



June 8, 2017

FOUNDATION INVESTIGATION AND DESIGN REPORT

**Deformation of Gabion Wall
Structure Site No. 21-647/W
Highway 401 and Graham Creek
Clarington, Ontario
W.O. 2016-11028**

Submitted to:

Ministry of Transportation, Ontario
Foundation Engineering Section
145 Sir William Hearst Avenue
Downsview, Ontario M3M 0B6



REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
DEFORMATION OF GABION WALL
STRUCTURE SITE NO. 21-647/W
HIGHWAY 401 AND GRAHAM CREEK
CLARINGTON, ONTARIO
W.O. 2016-11028**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by the Ministry of Transportation, Ontario (MTO) under MTO's Eastern Region Foundation Engineering Retainer (Agreement No. 4014-E-0012) to provide foundation engineering services to assess the deformation of a gabion basket retaining wall along the north side of Highway 401 at Graham Creek in Clarington, Ontario (Structure Site No. 21-647/W).

The terms of reference and scope of work for the foundation investigation are outlined in MTO Work Item Order Form No. 9 of Agreement No. 4014-E-0012, dated July 21, 2016 and in Golder's Understanding of Scope document submitted on July 25, 2016. Authorization to proceed was received from MTO via the executed Work Item Quote Form No. 9 on July 27, 2016.

This report presents the results of the investigation carried out by Golder at the site. The borehole results from the current investigation have been compared to and supplemented by previous geotechnical investigation data at this site.¹

2.0 SITE AND PROJECT DESCRIPTION

The site is located on the north side of Highway 401, approximately 550 m east of Mill Street in Clarington, Ontario, where Graham Creek crosses under the highway via a culvert (refer to the Key Plan on Drawing 1). A section of gabion basket retaining wall extends eastward from the east edge of the culvert along the north toe of the Highway 401 embankment, immediately adjacent to Graham Creek. The configuration of the embankment, retaining wall and creek is shown in the following photograph:



Photograph A1: Looking west along gabion wall, along north toe of Highway 401 embankment.

¹ Soil Site Investigation – Graham Creek – Hwy. No. 401 Bridge Near Newcastle, Ontario by E.M. Peto Associates Ltd., dated May 3, 1957. GEOCREs No. 30M15-38.



The Highway 401 embankment is approximately 10 m high, with the pavement surface at approximately Elevation 91 m at the north shoulder of the westbound lanes. The elevation of the natural ground surface at the toe of the embankment adjacent to Graham Creek ranges from approximately Elevation 81 m to 82 m, with the creek channel at approximately Elevation 80.8 m. The creek channel is slightly deeper at the base of the deformed gabion basket retaining wall. The water level in the creek was at approximately Elevation 81.1 m at the time of Golder's investigation in August and September 2016, with the water approximately 0.3 m deep east of the deformed wall section, and up to about 0.6 m deep in front of the deformed wall section.

The gabion basket retaining wall extends along the south (left) bank of the creek for a length of approximately 35 m, from the east side of the arch culvert to a grassy/treed area east of the culvert. North of Highway 401, Graham Creek generally flows from north to south; as the creek approaches the Highway 401 embankment, it changes flow direction toward the west (roughly parallel to the gabion wall), and then southwest as it continues through the arch culvert under Highway 401.

3.0 INVESTIGATION PROCEDURES

The field work was carried out on August 22, 2016, during which time a total of three boreholes (designated Boreholes 16-1, 16-2 and 16-2B) were advanced on the north side of Highway 401, east of the Graham Creek arch culvert: one borehole at the crest of the embankment on the right shoulder of the westbound lanes; and two boreholes at the toe of the embankment beyond the east end of the gabion basket retaining wall, at an accessible and safe drilling location. The locations of these boreholes, together with the location of one selected borehole advanced as part of the previous investigation, are shown on Drawing 1. The results from the current investigation are generally consistent with the results from the previous investigations, and only the previous borehole closest to the gabion basket wall (Borehole 38-6 from GEOCRETS No. 30M15-38) is included in this report and on the interpreted stratigraphic sections.

Borehole 16-1 was advanced through the Highway 401 shoulder with a truck-mounted drill rig supplied and operated by Fisher Environmental Drilling of Markham, Ontario. This borehole was advanced through the overburden using 160 mm diameter hollow stem augers, and into the bedrock using an NQ-sized core barrel. Boreholes 16-2 and 16-2B were advanced using portable drilling equipment supplied and operated by Ohlmann Geotechnical Services (OGS) Inc. of Ottawa, Ontario. Soil samples were obtained at 0.75 m and 1.5 m depth intervals in Borehole 16-1, and continuous sampling was completed in Boreholes 16-2 and 16-2B, using a nominal 50 mm outside diameter split-spoon sampler; the sampler was driven by an automatic hammer in Borehole 16-1 and by a manual hammer in Boreholes 16-2 and 16-2B in accordance with the Standard Penetration Test (SPT) procedure. Borehole 16-1 was advanced to a depth of 17.3 m (including bedrock coring) and Boreholes 16-2 and 16-2B were advanced to refusal at depths of 2.9 m and 3.0 m, respectively, below ground surface.

The open boreholes were backfilled with bentonite upon completion in accordance with Ontario Regulation 903 (as amended). The water levels in the open boreholes were observed during the drilling operations and are described on the borehole records in Appendix A.

The field work was observed by a member of Golder's engineering staff, who located the boreholes, arranged for the clearance of underground services, observed the drilling, sampling and in situ testing operations, logged the boreholes and examined the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers and core boxes, labelled and transported to Golder's geotechnical laboratory in



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Mississauga, where the samples underwent further visual examination and classification testing (water contents, grain size distributions and Atterberg limits on selected soil samples, and an unconfined compressive strength test and point load index tests on selected samples of the bedrock core). All of the laboratory tests were carried out to MTO LS and/or ASTM standards, as appropriate. The results of the geotechnical laboratory testing are included on the borehole records and in Appendix B.

The as-drilled borehole locations and elevations were measured relative to identifiable site features. The borehole locations provided on the borehole records and on Drawing 1 are positioned using MTM NAD83 (Zone 17) northing and easting coordinates, and the ground surface elevations are referenced to geodetic datum. The borehole locations, ground surface elevations and drilled depths are summarized below.

Borehole Number	Location (MTM NAD83)		Ground Surface Elevation (m)	Borehole Depth (m)	Drilling Method
	Northing (m)	Easting (m)			
16-1	4864003.9	378831.6	91.0	17.3	Power Auger / NQ Coring
16-2	4864046.7	378835.8	80.9	2.9	Portable Equipment
16-2B	4864047.7	318835.8	80.9	3.0	Portable Equipment

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This section of Highway 401 at Graham Creek is located within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984)² and *Urban Geology of Canadian Cities* (Brennand, 1998)³. The Iroquois Plain extends around the western shores of Lake Ontario. The Plain is comprised of the flat to undulating lakebed and beaches of the former glacial Lake Iroquois, which occupied this area during the last glacial recession.

The surficial soils in this area of the Iroquois Plain are typically comprised of glaciolacustrine clays, silts and sands to gravelly sands, which are underlain by an extensive till deposit that is mapped in this area as the Bowmanville Till. Within the area approximately bounded by Holt Road and Morgan's Road, the surficial glaciolacustrine deposits are absent or of limited thickness and the Bowmanville Till unit is frequently present immediately below the ground surface. Between these limits, an extensive surficial deposit of clayey silt to silty clay is present over the Bowmanville Till (Brennand, 1998). More recent alluvial deposits of gravel, sand, silt and/or clay are present in the valleys associated with Bowmanville Creek, Soper Creek, Wilmot Creek and Graham Creek. The overburden soils are underlain by limestone bedrock of the Lindsay Formation, Simcoe Group.

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes advanced as part of the current investigation, together with the results of in situ and laboratory testing, are presented on the borehole

² Chapman, L.J., and Putnam, D.F., 1984. *The Physiography of Southern Ontario*, 3rd Edition. Ontario Geological survey, Special Volume 2. Ontario Ministry of Natural Resources.

³ Brennand, T.A., 1998. Urban Geology note: Oshawa, Ontario. In P.F. Karrow and O.L. White (Eds.), Geological Association of Canada, Special Paper 42: *Urban Geology of Canadian Cities*, pp. 353-364.



records and on the laboratory test summary figures provided in Appendices A and B, respectively. The record for Borehole 38-6 from the 1957 investigation is also included in Appendix A.

The stratigraphic boundaries shown on the borehole records are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions in the area of the gabion basket wall at the Highway 401-Graham Creek crossing consist of cohesive embankment fill underlain by clayey silt till and silty sand to gravelly sand till. A deposit of sand and gravel is present within the creek bed adjacent to the gabion basket wall, and a layer of sandy silt is present immediately below ground surface near the east end of the wall, with both of these units underlain by silty sand to gravelly sand till. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt

A 250 mm thick layer of asphalt was encountered at ground surface in Borehole 16-1.

4.2.2 Topsoil

An approximately 150 mm thick layer of topsoil was encountered at ground surface of Boreholes 16-2 and 16-2B.

4.2.3 Clayey Silt to Silty Clay Fill

A 6.9 m thick deposit of clayey silt to silty clay fill was encountered below the asphalt in Borehole 16-1 at a depth of 250 mm below ground surface (Elevation 90.7 m). The clayey silt to silty clay fill contains some sand to sandy, trace gravel, and trace organics were noted to be present throughout the fill deposit.

An Atterberg limits test was carried out on one sample of sandy clayey silt fill and measured a liquid limit of 17 per cent, a plastic limit of 10 per cent and a plasticity index of about 7 per cent. The test result, which is plotted on a plasticity chart on Figure B1 in Appendix B, indicates that the material is a clayey silt of low plasticity. The result of a grain size distribution test completed on one sample of sandy clayey silt fill is shown on Figure B2 in Appendix B. The water content measured on three samples of the fill material ranges from 14 to 20 per cent.

The measured SPT "N"-values within the clayey silt to silty clay fill range from 5 blows to 7 blows per 0.3 m of penetration, suggesting a firm consistency.

4.2.4 Sandy Silt

A 0.4 m thick layer of sandy silt was encountered below the topsoil in Boreholes 16-2 and 16-2B, beyond the toe of the highway embankment adjacent to the creek, at a depth of 0.2 m below ground surface (Elevation 80.7 m). The sandy silt contains rootlets and trace organics.

The measured Standard Penetration Test (SPT) "N"-values within the sandy silt layer are 1 blow and 2 blows per 0.3 m of penetration, indicating a very loose relative density.

4.2.5 Clayey Silt Till

A deposit of clayey silt till was encountered below the fill in Borehole 16-1 at a depth of 7.2 m below ground surface (Elevation 83.8 m); this till deposit is 4.7 m thick. The clayey silt till contains some sand and trace gravel.



An Atterberg limits test was carried out on one sample of the clayey silt till and measured a liquid limit of 28 per cent, a plastic limit of 13 per cent and a plasticity index of 15 per cent. The test result, which is plotted on a plasticity chart on Figure B3 in Appendix B, indicates that the material is a clayey silt of low plasticity. The result of a grain size distribution test completed on one sample of the clayey silt till is shown on Figure B4 in Appendix B. The water content measured on two samples of the clayey silt till were 6 per cent and 11 per cent.

The measured SPT “N”-values within the clayey silt till deposit range from 73 blows per 0.3 m of penetration to 90 blows per 0.15 m of penetration, suggesting a hard consistency.

4.2.6 Silty Sand to Gravelly Sand Till

A 1.9 m thick deposit of silty sand till was encountered below the clayey silt till deposit in Borehole 16-1 at a depth of 11.9 m below ground surface (Elevation 79.1 m). The silty sand till deposit was also encountered below the sandy silt deposit in Boreholes 16-2 and 16-2B at a depth of 0.6 m below ground surface (Elevation 80.3 m), where it grades locally to silt and sand till or gravelly sand till. Boreholes 16-2 and 16-2B were terminated on refusal to advancement of the portable equipment casing at a depth of 2.9 m and 3.0 m below ground surface (Elevation 78.0 m and 77.9 m), after penetrating 2.3 m and 2.4 m into the deposit, in the respective boreholes.

An Atterberg limits test carried out on a one sample of the fines of the silty sand till measured a liquid limit of 11 per cent, a plastic limit of 10 per cent and a plasticity index of 1 per cent. The test result, which is plotted on a plasticity chart on Figure B5, indicates that the silty sand till exhibits a slight plasticity. The results of grain size distribution testing completed on four samples of the silty sand to gravelly sand till are shown on Figure B6 in Appendix B, showing the variation in the deposit from silty sand to gravelly sand till, containing trace clay. The water content measured on five samples of the silty sand to gravelly sand till deposit range from 4 to 13 per cent.

The measured SPT “N”-values in the silty sand to gravelly sand till deposit range from 11 blows to 149 blows per 0.3 m of penetration, and 100 blows per 0.23 m of penetration, indicating a compact to very dense relative density.

4.2.7 Limestone Bedrock

Bedrock was encountered in Borehole 16-1 at a depth of 13.8 m below ground surface (Elevation 77.2 m), and cored for a length of 3.5 m. Based on a review of the bedrock core, the bedrock consists of fine-grained limestone with shale interbeds. The bedrock is slightly weathered and medium strong.

The Total Core Recovery (TCR) and Solid Core Recovery (SCR) of the core samples recovered are between 75 and 100 per cent, and between 22 and 98 per cent, respectively. The Rock Quality Designation (RQD) based on the borehole data ranges from 0 to 85 per cent, indicating a rock mass of very poor to good quality as per Table 3.10 of the Canadian Foundation Engineering Manual (CFEM, 2006)⁴.

An Unconfined Compression (UC) test performed on a core sample of the bedrock from Borehole 16-1 measured a uniaxial compressive strength (UCS) of 49 MPa. Based on the laboratory UC Test, the bedrock is classified as medium strong (R3, 25 MPa < UCS < 50 MPa). The UC test result is presented on the Record of Drillhole 16-1, and the details of the test are presented on Figures B7A and B7B in Appendix B. Point load strength index tests were carried out on selected samples of the bedrock core and the corrected point load strength index values (Is_{50}) are presented below and included on the Record of Drillhole 16-1.

⁴ Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. BiTech Publisher Ltd., British Columbia.



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Borehole	Depth (Elevation)		IS ₅₀	Test Orientation	Rock Type
	From (m)	To (m)			
16-1	15.19	15.24	3.4	Diametral	Limestone with Shale Interbeds
	15.26	15.30	6.7	Axial	Limestone with Shale Interbeds
	16.15	16.25	2.1	Diametral	Limestone with Shale Interbeds
	16.25	16.31	3.9	Axial	Limestone with Shale Interbeds

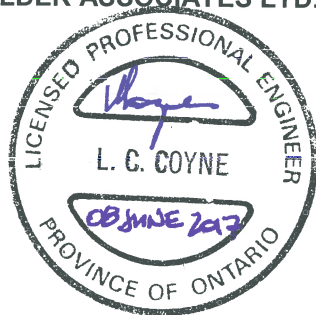
4.3 Groundwater Conditions

The groundwater conditions in the boreholes were observed during drilling, and the water level was measured in Boreholes 16-2 and 16-2B at a depth of 0.5 m and 0.8 m below ground surface (Elevation 80.4 m and 80.1 m), respectively, on completion of drilling. Borehole 16-1 was noted to be dry on completion of drilling prior to commencing bedrock coring. The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ted Beadle, P.Eng., and reviewed by Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal at Golder. Jorge Costa, P.Eng., Senior Consultant and a Designated MTO Foundations Contact for Golder, conducted an independent audit and quality control review of the report.

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PART B

**FOUNDATION DESIGN REPORT
DEFORMATION OF GABION WALL
STRUCTURE SITE NO. 21-647/W
HIGHWAY 401 AND GRAHAM CREEK
CLARINGTON, ONTARIO
W.O. 2016-11028**



6.0 DISCUSSION AND GEOTECHNICAL/FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

The following sections of this report provide a discussion, assessment and recommendations related to the observed deformation of the gabion basket retaining wall at Graham Creek on the north side of Highway 401, approximately 550 m east of Mill Street in Clarington, Ontario. The discussion, assessment of mitigation options and recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current geotechnical investigation at this site, supplemented with the results from the closest borehole advanced as part of a 1957 investigation at the site.

This Foundation Design Report, including the interpretations and recommendations contained herein, are intended for the use of the Ministry of Transportation, Ontario (MTO) to provide the designers with information to assess the feasible foundation/wall alternatives and to carry out the design of the mitigation or replacement of the wall, and shall not be used or relied upon for any other purpose or by any other parties, including the construction on design-build contractors. The contractor must make his own interpretation of the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the contract documents. Those requiring information on aspects of construction must make their own interpretations of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing gabion basket retaining wall is located at the north toe of the Highway 401 embankment on the northeast corner of the existing arch culvert at Graham Creek (Site 21-194/C). The retaining wall extends along the south (left) creek bank, northeasterly from the arch culvert footing on the east side of the arch for a length of approximately 35 m, to a grassy/treed area adjacent to the creek. Graham Creek generally flows from north to south in the area north of Highway 401, and as it approaches the Highway 401 embankment, it changes flow direction to a northeast-to-southwest orientation northeast of the culvert inlet end and continues through the arch culvert under Highway 401. It is understood that the gabion basket retaining wall is experiencing localized toppling/overturning along with soil loss within the western section of the wall. It is understood that this deformation was first noted by MTO in November 2014, and MTO completed site visits to document the wall condition in 2014, 2015 and 2016. Photographs taken during MTO's site visits show that the deformation of the gabion wall is progressing and a gap has formed behind the wall.

6.2 Assessment of Failure Mechanism(s)

6.2.1 Observations

The field work was carried out on August 22, 2016, during which time a total of three boreholes were advanced at the crest of the Highway 401 embankment and at the toe of the embankment, adjacent to Graham Creek. Following completion of the borehole drilling activities, a member of Golder's field staff completed a visual inspection and noted the surficial condition of the highway embankment north slope, the condition of the ground surface immediately adjacent to the top of the wall, and the wall itself, including taking photographs and measurements of the gabion basket retaining wall geometry and deformation. An additional site visit was completed on September 9, 2016 at which time additional observations and measurements of the wall and embankment geometry were noted and additional photographs taken.



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The existing gabion basket retaining wall extends from the east side of the arch culvert footing to approximately 35 m northeast of the footing. During the site inspections, it was noted that the deformed area of the wall extends from approximately 5 m to 22 m east of the arch culvert footing as shown in the photograph below.



Photograph B1: Deformed Area of Gabion Basket Retaining Wall – Looking South (Note image distortion due to panoramic view)

The height of the gabion basket retaining wall to the west of the deformed area is about 3.0 m above the creek bed, with the water being about 0.6 m deep at the time of inspection. The height of the wall to the east of the deformed area is about 2.5 m above the creek bed with the water being about 0.3 m deep at the time of inspection. The height of the wall midway along the slumping/overturned area of the wall is approximately 2.3 m above the creek bed with the water being about 0.7 m deep. Midway along the deformed area where the maximum translation has occurred, the wall has moved laterally approximately 1.4 m to the north of its original location as shown in the photograph below.



Photograph B2: Translation/Toppling of Gabion Basket Retaining Wall – Looking West



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At the base of the retaining wall, a 0.6 m wide “bench” was observed in the “intact” sections of the wall, and this may have been incorporated for global stability and/or to provide for erosion protection. However, the bench was not observed to be present (or it has been eroded/deformed such that it is no longer visible) over the extent of the deformed area of the wall at the time of Golder’s 2016 inspection, or in the photographs provided by MTO taken in 2014 and 2015. A geotextile (non-woven) fabric was visible behind the wall as observed during the inspection from the toe of the embankment/top of the wall, except for an approximately 1.3 m long section near the middle of the deformed area. The area in which the geotextile fabric is missing is shown in the photograph below.



Photograph B3: No Geotextile Fabric Visible Behind Wall – Looking West Along Top of Wall

During the wall inspection, a steel rod was used to probe for voids under the deformed portion of the wall. Voids of about 50 mm in depth, extending to as much as 1 m behind the front edge of the gabion basket retaining wall, were noted to be present at various locations along the base of the wall. Based on Golder’s visual observations and measurements, the base of the deformed section of the gabion basket retaining wall has dropped by as much as 500 mm relative to the base of the non-deformed wall sections.

The material on the embankment slope face immediately above/behind the wall did not show any major signs of sloughing; however, the material directly behind the wall has depressed by up to about 300 mm to 500 mm,



although there did not appear to be any ground loss from behind the wall. The highway embankment slope extending from behind the wall up to the Highway 401 shoulder did not show signs of sloughing or instability.

6.2.2 Conclusion Regarding Failure Mechanism

Graham Creek generally flows from north to south in the area north of Highway 401 and east of Mill Street; however, as the creek approaches the north toe of the Highway 401 embankment, it turns westerly along the embankment toe and flows from northeast to southwest. We infer that the gabion basket retaining wall was adopted to provide stability of the slope above and erosion protection at the toe of the slope at Graham Creek as it approached the culvert under the highway.

The wall deformation has manifested along the section of wall where Graham Creek turns from a north-south to a northeast-southwest flow direction, where the creek water flow is directed against and along the base of the gabion basket wall. The cause of deformation appears to be from ground loss under the wall (reference the voids below the wall noted during Golder's inspection) due to erosion of the immediately underlying clayey silt to silty sand till and/or sand and gravel deposits. It is assumed that when the wall was constructed, the water depth was similar along the full length of the wall to that noted at the west and east ends of the wall (approximately 0.1 m to 0.3 m as it approaches the wall and as it flows into the arch culvert). It is our opinion that after the gabion basket wall was constructed, the water flow began eroding the soil under the wall, resulting in the creek progressively becoming deeper locally in (approximately 0.6 m deep at the time of inspection).

The erosion of soil from below the base of the wall has caused the wall to sink toward the increasingly lower creek bed in this local area. The sinking of the base of the wall has caused the top of the wall to translate forward (north), toward the creek. It is expected that erosion activities increase at times of higher water flows through the creek such as during spring snow melts and following periods of significant rainfall. As shown in Photograph B3, while an approximately 1.3 m long gap was observed in the presence of the filter fabric behind the wall, there does not appear to be ground loss from behind the wall in this area. Therefore, the gap in the filter fabric does not appear to have contributed to any ground loss that could result in the deformation of the wall.

7.0 MITIGATION OPTIONS

The following sections provide mitigation options for the deformed gabion basket retaining wall on the north side of Highway 401 at Graham Creek. Table 1, following the text of this report, discusses the advantages, disadvantages, construction considerations and relative cost for each option. Based on this comparison, the preferred alternative(s) from a foundations perspective are as follows:

- **For a short-term repair scenario:** Repair the affected section with armour stone, or alternatively repair the affected section with a gabion basket wall.
- **For a long-term replacement option, should the timing for the repair be compatible with the timing for future highway widening in this area:** Replace the existing wall with a new, "top-down" wall such as an anchored soldier pile and concrete panel wall, which can also be designed to support a widened highway platform at that time.



7.1 “Do Nothing” – Leave Deformed Gabion Basket Retaining Wall in Place

At the Highway 401-Graham Creek crossing, the existing embankment is approximately 10 m high and the north embankment side slope is oriented at approximately 1.5 horizontal to 1 vertical (1.5H:1V) to 1.75H:1V towards Graham Creek and the gabion basket retaining wall. As indicated in Section 6.2.2, flow of the creek at the bend in the creek alignment is eroding the soil at the base of the gabion basket retaining wall, causing the wall to deform and translate/topple towards Graham Creek. Photographs provided by MTO from 2014 and 2015, together with the current borehole investigation and slope and wall inspection, suggest that the deformation has progressed fairly slowly. As such, it is anticipated that the creek water flow will continue to erode the soils from under the gabion basket retaining wall and it will continue to deform, potentially at a greater rate, eventually falling into Graham Creek. Given that the gabion basket sections are attached forming a continuous wall, continued deformation of the wall section will pull the adjacent attached sections away from the slope. If the retaining wall is not repaired or replaced, there is a longer-term high risk that continued erosion of the toe along the north side of the embankment could cause failure of the retaining wall and could cause the Highway 401 embankment to become unstable.

7.1.1 Global Stability

At the time of inspection, the existing embankment slope between Highway 401 and the retaining wall adjacent to Graham Creek did not show signs of instability or sloughing. The approximately 10 m high highway embankment is sloped at between 1.5H:1V and 1.75H:1V within the general extent of the deformed area of the gabion basket retaining wall.

Slope stability analyses have been completed for the inferred slope and wall configuration prior to the gabion basket retaining wall deformation, as well as for the existing slope configuration within the limits of the deformed gabion basket retaining wall area, using the commercially available program Slide by Rocscience to assess the factor of safety of the embankment in this area. Target minimum factors of safety of 1.3 and 1.5, for short-term and long-term stability, respectively, are normally adopted in the design of slopes and walls under static conditions, per the *Canadian Highway and Bridge Design Code* (CHBDC 2014); these values are based on a typical consequence factor, ψ , for Highway 401, and a typical degree of understanding, ϕ_{gu} , based on the borehole investigation at the site. In the past, the *Canadian Foundation Engineering Manual* suggested a minimum factor of safety of 1.3 to 1.5 be adopted for global stability for earthworks (CFEM, 2006 – Table 8.3), and that a minimum factor of safety of 1.5 be applied when designing for overall stability for retaining walls (CFEM, 2006 – Section 24.12.4.5); however, MTO's typical practice for flexible walls including gabion walls permitted the use of a factor of safety of 1.3 for global stability. It is therefore assumed that for the existing gabion basket retaining wall and the embankment slope above the wall, a minimum factor of safety of 1.3 would have been used in the design.

The soil parameters that have been used in the stability analysis for this area of the gabion basket retaining wall and Highway 401 embankment slope are presented below. The soil parameters are based on field and laboratory test data as well as accepted correlations as proposed by Bowles, (1984) and Kulhawy and Mayne, (1990).



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Soil Deposit	Short-term (Undrained) Analysis			Long-term (Drained) Analysis		
	Bulk Unit Weight (kN/m ³)	Effective Friction Angle Φ'	Undrained Shear Strength (kPa)	Unit Weight (kN/m ³)	Effective Friction Angle Φ'	Cohesion (kPa)
Existing embankment fill	21	30°	-	21	30°	-
Clayey silt till	21	-	200	21	35°	-
Silty sand till	21	36°	-	21	36°	-
Sand and gravel	21	36°	-	21	36°	-
Sandy silt	19	28°	-	19	28°	-

The global stability analysis for the 10 m high embankment was completed for the steepest configuration, 1.5H:1V, measured above the wall. The global stability analysis completed for the condition with the gabion basket retaining wall in place (i.e., original construction or pre-deformation) has a factor of safety of 1.3 against global instability, as shown on Figure 1. This calculated factor of safety for the embankment and retaining wall configuration meets the 2006 CHBDC standards, which would have applied at the time of design and construction of this embankment and wall section. This configuration would not meet the higher factor of safety of 1.54 required for the permanent condition under CHBDC 2014.

The global stability analysis completed for a section of the slope where the wall has failed (i.e. the wall has toppled or has been removed) yields a factor of safety of less than 1.0, as shown on Figure 2. These results indicate that if the deformed segment of the gabion basket retaining wall is not repaired or replaced, the Highway 401 embankment could become unstable in the future. It is noted that we consider that the current configuration of the slope remains stable due to soil arching within the embankment slope, relative to the intact sections of the wall, although this effect is difficult to model in a two-dimensional slope stability analysis.

7.1.2 Monitoring

It is understood that this section of Highway 401 will likely be widened in the future. Although the timing of such a widening is not known, it is anticipated that new or higher retaining walls would be required in conjunction with northward highway widening at that time. If a “do nothing” approach is adopted in the interim time period, it is recommended that monitoring of the deformed section of the wall be carried out until the construction of the highway widening and replacement of the gabion wall, and an instrumentation and monitoring plan should be prepared for implementation at the site.

Conceptually, this plan would consist of monitoring of the slope and existing gabion basket wall, with the installation of surface monitoring points on the top of the wall, at the top (crest) of the embankment, and along the sloped area of the embankment, both within the deformed area and in the adjacent areas where the wall has not deformed. Baseline measurements should be taken at the time of installation of the survey points, and monitoring measurements should be taken on a monthly or bi-monthly basis to monitor the progression of the wall deformation and for any indication of slope instability.



7.2 Replace Affected Section of Gabion Wall

7.2.1 Gabion or Armour Stone Repair

Given that two sections of the gabion basket retaining wall are performing adequately (that is, they satisfy the original global stability design requirement of a factor of safety greater than 1.3 – see Figure 1), the approximately 17 m long section of deformed wall could be repaired with new gabion baskets, or with armour stone. If this option is adopted, it will be necessary to remove the existing gabion baskets and found the base of the new gabion baskets or armour stone for the replacement section on a granular fill pad or on unshrinkable fill placed after removal of any loose/softened material from the footprint of the new wall section. Cofferdams and creek water/groundwater control are likely to be required, although the work could be completed in “wet” conditions with the placement of unshrinkable fill or tremie concrete within a flooded cofferdam area. In addition, a temporary protection system would be required behind the area to be removed and replaced, to maintain the stability of the embankment and protect the workers during construction.

For design of the repaired section of gabion wall using one of these two alternatives, the factored ultimate geotechnical resistance may be taken as 125 kPa, assuming that the gabion or armour stone wall has a minimum base width of 1.5 m. The factored serviceability geotechnical resistance, for 25 mm of settlement, may be taken as 100 kPa.

It is recommended that if a gabion wall repair alternative is adopted, reconstruction of this section of the gabion basket retaining wall be carried out in accordance with OPSS.PROV 512 (Installation of Gabions). It is recognized that the section of wall to be reconstructed is 2.3 m in height, whereas OPSS.PROV 512 has been established for gabion walls not exceeding 2 m in height; however, this specification is considered to set out appropriate requirements for the submittals, materials and construction procedures that remain applicable to the higher wall height.

Where an armour stone repair is adopted, the structural designer must incorporate appropriate transition details between the ends of the existing adjacent sections of gabion wall, and the new armour stone segment. This should also include placement of a geotextile fabric behind the gabion and armour stone sections, to ensure that no loss of backfill material through the face of the wall.

Repair of the gabion basket retaining wall must include toe protection across the entire retaining wall to prevent further scour and undermining of the structure foundation. Toe protection alternatives are discussed in Section 7.4.

7.2.2 Repair Using Other Wall Types

The affected section of gabion wall could also be replaced with a short section of concrete retaining wall, or with a “top-down” wall type such as a soldier pile and concrete panel wall, or secant pile (caisson) wall. While feasible, the footing founding level for a concrete retaining wall would present challenges related to excavation, groundwater control and potential undermining of the existing adjacent segments of the gabion wall, and as such, this option is not preferred for a short repair section from a foundations perspective. Top-down wall types are also technically feasible for this short section of repair, although they would likely require construction of access roadways and working platforms for the heavy equipment (piling or caisson rigs) at the base of the slope, in or near the creek, unless it is possible to complete this work from the shoulder of Highway 401.



If one of these wall types is selected as the preferred alternative for repair of the affected section of gabion wall, the geotechnical recommendations provided in Section 7.3 (for replacement of the entire length of the retaining wall) are also applicable for design of the repair segment. Depending on the type of wall adopted for the repair segment, the structural designer must incorporate appropriate transition details between the ends of the existing adjacent sections of gabion wall, and the new wall segment.

7.3 Replace Entire Retaining Wall

7.3.1 Concrete Retaining Wall

The entire section of gabion basket retaining wall, from the arch culvert footing extending approximately 35 m to the east, could be replaced by a concrete retaining wall. The ground surface and/or creek bed near the toe of the existing slope/retaining wall is between about Elevation 80 m and 81 m based on drawings provided by MTO and site inspection performed by Golder. At Elevation 80 m to 81 m, very dense sand and gravel, very dense silty sand till and hard clayey silt till deposits were encountered in the boreholes advanced during the current and previous investigations. These conditions are considered suitable for a concrete retaining wall supported on a strip footing founded on these overburden soils.

Given that the concrete footing for the proposed retaining wall will need to be founded below the depth of frost penetration as well as below the depth of erosion/scour, the foundation will be below the groundwater level measured in the boreholes and below the existing creek water level. The use of cofferdams in conjunction with dewatering will be required along retaining wall alignment to allow for construction of the wall foundation.

7.3.1.1 Founding Elevation and Factored Geotechnical Axial Resistances

The footings should be provided with a minimum 1.2 m of soil cover for frost protection as per OPSD 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically and perpendicular from the creek bed and toe of slope to the edge of the underside of the footing.

For design of a shallow strip footing and reinforced concrete retaining wall structure founded on the very dense sand and gravel and/or very dense silty sand till deposits at or below Elevation 79.8 m (or lower if needed to address frost protection requirements), a factored ultimate geotechnical resistance of 300 kPa and a factored serviceability geotechnical resistance of 200 kPa (for 25 mm of settlement) may be used, provided that all the existing fill materials and any loosened/softened soils are removed from the foundation footprint. This founding elevation should be checked by the hydraulic engineer during detail design relative to the depth of potential erosion/scour, depending on the toe protection treatment adopted.

These factored geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.4 of the *Canadian Highway Bridge Design Code (CHBDC 2014)* and its *Commentary*.

7.3.1.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the cast-in-place concrete footing and the subsoils should be calculated in accordance with Section 6.10.5 of the *CHBDC (2014)*. The following presents the coefficient of friction, $\tan \phi'$, for the interface between the concrete footing and the founding material.



Founding Material	Coefficient of Friction ($\tan \phi'$)
Cast-in-place concrete footing on native very dense sand and gravel and very dense silty sand till	0.70
Cast-in-place concrete footing on hard clayey silt till (if present)	0.62

7.3.1.3 Global Stability of Concrete Retaining Wall

The embankment fill is comprised of clayey silt and the native overburden at this site is generally comprised of a hard cohesive till deposit and compact to very dense non-cohesive soils. The factor of safety against global instability of a concrete retaining wall is 1.55, as shown on Figure 3; this is based on a footing founded at a minimum depth of 1.2 m below the ground surface in front of the wall, and assuming that has an overall width of at least 3.5 m (extending at least 2.5 m behind the back of the wall). This analysis has been completed based on the steepest observed slope configuration (1.5H:1V) observed above the wall, although it is noted that the slope varies from about 1.5H:1V to 1.75H:1V. This assessment of the global stability of the retaining wall should be reviewed and confirmed if this option is adopted by the detail designers.

As discussed in the Section 7.1, the stability of the new retaining wall will be dependent on the integrity of the base of the foundation. Adequate protection of the base of the retaining wall should be provided to prevent scour and undermining of the structure foundation. Toe protection alternatives are discussed in Section 7.4.

7.3.1.4 Lateral Earth Pressures for Design

The lateral pressure acting on the retaining wall will depend on the type and method of placement of backfill materials, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the wall and the drainage conditions behind the wall. The following recommendations are made concerning the design of the concrete retaining wall (if this option is adopted).

- The retaining wall backfill should consist of select, free draining granular fill meeting the material requirements of OPSS.PROV 1010 Granular A or Granular B Type II. Transverse drains and weep holes should be installed to provide positive drainage of the granular backfill.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC (2014) Section 6.12.3. Other surcharge loadings should be accounted for in the design as required.
- For an unrestrained wall, the granular fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Figure C6.20(b) within Section C6.12 of the Commentary to the CHBDC 2014).
- For an unrestrained structure, the lateral earth pressures acting against the wall are based on the granular fill as placed, and the following parameters (unfactored) may be used:



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

Assuming the wall support allows for lateral yielding of the wall, active earth pressures are to be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC (2014).

7.3.2 Driven Sheetpile Wall

An interlocking, driven steel sheetpile wall has been considered as an option for replacement of the existing gabion wall. Driving conditions would be very difficult due to the presence of hard or very dense soil (having Standard Penetration Test "N" values of greater than 100 blows per 0.3 m of penetration) immediately below the embankment toe, together with the potential presence of cobbles or boulders within the till deposits. Further, bedrock is present at a depth of about 2 m to 3 m below the creek bed, and the wall would likely not achieve sufficient penetration to develop adequate passive lateral resistance for the wall/embankment height. Therefore, this wall type is not considered feasible.

7.3.3 Soldier Pile and Concrete Panel Wall

A soldier pile and concrete panel wall could be considered as an option for replacement of the existing gabion basket wall. A soldier pile and concrete panel wall would minimize the extent of temporary excavation into the embankment slope compared to a concrete retaining wall. For this site, the soldier piles could be installed behind the existing gabion wall prior to removing the gabion wall via top-down excavation, with concurrent placement of the permanent concrete panels. It is noted that at this location, access to install the soldier piles and panel wall could be difficult considering Graham Creek is immediately adjacent to the Highway 401 embankment toe. It may be possible to construct the soldier piles for such a wall from the Highway 401 grade, although it may be necessary to construct a temporary widening/working platform at the top of the embankment to reach the wall alignment.

Such a wall would consist of soldier piles socketed to sufficient depth (i.e. into the underlying limestone bedrock) to provide the necessary axial and passive (lateral) resistance for the maximum retained soil height. Additional lateral support to the soldier pile and concrete panel wall system would be provided in the form of permanent soil anchors located along the retaining wall.

The concrete lagging panels should be installed as the trimming excavation and removal of the existing gabion basket wall is made such that the unsupported height of soil (embankment slope fill) does not exceed 1.2 m at any time, and the space behind the concrete panels should be immediately packed with granular material to ensure intimate contact of the soil with the back of the wall and to aid in achieving proper drainage. If sufficient thickness of free-draining granular soil is not provided behind the concrete panels to provide adequate drainage and frost protection, consideration should be given to using a geosynthetic drainage sheet. An insulation layer should also be provided immediately behind the wall to provide for frost protection, if required.



7.3.3.1 Passive Resistance for Soldier Pile Sockets

The factored passive resistance at ULS in front of the soldier piles below the base of the wall may be assessed using the equation and the design parameters provided below:

$$P_p = 1.5 K_p \gamma' z B$$

- where P_p is the factored lateral resistance at ULS (kN/m);
- K_p is the coefficient of passive earth pressure, which may be taken as 3.7;
- γ' is the effective unit weight of the soil in front of the soldier pile socket, which may be taken as 10 kN/m³ below the groundwater level;
- z is the depth from the ground surface in front of the pile to the base of the pile socket (m); and
- B is the diameter of the soldier pile socket (m).

The equation above assumes that the lateral resistance acts over a width equal to three times the socket diameter. The upper 1.2 m of soil (below the finished grade) in front of the soldier piles should be ignored in the calculation of the passive resistance, to account for frost effects. It is recommended that the soldier piles extend to a depth equal to at least the height of the wall plus the top 1.2 m of soil depth which is ignored in the calculation, or deeper if necessary to satisfy limiting equilibrium requirements.

7.3.3.2 Permanent Soil Anchors

If required, a soil anchor support system can be designed to accommodate the loads applied from lateral earth pressures and surcharge pressures from area, line or point loads and take into account any sloping ground behind the retaining wall system. For design, the soil anchors may be sized based on the following unfactored bond stresses acting between the grout and native soil.

Soil Deposit	Estimated Ultimate Load Transfer (kPa)
Existing clayey silt embankment fill above approximately Elevation 83.5 m	30
Hard clayey silt till between approximately Elevation 83.5 m and 79 m	100
Compact to very dense silty sand till between approximately Elevation 79 m and 77 m	150

In accordance with CHBDC (2014), a factor of 0.4 should be applied to the unfactored (ultimate) bond stress value for the ultimate limit state condition. The serviceability limit state value for 25 mm of displacement will not govern and may be greater than the ULS value. For design purposes, a factored serviceability value equal to the factored ultimate value should be used.

The sustained working load should not be greater than 60 per cent of the ultimate tensile strength of the anchor tendons or bars. The ground anchors should have their fixed length (bond zone) formed within the native hard clayey silt till deposit or compact to very dense silty sand till deposit, and should be installed at a downward angle



of 20 degrees or steeper. The top row of anchors should be installed not less than 1.5 m below the top of the wall face. A minimum of 4.5 m of overburden is required above the center of the fixed length (bond zone) to provide the necessary overburden pressure to develop anchor capacity in gravity-grouted anchors, to prevent grout leakage during installation of pressure grouted anchors, and to prevent heaving of the ground surface for higher grout pressure operations (FHWA, 1999). The fixed length (bond zone) of the anchors should be at least 3 m (and may be up to 8 m) and should be maintained behind a line drawn upward at 45 degrees from the toe of the proposed wall. The horizontal spacing between anchors will be dependent of the spacing of the soldier piles but should be greater than four times the diameter of the anchor diameter (grouted section) or 1.2 m. The permanent soil anchors should be provided with suitable corrosion protection.

Lateral earth pressures for design are based on the existing embankment fill material and the clayey silt till and the following parameters (unfactored) may be used:

	Existing Embankment Fill	Clayey Silt Till
Soil unit weight	20 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure		
Active, K_a	0.33	0.27
At rest, K_o	0.50	0.43

Anchor installation, grouting and testing should be carried out in accordance with OPSS 942 (*Pre-Stressed Soil and Rock Anchors*).

7.3.3.3 Global Stability

If a soldier pile and concrete panel wall is adopted, the global stability of the full system including permanent anchors must be assessed at the detailed design stage.

7.3.4 Secant Pile (Caisson) Walls

A secant pile wall (or “caisson wall”) is constructed by drilling holes between 0.9 m and 1.2 m diameter to the full depth of the wall, inserting steel reinforcement in the form of steel beams or reinforcing bars, and filling the holes with tremie concrete. The secant pile wall is formed by having each caisson overlap the adjacent caisson. A permanent secant pile wall often has a cast-in-place or pre-cast concrete facing attached to the front surface to fill any gaps between caissons and provide a smooth or architecturally appropriate surface finish. A secant wall can be designed as a cantilever wall or with permanent tie-backs. Permanent walls must also include provisions for frost protection and control of any groundwater seepage.

The main advantages of a secant pile wall are increased wall stiffness compared to a soldier pile and lagging wall. They are, however, more complex and expensive to construct compared to the other options presented for this site, and therefore such a wall type has not been evaluated further at this stage.

7.4 Retaining Wall Toe Protection

A key design component is the requirement for heavy riprap toe protection (often referred to as a “launching apron”) to prevent scour and undermining of the wall foundation/base. Inadequate toe protection is the most common design deficiency leading to failure of engineered wall/slope stabilization measures where flowing water



is present, and is considered the key factor in the deformation/partial failure of the gabion basket retaining wall at this site.

For this site, the particle size, thickness and geometry/height of the rip-rap toe protection should be designed based on appropriate inputs from a hydraulic engineer, including consideration of the Graham Creek water levels and velocities under the design storm and flood condition, considering the impacts of climate change per the MTO's current policy. The rip-rap should be designed and placed in accordance with OPSS 511 (Rip-Rap, Rock Protection and Granular Sheet piling).

8.0 CONSTRUCTION CONSIDERATIONS

The following sections identify construction considerations for the replacement of the deformed section or entire length of the gabion basket retaining wall.

8.1 Excavation

The foundation excavations for the levelling pad and lower-most course of the gabion basket retaining wall, or for concrete footings, would extend below the water level of the creek, through the very dense sand and gravel and into the compact to very dense silty sand to gravelly sand till. While the lower-most course of the replacement section of gabion basket retaining wall could be placed to match the elevation of the base of the existing wall, it is recommended that the lower most course be embedded below the creek bed to provide additional protection against erosion and scour within this section, and hence the excavations for the lower-most course of gabion baskets could be nearly as deep as those for concrete footings.

The excavation should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing embankment fill and any surficial deposits (i.e. topsoil, loose sandy silt) are classified as Type 3 soil and the compact to very dense silty sand to gravelly sand till are classified as Type 2 soils. Temporary excavations (i.e. those open for a relatively short time period) should be sloped no steeper than 1H:1V and may have to be flattened to 3H:1V below the creek bed to remain stable in the short term in the presence of localized seepage.

8.2 Temporary Protection Systems

Where a temporary protection system is required within the Highway 401 slope behind the existing retaining wall, it should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection Systems). The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539, provided that any existing adjacent utilities can tolerate this magnitude of deformation. It is considered that while a driven interlocking sheet pile system would be most suitable for the temporary excavation support associated with the strip subexcavation work at this site, the presence of hard clayey silt till and very dense silty sand to gravelly sand till will make it difficult to drive the sheet piles to an adequate depth. Therefore, a soldier pile and lagging system would likely be used, with the soldier piles driven or socketed to a sufficient depth to provide the necessary passive resistance for the retained soil height under the temporary excavation works, including any surcharge loads behind the protection system within at least a 1H:1V zone relative to the base of the excavation.



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While the selection and design of the temporary protection system will be the responsibility of the Contractor, the following information is provided to MTO and its designers to assess the approximate construction costs during detailed design. The temporary support system may be designed using the following parameters:

Soil Type	Earth Pressure Coefficient			Effective Friction Angle (ϕ , degrees)	Unit Weight (γ , kN/m ³)	Undrained Shear Strength (S_u , kPa)
	Active, K_a	At Rest, K_o	Passive, K_p			
Existing Embankment Fill	0.33	0.50	3.00	30	21	-
Clayey Silt Till	0.28	0.44	3.54	34	21	200
Silty Sand to Gravelly Sand Till	0.26	0.41	3.85	36	21	-
Sand and Gravel	0.26	0.41	3.85	36	21	-

The earth pressure coefficients noted above are based on a horizontal surface adjacent to the excavation. Depending on the location of the wall relative to the top of the highway embankment, the coefficients should be adjusted to take account of the sloping ground surface above the wall. Further, hydrostatic pressures must be added to the earth pressure where groundwater is not fully lowered to below the excavation level.

It should be noted that the pressure distributions given above are the minimum for the ultimate stress condition. A stiffer design may be required than predicted by these distributions in order to maintain displacements within an acceptable range.

8.3 Groundwater Control

The excavation for the removal of some or all of the existing gabion basket retaining wall, and for its replacement with either a new gabion basket or concrete retaining wall, will extend below the groundwater level (as high as approximately Elevation 81.3 m) and below the Graham Creek water level. The excavations will extend through the existing fill, clayey silt till, silty sand to gravelly sand till, and sand and gravel. The groundwater seepage through the existing embankment fill and clayey silt till, as well as potentially the silty sand to gravelly sand till, is expected to be minor; however, significant groundwater seepage is expected through the existing sand and gravel deposit within the Graham Creek bed. Temporary construction dewatering in conjunction with a cofferdam will be required to maintain the integrity of the excavation and subgrade during construction.



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9.0 CLOSURE

This Foundation Design Report was prepared by Ted Beadle, P.Eng., with technical input and review by Lisa Coyne, P.Eng., a senior geotechnical engineer and Principal of Golder. Jorge Costa, P.Eng., Senior Consultant and a Designated MTO Foundation Contact for Foundations for Golder, conducted an independent quality review of the report.

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Ontario Provincial Standard Specifications (OPSS)

OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS.PROV 512	Construction Specification for Installation of Gabions
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS 942	Construction Specification for Pre-Stressed Soil and Rock Anchors
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 3090.101	Foundation Frost Depths for Southern Ontario
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Other:

Ontario Regulation 213	Construction Projects (as amended)
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FOUNDATION REPORT - DEFORMATION OF GABION WALL AT HIGHWAY 401 AND GRAHAM CREEK, W.O. 2016-11028

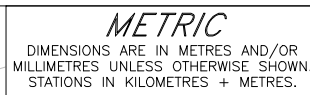
TABLE 1 – COMPARISON OF REPLACEMENT RETAINING WALL TYPES AND FOUNDATION ALTERNATIVES

Wall Type and Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
Repair of deformed section with gabion basket or armour stone wall	<ul style="list-style-type: none"> Feasible, removal of only deformed section required 	<ul style="list-style-type: none"> Lesser construction activities required for replacement of deformed section only, compared with all other options Relatively quick repair compared to full replacement options Requires lesser amount/depth of excavation compared to replacement of deformed section with concrete wall Armour stone has a longer design life than gabion basket wire Lowest cost alternative 	<ul style="list-style-type: none"> Removal of deformed section of wall is required Does not accommodate future Highway 401 widening Groundwater and creek water control required Durability of gabion basket wire is shorter than concrete or armour stone, leading to shorter design life 	<ul style="list-style-type: none"> Conventional construction techniques However, difficult access for construction equipment Environmental constraints will apply for construction in proximity to creek 	<ul style="list-style-type: none"> Significantly lower cost than other options
Replace full length with new gabion basket retaining wall	<ul style="list-style-type: none"> Feasible; requires removal of full wall with protection system 	<ul style="list-style-type: none"> Likely still shorter construction schedule to replace entire wall compared to other wall types Requires lesser amount/depth of excavation and water control compared to concrete wall which has deeper foundations 	<ul style="list-style-type: none"> Requires complete removal of the existing gabion basket retaining wall; temporary protection system would be required to maintain integrity of existing embankment and Highway 401 traffic Groundwater and creek water control required, although less than for concrete retaining wall 	<ul style="list-style-type: none"> Conventional construction techniques Difficult access for construction equipment; however, this equipment is generally smaller than for other wall types Environmental constraints will apply for construction in proximity to creek 	<ul style="list-style-type: none"> Lower costs than other full replacement options

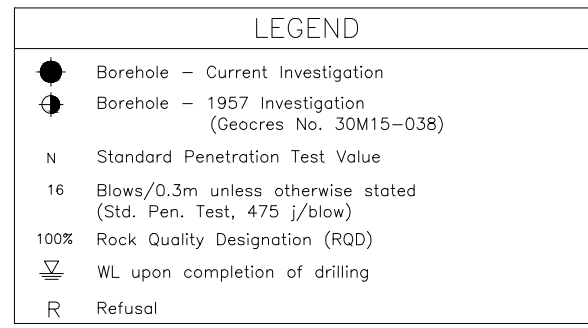


FOUNDATION REPORT - DEFORMATION OF GABION WALL AT HIGHWAY 401 AND GRAHAM CREEK, W.O. 2016-11028

Wall Type and Foundation Option	Feasibility	Advantages	Disadvantages	Constructability/ Risks	Estimated Costs
Concrete retaining wall on shallow foundations	<ul style="list-style-type: none"> Feasible; requires removal of full wall with protection system 	<ul style="list-style-type: none"> Could build the retaining wall to accommodate future widening of Highway 401 Standard retaining wall construction 	<ul style="list-style-type: none"> Requires complete removal of the existing gabion basket retaining wall; temporary protection system would be required to maintain integrity of existing embankment and Highway 401 traffic Cofferdam and dewatering required to keep base of excavation dry Difficult access for construction equipment 	<ul style="list-style-type: none"> Conventional excavation and construction techniques Difficult to construct at edge of the creek Environmental constraints will apply for construction in proximity to creek 	<ul style="list-style-type: none"> Higher cost relative to gabion basket repair
Driven steel sheetpile wall	<ul style="list-style-type: none"> Not feasible, due to "100-blow" soils and shallow bedrock depth 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
Soldier pile and concrete panel wall	<ul style="list-style-type: none"> Feasible and would mitigate requirement for temporary protection system 	<ul style="list-style-type: none"> Could build the retaining wall to accommodate future widening of Highway 401 "Top-down" construction; eliminates over-excavation behind existing retaining wall and temporary protection system, which would be required for other options 	<ul style="list-style-type: none"> Likely more time-consuming than other wall types due to steps involved (pre-augering for socket holes, placing soldier piles, placing backfill in lifts, installing concrete panels, installing, pre-stressing and testing tie-backs) Specialized equipment and skilled labour required Would have to install soldier piles adequately deep, possibly socketted into bedrock, to resist lateral earth pressures; at least one row of permanent ground anchors likely to be required, but potentially more rows of anchors depending on wall location and height 	<ul style="list-style-type: none"> Construction costs and time may escalate if cobbles and boulders are encountered in soldier pile installation Difficult access for larger equipment needed to complete the construction 	<ul style="list-style-type: none"> Considerable cost compared to other options



**Golder
Associates**



BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-1	91.0	4864003.9	378831.6
16-2	80.9	4864046.7	378835.8
16-2B	80.9	4864047.7	378835.8
38-6	81.3	4864016.3	378801.9

NOTES

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. X-Base-1.dwg and Surface_401Courtice-Central.dwg, received Aug. 16, 2012.

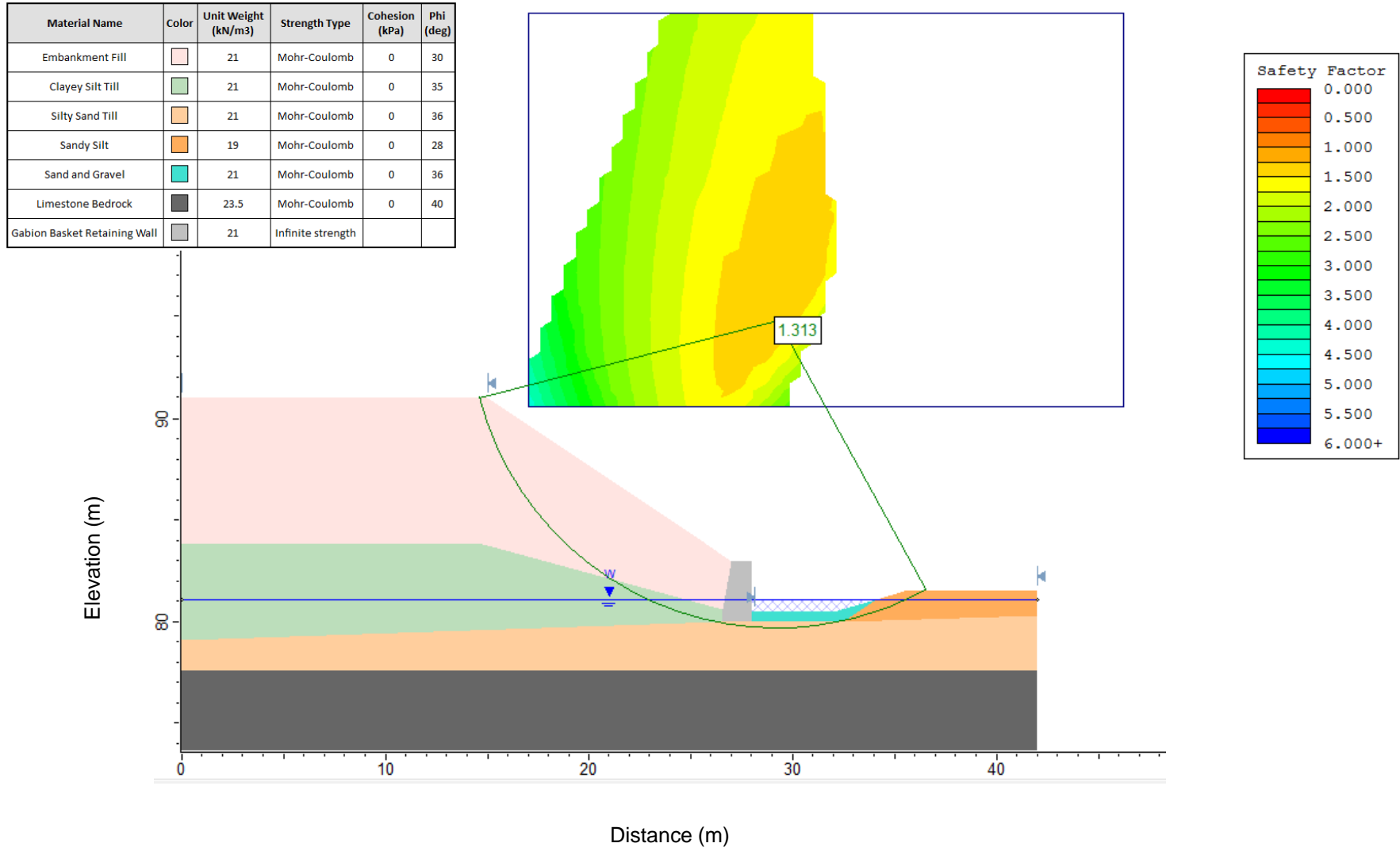
NO.		DATE		BY	
				REVISION	
Geocres No. 30M15-306					
HWY. 401			PROJECT NO. 1413191-9		DIST. CENTRAL
SUBM'D. TWB		CHKD. LCC		DATE: 9/16/2016	SITE: 21-647/
DRAWN: MR		CHKD. TWB		APPD. LCC/JMAC	DWG. 1





Static Global Stability Initial Construction (Pre-Deformation) Long-Term (Drained) Conditions

Figure 1

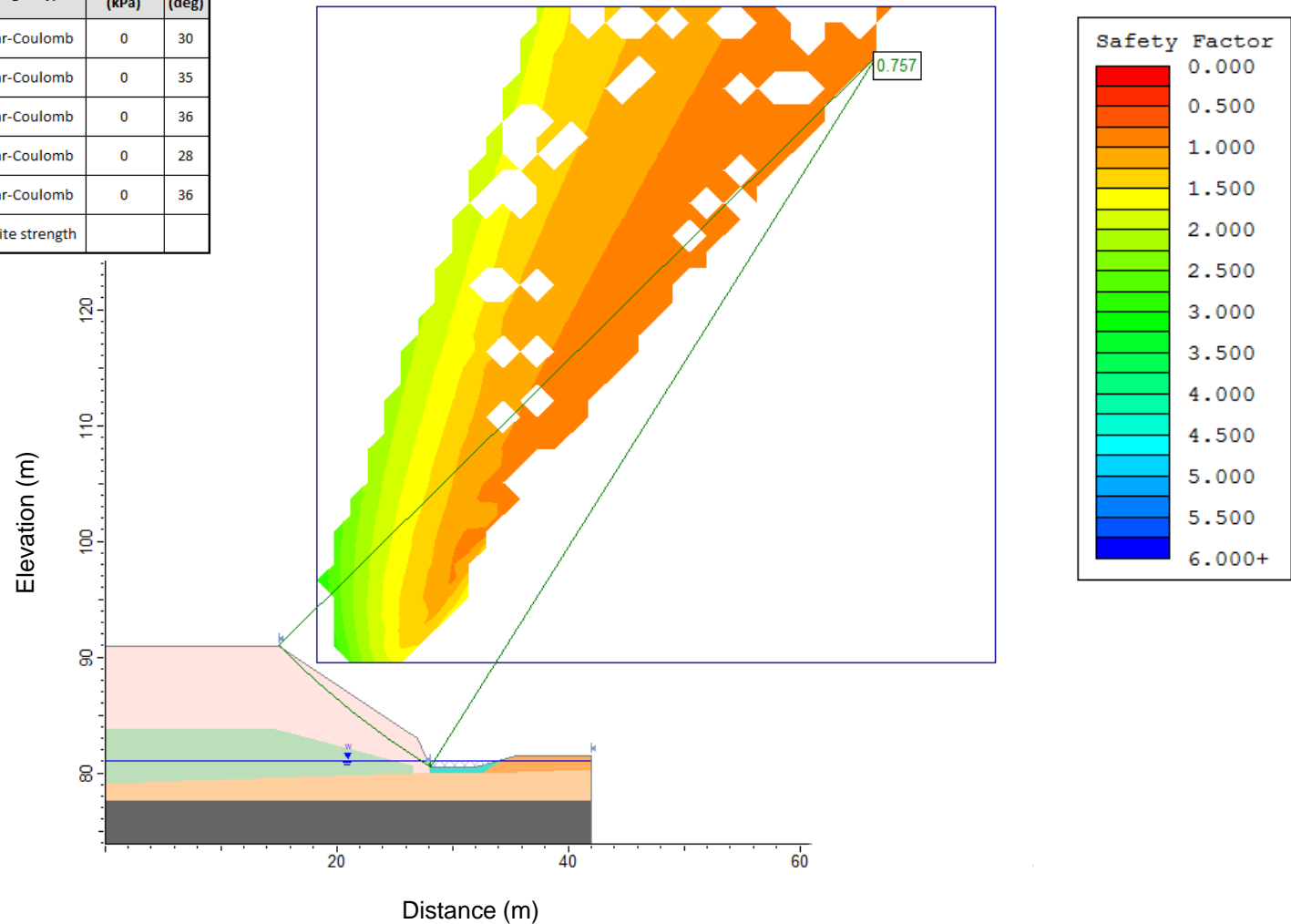




Static Global Stability Gabion Wall Toppled/Removed Long-Term (Drained) Conditions

Figure 2

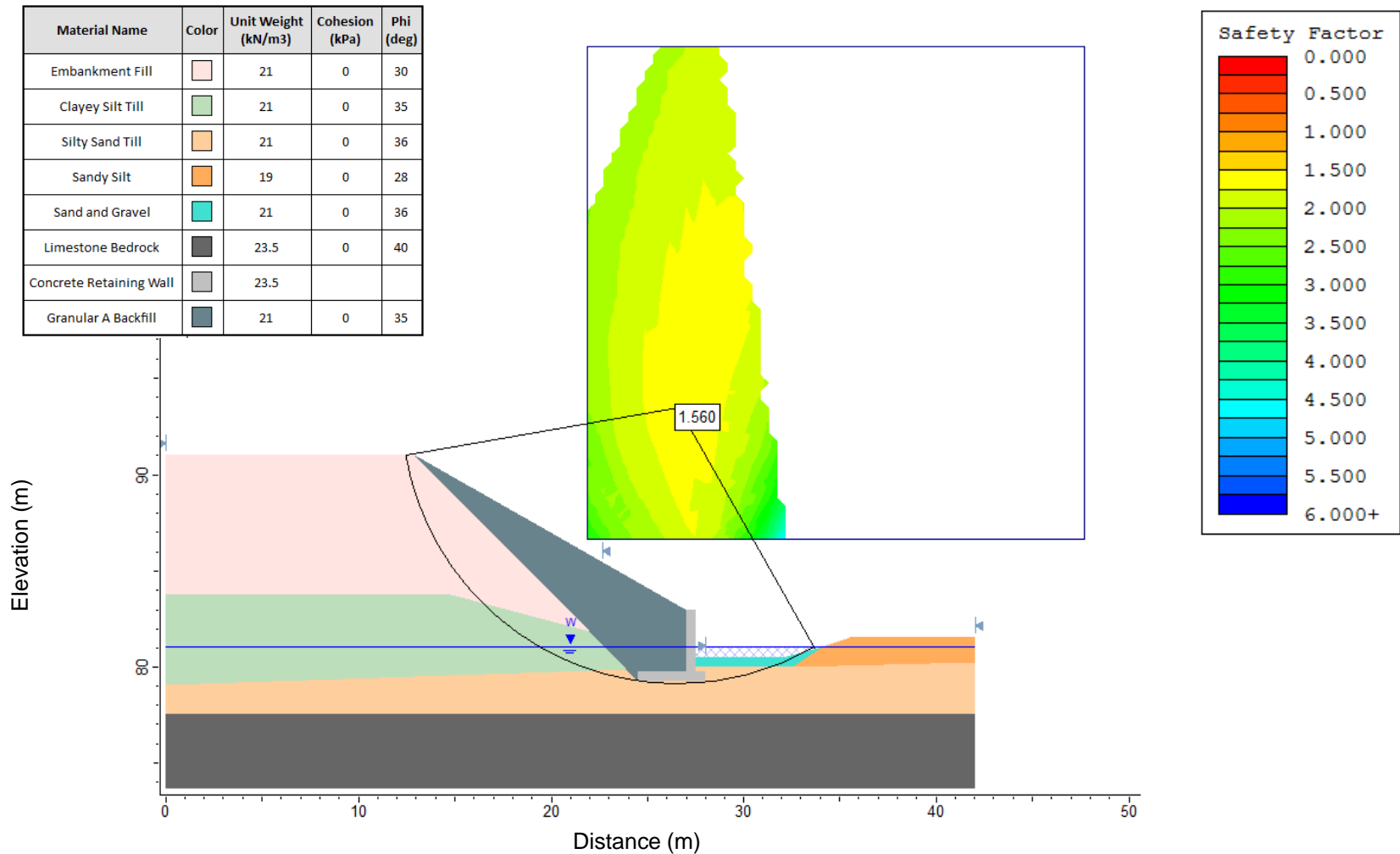
Material Name	Color	Unit Weight (kN/m ³)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment Fill		21	Mohr-Coulomb	0	30
Clayey Silt Till		21	Mohr-Coulomb	0	35
Silty Sand Till		21	Mohr-Coulomb	0	36
Sandy Silt		19	Mohr-Coulomb	0	28
Sand and Gravel		21	Mohr-Coulomb	0	36
Limestone Bedrock		23.5	Infinite strength		





Static Global Stability Concrete Retaining Wall Long-Term (Drained) Conditions

Figure 3





APPENDIX A

Borehole Records



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: * Grains greater than 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.


Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	

GTA-MTO 001 S:\CLIENTS\MTO\HWY 401\02 DATA\GINT\1413191 MTO RETAINER ASSIGNMENT\HWY400 MILLST CLARINGTON 1090.GPJ GAL-GTA.GDT 06/07/17

PROJECT <u>1413191 (1090)</u>				RECORD OF BOREHOLE No 16-1				SHEET 2 OF 2				METRIC					
W.P. <u>4014-E-0012</u>				LOCATION <u>N 4864003.9; E 378831.6 MTM ZONE 10 (LAT. 43.9122941; LONG. -78.5782505)</u>				ORIGINATED BY <u>PKS</u>									
DIST <u>Central</u> HWY <u>401</u>				BOREHOLE TYPE <u>CME-55, 159 mm Outer Diameter Hollow Stem Augers</u>				COMPILED BY <u>MR</u>									
DATUM <u>Geodetic</u>				DATE <u>August 22, 2016</u>				CHECKED BY <u>TWB</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/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
	LIMESTONE (BEDROCK)		2	RC	REC 98%												
	Bedrock cored from depths of 13.8 m to 17.3 m.																
	For bedrock coring details refer to Record of Drillhole 16-1.																
73.7							75										
17.3			3	RC	REC 100%		74										
	END OF BOREHOLE																
	NOTE: 1. Borehole dry on completion of overburden drilling operations, prior to commencement of rock coring.																

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PROJECT: 1413191 (1090)

RECORD OF DRILLHOLE: 16-1

SHEET 1 OF 1

LOCATION: N 4864003.9 ;E 378831.6

DRILLING DATE: August 22, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME

DRILLING CONTRACTOR: Fisher Environmental Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	COLOUR % RETURN	UN - Joint FLT - Fault SH - Shear VN - Vein CJ - Conjugate BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular PO - Polished K - Slickensided SM - Smooth RO - Rough VR - Very Rough MB - Mechanical Break BR - Broken Rock NOTE: For additional abbreviations refer to list of abbreviations & symbols.																NOTES
							FLUSH	RECOVERY		R.Q.D. %	FRACT. INDEX PER 0.25 m	DISCONTINUITY DATA					HYDRAULIC CONDUCTIVITY K, cm/sec			Diametral Point Load Index (MPa)	RMC -Q AVG		
								TOTAL CORE %	SOLID CORE %			B Angle	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	10°	10°			10°	

| 14 | NW Casing | Continued from Record of Borehole 16-1 | | 77.21 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 50



LOGGED: PKS

CHECKED: TWB

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PROJECT 1413191 (1090)			RECORD OF BOREHOLE No 16-2			SHEET 1 OF 1			METRIC									
W.P. 4014-E-0012			LOCATION N 4864046.7; E 378835.8 MTM ZONE 10 (LAT. 43.9126796; LONG. -78.5781916)			ORIGINATED BY PKS												
DIST Central HWY 401			BOREHOLE TYPE Portable Tripod			COMPILED BY MR												
DATUM Geodetic			DATE August 22, 2016			CHECKED BY TWB												
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
80.9	GROUND SURFACE							20	40	60	80	100						
0.0	TOPSOIL																	
80.3	Sandy SILT, containing rootlets and trace organic matter Very loose Dark brown Moist		1	SS	1	▽												
0.6	Silty SAND to SILT and SAND, trace to some gravel, trace clay (TILL) Compact to very dense Grey Wet		2	SS	11													
			3	SS	22													
			4	SS	96													
			5	SS	138													
78.0	END OF BOREHOLE CASING REFUSAL																	
2.9	NOTE: 1. Water level in open borehole at a depth of 0.5 m below ground surface (Elev. 80.4 m) upon completion of drilling.																	

GTA-MTO 001 S:\CLIENTS\MTOWHY_40102_DATA\GINT\1413191_MTO_RETAINER\ASSIGNMENT\HWY400_MILLST_CLARINGTON_1090.GPJ GAL-GTA.GDT 06/07/17

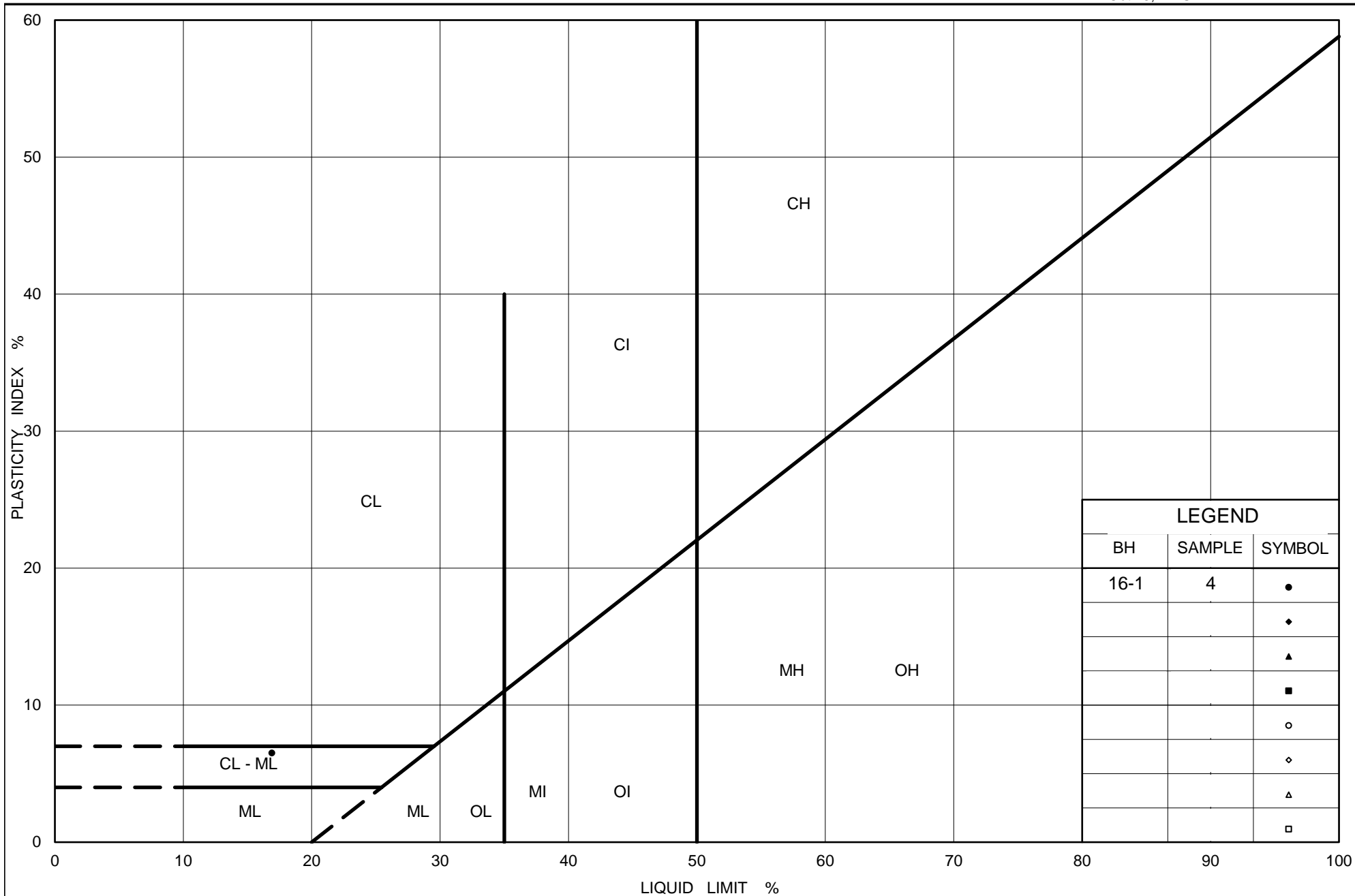
PROJECT 1413191 (1090)			RECORD OF BOREHOLE No 16-2B			SHEET 1 OF 1			METRIC								
W.P. 4014-E-0012			LOCATION N 4864047.7; E 378835.8 MTM ZONE 10 (LAT. 43.9126886; LONG. -78.5781915)			ORIGINATED BY PKS											
DIST Central HWY 401			BOREHOLE TYPE Portable Tripod			COMPILED BY MR											
DATUM Geodetic			DATE August 22, 2016			CHECKED BY TWB											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
80.9	GROUND SURFACE							20	40	60	80	100					
0.0	TOPSOIL																
80.3	Sandy SILT, containing rootlets and trace organic matter		1	SS	2	▽											
0.6	Very loose Dark brown Moist		2	SS	13												
	Gravelly SAND, some silt, trace clay (TILL)		3	SS	48												
	Compact to very dense Grey Wet		4	SS	149												
	- containing a pocket of silty sand between depths of 1.2 m and 1.5 m		5	SS	135												
77.9	END OF BOREHOLE CASING REFUSAL																
3.0	NOTE: 1. Water level in open borehole at a depth of 0.8 m below ground surface (Elev. 80.1 m) upon completion of drilling.																

GTA-MTO 001 S:\CLIENTS\MTOWHY_40102_DATA\GINT\1413191_MTO_RETAINER\ASSIGNMENT\HWY400_MILLST_CLARINGTON_1090.GPJ GAL-GTA.GDT 06/07/17



APPENDIX B

Geotechnical Laboratory Test Results



Ministry of Transportation

Ontario

PLASTICITY CHART Sandy Clayey Silt Fill

Figure No. B1

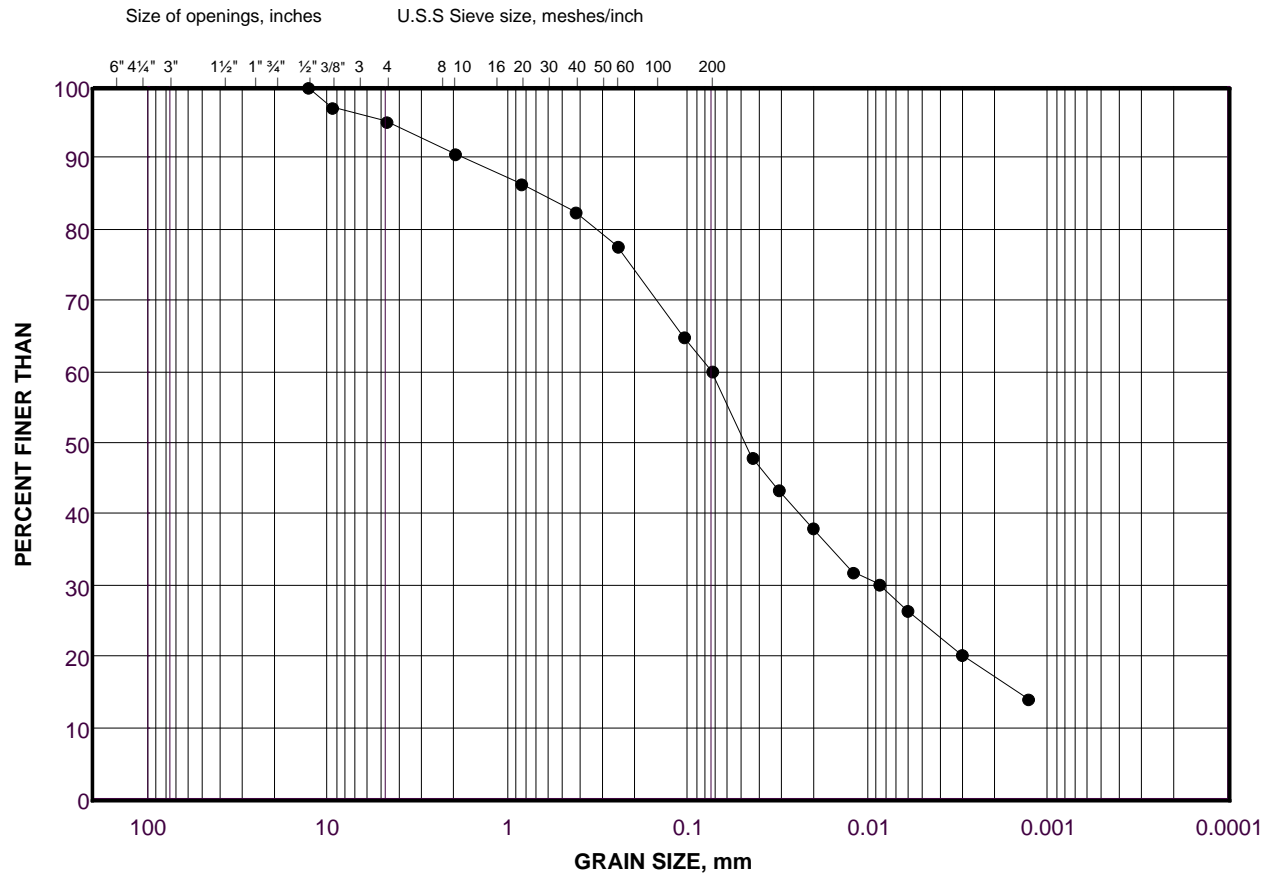
Project No. 1413191 (1090)

Checked By: TWB/LCC

GRAIN SIZE DISTRIBUTION

Sandy Clayey Silt Fill

FIGURE B2



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

LEGEND

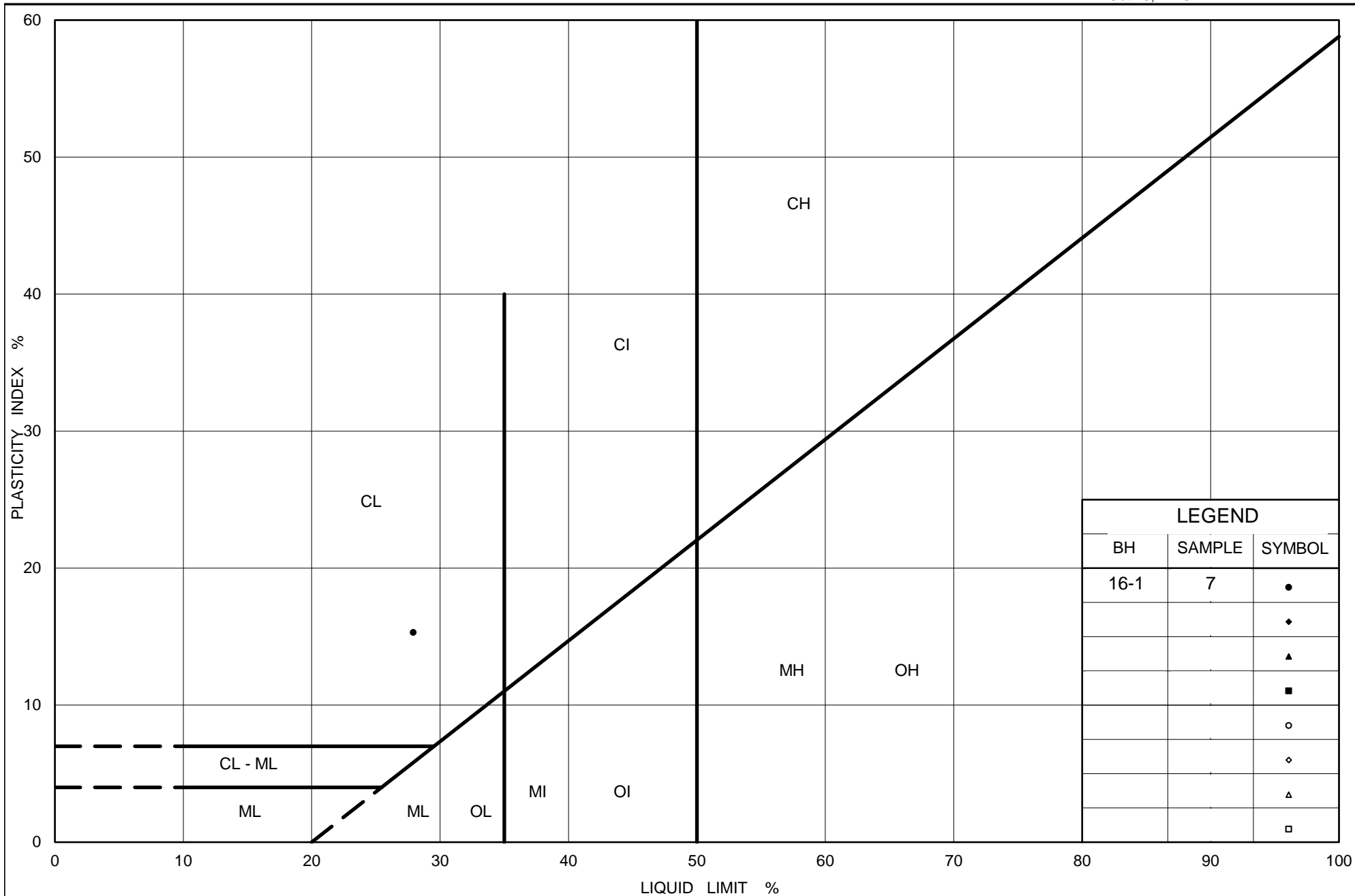
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	16-1	4	84.6

Project Number: 1413191

Checked By: TWB/LCC

Golder Associates

Date: 14-Sep-16



Ministry of Transportation

Ontario

PLASTICITY CHART

Clayey Silt Till

Figure No. B3

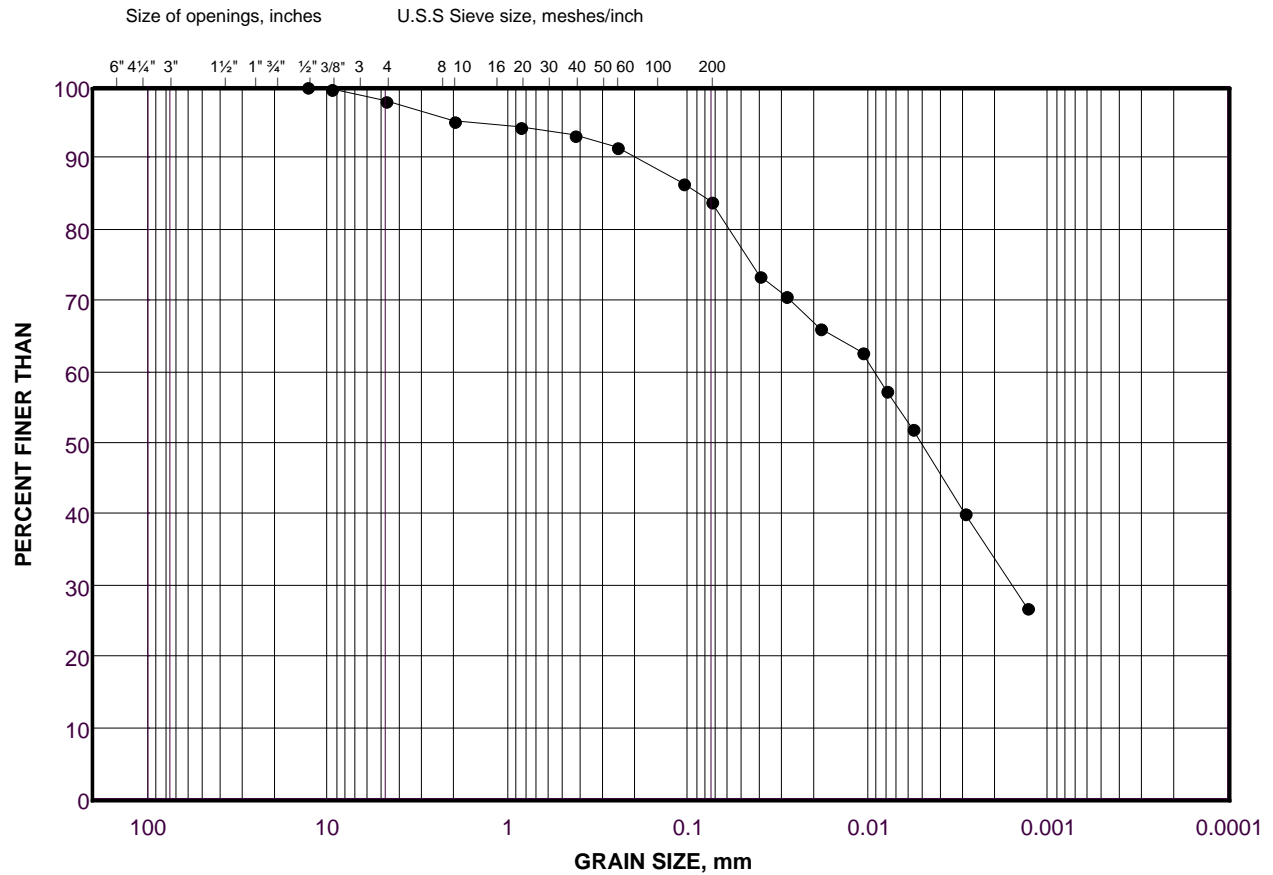
Project No. 1413191 (1090)

Checked By: TWB/LCC

GRAIN SIZE DISTRIBUTION

Clayey Silt Till

FIGURE B4



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

LEGEND

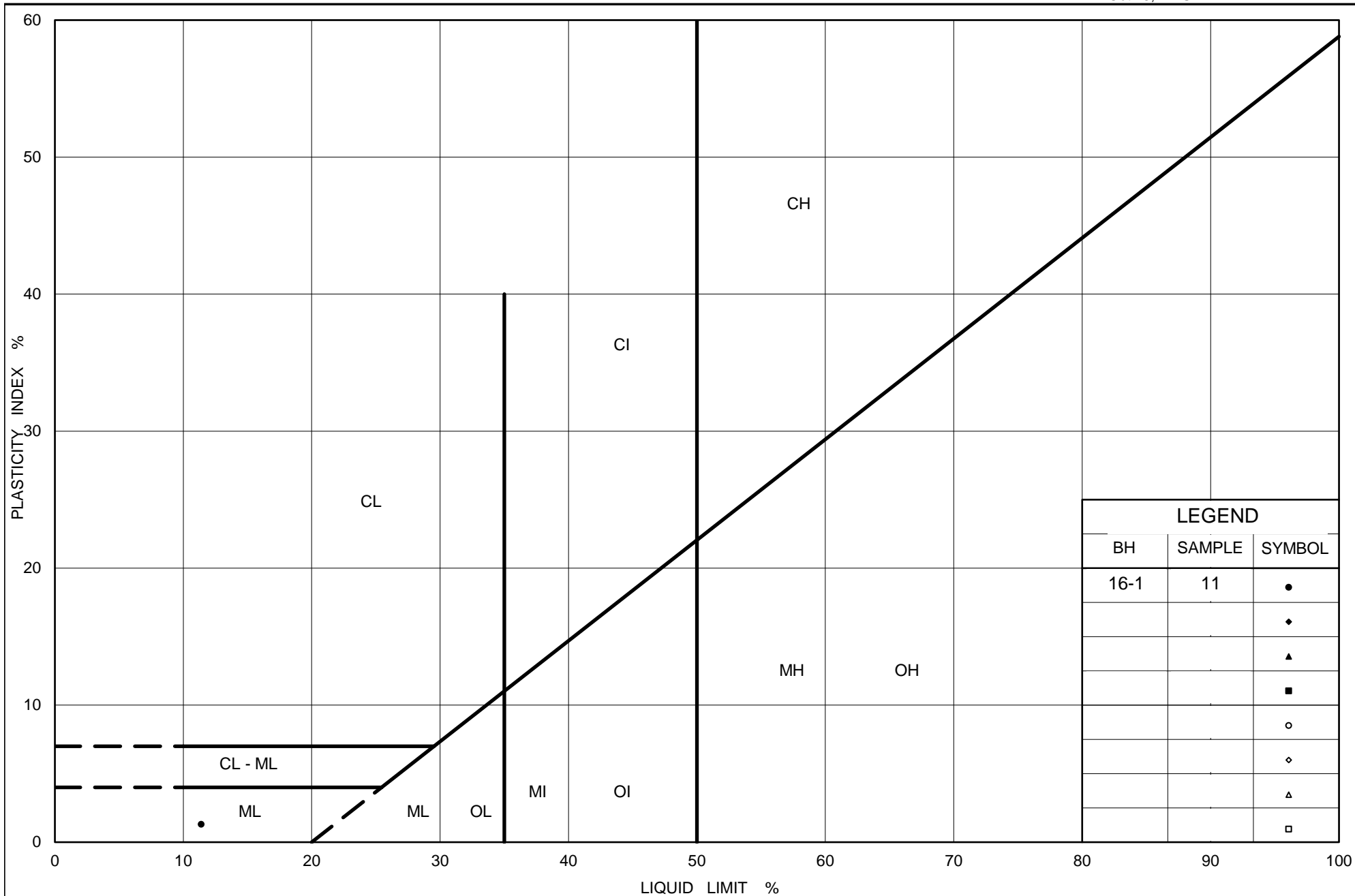
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	16-1	7	81.7

Project Number: 1413191

Checked By: TWB/LCC

Golder Associates

Date: 14-Sep-16



Ministry of Transportation

Ontario

PLASTICITY CHART

Silty Sand Till

Figure No. B5

Project No. 1413191 (1090)

Checked By: TWB/LCC

FIGURE B7A

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS

SAMPLE IDENTIFICATION

PROJECT NUMBER	1413191 (1090)	SAMPLE NUMBER	U1
PROJECT NAME	MTO/EOI-4014-E-0012&4014-E-0013	SAMPLE DEPTH, m	14.36-14.46
BOREHOLE NUMBER	16-1	DATE:	09/14/16

TEST CONDITIONS

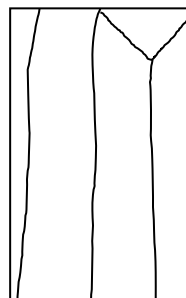
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.02

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	9.58	WATER CONTENT, (specimen) %	0.57
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m ³	25.89
SAMPLE AREA, cm ²	17.60	DRY UNIT WT., kN/m ³	25.75
SAMPLE VOLUME, cm ³	168.60	SPECIFIC GRAVITY	-
WET WEIGHT, g	445.37	VOID RATIO	-
DRY WEIGHT, g	442.85		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	49.3
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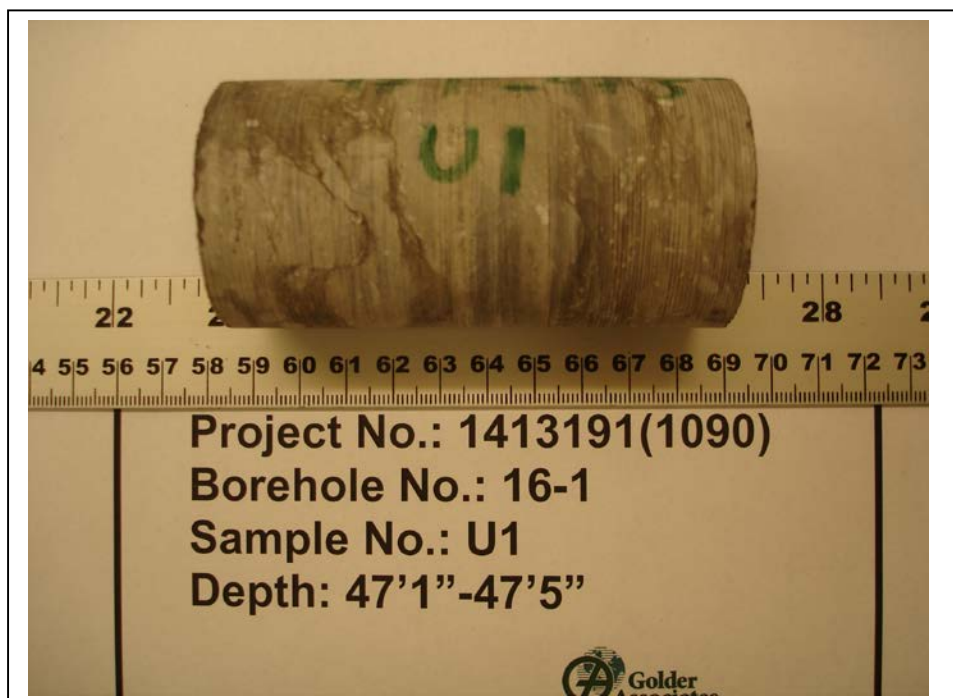
REMARKS:

Checked By:

Golder Associates

UNCONFINED COMPRESSION TEST (UC) OF INTACT ROCK CORE SPECIMENS
ASTM D7012

FIGURE B7B



BEFORE COMPRESSION



AFTER COMPRESSION

Date Sept. 14, 2016
Project 1413191(1090)

Golder Associates

Drawn Frank
Chkd. TWB

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

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