundations are to remain permanently in place) and a PPV of 100 mm/s would be appropriate at the existing structure for the replacement option (i.e. if the existing structure and foundations are being used temporarily during construction and will eventually be completely removed and replaced); however, this requires further assessment by the structural engineer. The piles further from the existing underpass structure should be driven first, in order to check the vibration level at the existing structure and, if necessary, alter the pile driving criteria for the remaining piles.

In the event that vibration monitoring is determined to be necessary, an example NSSP for such monitoring is provided in Appendix D for inclusion in the Contract Documents.



## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

### 7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, EIT, and reviewed by Mr. Kevin Bentley, P.Eng., an Associate and Senior Geotechnical Engineer at Golder. Mr. Jorge Costa, P.Eng., a Designated MTO Contact and Principal with Golder, conducted an independent review and quality control audit of this report.

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## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

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### Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specifications for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 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 Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls, Abutments, Backfill, Minimum Granular Requirements
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirements



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## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

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### **Construction Design Estimating and Documentation (CDED) Special Provisions (SP)**

SP110S13      Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

### **ASTM International**

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





## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

**TABLE 1- COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES  
REHABILITATION / REPLACEMENT OF HIGHWAY 401 / KEELE STREET UNDERPASS  
G.W.P. 2368-09-00**

Bridge Rehabilitation Alternative	Foundation Option	Advantages	Disadvantages	Relative Costs	Risks/Consequences
1. Full or Partial (i.e. Piers only) Bridge Replacement	Spread footings founded on the clayey silt till.	<ul style="list-style-type: none"> <li>Lower costs than deep foundations</li> <li>Standard construction methods; no specialized construction equipment required.</li> <li>Allows for semi-integral abutment design (in case of full replacement)</li> </ul>	<ul style="list-style-type: none"> <li>Low geotechnical resistances and potential settlement up to 25 mm</li> <li>No advantage in widening the new footings (i.e. reduced SLS geotechnical resistances)</li> <li>Traffic disruption during construction</li> <li>Requirement for temporary supports</li> </ul>	<ul style="list-style-type: none"> <li>least expensive foundation option for the full or partial (i.e. piers only) replacement alternatives</li> <li>Full replacement option more expensive than partial (i.e. piers only) replacement alternative</li> <li>(4.5 m x 22 m x 1 m) at \$600/m<sup>3</sup> = \$60,000/foundation element</li> </ul>	<ul style="list-style-type: none"> <li>Potential significant traffic disruption during construction if construction cannot be restricted to width of highway shoulder</li> </ul>
	Steel H-piles (or steel tube piles) driven to found within 100-blow Sand and Silt deposit	<ul style="list-style-type: none"> <li>Allows for integral or semi-integral abutment design (for full replacement alternative)</li> <li>Provides for higher geotechnical resistances than shallow foundations and minimal settlement</li> </ul>	<ul style="list-style-type: none"> <li>Potential vibrations induced on existing abutment footings from pile driving operations (for the piers replacement-only alternative)</li> <li>Long piles will be required to reach "100-blow" materials</li> <li>Significant traffic disruption during construction with complex construction staging</li> <li>Requirement for temporary supports</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than shallow foundations</li> <li>Full replacement option more expensive than partial (i.e. piers only) replacement alternative</li> <li>(18 steel H-piles x 30 m) at \$250/m = \$135,000/pier + \$70,000/abutment</li> </ul>	<ul style="list-style-type: none"> <li>Significant traffic disruption during construction due to space requirements for pile driving equipment</li> <li>Potential vibrations induced on existing footings (for partial/pier replacement alternative)</li> </ul>
	Caissons founded within 100-blow Sand and Silt deposit	<ul style="list-style-type: none"> <li>Relatively higher bearing resistances than for steel H-piles requires fewer units than H-piles, and minimal settlement</li> </ul>	<ul style="list-style-type: none"> <li>Deep caissons will be required in order to be founded within "100-blow" materials</li> <li>Liner would be required due to soil and groundwater conditions. Permanent liner recommended over temporary liner, to avoid difficulties with withdrawal of temporary liner due to length of caissons and presence of hard/very dense soils near caisson base, and to avoid "necking" of the caissons.</li> <li>Significant traffic disruption during construction with complex construction staging</li> <li>Requirement for temporary supports</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than steel H-piles, plus cost of permanent liner</li> <li>Full replacement option more expensive than partial (i.e. piers only) replacement alternative</li> <li>(4 caissons (36 in. Φ) x 30 m) at \$2,500/m = \$300,000/foundation element</li> </ul>	<ul style="list-style-type: none"> <li>Significant traffic disruption during construction due to space requirements for caisson drilling equipment</li> </ul>



## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

2. Supplement Existing Pier Foundation Resistances	New Spread Footing Section between the two existing pier footings	<ul style="list-style-type: none"> <li>Lower cost than new foundations associated with full or partial replacement of footings replacement alternatives</li> <li>Standard construction method</li> </ul>	<ul style="list-style-type: none"> <li>Lower geotechnical resistance than deep foundations</li> <li>Differential settlement between new and existing footing sections (up to 25 mm)</li> <li>Construction may possibly impact existing foundations</li> <li>Traffic disruption during construction</li> <li>Temporary supports likely required</li> </ul>	<ul style="list-style-type: none"> <li>Considered least expensive option for this bridge rehabilitation alternative</li> <li>(4.5 mx4.5 mx1 m) at \$600/m<sup>3</sup> = \$12,500/pier</li> </ul>	<ul style="list-style-type: none"> <li>Differential settlement between existing/new foundations</li> <li>Traffic disruption during construction</li> <li>Potential impact on existing foundations</li> </ul>
	New Caisson(s) in the Section between the two existing pier footings	<ul style="list-style-type: none"> <li>Provides a higher geotechnical resistance than footing section</li> <li>Minimizes differential settlement between the new and existing foundations</li> </ul>	<ul style="list-style-type: none"> <li>Deep caisson(s) required in order to be founded within "100-blow" materials</li> <li>Higher cost than spread footing section</li> <li>Liner would be required due to soil &amp; groundwater conditions</li> <li>Significant traffic disruption during construction</li> <li>Temporary supports likely required</li> </ul>	<ul style="list-style-type: none"> <li>More expensive than spread footing section</li> <li>(1-36 in. <math>\Phi</math> x 30 m) at \$2,500/m = \$75,000/pier</li> </ul>	<ul style="list-style-type: none"> <li>Significant Traffic disruption during construction due to space required for caisson drilling equipment</li> <li>Potential difficulties drilling caisson between existing footings at each pier</li> </ul>
	Caissons on each side of footings to Support Existing Footings on grade beams or caisson caps	<ul style="list-style-type: none"> <li>Higher bearing resistances</li> </ul>	<ul style="list-style-type: none"> <li>Caissons require to be installed at great depths to support the design loads</li> <li>Liners would be required due to soil &amp; groundwater conditions</li> <li>Significant traffic disruption during construction</li> <li>Temporary supports likely required</li> </ul>	<ul style="list-style-type: none"> <li>Significantly higher cost than the addition of a new footing section or caisson(s) between the two existing footings at each pier location</li> <li>(\$150,000 + Grade Beam + \$75,000 for central caisson)/pier</li> </ul>	<ul style="list-style-type: none"> <li>Significant Traffic disruption during construction due to space required for caisson drilling equipment</li> </ul>
	Micropiles	<ul style="list-style-type: none"> <li>Will provide increased geotechnical resistances</li> <li>Small diameter drilled piles avoid significant disturbance to existing abutment foundations as compared to driving full size piles or larger diameter caissons</li> </ul>	<ul style="list-style-type: none"> <li>Requires specialized contractor and equipment and a site-specific design and construction specification</li> <li>Micropiles may require to be drilled to great depth and/or a significant number of micropiles would be required</li> <li>Significant traffic disruption during construction</li> <li>Temporary supports likely required</li> </ul>	<ul style="list-style-type: none"> <li>Higher costs than new footing or new caisson section</li> <li>((18 piles x 1.5 factor)/3 Footing Factor) x 30 m at \$1,200/m = \$324,000 + design cost/pier</li> </ul>	<ul style="list-style-type: none"> <li>Significant traffic disruption during construction due to space required for drilling and grouting equipment</li> </ul>



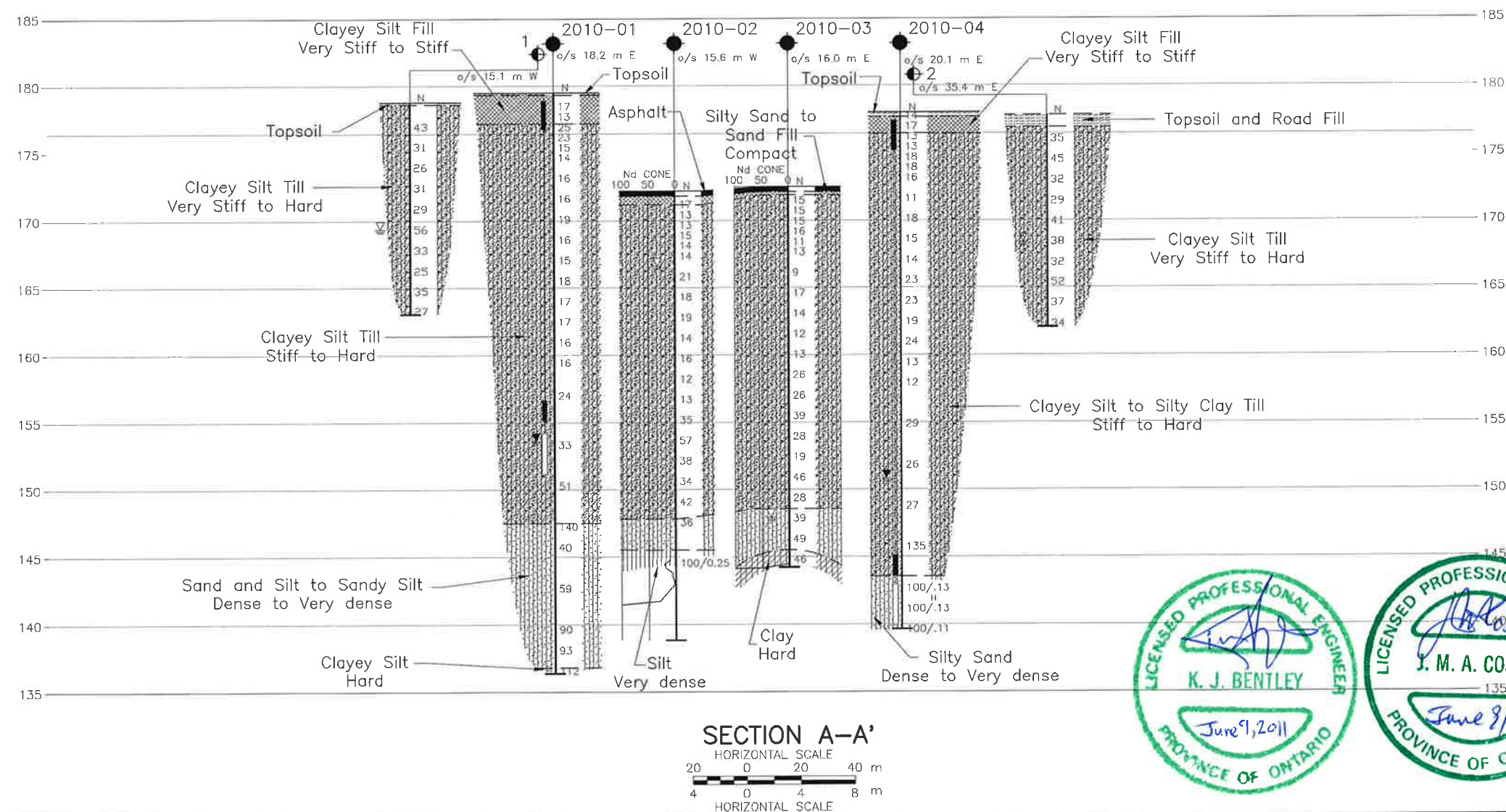
## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

3. Strengthen Pier Superstructure with No Foundation Improvement	Keep existing foundation system in conjunction with implementing a monitoring program and preparing a contingency plan and if possible, minimize additional loads	<ul style="list-style-type: none"><li>• Reduced traffic disruption during construction</li><li>• No additional foundation improvement costs</li></ul>	<ul style="list-style-type: none"><li>• Existing foundations under proposed load conditions do not adhere to the required design Factors of Safety per the CHBDC (i.e. reduced factors of safety) (est. FS = 2.6)</li><li>• Requires implementation of a monitoring program during and after construction to assess movement of the existing foundations</li><li>• Requires development of Review and Alert Levels suitable to ensure the integrity of the structure in the event rehabilitation work has to be suspended</li><li>• Requires a contingency plan, which may need to be put in action, if unacceptable foundation movements are observed.</li><li>• Temporary support may still be required during construction</li></ul>	<ul style="list-style-type: none"><li>• Least expensive rehabilitation alternative. However, if unacceptable foundation movements occur, the contingency plan may need to be implemented, which would make this alternative ultimately most expensive</li><li>• (No Foundations Cost but monitoring program ≈\$750/day for 1 year)</li></ul>	<ul style="list-style-type: none"><li>• MTO acceptance of reduced geotechnical resistance design factor of safety than specified in the CHBDC. Although this risk would be reduced if the additional loads are minimized</li><li>• Potential that some other alternative would still have to be implemented if this alternative fails (i.e. Alert Levels reached and safety of the structure is compromised)</li></ul>
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### NOTES:

1. Shorter friction piles or shorter caissons with variably smaller diameters are also feasible but will provide lower geotechnical resistances and result in a higher number of piles or caissons per foundation element.





NO.	DATE	BY	REVISION		
Geocres No. 30M11-237					
HWY. 401		PROJECT NO. 09-1111-6007		DIST.	
SUBM'D. NK	CHKD. NK	DATE: Jun. 8, 2011		SITE.	
DRAWN: JFC	CHKD. KJB	APPD. JMAC		DWG. 1	



# **APPENDIX A**

## **Records of Boreholes from Current Investigation**



## 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
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
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.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	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 cm<sup>2</sup> 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) 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

Percent 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 (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand





## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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
F	factor of safety
V	volume
W	weight

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\epsilon$	linear strain
$\epsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - \mu$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
$\mu$	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
$\gamma'$	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
$w_l$	liquid limit
$w_p$	plastic limit
$I_p$	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
$C_a$	coefficient of secondary consolidation
$m_v$	coefficient of volume change
$c_v$	coefficient of consolidation
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$T_p, T_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  $\tau = c' + \sigma' \tan \phi'$   
2 shear strength = (compressive strength)/2



**RECORD OF BOREHOLE No 2010-1**

1 OF 4 **METRIC**

PROJECT 09-1111-6007  
 G.W.P. 2368-09-00 LOCATION N 4842588.7 ; E 306315.7 ORIGINATED BY PKS  
 DIST Central HWY 401 BOREHOLE TYPE D-50 Turbo, 210 mm Diameter Hollow Stem Auger COMPILED BY TT  
 DATUM Geodetic DATE October 6,7 and 12, 2010 CHECKED BY NK/HJ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
179.5	GROUND SURFACE						20	40	60	80	100							
0.0	TOPSOIL																	
0.2	Clayey silt, some sand, trace gravel, containing occasional rootlets and asphalt pieces (FILL) Very stiff to stiff Brown Moist		1	SS	17													
			2	SS	13													
177.2																		
2.3	CLAYEY SILT with sand, trace gravel (TILL) Stiff to hard Brown becoming grey below 3.8m Moist		3	SS	25													
			4	SS	23													
			5	SS	15													
			6	SS	14													
			7	SS	16													
			8	SS	16													
			9	SS	19													
			10	SS	16													
			11	SS	15													
			12	SS	18													

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

**RECORD OF BOREHOLE No 2010-1**

2 OF 4 **METRIC**

PROJECT 09-1111-6007  
 G.W.P. 2368-09-00 LOCATION N 4842588.7 ; E 306315.7 ORIGINATED BY PKS  
 DIST Central HWY 401 BOREHOLE TYPE D-50 Turbo, 210 mm Diameter Hollow Stem Auger COMPILED BY TT  
 DATUM Geodetic DATE October 6,7 and 12, 2010 CHECKED BY NK/HJ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)	GR	SA	SI	CL			
								○ UNCONFINED	+ FIELD VANE										● QUICK TRIAXIAL	× REMOULDED	20
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	CLAYEY SILT with sand, trace gravel (TILL) Stiff to hard Brown becoming grey below 3.8m Moist		13	SS	17																
			14	SS	17																
			15	SS	16																
			16	SS	16																
			17	SS	24																
			18	SS	33																

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO.001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT 09-1111-6007		<b>RECORD OF BOREHOLE No 2010-1</b>		3 OF 4 <b>METRIC</b>	
G.W.P. 2368-09-00		LOCATION N 4842588.7 ; E 306315.7		ORIGINATED BY PKS	
DIST Central HWY 401		BOREHOLE TYPE D-50 Turbo, 210 mm Diameter Hollow Stem Auger		COMPILED BY TT	
DATUM Geodetic		DATE October 6,7 and 12, 2010		CHECKED BY NK/HJ	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>						
	--- CONTINUED FROM PREVIOUS PAGE ---																				
147.5	CLAYEY SILT with sand, trace gravel (TILL) Stiff to hard Brown becoming grey below 3.8m Moist						149														
32.0	SAND and SILT to Sandy SILT, trace clay, trace gravel Dense to very dense Grey Moist to wet		20	SS	140		148														
							147														
			21	SS	40		146														
							145														
							144														
							143														
			22	SS	59		142														
							141														
							140														
			23	SS	90		139														
							138														
			24	SS	93		137														
136.8																					
42.7	CLAYEY SILT, some sand, containing occasional silty sand seams		25	SS	112																
136.4	Hard																				
43.1	Grey Wet																				

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO 001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT <u>09-1111-6007</u>		<b>RECORD OF BOREHOLE No 2010-1</b>		4 OF 4 <b>METRIC</b>	
G.W.P. <u>2368-09-00</u>		LOCATION <u>N 4842588.7 ; E 306315.7</u>		ORIGINATED BY <u>PKS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>D-50 Turbo, 210 mm Diameter Hollow Stem Auger</u>		COMPILED BY <u>TT</u>	
DATUM <u>Geodetic</u>		DATE <u>October 6,7 and 12, 2010</u>		CHECKED BY <u>NK/HJ</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
								20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>						
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	END OF BOREHOLE																				
	NOTE:  1. Water level in piezometer at a depth of 27.1 m below ground surface (Elevation 152.4 m) on completion of installation.  2. Water level in piezometer at a depth of 14.4 m below ground surface (Elevation 165.1 m) on Oct. 20, 2010.  3. Water level in piezometer at a depth of 25.9 m below ground surface (Elevation 153.6 m) on June 3, 2011.																				

PROJECT 09-1111-6007		RECORD OF BOREHOLE No 2010-2		1 OF 3	METRIC
G.W.P. 2368-09-00		LOCATION N 4842539.0 ;E 306289.7		ORIGINATED BY PKS/MS	
DIST Central HWY 401		BOREHOLE TYPE D-120 Track-mount, 108 mm Diameter Solid Stem Auger		COMPILED BY TT	
DATUM Geodetic		DATE September 29-30, 2010		CHECKED BY NK/HJ	


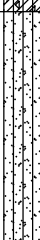

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT  NATURAL MOISTURE CONTENT  LIQUID LIMIT	UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100					SHEAR STRENGTH kPa		W <sub>P</sub> W W <sub>L</sub>	
								○ UNCONFINED + FIELD VANE					● QUICK TRIAXIAL × REMOULDED		WATER CONTENT (%)	
172.2	ROAD SURFACE						172									
0.0	ASPHALT						171									
171.8							170									
0.4	Sand, some gravel, some silt (FILL) Compact Brown Moist		1	SS	17		169									
171.1	CLAYEY SILT with to some sand, trace to some gravel (TILL) Stiff to hard Grey Moist		2	SS	13		168									
1.1			3	SS	13		167									
			4	SS	15		166									
			5	SS	14		165									
			6	SS	14		164									
							163									
			7	SS	21		162									
							161									
			8	SS	18		160									
							159									
			9	SS	19		158									
			10	SS	14											
			11	SS	16											
			12	SS	12											

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

MIS-MTO 001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT <u>09-1111-6007</u>		<b>RECORD OF BOREHOLE No 2010-2</b>		2 OF 3 <b>METRIC</b>	
G.W.P. <u>2368-09-00</u>		LOCATION <u>N 4842539.0 ; E 306289.7</u>		ORIGINATED BY <u>PKS/MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>D-120 Track-mount, 108 mm Diameter Solid Stem Auger</u>		COMPILED BY <u>TT</u>	
DATUM <u>Geodetic</u>		DATE <u>September 29-30, 2010</u>		CHECKED BY <u>NK/HJ</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	w <sub>p</sub> w w <sub>L</sub>	WATER CONTENT (%)			GR	SA	SI	CL	
										10 20 30								
	--- CONTINUED FROM PREVIOUS PAGE ---																	
	CLAYEY SILT with to some sand, trace to some gravel (TILL) Stiff to hard Grey Moist		13	SS	13		157											
								156										
			14	SS	35			155										
								154										
	Silty sand seam between 18.5 m and 19.0 m depth		15	SS	57			153										
								152										
			16	SS	38			151										
								150										
							149											
							148											
147.8							147											
24.4	SAND AND SILT, trace clay and gravel Dense Grey Wet		19	SS	36		146											
								145										
145.5							144											
26.7	SILT, trace sand and clay Very dense Grey Wet						143											
								142										
144.4							141											
27.8	END OF BOREHOLE START OF DCPT at Elevation 144.8 m						140											

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO 001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT <u>09-1111-6007</u>		<b>RECORD OF BOREHOLE No 2010-2</b>		3 OF 3 <b>METRIC</b>	
G.W.P. <u>2368-09-00</u>		LOCATION <u>N 4842539.0 ; E 306289.7</u>		ORIGINATED BY <u>PKS/MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>D-120 Track-mount, 108 mm Diameter Solid Stem Auger</u>		COMPILED BY <u>TT</u>	
DATUM <u>Geodetic</u>		DATE <u>September 29-30, 2010</u>		CHECKED BY <u>NK/HJ</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL LIMIT   MOISTURE   CONTENT LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W <sub>p</sub>	W	W <sub>L</sub>		
								○ UNCONFINED   + FIELD VANE	WATER CONTENT (%)					
							● QUICK TRIAXIAL   × REMOULDED							
	--- CONTINUED FROM PREVIOUS PAGE ---							20   40   60   80   100						
										</				



1 OF 3 **METRIC**

CHECKED BY NK/HJ

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



## 2 OF 3 METRIC

PROJECT 09-1111-6007		RECORD OF BOREHOLE NO 2010-3		2 OF 3 METRIC	
G.W.P. 2368-09-00		LOCATION N 4842502.5 ;E 306327.7		ORIGINATED BY MS	
DIST Central HWY 401		BOREHOLE TYPE D-120 Track-mount, 57 mm Diameter Solid Stem Auger		COMPILED BY TT	
DATUM Geodetic		DATE October 1, 2010		CHECKED BY NK/HJ	

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+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE

MIS-MTO 001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT <u>09-1111-6007</u>		<b>RECORD OF BOREHOLE No 2010-3</b>		3 OF 3 <b>METRIC</b>	
G.W.P. <u>2368-09-00</u>		LOCATION <u>N 4842502.5 ; E 306327.7</u>		ORIGINATED BY <u>MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>D-120 Track-mount, 57 mm Diameter Solid Stem Auger</u>		COMPILED BY <u>TT</u>	
DATUM <u>Geodetic</u>		DATE <u>October 1, 2010</u>		CHECKED BY <u>NK/HJ</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE   LIQUID CONTENT   LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		GR	SA	SI	CL	
								○ UNCONFINED	+	FIELD VANE	● QUICK TRIAXIAL	×	REMOULDED	WATER CONTENT (%)							
	--- CONTINUED FROM PREVIOUS PAGE ---  DCPT REFUSAL at Elevation 144.2 m  NOTE:  1. Water level in open borehole at at depth of 24.6 m below ground surface (Elevation 147.9 m) on completion of drilling.																				

MIS-MTO.001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT 09-1111-6007		<b>RECORD OF BOREHOLE No 2010-4</b>		1 OF 3 <b>METRIC</b>	
G.W.P. 2368-09-00		LOCATION N 4842462.1 ; E 306338.6		ORIGINATED BY MS	
DIST Central HWY 401		BOREHOLE TYPE D-90 Track-mount, 108 mm Diameter Hollow Stem Auger		COMPILED BY TT	
DATUM Geodetic		DATE October 6-7, 2010		CHECKED BY NK/HJ	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED						
178.0	GROUND SURFACE														
0.0	TOPSOIL														
177.7															
0.3	Silty clay, trace to some sand and gravel, containing rootlets (FILL) Stiff to very stiff Brown Moist		1	SS	14						○				
			2	SS	17										
176.5															
1.5	CLAYEY SILT to SILTY CLAY with sand, trace to some gravel (TILL) Stiff to very stiff to Hard below about 32 m depth Brown becoming grey at depth of 4.4 m Moist		3	SS	13						○				
			4	SS	13										
	Containing sand pockets to a depth of 2.9m		5	SS	18						○				
			6	SS	18						○	—			
			7	SS	16						○	—			
			8	SS	11										
			9	SS	18										
			10	SS	15						○				
			11	SS	14						○	—			
			12	SS	23										
			13	SS	23						○				

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO 001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

PROJECT <u>09-1111-6007</u>		<b>RECORD OF BOREHOLE No 2010-4</b>		2 OF 3 <b>METRIC</b>	
G.W.P. <u>2368-09-00</u>		LOCATION <u>N 4842462.1 ; E 306338.6</u>		ORIGINATED BY <u>MS</u>	
DIST <u>Central</u> HWY <u>401</u>		BOREHOLE TYPE <u>D-90 Track-mount, 108 mm Diameter Hollow Stem Auger</u>		COMPILED BY <u>TT</u>	
DATUM <u>Geodetic</u>		DATE <u>October 6-7, 2010</u>		CHECKED BY <u>NK/HJ</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × REMOULDED	w <sub>p</sub>	w	w <sub>L</sub>		
	--- CONTINUED FROM PREVIOUS PAGE ---													
	CLAYEY SILT to SILTY CLAY with sand, trace to some gravel (TILL) Stiff to very stiff to Hard below about 32 m depth Brown becoming grey at depth of 4.4 m Moist		14	SS	19									
			15	SS	24									
			16	SS	13									
			17	SS	12									
			18	SS	29									
			19	SS	26									
			20	SS	27									

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

MIS-MTO.001 09-1111-6007.GPJ GAL-MISS.GDT 6/8/11

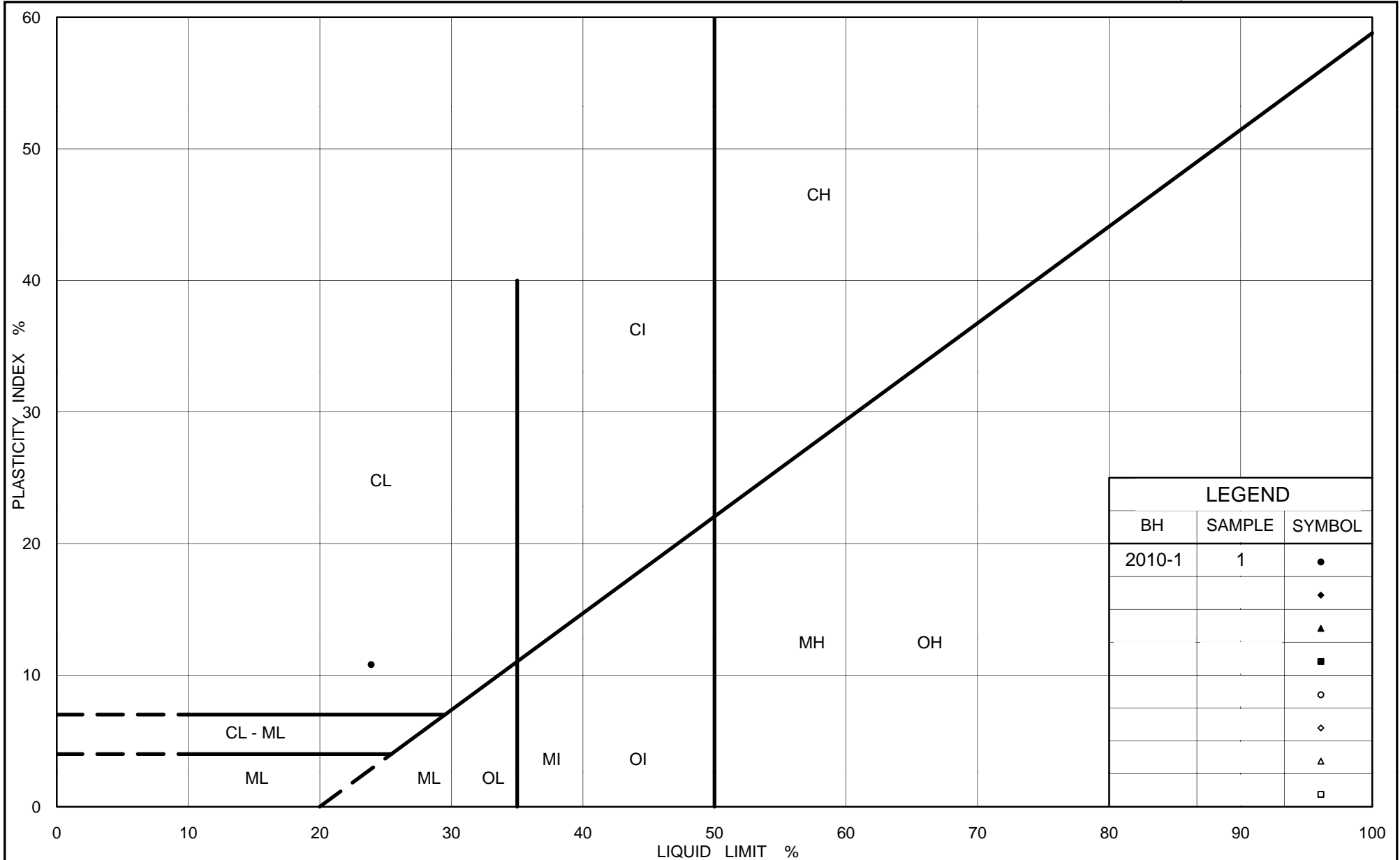
PROJECT		RECORD OF BOREHOLE No 2010-4		3 OF 3 METRIC							
G.W.P. 09-1111-6007		LOCATION N 4842462.1 ; E 306338.6		ORIGINATED BY MS							
DIST Central HWY 401		BOREHOLE TYPE D-90 Track-mount, 108 mm Diameter Hollow Stem Auger		COMPILED BY TT							
DATUM Geodetic		DATE October 6-7, 2010		CHECKED BY NK/HJ							
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES	SHEAR STRENGTH kPa			
	--- CONTINUED FROM PREVIOUS PAGE ---										
	CLAYEY SILT to SILTY CLAY with sand, trace to some gravel (TILL) Stiff to very stiff to Hard below about 32 m depth Brown becoming grey at depth of 4.4 m Moist										
	Containing rock fragments from 32.0 m to 32.6 m		21	SS	135						
143.6											
34.4	Silty SAND, trace clay Very dense Wet Grey		22	SS	100/13						0 67 29 4
			23	SS	100/13						
139.6			24	SS	100/11						
38.4	END OF BOREHOLE										
	Notes: 1. Water level in piezometer at a depth of 27.1 m below ground surface (Elevation 150.9 m) on completion of installation. 2. Water level in piezometer at a depth of 27.8 m below ground surface (Elevation 150.2 m) on October 20, 2010. 3. Water level in piezometer at a depth of 27.2 m below ground surface (Elevation 150.8 m) on June 3, 2011.										



# **APPENDIX B**

## **Laboratory Test Results**





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## PLASTICITY CHART

### Clayey Silt Fill

Figure No. B1

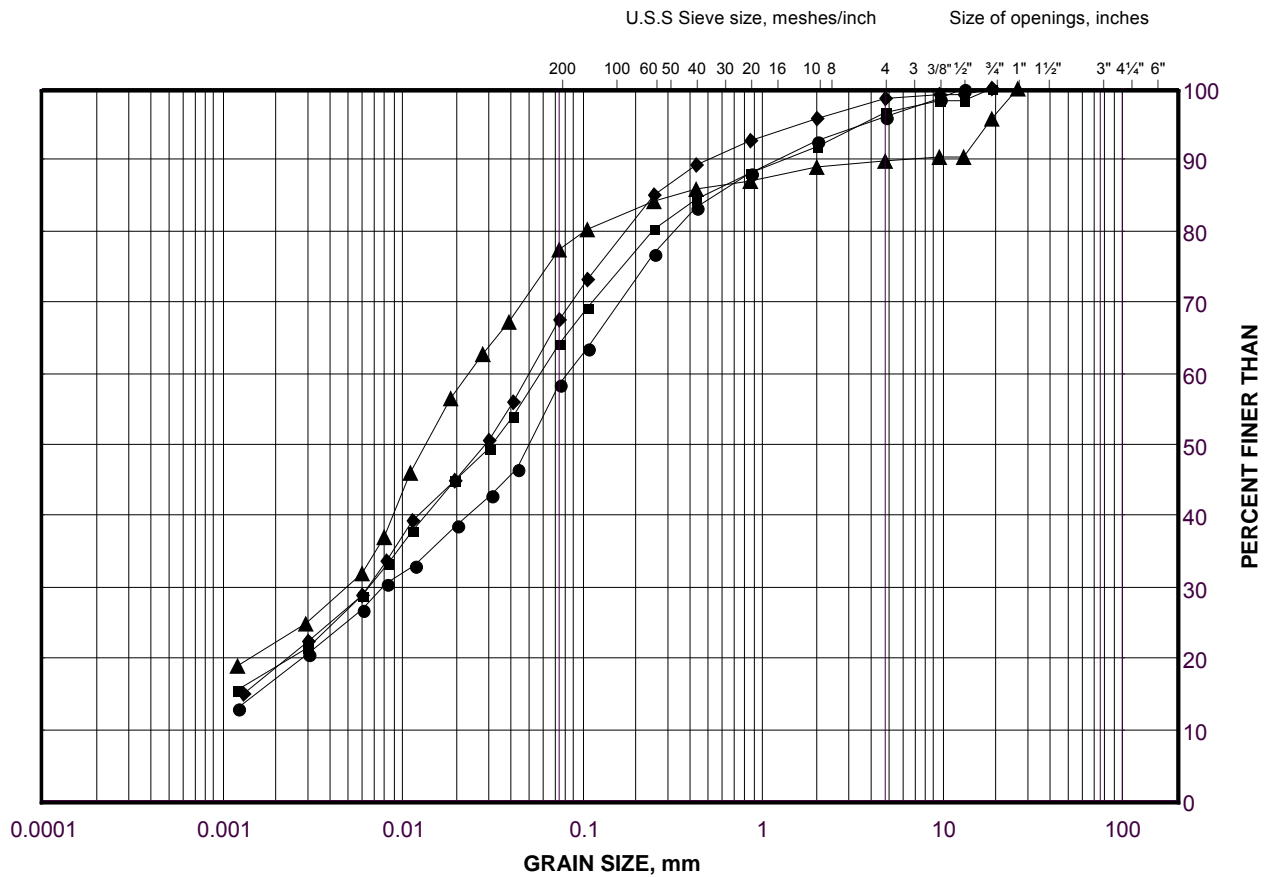
Project No. 09-1111-6007

Checked By:

# GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B2A



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2010-1	10	168.5
■	2010-1	13	164.0
◆	2010-2	3	169.6
▲	2010-2	9	162.8

Project Number: 09-1111-6007

Checked By: \_\_\_\_\_

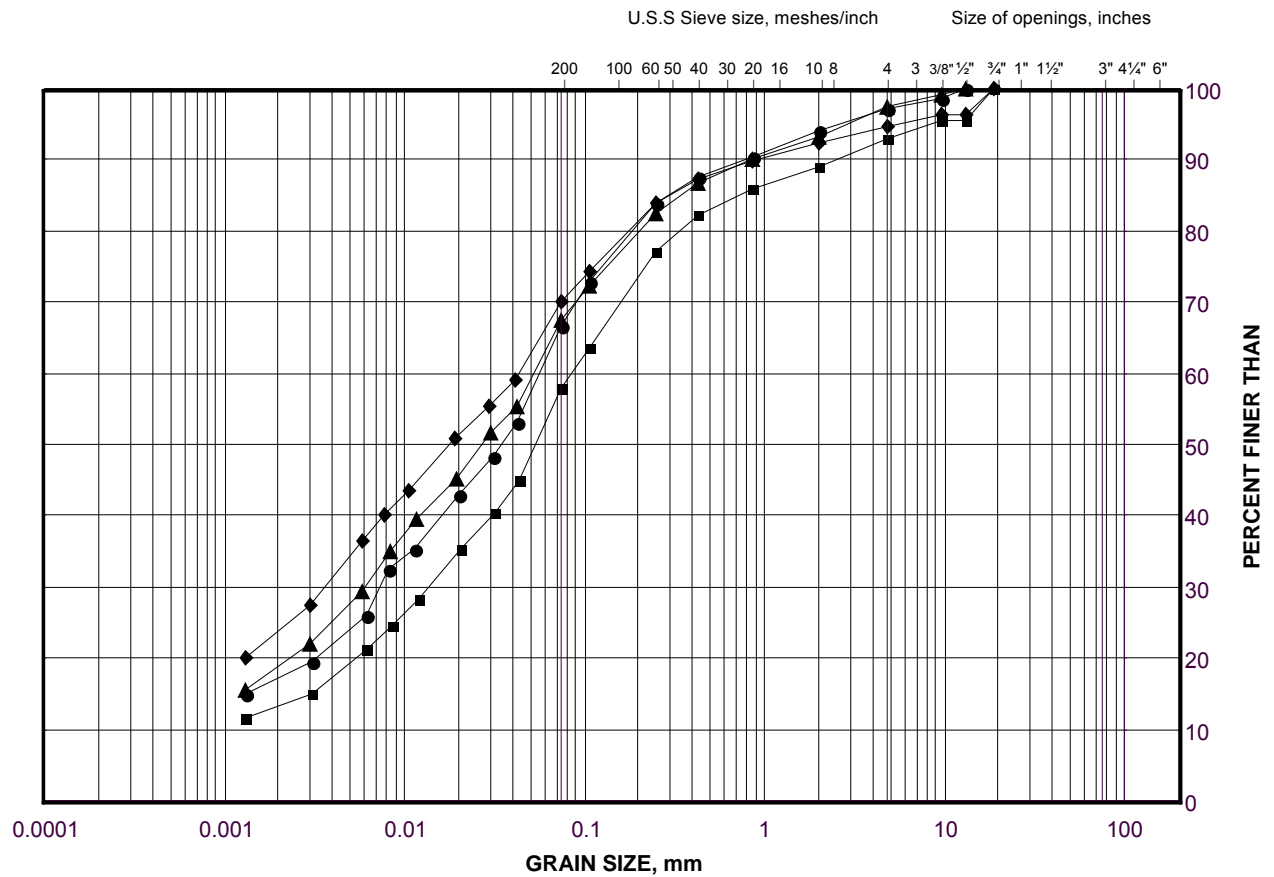
**Golder Associates**

Date: 03-Mar-11

# GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B2B



## LEGEND

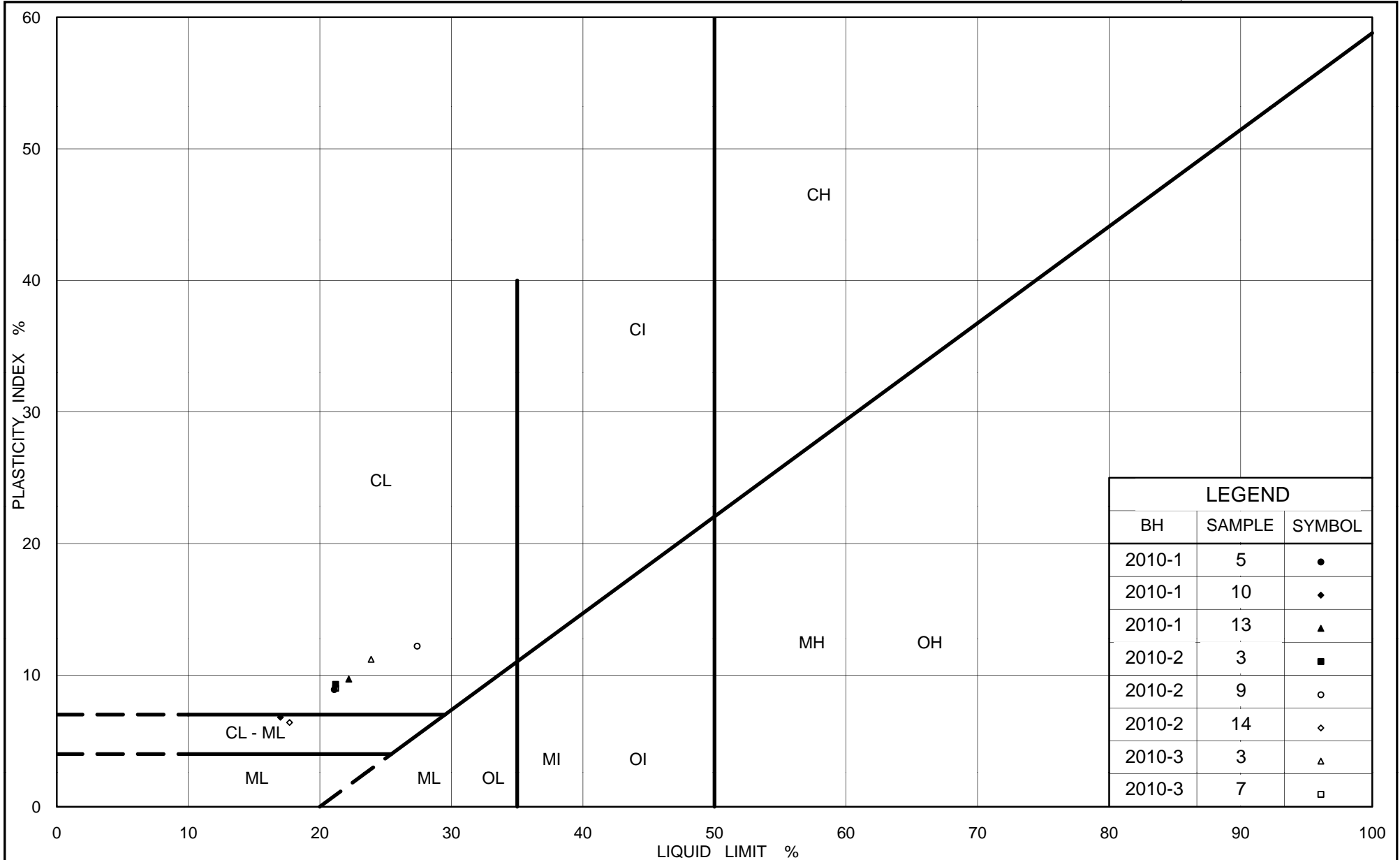
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2010-4	16	159.4
■	2010-3	16	152.4
◆	2010-4	7	173.1
▲	2010-3	7	166.1

Project Number: 09-1111-6007

Checked By: \_\_\_\_\_

**Golder Associates**

Date: 02-Mar-11



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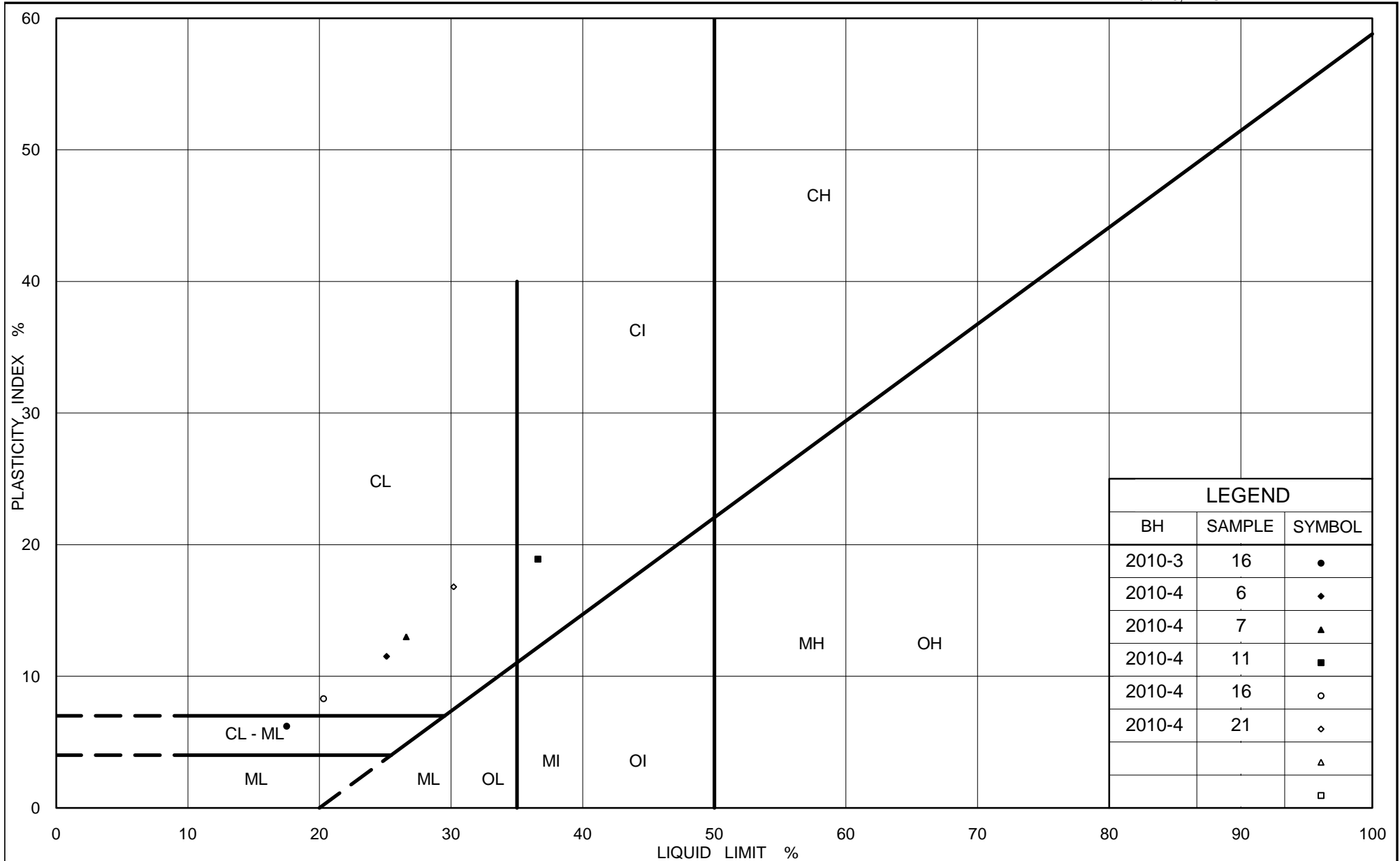
## PLASTICITY CHART

### Clayey Silt Till

Figure No. B3A

Project No. 09-1111-6007

Checked By:



Ministry of Transportation

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## PLASTICITY CHART

### Clayey Silt to Silty Clay/Till

Figure No. B3B

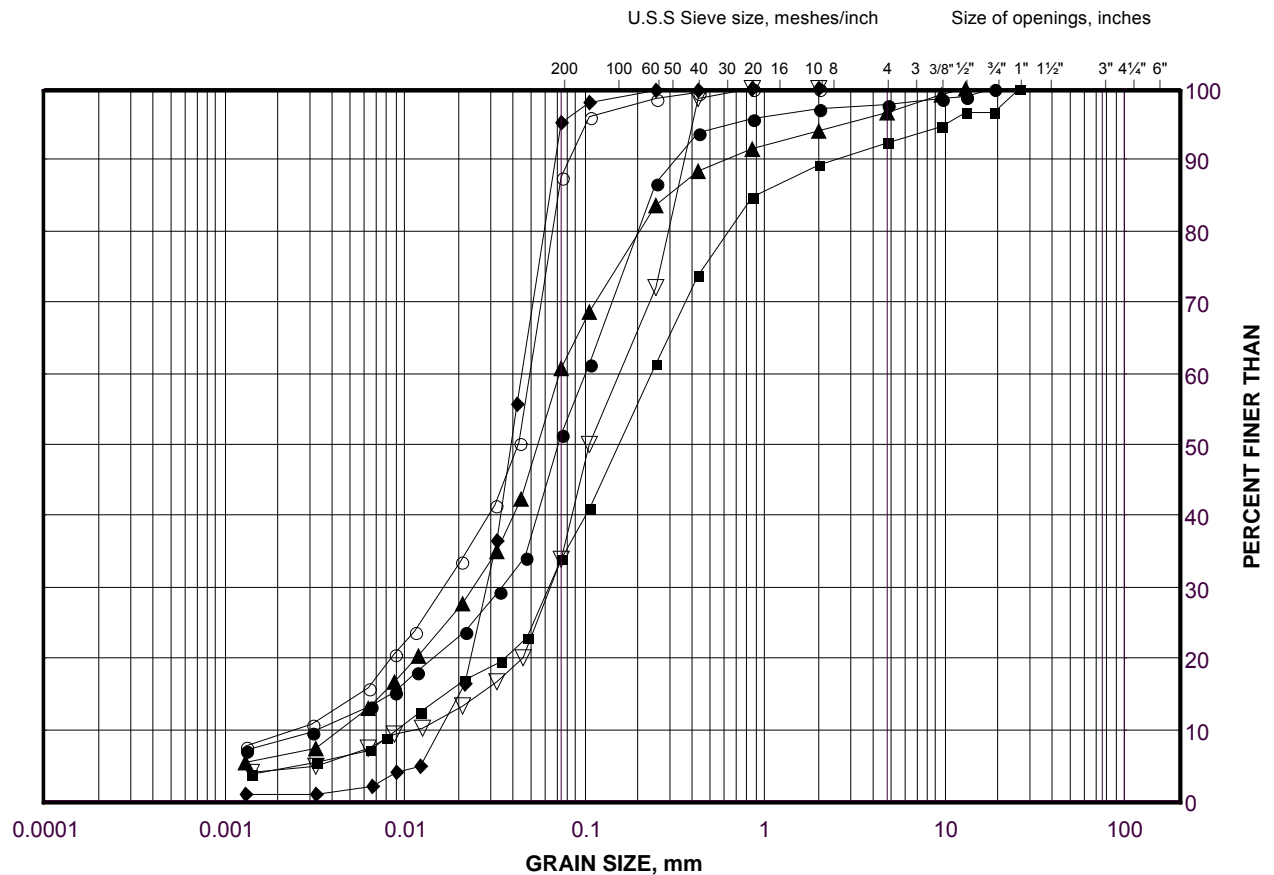
Project No. 09-1111-6007

Checked By:

# GRAIN SIZE DISTRIBUTION

Sand and Silt to Sandy Silt to Silt

FIGURE B4



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

## LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	2010-2	19	147.5
■	2010-3	20	146.3
◆	2010-2	20	144.6
▲	2010-1	21	145.7
▽	2010-4	22	142.8
○	2010-1	24	138.0

Project Number: 09-1111-6007

Checked By: \_\_\_\_\_

**Golder Associates**

Date: 10-Jan-11

LEGEND		
BH	SAMPLE	SYMBOL
2010-1	25	●
2010-3	21	◆
		▲
		■
		○
		◇
		△
		□



Ontario

## PLASTICITY CHART

### Clayey Silt and Clay

Figure No. B5

Project No. 09-1111-6007

Checked By:





# **APPENDIX C**

## **Records of Boreholes from Previous Investigation**

RECORD OF BOREHOLE NO. 1

JOB 63-F-87 LOCATION Stn. 229+23 and 227' to rt. of E. Hwy. 401 ORIGINATED BY B.M.G.  
 W.P. 231-60 BORING DATE Aug. 13, 1963. COMPILED BY B.M.G.  
 DATUM G.S.C. BOREHOLE TYPE Pennsylvania Auger - 3 1/2" Ø CHECKED BY A.G.S.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE			LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT	20 40 60 80 100	WP	WL	W			
						SHEAR STRENGTH P.S.F.			WATER CONTENT % 10 20 30					
586.7	Groundlevel					590								
0.6	Topsoil													
	Clayey silt-some sand and gravel.  (Glacial till).  V. stiff to hard.  Brown changing to grey at Elev. 575.7		1	SS	43	580								
			2	SS	31									
			3	SS	26	570								
			4	SS	31									
			5	SS	29	560								
			6	SS	56									
			7	SS	33	550								
			8	SS	25									
			9	SS	35	540								
			10	SS	27									
535.2														
51.6	End of borehole.					530								

WL  
Elev. 555.7

# RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 63-F-87

LOCATION Stn. 228+40 and 219' to left of E. Hwy. 401

ORIGINATED BY B.M.G.

W.P. 231-60

BORING DATE Aug. 14, 1963.

COMPILED BY B.M.G.

DATUM G.S.C.

BOREHOLE TYPE Pennsylvania Auger - 3 1/2" Ø

CHECKED BY A.G.S.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT — WL PLASTIC LIMIT — WP WATER CONTENT — W			BULK DENSITY P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BLOWS / FOOT					WATER CONTENT %				
							20	40	60	80	100	WP	W			WL
							SHEAR STRENGTH P.S.F.									
583.3	Groundlevel															
	Topsoil & road fill.															
3.0	Clayey silt - some sand and gravel (Glacial till)  V. stiff to hard.  Brown changing to grey at Elev. 570		1	SS	35	580										
			2	SS	45	570										
			3	SS	32											
			4	SS	29	560										
			5	SS	41											
			6	SS	38	550										
			7	SS	32											
			8	SS	52	540										
			9	SS	37											
			10	SS	34											
31.8																
51.6	End of borehole.					530										

WD

Elev. 552.3

WL  
Elev. 552.3



# **APPENDIX D**

## **Non-Standard Special Provisions**



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## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

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### CONCRETE WORKING SLAB – Item No.

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#### Special Provision

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The subgrade for the Highway 401-Keele Street structure foundations will be susceptible to disturbance and softening/loosening from construction traffic and ponded water. Within four hours following inspection and approval of the prepared subgrade, a concrete working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade.

The concrete shall have a compressive strength of at least 20 MPa, and be placed in accordance with OPSS 904.

### BASIS OF PAYMENT

Payment at the contract price for the above tender item shall include full compensation for all labour and materials to complete the work.

### END OF SECTION



## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

### **VIBRATION MONITORING - Item No.**

Special Provision

#### ***Scope***

This special provision describes requirements for vibration monitoring during piling installation works for the Rehabilitation of the Highway 401 Keele Street Underpass structure.

#### ***References***

The subsurface conditions at the site are described in the following Foundation Investigation Report for G.W.P. 2368-09-00:

Foundation Investigation Report, Keele Street Underpass, Highway 401 EBL Collector Rehabilitation from Jane Street to Avenue Road, Toronto, Ontario, G.W.P. 2368-09-00.

#### ***Definitions***

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the contract. The QVE shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificates of conformance.

#### ***Submission Requirements***

The Contractor/QVE shall submit details of the vibration monitoring plan to the Contract Administrator for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibration monitoring specialist.
- Details regarding proposed instrumentation.
- Proposed location of instruments on existing Highway 401 structures.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

#### ***Monitoring***

The vibration monitoring equipment shall be placed on the existing Highway 401 Keele Street structure, as close as possible to the piling works. The Contractor/QVE shall take readings on the existing structures during driving of each pile, starting with the pile furthest away from the Highway 401 Keele Street structure for each widening area.

The vibrations measured on the existing structure shall not exceed 50 mm/s (peak particle velocity) for permanent components of the bridge that will remain as part of the bridge rehabilitation option. For portions of



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## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

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the bridge that will eventually be removed and replaced as part of the bridge replacement option, vibration levels shall not exceed 100 mm/s (peak particle velocity).

The results shall be submitted to the Contract Administrator after each pile has been driven, prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next pile(s) with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations at the existing structures are within acceptable levels. The above process must be repeated for each pile.

### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

### **END OF SECTION**



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**CSP FOR INTEGRAL ABUTMENTS – Item No**

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Special Provision

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***SCOPE***

This specification covers the requirements for the installation of the corrugated steel pipes (CSPs) at the integral abutments.

***SUBMISSION AND DESIGN REQUIREMENTS***

All submissions shall bear the seal and signature of an Engineer.

At least two weeks prior to commencement of installation of the abutment piles, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times. Working drawings shall include at least the following:

1. Layout and elevations of the CSPs;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points;
3. Construction sequence and details;
4. Source of the sand fill, and description of placing methods and equipment;
5. Location and details of all temporary bracing and spacers for the piles and CSPs;
6. Method for preventing water and debris from entering the CSP prior to placing sand; and
7. Method for preventing concrete from abutment pours from entering the CSPs during placement.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including spacers required to maintain the piles, CSP spacing and abutment stems in their specified positions through all stages of construction until the CSPs have been backfilled. All temporary bracing shall be removed.

***MATERIAL***

***Corrugated Steel Pipe***

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM#4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

CSPs shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSPs will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSPs shall be in accordance with the manufacturer's recommendations. Damaged CSPs shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSPs shall be repaired with two coats of zinc-rich paint.





## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

### *Sand Fill*

The sand fill for backfilling the CSP shall meet the gradation requirements of Table 1 below:

**Table 1 – Sand Fill Gradation Requirements**

MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100%
600 $\mu\text{m}$	#30	80% to 100%
425 $\mu\text{m}$	#40	40% to 80%
250 $\mu\text{m}$	#60	5% to 25%
150 $\mu\text{m}$	#100	0% to 6%

### **CONSTRUCTION**

The sequence of construction shall be in accordance with the working drawings and as follows, unless otherwise approved:

1. Construct levelling pad and place CSPs and spacers.
2. Install piles by driving to design criteria.
3. Place loose sand into 600 diameter CSP.
4. Remove temporary spacers.

The CSP shall be positioned such that the piles are centrally positioned within the CSP. Temporary blocking and bracing shall be used to hold the CSP in position.

The Contractor shall ensure the full perimeters of the tops of all CSPs at each abutment are at the elevation and orientation shown on the working drawings.

The CSP at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	+/- 50 mm
Maximum deviation of any point on the top perimeter of the CSP from the specified elevation	+/- 10 mm



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## FOUNDATION REPORT - KEELE STREET UNDERPASS, HIGHWAY 401 EBC REHABILITATION

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The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and pile. No additional compaction effort other than the action of placing the sand itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP.

### ***BASIS OF PAYMENT***

Payment at the contract price for the above tender item shall include all labour, equipment and material required to do the work.

**END OF SECTION**

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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