



April 23, 2018

## FOUNDATION INVESTIGATION AND DESIGN REPORT

**Thorold Tunnel Rehabilitation, Site No. 34-177/T  
Highway 58, City of Thorold,  
Ministry of Transportation, Ontario  
G.W.P. 2370-16-00**

**Submitted to:**

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REPORT

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**FOUNDATION REPORT  
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# **PART A**

**FOUNDATION INVESTIGATION REPORT  
THOROLD TUNNEL REHABILITATION (SITE NO. 34-177/T)  
HIGHWAY 58, CITY OF THOROLD  
MINISTRY OF TRANSPORTATION, ONTARIO  
G.W.P. 2370-16-00**



## 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield Limited (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation, hydrogeological and tunnelling engineering services for the proposed rehabilitation of the Thorold Tunnel (Highway 58), in the City of Thorold, Regional Municipality of Niagara, Ontario.

The purpose of this study is to carry out an assessment of the water seepage/leakage into the tunnel and to assist the MH team in developing mitigation measures to minimize the amount of future seepage into the tunnel and along the exposed rock face on the north and south sides of Highway 58, east of the east portal of the Thorold Tunnel. The field investigation consisted of an inspection of the rock faces on the north and south sides of Highway 58, an inspection of the interior walls of the Thorold Tunnel, and a field investigation comprising one borehole advanced west of the West Service building to the north of the Thorold Tunnel.

As part of this assignment, Golder initially completed a desktop study to summarize existing information, and this is presented in the following report:

- “Foundation Desktop Study Report, Thorold Tunnel Rehabilitation, Site No. 34-177/T, Highway 58, Thorold, Ontario Assignment No 2016-E-0001, G.W.P. 2370-16-00”, Geocres No 30M3-300, dated October 13, 2017.

The desktop study, referenced above, summarizes the background/site history of the tunnel investigations and construction/performance issues, a reconnaissance/site visit, and development of context for the hydrogeological setting of the tunnel based on both existing information and site reconnaissance. The desktop study also summarizes the subsurface conditions from the previous investigation reports prepared between 1964 and 1982.

The Terms of Reference and the scope of work for the foundation engineering services are outlined in MTO's Request for Proposal, dated October 2016, which forms part of the Consultant's Assignment No. 2016-E-0001 for this project. Due to legal/insurance challenges related to working on the St. Lawrence Seaway Authority property, the field investigation program was reduced as outlined in Golder's Revised Scope letter dated April 12, 2018. The work has been carried out in accordance with Golder's Project Specific Supplementary Specialty Plan for foundation engineering services for this project, dated May 9, 2017.

## 2.0 SITE DESCRIPTION

### 2.1 Thorold Tunnel

The Thorold Tunnel consists of two separate tube structures, each accommodating two lanes that carry the eastbound (EB) and westbound (WB) traffic of Highway 58 beneath the Welland Canal. The EB and WB tubes are separated by a service tunnel accommodating a walkway and utilities. A public sidewalk runs along the north side of the WB tube. The Thorold Tunnel extends from Ormond Street on the west side of the Welland Canal (West Portal) to just east of Seaway Haulage Road on the east side of the Welland Canal (East Portal), and is approximately 700 m long (see Drawing 1). The Highway 58 grade at the West Portal is at about Elevation 164.6 m, sloping downwards to the low point of the tunnel located approximately below the east side of the Welland Canal at Elevation 150.8 m, and then sloping upward toward the East Portal to about Elevation 163.6 m. The vertical outside height of the tunnel is about 9.6 m, and the inside height of the tunnel is about 6.0 m. The exterior concrete walls of the tunnel are about 1.8 m thick and are covered with a waterproofing zone consisting of 5 mm thick bentonite panels in turn protected by fibreboard. The base slab and top slab are about 2 m thick and 1.4 m



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thick, respectively; it is understood that the base slab contains a waterproofing steel membrane between the concrete and the bedrock, whereas the top slab is covered with a four-ply fibreglass waterproof membrane in turn covered by a 25 mm thick fibreboard and 127 mm concrete slab.

At the West and East Portals, service buildings contain the ventilation shafts and electrical controls. The West Service Building contains the pump controls and security cameras, in addition to the ventilation and electrical controls.

The land area between the west bank of the Welland Canal and the East Portal is owned by the St. Lawrence Seaway Authority (SLSA). The Welland Canal is about 107 m wide and is positioned such that its west bank is about 70 m west of the midpoint of the tunnel. The water level in the Canal is at about Elevation 171.3 m. A gravel road runs along the west side of the Canal, with a City of Thorold multi-use path located west of the gravel road. About 10 m west of the multi-use path, Trillium Railway owns and operates a single rail track. The gravel road, multi-use path and rail track are at about Elevation 176 m; further west, the ground surface is covered by low shrub and grass vegetation and slopes up to Elevation 179.5 m in the vicinity of the Tunnel's West Service Building. A double-set of railway tracks is present east of the West Service Building.

Between the east side of the Canal and the East Portal, the land north and south of the tunnel is used by the SLSA as a dredgate disposal area, and therefore the ground surface contours are variable. It is understood from SLSA that this dredgate disposal area is unlined. The tunnel's East Service Building is bounded by Seaway Haulage Road to the west. The ground surface at the East Service Building is at about Elevation 179.8 m.

The cover over the roof of the tunnel is about 7.5 m thick at the West Portal, then decreases to about 4.5 m below the Welland Canal, and about 13.5 m thick at the East Portal. The cover material above the roof of the tunnel and below the Welland Canal is comprised of rock fill.

## 2.2 Approach Side Slopes East and West of the Tunnel

At the West Portal, the north and south cut slopes of the Highway 58 approach embankment are inclined at about 2 horizontal to 1 vertical (2H:1V) and are vegetated with trees which are standing vertically (i.e., no evidence of curvature on the tree trunks).

At the East Portal there are vertical rock face cuts on the north side of the WB tube and on the south side of the EB tube, and the rock face is about 7.3 m high at the portal. The Highway 58 pavement surface gradually rises to the east to Davis Road where it is about coincident with the surrounding ground surface. At a distance of about 130 m east of the east portal, the surface of Highway 58 is approximately coincident with the near-horizontal bedrock surface. There is evidence of seepage between the bedding planes on the rock faces, and vegetation growth is present at locations of seepage. The seepage amount is greater on the north rock face compared to the south rock face. There is also rock debris present at the base of the rock faces, suggesting that rock has fallen from the face.

On the north side of Highway 58, the rock face extends a distance of about 75 m from the east portal. Rock bolts installed into the north rock cut face close to the east portal are evident, and there is also evidence of a formed concrete cap on the bedrock surface, although the concrete cap is not continuous. At a distance of about 28 m from the east portal, a more substantial concrete wall that is about 2 m wide extends from the sidewalk level to the full height of the rock face. East of the 2 m wide concrete wall, the rock face/concrete wall steps back by about 1 m (see Photograph 1 below). From this location to a distance of about 43 m easterly, a concrete wall extends





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from the bedrock surface and downwards (partially covering the rock face) over a distance of about 1.5 m. A light pole is located on the north side of Highway 58, in close proximity to the rock face. At the base of the light pole, fallen rocks have accumulated. The distance from the rock face to the light pole varies vertically from about 1 m to 1.2 m (see Photograph 1), and the light pole is located about 1.9 m east of the full-height concrete wall (see Photograph 2).



Photograph 1: Looking west at the light pole (north side)



Photograph 2: Looking north at the light pole (north side)

On the south side of Highway 58, the rock face extends a distance of about 132 m east of the east portal. At a distance of about 85 m from the east portal, a concrete cap/wall extends from the bedrock surface and mostly covers the rock face but has an irregular shape. Near the east portal, the rock face is undercut and it is understood that pieces of rock occasionally fall from the face onto the curb.

The 1967 construction drawings indicate that 250 mm diameter subdrains are present at the base of the south and north rock faces, at a depth of about 1.2 m below the grade shown on the drawing.

### 3.0 SITE RECONNAISSANCE AND ROCK FACE INSPECTION

#### 3.1 Inspection of Thorold Tunnel

On December 20, 2016 and May 11, 2017, Golder's foundation engineer inspected the north tunnel wall from the sidewalk, examining the vertical cracks in the tunnel wall, and the cracking through both the original and patched concrete where the tunnel wall meets the roof. Icicles (representing frozen seepage water) were observed on the vertical cracks and some of the joints in the south tunnel wall in the winter 2016 inspection. At the contraction joint at Station 13+002 in the south tunnel (located about 70 m west of the east portal in the area of the Pondage Canal), it is understood that several years ago MTO Maintenance placed a rubber mat over the joint from the curb to the corbel, in order to minimize the potential for ice adhering to the wall. Small, conical or flattened piles of soil were observed on the sidewalk at three of the existing crack locations (at approximately 55 m, 120 m and 175 m from the West Portal) in the north (WB) tube, and some fine soil particles were also observed on the sidewalk below



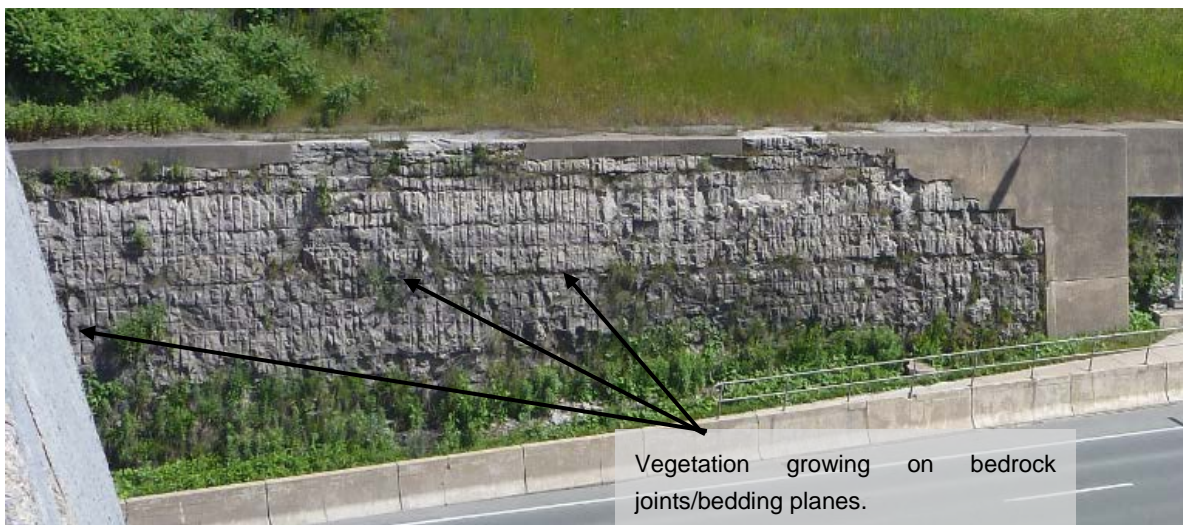


cracks at about 390 m and 530 m from the West Portal; these observations may indicate that fine soil particles are being carried with the water seepage through the cracks. Wet pavement areas were also observed, and these areas are believed to be due to water emanating from beneath the pavement.

### 3.2 Rock Face Inspection

A senior hydrogeologist and rock mechanics engineer conducted a site reconnaissance of the north and south bedrock cut faces adjacent to the East Portal on June 22, 2017 to evaluate the local site conditions.

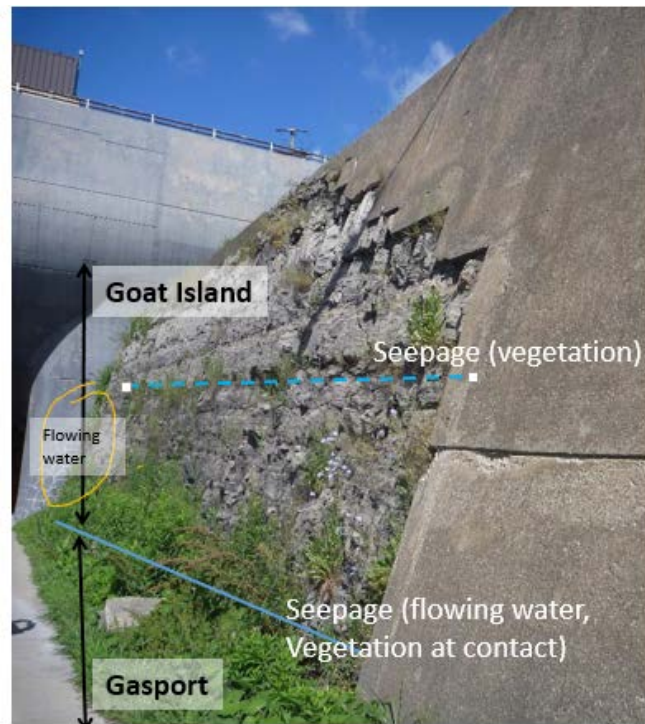
An area of exposed bedrock along the rock cut immediately east of the East Portal on the northern side of Highway 58 was examined. Based on bedrock mapping, the bedrock at the exposed rock face consists of dolomitic limestone of the Goat Island Member overlying the Gasport Member, both of which are part of the Lockport Formation. The rock face was observed to be damp with diffuse seepage occurring along the bedrock surface, and from a horizon midway up the rock cut. Vegetation (grass and small shrubs) was noted to be present along the bedding partings suggesting that seepage commonly occurs at this location. In addition, a damp rock surface and emanation of a low rate of seepage were observed at the interface of the bedrock cut and the northeast edge of the East Portal. During the winter the water seepage on the rock face freezes and forms icicles and water also seeps onto the sidewalk and freezes. There are wet swampy areas present behind the rock cut as well as standing water further north, which are inferred to be the source of the seepage water observed on the rock cut face. Rock bolts installed into the north rock cut face close to the East Portal are evident and there is also evidence of formed concrete near the crest of the slope. Rock debris has accumulated at the base of the rock face indicating that rock has fallen from the face. Selected photographs of these observed conditions are presented in Photographs 3 and 4.



**Photograph 3: Looking north at rock face east of the East Portal.**



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**Photograph 4: Looking westward at rock face on north side of Highway 58 east of the East Portal.**

A bedrock cut is also exposed along the south side of Highway 58 east of the East Portal. This face was observed to be dry to damp, but no groundwater seepage was noted at the time of the site visit. Vegetation growth (grass) was also observed to be present along a bedrock bedding plane about halfway up the rock cut face. There is evidence of some undercutting of the rock face leaving an overhang, and rock debris has accumulated at the base of the rock face indicating that rock has fallen from the face. It is understood that historically, water seepage from the face sometimes seeps out onto the road surface and has the potential to freeze during winter conditions. The conditions on the south face east of the East Portal are shown in Photograph 5.



**Photograph 5: Looking south at south rock cut face east of the East Portal.**



## 4.0 INVESTIGATION PROCEDURES

### 4.1 1964 Investigation

In 1964 H.G. Acres & Company Limited, Consulting Engineers (Acres) completed a geotechnical field investigation for the Thorold Tunnel and the results are presented in a report titled “Thorold Tunnel, Feasibility Report Appendices, Appendix A – Field and Laboratory Investigations and Geotechnical Design Considerations, Tunnel and East Approach,” dated October 30, 1964. Several horizontal tunnel alignments were under consideration during the field investigation:

- four boreholes (1001, 1002, 1004 and 1005) were advanced for Scheme 6 alignment, located 300 m to the north of the existing alignment;
- one borehole (1003) was advanced for Scheme 4 alignment, located 1,525 m to the south of the existing alignment; and
- twenty-seven boreholes and three test pits were advanced for the recommended location of the tunnel which was designated as the Scheme 3 alignment (see Drawing 1).

The locations of the boreholes from the 1964 investigation are shown on Plates A1 to A4 of the Acres (1964) report, copies of which are included in Appendix A; the locations of the boreholes from the 1964 investigation specifically for the recommended Scheme 3 alignment are also shown on Drawing 1, following the text of this report. It is understood that the bedrock contours shown on Plate A1 were developed using data from the SLISA.

The boreholes for the Scheme 3 alignment were advanced with diamond drills, supplied and operated by F.E. Johnston Drilling Co. Ltd. The test pits were excavated to the west of the current location of the West Portal with a tractor-mounted backhoe supplied and operated by W. Duffin Construction to a depth of about 2.7 m to expose the bedrock surface. In a total of twelve boreholes, samples of the overburden were obtained using a 50 mm diameter Shelby tube, or a 50 mm diameter split spoon, at approximately 1.5 m intervals of depth. The Acres (1964) report indicates that when attempting to obtain Shelby tube samples from the very stiff portions of the overburden, it was typically not possible to push the Shelby tube more than 150 mm and it was often necessary to drive the Shelby tubes using the 63.5 kg weight hammer. Bedrock core in NX-size was obtained from sixteen boreholes, of which twelve boreholes were drilled vertically and four boreholes were drilled inclined at 45 degrees. Several boreholes were advanced without sampling for the purpose of determining the bedrock surface; at these locations 3 m of bedrock cored was obtained.

The Acres (1964) report notes that, in every borehole advanced, drilling water was lost over a short depth below the bedrock surface, and that the static water level was typically above the bedrock surface and usually at an elevation equal to or higher than the water level in the Welland Canal at the time of the investigation.

In boreholes where the bedrock was cored for greater than a 3 m length, a packer test was carried out after each 1.5 m length of bedrock coring. The tests were carried out by lowering a 1.5 m long perforated pipe to the bottom of the borehole (at each test) with a single packer positioned at the top of the pipe. The tests were carried out by applying water pressure to the test section equal to 23 kPa per metre (1 psi/ft) of depth to the section being tested. Water level readings were taken at one minute intervals until steady state rate of flow was achieved, which typically took between 5 minutes and 10 minutes. Standpipes were installed in five of the boreholes (1001, 1009, 1014, 1016 and 1022).



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The Acres (1964) report indicates that pockets of gas were encountered while coring the bedrock; the gas pressure was typically relieved fairly rapidly, with the exception of Boreholes 1016 and 1018, located near the East Portal, and in Borehole 1025 where “considerable volumes of gas were released.”

A summary of the soil sampling, bedrock coring procedures, bedrock coring depths, packer testing and piezometer installations at each borehole advanced for the Scheme 3 alignment is presented below.

Borehole No.	Overburden Sampling	Bedrock Coring Depth/Elevation ** (m)	Packer Testing Carried Out	Standpipe Piezometer Installed
1006	Split-spoon and Shelby to bedrock surface	6.3 - 34.2 (172.5 - 144.6)	Yes	No
1007	Split-spoon to bedrock surface	3.0 - 29.4 (171.6 - 145.2)	Yes	No
1008	Drill to bedrock surface	None	No	No
1009	Shelby to bedrock surface	7.1 - 33.9 (171.8 - 144.9)	Yes	Yes
1010	Split-spoon, Shelby and Core Barrel to bedrock surface	11.6 - 13.4 (167.43 - 165.66)	No	No
1011	Split-spoon, Shelby and Core Barrel to bedrock surface	11.8 - 34.0 (167.3 - 145.1)	Yes	No
1012	Split-spoon and Shelby to bedrock surface	10.5 - 34.8 (170.4 - 146.0)	Yes	No
1013	Shelby to bedrock surface	6.4 - 31.7 (171.02 - 145.72)	Yes	No
1014	Shelby to bedrock surface	7.8 - 39.4 (172.1 - 140.5)	Yes	Yes
1015	Shelby to bedrock surface	7.0 - 32.0 (171.8 - 146.8)	Yes	No
1016	Shelby to bedrock surface	5.9 - 31.5 (171.3 - 145.8)	Yes	Yes
1017	Split-spoon to bedrock surface	6.7 - 10.6 (171.15 - 167.27)	No	No
1018	Shelby to bedrock surface	6.6 - 30.5 (170.81 - 146.88)	Yes	No
1019	Drill to bedrock surface	8.8 - 11.9 (169.9 - 166.8)	No	No
1020	Split-spoon and Shelby to bedrock surface	8.2 - 11.5 (170.44 - 167.15)	No	No
1021	Shelby to bedrock surface	8.0 - 24.7 (169.6 - 152.9)	Yes	No
1022	No sampling; advance casing to bedrock surface	7.9 - 25.8 (168.8 - 150.8)	No	Yes
1023	Split-spoon and Shelby to bedrock surface	None	No	No





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Borehole No.	Overburden Sampling	Bedrock Coring Depth/Elevation ** (m)	Packer Testing Carried Out	Standpipe Piezometer Installed
1024	Split-spoon and Shelby to bedrock surface	None	No	No
1025*	No sampling; advance casing to bedrock surface	4.0 - 42.9* (170.7 - 131.8)*	Yes	No
1026*	No sampling; advance casing to bedrock surface	9.8 - 44.0* (168.8 - 134.6)*	Yes	No
1027*	No sampling; advance casing to bedrock surface	4.3 - 42.7* (170.5 - 132.1)*	Yes	No
1028*	No sampling; advance casing to bedrock surface	9.1 - 43.1* (168.8 - 134.8)*	Yes	No
1029	Drill to bedrock surface	8.9 - 11.7 (170.3 - 167.5)	No	No
1037	Drill to bedrock surface	8.2 - 11.0 (171.15 - 168.37)	No	No
1045	No sampling; advance casing to bedrock surface	None	No	No
1046	No sampling; advance casing to bedrock surface	None	No	No

\* Depth to / Elevation of bedrock surface is not consistent with the bedrock surface elevation in the adjacent boreholes in the general area (as shown on Plate A4 in Appendix A of the Acres (1964) report).

\*\* Elevations given in this table are consistent with those shown on the boreholes records and are referenced to SLISA datum. The SLISA datum is 0.35 m higher than Geodetic datum.

The ground surface and bedrock surface contour elevations presented in the Acres (1964) report are referenced to the SLISA datum using Benchmark 210A (Elevation 573.03 ft.). The borehole locations were tied-in to stations on the Canal using then-existing Canal survey baselines; the borehole locations shown in Drawing 1 have been established relative to various streets / identifiable features on Plate 1 of the Acres (1964) report and transposed into the MTM NAD 83 (Zone 10) locations/coordinates presented on Drawing 1. The ground surface elevations as depicted on Drawing 1 have been adjusted from the SLISA datum to be consistent with Geodetic datum. This adjustment is -0.35 m relative to the SLISA datum as referenced on the borehole records from the Acres 1964 report.

### 4.2 Current Investigation

The field work for the current foundation investigation was carried out between October 17 and November 7, 2017, during which time Borehole GT17-01 was advanced at the location shown on Drawing 1. The borehole records and the lists of abbreviations and symbols to assist in the interpretation of the borehole and records are provided in Appendix A.

The borehole investigation was carried out using a truck-mounted CME 75 drill rig, supplied and operated by Aardvark Drilling of Guelph, Ontario. The borehole was advanced through the overburden using 210 mm outside diameter hollow-stem augers and HW casing. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth,



## FOUNDATION REPORT THOROLD TUNNEL REHABILITATION

using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-08)<sup>1</sup>. Samples of the bedrock were obtained using an 'HQ' size rock core barrel and coring techniques. The borehole was advanced to auger refusal (i.e. inferred bedrock) then further advanced to a depth of 36.2 m (Elevation 142.5 m) below existing ground surface, including coring of bedrock for core lengths of 29.1 m. Photographs of the recovered bedrock core samples are provided in Appendix B.

### ***Blow-Out Prevention Procedures***

Due to the naturally occurring gas that was previously encountered in the 1964 boreholes, and the known potential for naturally occurring gas in these bedrock formations, a well licence was required from the Ministry of Natural Resources and Forestry (MNRF). Golder submitted the well licence application and Well Licence 12542 was issued by the MNRF (see Appendix A for a copy of the well licence). In order to seal the rock formation where the gas was expected, the borehole was advanced to a depth of about 7 m below the bedrock surface (14 m below ground surface) and a 114 mm diameter conductor casing was cemented inside a 159 mm diameter cored borehole. Figure 1, following the text of this report, presents the installation and size details for the conductor casings. After the casings were grouted, the blow-out prevention (BOP) system was installed.

Photographs 6 and 7 illustrate the BOP system setup. The annular BOP, which is attached to the borehole casings, is connected to the degasser and the control unit. In the event of a blowout when drilling, the drill was immediately stopped and the annular BOP was initiated. This would seal the space around the drill rods and force the gas and water to come up around the drill rods and towards the degasser. Once separated, the gas vented out through a 7 m stack and the water accumulated in a holding basin.



**Photograph 6: Blow-out prevention system showing the degasser and venting stack (left) and drill (right)**

<sup>1</sup> ASTM D1586-08a – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of the soil.





**Photograph 7: Blow-out prevention system on the borehole**

### ***Packer Testing***

In situ hydrogeological testing, in general accordance with procedures defined in ASTM D4630, was conducted at six depths in Borehole GT17-01 using a dual pneumatic packer setup connection to an on-surface nitrogen tank through an inflation line. Upon completion of drilling, the packer assembly was lowered into the borehole to isolate a select depth interval within the rock and a constant pressure head (CH) test was performed. The test results were then used to evaluate the hydraulic conductivity within the isolated packer interval. A pressure gauge data logger, manufactured by In Situ Inc., was used to monitor water pressure responses in the isolated interval during the tests. Flow rates and test pressures in the isolated interval were recorded during CH tests as well as being recorded by the data logger. The water pressure profiles obtained were used to calculate estimates of hydraulic conductivity using standard steady-state analysis methods.

The groundwater conditions and water levels in the open borehole were observed during the drilling operations. Two vibrating wire piezometers were installed at depths of 16.1 m and 23.2 m below ground surface. The borehole was backfilled with either grout or bentonite upon completion in accordance with Ontario Regulation 903, Wells (as amended).

The field work was observed by members of Golder's engineering and technical staff, who located the boreholes, arranged for the clearance of underground services including both public and private locates, observed the drilling, sampling and in situ testing operations, and logged the boreholes. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM Standards, as appropriate. Classification testing (water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. Unconfined compression (UC) tests were carried



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out on selected specimens of the bedrock core samples by Golder's Mississauga Laboratory. Two rock core samples were submitted to the University of Western Ontario for free swell testing. The results of the geotechnical laboratory testing on soil and rock samples from the current investigation are included in Appendix B.

The borehole location and the ground surface elevation were obtained using a GPS (Trimble XH 3.5G), having an accuracy of 0.1 m in the vertical and horizontal directions. The location provided on the borehole and drillhole records and shown on Drawing 1 are positioned relative to MTM NAD 83 (Zone 10) coordinates system, and the ground surface elevations are referenced to Geodetic datum. The borehole location and ground surface elevation are summarized below.

Borehole No.	Location (MTM NAD 83)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (Latitude, °)	Easting (Longitude, °)		
GT17-1	4,775,177.9 (43.11604)	329,361.1 (-79.19819)	178.7	36.2*

\* includes 29.1 m of bedrock core

## 5.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

### 5.1 Regional Geology

This section of Highway 58 is located within the Iroquois Plain physiographic region, as delineated in the *Physiography of Southern Ontario* (Chapman and Putnam, 1984). The Iroquois Plain extends around the western shores of Lake Ontario and 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. This site is bound to the north by shoreline beach deposits from Glacial lake Iroquois such as the Homer Bar on which downtown St. Catharines is located, and the Niagara Escarpment located some 3 km to the south.

Surficial sediments are typically comprised of silty and clayey till of the Halton Till sheet according to the *Quaternary Geology of the Niagara-Welland Area* (Ontario Geological Survey Map 2496; Feenstra, 1984). The Halton Till sheet is underlain by an older, red, sandy and silty till, possibly the Wentworth Till sheet (OGS Preliminary Map 764, Feenstra 1972). Shallow depressions on the surface of the clay plain upslope of the Homer Bar are infilled with bog sediments, while fill materials comprised of earth and rock fill associated with the canal construction occur in the vicinity of the former Welland Canal (OGS Preliminary Map 764, Feenstra 1972).

The Niagara Escarpment is the major topographic feature in the region (Karrow and White, 1998). The bedrock escarpment runs in an east-to-west direction along the southern shore of Lake Ontario and extends northward towards the Bruce Peninsula. The Niagara Escarpment is located approximately 2 km north of the Site, and rises up to 120 m above the lake plain to the north (Menzies and Taylor, 1998). The escarpment is discontinuous north of the site as it has a few re-entrant features which could possibly mark the sites of ancient tunnel valleys that have been infilled by glacial sediments (Menzies and Taylor, 1998). Immediately south of the Niagara Escarpment are a series of linear ridges that have been interpreted as terminal moraines (Chapman and Putnam, 1984).

The Niagara peninsula south of the Niagara Escarpment consists of Palaeozoic sedimentary strata bedrock of the Silurian and Devonian age. The beds dip to the south under Lake Erie with a shallow inclination of approximately



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5.7 m/km. The massive dolomitic limestone is from the Salina Formation in the Welland area and is comprised of siltstone and calcareous shaly interbeds with occasional limestone layers and inclusions of gypsum within the dolomite. These strata occur within the middle part of the Salina formation and are of Silurian age. Two escarpments, the Niagara Escarpment and Onondaga Escarpment, run east-west in the northern area of Port Colborne and south of the Thorold Tunnel under the Welland Canal, respectively. The bedrock depression between the Onondaga Escarpment and Niagara Escarpment is infilled with overburden with elevations ranging between 460 m and 500 m.

Near the Thorold Tunnel, beneath the glacial till lies the Goat Island member of the Lockport Formation, which is comprised of massive to thickly bedded dolomite and is approximately 6 m thick at the site. The Gasport member lies below the Goat Island member and is a similar dolomitic limestone with occasional gypsum pockets. However, within this member there is a 3 m thick, black shaly zone that has been interpreted to have contributed to distress at the tunnel alignment due to the weak rock mass. The thickness of the Gasport member varies but is approximately 10 m thick at the tunnel site and is underlain by the Decew Formation which is typically thin to thickly bedded dolomite and has a thickness of about 2 m to 4 m. The Decew Formation is underlain by the Rochester Formation which consists of dark grey shale.

Throughout the Welland area, the surface topography of the overburden overlying the bedrock contact is characterized by low relief and poor drainage (Menzies and Taylor, 1998). Due to the presence of the weathered and fractured overburden-bedrock interface, a major source of groundwater is found in an aquifer contained within this upper zone and within pervious zones that occur locally at the overburden-bedrock interface. There is no known faulting in the immediate vicinity of the site; however a significant thrust fault is present in the region approximately 100 km east of the tunnel location.

### 5.2 Subsurface Conditions

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the borehole advanced during the current investigation are presented on the borehole and drillhole records provided in Appendix A. The results of the geotechnical laboratory tests carried out on selected soil and bedrock core samples from the current investigation are contained in Appendix B. The results of the in-situ field tests (i.e. SPT “N” values) as presented on the borehole records and in subsections of Section 5.2 are uncorrected.

The stratigraphic boundaries in the overburden, shown on the borehole records and on the stratigraphic profile on Drawing 1, are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests. These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. The bedrock formation contacts as presented on the drillhole records are based on visual examination of the bedrock core samples. Furthermore, subsurface conditions will vary between and beyond the borehole location. It should be noted that the interpreted stratigraphy shown on Drawing 1 is a simplification of the subsurface conditions from the current investigation and the investigation carried out by Acres in 1964.

In summary, the subsurface conditions in the borehole advanced during the current investigation consists of sandy gravel fill material with layers of silty clay fill. The fill material is underlain by the Goat Island member and Gasport member dolomitic limestone of the Lockport Formation. The Lockport Formation is underlain by dolomitic limestone of the Decew Formation which in turn is underlain by shale of the Rochester Formation. A more detailed description of the subsurface conditions encountered in the borehole advanced during the current investigation is provided in the following sections.



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Golder's Foundation Desktop Study Report, referenced in Section 4.1, presents a detailed description of the subsurface conditions encountered in the boreholes advanced by Acres (1964). Information on the bedrock conditions, details about the gas encountered and the results of the packer testing from the Acres (1964) report are also presented in sections below.

All elevations given in the following sections as it relates to the Acres (1964) report are referenced to the SLSA datum, obtained relative to Benchmark 210A (Elevation 573.03 feet). The SLSA datum is 0.35 m higher than Geodetic datum.

### 5.2.1 Fill

In Borehole GT17-01, fill consisting primarily of gravelly sand to sandy gravel was encountered and extended to a depth of 7.1 m (Elevation 171.6 m) below ground surface. Within the granular fill, layers of silty clay fill were encountered at depths of 0.7 m (Elevation 178.0 m) and 1.5 m (Elevation 177.2 m) and were 0.5 m and 0.7 m thick, respectively.

The SPT "N" values within the granular fill range from 13 blows to 47 blows per 0.3 m of penetration with two samples having an SPT "N" value of 50 blows per 30 mm and 80 mm penetration, indicating that the granular fill generally has a compact to dense compactness condition. The two higher SPT "N" values may be more reflective of the split-spoon sampler encountering gravel. Two SPT "N" values measured within the cohesive fill were 12 blows and 17 blows per 0.3 m of penetration, indicating that the cohesive fill has a stiff to very stiff consistency.

The result of a grain size distribution test completed on one sample of the granular fill is presented on Figure B1 in Appendix B. The sandy gravel fill contains trace to some sand and trace clay. The water content measured on a sample of the granular fill material was about 3 per cent.

The result of a grain size distribution test completed on one sample of the cohesive fill is presented on Figure B2 in Appendix B. The silty clay fill contains trace to some sand and trace gravel. An Atterberg limits test was carried out on the fines portion of one sample of the cohesive fill, and measured a liquid limit of about 38 per cent, a plastic limit of about 19 per cent, and a plasticity index of about 19 per cent; as shown on the plasticity chart on Figure B3 in Appendix B, this result indicates that the fines portion of the cohesive fill can be classified as a silty clay of medium plasticity. The water content measured on a sample of the cohesive fill was 18 per cent, near the plastic limit of the material.

### 5.2.2 Bedrock

In Borehole GT17-01, bedrock was encountered at a depth of about 7.1 m (Elevation 171.6 m) below ground surface and core samples of the bedrock were recovered. The contacts between the formations and their respective members were selected based on visual examination of the core. Summarized below are the bedrock surface elevations for the major bedrock formations and members encountered in the current investigation, and their relative thickness.





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Formation	Member	Depth to Surface of Formation (m)	Elevation of Surface of Formation (m)	Approximate Thickness of Formation (m)
Dolomitic limestone of Lockport Formation	Goat Island Member	7.1	171.6	8.7
	Gasport Member	15.8	162.9	6.3
Dolomitic limestone of Decew Formation	--	22.1	156.6	3.6
Shale of Rochester Formation	--	25.7	153.0	>10.5*

\* Borehole GT17-01 terminated in the Rochester Formation at a depth of 36.2 m (Elevation 142.5 m) below ground surface.

Summarized below are the bedrock surface elevations for the major bedrock formations and members encountered during the Acres (1964) borehole drilling investigation, and their relative thickness.

Formation	Member	Approximate Range of Elevation* of the Surface of the Formation (m)	Range of Thickness (m)
Lockport	Goat Island	172.5 – 167.3	5.2 – 7.6
	Gasport	165.0 – 159.5	6.4 – 8.5
Decew	-	158.2 – 148.5	2.1 – 4.0
Rochester	-	154.5 – 142.3	13.4+

\* Elevations given in this table are consistent with those shown on the boreholes records and are referenced to SLISA datum. The SLISA datum is 0.35 m higher than Geodetic datum.

As presented above, the elevation of the surface of the formations encountered in the borehole advanced for the current investigation is within the range of elevation from the Acres (1964) report.

In general, the dolomitic limestone bedrock core samples of the Lockport and Decew Formations are described as fresh, medium to thickly bedded, fine to medium grained, slightly to moderately porous, medium strong to strong, grey, and containing limestone and shale partings or thin interbeds at varying intervals. In general, the shale bedrock core samples of the Rochester Formation are described as fresh, thickly bedded, fine grained, slightly porous, medium strong, grey to black, shale to dolomitic shale. The drillhole records are presented in Appendix A, and photographs of the recovered bedrock core samples are shown on Figure B4 in Appendix B. The degree of weathering of the bedrock core samples (i.e. fresh – W1), and the strength classification of the intact rock mass based on field identification (i.e. medium strong to strong – R3 to R4) are described in accordance with the International Society for Rock Mechanics (ISRM<sup>2</sup>) standard classification system.

The Rock Quality Designation (RQD) measured on the bedrock core samples generally ranges from about 94 per cent to 100 per cent, indicating a rock mass of excellent quality as per Table 3.10 of CFEM (2006)<sup>3</sup>. The Total

<sup>2</sup> International Society for Rock Mechanics Commission on Test Methods, 1985. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr. Vol 22, No. 2, pp. 51-60.

<sup>3</sup> Canadian Geotechnical Society, 2006. Canadian Foundation Engineering Manual (CFEM), 4<sup>th</sup> Edition. The Canadian Geotechnical Society, BiTech Published Ltd., British Columbia.



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Core Recovery (TCR) and Solid Core Recovery (SCR) of samples recovered are between 96 per cent and 100 per cent and between 68 per cent and 100 per cent, respectively.

Two Unconfined Compression (UC) tests (ASTM D7012)<sup>4</sup> were carried out on selected core samples of the Goat Island bedrock obtained in Borehole GT17-01 and measured uniaxial compressive strengths of about 71 MPa and 101 MPa, as summarized on Figures B5 and B6 in Appendix B. Based on the laboratory UC tests, in accordance with Table 3.5 in CFEM (2006)<sup>4</sup>, the limestone bedrock is classified as medium strong to strong (R3, 50 MPa < UCS < 75 MPa to R4, 75 MPa < UCS < 100 MPa).

### Free Swell Test Results

Two rock core samples from Borehole GT17-01 were submitted to the University of Western in London, Ontario for free swell testing to provide information on the swelling potential of the shale of the Gasport Member of the Lockport Formation and the Rochester Formation. The following provides the depth, elevation and Formation for the two bedrock core samples tested:

Borehole	Depth (m)	Elevation (m)	Bedrock	Member/Formation
GT17-01	17.2	161.5	Shale	Gasport Member, Lockport Formation
	28.3	150.4	Shale	Rochester Formation

For the free swell tests, the bedrock core samples are tested without any applied pressure and are allowed to swell freely in all directions. Free swelling was tested on rock samples in vertical and horizontal direction (sample axis perpendicular to the bedding planes). The testing also included the determination of the moisture content, salinity of the pore fluid in the test specimen, and calcite content of the samples. The swell testing results for the free swell tests are given as swelling strains (in %) versus time (in log scale to base 10) curves. The results are presented in Appendix B.

Golder's analysis of the test results from horizontal and vertical free swell tests indicates the following free swelling potential of the shale bedrock. The results indicate that the free swelling potential of the Rochester shale is much greater than the Gasport shale and for both shales the free swell potential in the vertical direction is higher than in the horizontal direction.

	HSP <sup>1</sup> (X)	HSP <sup>1</sup> (Y)	Average HSP <sup>1</sup> (X+Y)/2	VSP <sup>2</sup> (Z)	Calcite Content
	[% per log cycle]	[% per log cycle]	[% per log cycle]	[% per log cycle]	[%]
Range	0.02 - 0.1	0.07 – 0.1	0.05 - 0.1	0.05 – 0.32	28.5 and 30.5
Average	--	--	0.07	0.19	--

<sup>1</sup> HSP Horizontal Swelling Potential

<sup>2</sup> VSP Vertical Swelling Potential

<sup>4</sup> ASTM D7012 – Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens





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### 5.2.3 Bedrock Hydraulic Conductivity

Six packer tests were carried out during the current investigation, in accordance with the procedures presented in Section 4.2. The following summarizes the depth and elevation of the interval of the test, the bedrock formation and the estimated hydraulic conductivity. The results confirm the tight nature of the rock mass, with intermediate hydraulic conductivities in the range of  $10^{-6}$  to  $10^{-8}$  m/s for the lower rock mass of the Gasport, Decew, and Rochester formations. The upper rock mass in the Goat Island formation shows high hydraulic conductivities in the range of  $10^{-4}$  m/s.

Borehole	Packer Test Interval Depth (Elevation)		Rock Member and Formation	Water Level (mbgs)	Estimated Hydraulic Conductivity (m/s)
	From (m)	To (m)			
GT17-01	7.72 (170.98)	11.02 (167.68)	Goat Island member, Lockport Formation	N/A <sup>1</sup>	$1 \times 10^{-4}$
	9.72 (168.98)	13.02 (165.68)	Goat Island Member, Lockport Formation	N/A <sup>1</sup>	N/A <sup>1</sup>
	16.75 (161.95)	20.05 (158.65)	Gasport	9.90	$9 \times 10^{-6}$
	19.75 (158.95)	23.05 (155.65)	Gasport-Decew	9.95	$2 \times 10^{-7}$
	22.78 (155.92)	26.08 (152.65)	Decew and Rochester Formation	10.96	$3 \times 10^{-6}$
	25.75 (152.95)	29.05 (149.65)	Rochester Formation	9.20	$8 \times 10^{-8}$

1. Packer test was conducted above the water table in a dry condition. Water level did not rise above the instrumentation for the duration of the testing phase; therefore, the test results are not valid.

### Summary of Packer Testing from the Acres (1964) Report

Packer testing was carried out in 17 of the 32 boreholes advanced by Acres for the recommended alignment. After each 1.5 m of bedrock coring, a packer test was carried out in the boreholes where the bedrock was cored greater than a 3 m length. The test was carried out by lowering a 1.5 m long perforated pipe to the bottom of the borehole (at each test) with a single packer above it. The tests were carried out using gauge pressures equal to about 23 kPa/m (1 psi/ft) of depth to the section being tested. Water level readings were taken at one minute intervals until a steady state rate of flow was achieved, which typically took between 5 minutes and 10 minutes.

Borehole No.	Number of Tested Intervals	Top Interval (m)	Bottom Interval (m)	Average Bedrock Hydraulic Conductivity (m/sec)
1001	12	14.0	32.2	$4.0 \times 10^{-7}$
1003	10	15.8	31.4	$3.1 \times 10^{-6}$
1006	10	9.4	34.2	$6.2 \times 10^{-6}$
1007	17	5.4	29.4	$4.1 \times 10^{-6}$



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Borehole No.	Number of Tested Intervals	Top Interval (m)	Bottom Interval (m)	Average Bedrock Hydraulic Conductivity (m/sec)
1009	17	7.1	33.9	$4.6 \times 10^{-6}$
1011	13	13.2	34.0	$3.0 \times 10^{-6}$
1012	15	11.0	34.8	$3.8 \times 10^{-6}$
1013	16	7.1	31.7	$4.5 \times 10^{-6}$
1014	18	8.3	37.5	$2.1 \times 10^{-6}$
1015	15	7.1	32.0	$2.0 \times 10^{-6}$
1016	17	5.9	31.5	$1.3 \times 10^{-6}$
1018	15	7.7	30.5	$1.9 \times 10^{-6}$
1021	10	8.2	24.7	$2.9 \times 10^{-6}$
1025	23	4.6	43.0	$5.5 \times 10^{-6}$
1026	17	10.4	44.0	$1.8 \times 10^{-6}$
1027	16	5.3	42.7	$6.5 \times 10^{-6}$
1028	14	10.1	43.1	$1.8 \times 10^{-6}$

Coefficients of permeability were established from the test results in each borehole such that maximum and average coefficients were produced for each formation member. A summary of the bedrock permeability of the four bedrock formations as derived from the 255 individual packer tests is presented below:

Formation	Member	Co-efficient of Permeability of $10^{-6}$ (m/sec)		
		Maximum	Minimum	Average
Lockport	Goat Island	9.9	0	5.2
	Gasport	6.8	0	1.8
Decew	-	3.6	0	0.9
Rochester	-	3.0	0	0.5

### 5.2.4 Gas

During drilling of Borehole GT17-01, concentrations of methane and hydrogen sulphide were measured at the drill head using a RKI GX-2015 4 sensor gas monitor ( $\text{CH}_4/\text{O}_2/\text{H}_2\text{S}/\text{CO}$ ). During the drilling program, gas was encountered while advancing the borehole within the Rochester Formation, between a depth of 27.8 m (Elevation 150.9 m) and 29.4 m (Elevation 149.3 m) below ground surface and a methane concentration of 29 per cent of the Lower Explosive Limit and a CO concentration of 150 ppm was measured within this interval.



### *Summary of Gas Encountered from the Acres (1964) Report*

The Acres (1964) report indicates that pockets of gas were encountered while coring the bedrock and that the pressure was typically relieved fairly rapidly, with the exception of Boreholes 1016 and 1018, located near the East Portal and in Borehole 1025 where “considerable volumes of gas were released.” The Acres (1964) report further indicates that the pressure at some depths was sufficient to blow the drilling water inside the casing about 8 m above the ground surface, that the gas had an odour similar to that of hydrogen sulphide and that it “burned with a blue flame and deposited a black coating on the drill rods.” A summary of the measured back pressure (which is the residual gauge pressure remaining after packer testing) is provided in Table A4 of the Acres (1964) Feasibility Report – Appendices, included in Appendix A to the desktop report.

### **5.2.5 Groundwater**

During the drilling operations for Borehole GT17-01, the water level inside the casing was measured at the start and end of each day and ranged from depths of between 9.8 m and 11.5 m (between Elevation 168.9 m and 167.2 m) below ground surface. Because water was required to advance the drill bit, the water levels are not considered to represent the stabilized groundwater level at the site. It is noted that during bedrock coring operations, water used to advance the core barrel did not recirculate back to ground surface, indicating that the water was dissipating into the bedrock formation.

Two vibrating wire piezometers (VWP) were installed in Borehole GT17-01 and details of the VWP installations and measured groundwater levels are shown on the borehole records in Appendix A. The following summarizes the VWP tip depth and elevation and the water levels recorded in the VWPs.

VWP No.	Bedrock Member, Formation	VWP Tip Depth	VWP Tip Elevation	Depth to Water Level (Groundwater Elevation)
1	Goat and Gasport Island Member, Lockport Formation	16.1 m	162.6 m	12.6 m (Elev 166.1 m) (February 1, 2018)
2	Decew Formation	23.2 m	155.5 m	12.0 m (Elev.166.7 m) (February 1, 2018)

It should be noted that the groundwater level in the area is subject to seasonal fluctuations and precipitation events, and should be expected to be higher during wet periods of the year. The shallow groundwater conditions in the immediate vicinity of the Welland canal may be influenced by seasonal draining of the canal, which occurs between January and March.

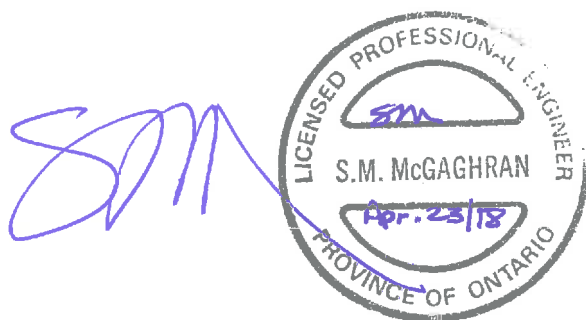


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### 6.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Alex Champigny, E.I.T. who also supervised the drilling operations. Ms. Sandra McGaghran, M.Eng., P.Eng., a geotechnical engineer and Associate with Golder reviewed the report and Mr. Mark Telesnicki, P.Eng. a rock mechanics engineer and Principal with Golder provided Technical Input. Ms. Lisa Coyne, P.Eng., Golder's Designated MTO Foundation Contact for this project and Principal with Golder, conducted an independent technical and quality control review of the report.

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SMM/BL/MJT/LCC/sm

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**FOUNDATION REPORT  
THOROLD TUNNEL REHABILITATION**

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# **PART B**

**FOUNDATION DESIGN REPORT  
THOROLD TUNNEL REHABILITATION (SITE NO. 34-177/T)  
HIGHWAY 58, CITY OF THOROLD  
MINISTRY OF TRANSPORTATION, ONTARIO  
G.W.P. 2370-16-00**



## 7.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides detail foundation engineering design recommendations for the proposed rehabilitation of Thorold Tunnel (Site No. 34-177/T) and the rock faces on the north and south sides of Highway 58 east of the East Portal. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the previous investigation and the borehole advanced during the current subsurface investigation. The discussion and recommendations presented are intended to provide the designer with sufficient information to assess the feasible alternatives and develop the detail design and contract documents for the rehabilitation of the Thorold Tunnel and the rock faces on the north and south sides of Highway 58 east of the East Portal.

The discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and their designers for G.W.P. 2370-16-00, and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. 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 may be required in the Contract Documents. The contractor must make their own interpretation of the factual information provided in Part A (Foundation Investigation) of the report, as such interpretation may affect equipment selection, proposed construction methods, scheduling, and the like.

### 7.1 General

This project involves the detail design for the rehabilitation of the Thorold Tunnel. Foundation design recommendations are required to address the following components of the rehabilitation work:

- Grouting of the cracks and leaking joints of the tunnel walls; and,
- Treatment for the rock faces north and south of Highway 58.

#### *Thorold Tunnel*

As discussed in Section 2.0, the Thorold Tunnel consists of two separate tube structures, each accommodating two lanes that carry the eastbound (EB) and westbound (WB) traffic of Highway 58 beneath the Welland Canal. The EB and WB tubes are separated by a service tunnel accommodating a walkway and utilities. A public sidewalk runs along the north side of the WB tube. Reports by Acres (see references in Desktop Study for a list of reports) indicate that shortly after the tunnel was constructed in 1968, cracking was observed in the tunnel walls. Since then there have been several rehabilitation contracts to grout the cracks. As discussed in Section 3.1, icicles (representing frozen seepage water) were observed on the vertical cracks in the tunnel wall. Small conical piles of soil were also observed on the sidewalk at three of the existing crack locations, with lesser quantities of fine soil particles observed adjacent to two additional crack locations; these observations may indicate that fine soil particles are being carried with the water seepage through the crack(s). Wet pavement areas were also observed, and these areas are believed to be due to water emanating from beneath the pavement.

#### *Rock Faces East of the East Portal*

Section 2.2 provides a detailed description of the north and south rock faces east of the East Portal. Current challenges with the rock faces include fallen rock debris, rockfalls, and water seeping onto the sidewalk and road and freezing, all of which pose safety hazards to traffic and pedestrians. It is understood that throughout the year, a fairly significant amount of rock falls from the face and on the south side of Highway 58 some of the rock that falls from the face has reached the road surface of Highway 58. Near the portal on the south side, some undercutting of the rock face has occurred, leaving an overhang; therefore the rock above the undercut is at





greater risk for falling onto the road surface. In addition to the rockfalls, ice builds up on the rock faces and can fall as it melts. Water also seeps onto the sidewalk and freezes during winter conditions; this requires a maintenance crew to remove the ice that builds up on sidewalk on an as-required basis. On the south side of Highway 58, only a curb is present, and any water that seeps onto the road surface has the potential to freeze during winter conditions.

The treatment of the rock face needs to take into account the existing light pole on the north side of Highway 58 which is in close proximity to the rock face. The distance from the rock face to the light pole varies vertically from about 1 m to 1.2 m. Just west of the light pole the vertical concrete wall is generally about 1.9 m from the light pole.

## 7.2 Thorold Tunnel Rehabilitation Options

Various rehabilitation options for the Thorold tunnel were considered, and a comparison of the options based on advantages, disadvantages and risks/consequences is provided in Table 1 following the text of this report, and summarized below.

- **Do Nothing:** The “do nothing” option serves as the base case and would involve not repairing the cracks in the in the tunnel walls. There is risk associated with this option as cracks may potentially widen and/or deteriorate further, and it is therefore recommended that as a minimum an annual or semi-annual visual inspection of cracking be carried out to assess the condition. In addition, this option requires continued, ongoing weekly/daily inspection and routine maintenance to confirm that ice has not formed on the sidewalk, and to remove such ice where applicable. Continued water seepage from cracks in the tunnel wall, and formation of ice at these locations in the winter, may contribute to public perception that the tunnel is deteriorating.
- **Partial Penetration of Tunnel Wall with Injection of Chemical Grout:** This option involves the injection of a flexible polyurethane chemical grout into joints/cracks, only partially (not fully) penetrating the tunnel walls. This material complements the existing concrete wall and would allow the concrete to continue to expand or contract. Because the path of the crack behind the surface of the tunnel wall is not known, the crack should be grouted from injection ports on either side of the crack during the attempt to seal the crack. An advantage of this option is that the chemical grout is long lasting and has good resistance to chemical and biological degradation. It is noted that even with partial penetration grouting of the existing cracks, there is the potential that new cracks could form or joints could open up and permit alternate pathways for water seepage into the tunnel. A yearly inspection will be required, and maintenance/additional grouting is expected to be required in the future, in subsequent rehabilitation contracts. This option requires closure of the tunnel to permit injection of the grout into the cracks.
- **Backwall Grouting:** This option involves fully penetrating the tunnel wall with the injection of grout. This option would result in puncturing the existing tunnel waterproofing membrane on the outside of the concrete walls. Behind the waterproofing membrane, the tunnel was backfilled with rock fill. Due to the large void space in the rock fill, backwall grouting at this site will result in the grout migrating through the rock fill, and a large volume of grout will be used at each grouting location. It is difficult to estimate the volume of grout required to fill the voids in the rock fill as well the rock mass, creating a quantity/cost and schedule risk. In addition, heavy drilling equipment will be required and this will impact construction and traffic staging as the tunnel(s) will need to be closed for drilling and grouting activities; however, this could be completed at the same time as other repairs to be carried out.



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- **Grouting the Rock Formation from Ground Surface:** This option would minimize closure of the tunnel for construction; however, closure of the tunnel is still required for other repairs. Grouting of rock formation does not specifically address the cracks in the tunnel walls. Water from the Welland Canal could still infiltrate the rock fill behind the wall, and contribute to seepage into the tunnel.
- **Waterproofing Membrane on the Interior Tunnel Wall:** This option would consist of the application of a waterproofing membrane which is then sprayed with shotcrete. This option would be less intrusive, faster and would likely have a lower impact on traffic than the backwall grouting option; however, it does not stop water from migrating through the existing concrete walls, floor and ceiling slab, leading to further deterioration of the concrete structure.

Based on the above considerations, partial penetration of tunnel wall with injection of chemical grout is recommended as the preferred option from a geotechnical/foundations perspective.

### 7.2.1 Preferred Option – Partial Wall Grouting

It is recommended that the existing cracks and leaking joints within the tunnel walls of the north and south tube be injected with flexible polyurethane chemical grout, and that the injections only partially penetrate the 1.8 m thick outer concrete walls of the Thorold tunnel. It is understood that MH's rehabilitation design also includes grouting all of the joints in the base slab under the pavement, and that heat tracer wire will be installed to reduce the potential for ice forming on the road surface during winter conditions.

The grouting should be carried out in accordance with OPSS 932. For each crack exposed at the face of the tunnel wall, the path of the crack at depth within the wall is unknown; therefore in order to intersect the crack path it is recommended that small diameter holes be drilled at a 45 degree angle. The holes should be advanced in good quality concrete and located such that they intersect the crack section at approximately the midpoint and extend through the crack section. Figure 2 presents a conceptual sketch of the proposed method. It is recommended that, following the initial grouting stage at each crack and joint, a second iteration be carried out to confirm that the hole intersects the crack behind the face of the concrete wall. This requirement has been addressed in a Non-Standard Special Provision (NSSP) that amends OPSS 932.

The Contractor should be required to verify that the methodology for grouting, as proposed by the Contractor, will not fully penetrate to the back wall or the existing waterproofing membrane. The Contractor should take into account the angle of the drill hole, the drill length and the thickness of the concrete wall. It is understood that the joints were constructed with a waterstop and it is recommended that the drill holes advanced through the concrete wall for grouting should not intersect or damage the waterstop. It is understood that MH have addressed this requirement in the NSSP that amends OPSS 932.

Prior to installation of the grout, each hole should be flushed with an air-water blast to ensure that it is cleaned of all deleterious material and to remove drill cuttings from the hole. Additional drill holes may be required as determined by water flushing techniques, if connections are not made between all of the drill holes.

Concrete preparation and parging of the crack/joint faces may be required to provide confinement for the injection grout. After cleaning drill dust and debris from the drill holes, plastic injection packers or small diameter mechanical packers are to be installed. After the grout is installed the injection packers should be removed and drill holes are to be filled flush with parging cement to match the existing concrete surface.

It is recommended that the Contract Administrator (CA) inspect the drilling and flushing of the holes and the installation of the grout.



### 7.3 Rock Face Treatment Options

Various rock face treatment options were considered and a comparison of the options based on advantages, disadvantages and risks/consequences is provided in Table 2 following the text of this report, and summarized below.

- **Do Nothing:** The “do nothing” option serves as the base case and would involve leaving the rock cuts as they are. The advantage of this option is that it is low cost, aside from the daily/weekly maintenance of sidewalk and road surface of Highway 58. In addition, the do nothing approach would minimize the time that the lane(s) of the tunnel are closed to vehicular and/or pedestrian traffic during the rehabilitation works. However, if the existing rock face is not treated there is the potential that rock will continue to ravel and fall on the existing sidewalk and roadway due to ongoing weathering and freeze-thaw cycles, and that water will continue to seep from the rock face resulting in ice on the sidewalk and roadway, both of which are a hazard to vehicular and pedestrian traffic. As a result of the falling rock and ice, weekly/daily inspection and routine maintenance are generally required. This option has the lowest cost, but the greatest risk of liability for the MTO.
- **Shotcrete Facing:** This option involves the application of composite drainage panels, rock bolts and shotcrete to the rock faces. The advantages with this option are that it is relatively easy to construct and has a lower cost compared to precast panel wall or cast-in-place concrete retaining wall options. The shotcrete can be coloured with pigments with a minimal increase in cost and/or it can be sculpted to improve the aesthetic appearance, but this would increase the cost significantly. Application of the shotcrete facing uses conventional construction techniques, does not require formwork and has a shorter construction duration period compared to the cast-in-place wall option, which would mean a reduced lane closure requirement during construction. A disadvantage is that the application of shotcrete is limited to non-winter months and the shotcrete would typically have a shorter design life compared to the pre-cast or cast-in-place wall. One further disadvantage common to the shotcrete facing as well as to pre-cast panel wall and cast-in-place wall options is the potential for graffiti.
- **Pre-cast Panel Wall or Cast-in-place Concrete Wall:** A pre-cast panel wall and a cast-in-place concrete wall were considered as a treatment option over the rock face. The primary advantage of pre-cast panel walls and cast-in-place concrete walls is that they will have a longer design life compared to a shotcrete wall. Pre-cast panel walls have advantages over cast-in-place concrete walls for this site: formwork is not required, and the panels are formed off-site in a controlled environment, and therefore can be installed at any time of the year, whereas cast-in-place construction must be carried out in temperate conditions or in conjunction with heated enclosures while the concrete cures. However, there are several challenges and disadvantages with both of these wall options. Firstly, from an aesthetic perspective the panels should be placed in a relatively straight alignment along the rock face; however, the rock face is very irregular which will result in a highly variable gap between the wall and the rock face which would need to be backfilled, with the wall designed for the earth pressures from the backfill. Posts would be required to support the panels for a pre-cast panel wall option, and the posts will have pre-formed holes for the rock bolts; if there are any adjustments/changes based on the field conditions, these can be difficult to accommodate. It will be difficult to construct either wall type around the existing light pole on the north rock face. In addition, a level bearing surface between posts will be required to support the bottom panels. As discussed above, there is potential for aesthetic damage (i.e., graffiti) to both wall types.



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- **Grout Curtain Wall:** In order to control/minimize the water seepage from the rock face, consideration was given to construction of a grout curtain wall behind the rock face. This would require drilling a row of grout holes about 3 m to 4 m behind the rock face. Initially, primary grout holes would be drilled at a spacing of about 5 m and these would be pressure grouted. Then, a second set of secondary grout holes would be drilled in a row between the primary holes and these would be water-tested to verify that the grout from the primary holes was effective in reducing the hydraulic conductivity of the rock mass. If required (based on the water test results), the secondary grout holes would be pressure grouted and tertiary grout holes drilled between the secondary holes. These holes would be water-tested and grouted if required, and the process repeated until the rock mass is adequately grouted. Specialized equipment will be required to carry out line drilling behind the rock face. This option requires permission for construction equipment on the St. Lawrence Seaway Authority (SLSA) property during construction; however, an advantage of this option is that it does not require closure or reduction of lanes of the highway to implement solution. Although the grouting will minimize the seepage of water and therefore the formation of ice on the face, it would not directly address the risk of ravelling and falling rocks from the rock face; because this option does not address this risk, it is not considered further.

Based on the above considerations, the shotcrete facing (with composite drainage panel and rock bolts) is recommended as the preferred option from a geotechnical/foundations perspective.

### 7.3.1 Preferred Option – Shotcrete Facing

The recommended treatment of the rock faces is the application of shotcrete to the rock face. Drawing 3 illustrates the proposed option and Drawing 4 provides typical sections and details. This alternative has been used with success on a previous MTO project (the west access road for the Queen Elizabeth Way (QEW) Credit River project). With good workmanship, this option should require little to no maintenance over a 10- to 15-year design life.

Prior to applying the shotcrete, the rock face must be scaled by machine in accordance with OPSS.PROV 202, in order to remove all loose and unstable rock. Recommendations for rock scaling by machine are discussed in Section 7.3.1.1. The shotcrete should also be applied to the existing concrete wall faces, such that the rock face and concrete are all covered in shotcrete for a more consistent appearance. The shotcrete facing option consists of installation of composite drainage panel strips for groundwater drainage behind the shotcrete, and rock bolts between the drainage panels. At the location of the existing concrete wall, composite drainage panels are not required; however, a welded wire mesh will need to be pinned to the concrete wall in order to provide greater adherence of the shotcrete to the concrete wall. The existing buried subdrain at the toe of the rock face is recommended to be replaced with a new 150 mm diameter subdrain, as the condition of the old subdrain is unknown. The following provides further details for the composite drainage panels, rock bolts, application of the shotcrete to the rock face and concrete wall, and long-term drainage.

#### 7.3.1.1 Rock Scaling (Machine)

Machine scaling should be carried out in accordance with OPSS.PROV 202. The entire length of the rock cuts must be scaled using machine scaling methods in order to remove all loose and unstable rock. All scaling work should be carried out by starting at the top of the rock cut and working down toward the base of the rock cut. This will reduce the risk of working under unstable rock, which could result in damage to the equipment or personal injury to the operator.

Machine scaling involves the use of a large tracked excavator with sufficient reach to allow the operator to work off to one side so that scaled rock can fall safely down into the ditch away from the machine. Machine scaling



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includes both hoe ramming and rock removal with a narrow “ditching” bucket. It is recommended that the excavator bucket be used as the primary tool for scaling at this site, and that the hoe ram be used cautiously only where loose blocks of rock are difficult to remove. In some locations, there is a potential risk of over-excavation beyond the loose, fractured material that is required to be removed if the hoe ram is used too aggressively. In most cases it is recommended that the machine scaling be carried out after hoe ramming to remove any loose rock that has been broken up by the hoe ram but has not fallen from the face to the ditch area.

### **7.3.1.2 Composite Drainage Panels**

To address the seepage of water on the rock face, it is recommended that composite drainage panel strips, such as MiraDRAIN 6000 or equivalent, having a width of 0.3 m be installed at 3 m centre-to-centre spacings over the full height of the rock face. It is not recommended to cover the entire rock face with the composite drainage panels because the shotcrete will not bond as well to the panels as to the rock face. The composite drainage panels shall be positioned with the dimples toward the rock/drainage side and they shall extend to the toe of the rock faces where they must be connected to the site drainage system in such a way that the flow of water to the drainage system is unimpeded. A Non-Standard Special Provision (NSSP) to address this item is included in Appendix C, and this NSSP should be included in the Contract Documents.

### **7.3.1.3 Rock Bolts**

The shotcrete can be secured with a grid pattern of 1 m long rock bolts that are installed at 3 m centre-to-centre spacings horizontally, and about 2 m centre-to-centre vertically, between the composite drainage panels. The rock bolts are required to be 25M (25 mm diameter) galvanized reinforcing steel bars in accordance with OPSS.PROV 1440, with a minimum yield strength of 400 MPa. Each rock bolt must be provided with a spider plate of at least 125 mm x 125 mm x 10 mm with two #3 rebar “ears” extending from the steel plate (see Details B and C on Drawing 4) to tie the rock bolts to the shotcrete. The rock bolts should be fully grouted in pre-drilled holes, but are not required to be pre-stressed. The rock bolts must be installed in accordance with OPSS.PROV 203 and a Non-Standard Special Provision (NSSP), which is an amendment to OPSS.PROV 203, is included in Appendix C, for inclusion in the Contract Documents.

Because the height of the rock face varies along Highway 58 due to the road surface east of the East Portal gradually rising to the east, some site adjustments to the vertical spacing of the rock bolts will be required. In addition, existing rock bolts are located adjacent to the Portal and the installation of the new rock bolts will need to be field-fitted so as to not conflict with the existing rock bolts. During construction it is recommended that the Contract Administrator retain the services of a Foundations Specialist in Rock Engineering on a full-time basis to observe the installation of the rock bolts to ensure that the requirements of the contract are being met.

### **7.3.1.4 Shotcrete**

For the shotcrete facing option, the intent is that the shotcrete would be applied to the rock face as well as the existing concrete wall and that a smooth transition be provided. Application of the shotcrete shall be in accordance with OPSS.PROV 203 and the NSSP developed as an amendment to OPSS.PROV 203 (included in Appendix C) which is to be included in the Contract Documents. Application of shotcrete is limited to when the rock/concrete surface temperature is above 10 degrees Celsius or below 30 degrees Celsius (OPSS.PROV 203.07.02.01). The shotcrete must be of good quality and applied by a specialist contractor experienced in such construction methods (OPSS.PROV 203.07.02.02). The addition of silica fume to the shotcrete mix will assist in the likelihood of the shotcrete being applied in one application; however, multiple passes may be required particularly in the transition of the shotcrete from the rock face to the concrete wall. The applied shotcrete must be kept moist by water spraying at the start, middle and end of each working day for the first three days after placement. During





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construction it is recommended that the Contract Administrator retain the services of a Foundations Specialist in Rock Engineering on a full-time basis to observe the application of shotcrete to ensure that the requirements of the contract are being met.

### ***Application of Shotcrete to Existing Concrete Wall***

In order to secure the shotcrete to the existing concrete wall, it is recommended the concrete wall first be roughened and then a welded wire mesh be secured to the existing concrete wall using galvanized “L” bolts. Welded wire mesh shall have 100 mm x 100 mm openings fabricated from 4 mm diameter galvanized wires conforming to CSA G30.5; the wires shall be welded at the joints. The “L” bolts shall be galvanized and have a minimum 12 mm diameter, with a minimum length of 200 mm in one direction and 100 mm in the perpendicular direction. The 200 mm long portion of the bolt shall be drilled into the existing concrete wall and epoxy-grouted into the concrete. The 100 mm long portion shall be tied with wire to the welded wire mesh for the purpose of securing the welded wire mesh to the existing concrete wall. The “L” bolts shall be installed at 1.5 m centre-to-centre as specified in Detail D on Drawing 4. Steel fibre-reinforced shotcrete having a minimum thickness of 50 mm shall be applied over the existing concrete wall at the locations as specified on Drawing 3. At the juncture between the rock and concrete wall, the shotcrete shall be tapered over a distance of 2 m to provide a smooth transition between the 100 mm thick and 50 mm thick shotcrete (see Detail E on Drawing 4).

### ***Application of Shotcrete to Rock Face***

Following machine scaling of the rock face, application of the composite drainage panels and installation of the rock bolts, the rock face should be washed and kept damp while the shotcrete is placed to create a minimum 100 mm thick layer on the rock at the locations as specified on the Contract Drawings. Where an undercut is present on the existing rock face, the shotcrete shall have a minimum thickness of 100 mm and shall follow the contours of the rock face.

### ***7.3.1.5 Long Term Drainage***

The 1967 Contract Drawings indicate that there is an existing 250 mm diameter buried subdrain at the base of the rock face on the north and south side of Highway 58. Considering the existing subdrains have been in operation for approximately 50 years, it is recommended that they be replaced with a 150 mm diameter perforated subdrain as shown in Details A, C and D on Drawing 4. The subdrain invert shall be at a depth of 1.2 m below the finished grade and backfilled with 19 mm clear stone to 50 mm above the obvert in accordance with OPSS.PROV 1004. The clear stone must be fully wrapped in a geotextile meeting the requirements of Terrafix 360R or equivalent. The remainder of the trench can be backfilled with OPSS.PROV 1010 (Aggregates) Granular B Type II to the finished grade. The subdrain is to be connected to the existing catch basins at the locations as shown on Drawing 3 and in accordance with OPSD 216.021.

### ***7.3.2 Pre-Cast Panel Wall***

Prior to installing a pre-cast panel wall, the rock face must be scaled by machine in accordance with OPSS.PROV 202, in order to remove all loose and unstable rock. Recommendations for rock scaling by machine are discussed in Section 7.3.1.1. The pre-cast panel wall option consists of installation of composite drainage panel strips for groundwater drainage behind the panels. At the location of the existing concrete wall, composite drainage panels are not required. The existing buried subdrain at the toe of the rock face is recommended to be replaced with a new 150 mm diameter subdrain as the condition of the existing subdrain is unknown. The following provides further details for the composite drainage panels, installation of the pre-cast panel wall to the rock face and concrete wall, and long-term drainage.





### **7.3.2.1     *Composite Drainage Panels***

The composite drainage panels should be installed as per the recommendations provided in Section 7.3.1.2.

### **7.3.2.2     *Installation of Pre-Cast Panel Wall***

From an aesthetic perspective, the panels should be placed in a relatively straight alignment along the rock face and existing concrete wall; however, the rock face is very irregular and the existing concrete wall protrudes from the rock face, which will result in a highly variable gap between the panel wall and the rock face. Prior to installation of the panel wall, the base of the rock face will need to be cleaned of all rock debris to create a level bearing surface. The area behind the panels would need to be backfilled with select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular B Type II. Posts will be required to support the panels, and the posts will have pre-formed holes for the rock bolts; therefore, field adjustments/changes will be required and provision for this should be made in the Contract. Care will be required to install the panels behind the light pole on the north rock face and a custom panel will likely be required to create a notch around the base of the light pole.

### **7.3.2.3     *Long Term Drainage***

For the pre-cast panel wall option it is recommended that the existing subdrain be replaced and the new subdrain installed as per the recommendations provided in Section 7.3.1.5 and Section 7.4.1.

## **7.3.3     *Cast-in-Place Concrete Wall***

Prior to constructing a cast-in-place concrete wall, it is recommended that the rock face be scaled by machine in accordance with OPSS.PROV 202, in order to remove all loose and unstable rock. Recommendations for rock scaling by machine are discussed in Section 7.3.1.1. The cast-in-place concrete wall option consists of installation of composite drainage panel strips for groundwater drainage behind the cast-in-place panels, and rock bolts between the drainage panels. At the location of the existing concrete wall, composite drainage panels are not required. As with the other options, the existing buried subdrain at the toe of the rock face is recommended to be replaced with a new 150 mm diameter subdrain, as the condition of the existing subdrain is unknown. The following provides further details for the composite drainage panels, installation of the cast-in-place concrete wall panels to the rock face and concrete wall, and long-term drainage.

### **7.3.3.1     *Composite Drainage Panels***

The composite drainage panels should be installed as per the recommendations provided in Section 7.3.1.2.

### **7.3.3.2     *Cast-in-Place Concrete Wall***

Similar to the pre-cast panel wall, the concrete wall must be constructed in a relatively straight alignment along the rock face and existing concrete wall, but considering that the rock face is very irregular and the concrete wall protrudes from the rock face, this will result in a highly variable thickness for the concrete wall. Prior to construction of the concrete wall, the base of the rock face will need to be cleaned of all rock debris to create a level bearing surface. In order to permit construction of the cast-in-place concrete wall in winter months, the formwork would need to be wrapped with tarps and heating units used between the wall and the tarps to allow the concrete to cure properly.



### 7.3.3.3 *Rock Bolts*

The rock bolts for the cast-in-place panels should be installed as per the recommendations provided in Section 7.3.1.3.

### 7.3.3.4 *Long Term Drainage*

For the cast-in-place concrete wall option, it is recommended that the existing subdrain be replaced and the new subdrain installed as per the recommendations provided in Section 7.3.1.5 and Section 7.4.1.

## 7.4 Construction Considerations

### 7.4.1 Open-Cut Excavations and Groundwater Control

Removal of the existing subdrain and installation of the new subdrain will require excavation in front of the rock face/concrete wall, into a previously excavated trench/slot in the bedrock that extends below the Highway 58 pavement and sidewalk grade. Based on the 1967 design drawings, the subdrain is anticipated to be at a depth of approximately 1 m to 1.5 m below the pavement grade; the subdrain was placed within granular backfill in the rock trench/slot.

During rehabilitation of each tube of the Thorold tunnel, it is understood that the tube will be closed to vehicular and pedestrian traffic and therefore space may permit open-cut excavations for the removal and installation of the subdrain. This open-cut approach would minimize risks associated with encountering obstructions, including uneven bedrock within the cut, during installation of temporary protection systems.

Open-cut excavations into fill 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 materials are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.

### 7.4.2 Temporary Protection Systems

If space does not permit open-cut excavations as discussed above, temporary protection systems will likely be required to permit excavation for the removal and installation of subdrains adjacent to Highway 58. The temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*). The lateral movement of the protection systems should meet Performance Level 2 as specified in OPSS.PROV 539, to protect the adjacent pavement surface.

The protection systems are expected to be required for a maximum excavation depth of approximately 1.5 m below the pavement grade. However, their installation may be complex at this site, as the sides and base of the bedrock excavation are likely uneven, and obstructions such as pieces of bedrock may be present within the granular fill. Trench boxes or short sections of driven sheetpile, in conjunction with lateral braces or inclined rakers, may be feasible to protect the adjacent pavement surface and prevent/minimize sloughing of granular soils from below the pavement surface. It is unlikely that the contractor would implement a soldier pile and lagging system at this site given the relatively shallow depth of excavation, and the shallow depth to bedrock. However, ultimately, the selection and design of the protection system will be the responsibility of the Contractor.



## **8.0 CONSIDERATIONS FOR FUTURE WORK**

As discussed in Section 7.3.1, the shotcrete treatment on the rock faces east of the East Portal has a design life of about 10 to 15 years, depending on the quality of the shotcrete and the workmanship during the application. It is recommended that an annual or bi-annual inspection be carried out to observe for any signs of deterioration. Near the end of the design life, an evaluation will need to be undertaken to assess whether application of additional shotcrete, or removal and re-application of shotcrete or other wall treatment alternatives, is required.



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### 9.0 CLOSURE

This Foundation Design Report was prepared by Ms. Sandra McGaghran, M.Eng., P.Eng., a geotechnical engineer and Associate with Golder. Mr. Bill Lillico, a grouting specialist with Golder, provided input on the technical aspects of grouting of the Thorold Tunnel. Mr. Mark Telesnicki, P.Eng., a Principal with Golder and RAQS-registered Rock Hazard Specialist, reviewed the technical recommendations in this report. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent technical and quality control review of the report.

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#### **ASTM International:**

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

ASTM D4630-96 (2008)      Standard Test Method for Determining Transmissivity and Storage Coefficient of Low-Permeability Rocks by In Situ Measurements Using the Constant Head Injection Test

ASTM D7012                    Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

#### **Ontario Provincial Standard Specification:**

OPSS.PROV 202                Construction Specification for Rock Removal by Manual Scaling, Machine Scaling, Trim Blasting or Controlled Blasting

OPSS.PROV 203                Construction Specification for Rock Stabilization

OPSS.PROV 501                Construction Specification for Compacting

OPSS.PROV 539                Construction Specification for Protection Systems

OPSS.PROV 932                Construction Specification for Crack Repair - Concrete

OPSS.PROV 1004               Material Specification for Aggregates – Miscellaneous

OPSS.PROV 1010               Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

OPSS.PROV 1440               Material Specification for Steel Reinforcement for Concrete

#### **Ontario Water Resources Act:**

Ontario Regulation 903 Wells (as amended)





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### **Ontario Occupational Health and Safety Act:**

Ontario Regulation 213/91    Construction Projects (as amended)



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**Table 1: Comparison of Alternatives, Thorold Tunnel Rehabilitation**

<b>Tunnel Rehabilitation Option</b>	<b>Feasibility</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Constructability</b>	<b>Estimated Costs</b>
Do nothing (without or with periodic inspection/monitoring)	<ul style="list-style-type: none"> <li>Feasible</li> </ul>	<ul style="list-style-type: none"> <li>Lowest cost</li> </ul>	<ul style="list-style-type: none"> <li>Cracks may potentially widen and/or deteriorate further; annual or semi-annual visual inspection of cracking may be considered, in addition to inspection and maintenance outlined below.</li> <li>Requires weekly/daily inspection and routine maintenance to confirm that ice hasn't formed on the sidewalk.</li> <li>Continued water seepage from cracks in the tunnel wall and formation of ice on the wall at the location of the cracks in the winter contributes to public perception that the tunnel is deteriorating.</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>Cost of maintenance staff to periodically inspect the tunnel walls and service tunnel.</li> </ul>
Inject flexible, polyurethane chemical grout into joints/cracks from within the tunnel to partially penetrate and seal the concrete wall	<ul style="list-style-type: none"> <li>Feasible and recommended</li> </ul>	<ul style="list-style-type: none"> <li>Lower cost compared to backwall grouting (i.e., grouting that is done from outside the tunnel, or that penetrates fully through the tunnel walls to permeate the rock mass/backfill)</li> <li>Complements the concrete; allows the concrete to continue to expand or contract (allows for concrete movement)</li> <li>Minimizes water infiltration through cracks and joints</li> <li>Long lasting, good resistance to chemical and biological degradation.</li> </ul>	<ul style="list-style-type: none"> <li>Existing cracks/joints grouted; however, there is the potential that new cracks could form or joints could open up and permit alternate pathways for water seepage into the tunnel</li> <li>A yearly inspection will be required and provision should be made for carrying out additional grout in the event that additional cracks form.</li> <li>Flushing of the hole prior to injecting the grout is a critical part of the procedure; applicators have been known to skip this step. Quality control will be required.</li> <li>Tunnel(s) will need to be closed for rehabilitation grouting activities; however, could be carried out at a later date with lane closures at night if it interferes with other repairs to be carried out at the same time.</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction techniques.</li> <li>Can be done with small equipment (hand held drills) and easily mobilized.</li> <li>Requires coordination with other in-tunnel rehabilitation activities.</li> </ul>	<ul style="list-style-type: none"> <li>Lower cost compared to other alternatives; but higher compared to the do nothing option.</li> </ul>
Grout cracks from within the tunnel to fully penetrate the concrete wall (backwall grouting) and waterproofing membrane to permeate and displace water away behind the tunnel wall.	<ul style="list-style-type: none"> <li>Not recommended</li> </ul>	<ul style="list-style-type: none"> <li>Potentially reduced seepage into cracks in tunnel, albeit at significant cost and schedule duration</li> </ul>	<ul style="list-style-type: none"> <li>Backwall grouting results in puncturing the tunnel waterproofing membrane (5 mm thick). Behind the waterproofing membrane the tunnel was backfilled with rockfill. Backwall grouting results in the grout penetrating the rockfill in an uncontrolled manner. Due to the large void space in the rockfill the grout will migrate through the rockfill and a large volume of grout will be used at each grouting location. It is difficult to estimate the volume of grout required to fill the voids in the rock fill as well the rock</li> </ul>	<ul style="list-style-type: none"> <li>Requires larger equipment, impacting construction staging</li> <li>Requires highest level of coordination with other in-tunnel rehabilitation activities.</li> </ul>	<ul style="list-style-type: none"> <li>Higher costs</li> </ul>



## FOUNDATION REPORT THOROLD TUNNEL REHABILITATION

Tunnel Rehabilitation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
			<p>mass, creating a quantity/cost and schedule risk.</p> <ul style="list-style-type: none"> <li>• Heavy drilling equipment required and this will impact staging. Tunnel(s) will need to be closed for drilling and grouting activities; however, this could be completed at the same time as other repairs to be carried out.</li> <li>• Steel reinforcement embedded inside the concrete may be compromised.</li> <li>• Waterproofing systems may be compromised.</li> <li>•</li> </ul>		
Grout the rock formation from ground surface	<ul style="list-style-type: none"> <li>• In the area of the Welland Canal only potentially feasible during periods when the Welland Canal is drained.</li> <li>• Not recommended as it does not address the cracks in the concrete wall, and seepage into the tunnel would likely still occur.</li> </ul>	<ul style="list-style-type: none"> <li>• Minimizes closure of the tunnel for construction; however, closure of the tunnel is still required for other repairs.</li> </ul>	<ul style="list-style-type: none"> <li>• Grouting of rock formation does not address the cracks in the wall. Water from the Welland Canal can still infiltrate the rockfill behind the wall; therefore, there is still a source of water behind the tunnel wall to contribute to seepage into the tunnel.</li> </ul>	<ul style="list-style-type: none"> <li>• May require drilling from barges as the canal base won't be completely dry.</li> </ul>	<ul style="list-style-type: none"> <li>• Higher costs</li> </ul>
Lowering of the groundwater table to below the invert (Active drainage System)	<ul style="list-style-type: none"> <li>• Not feasible or recommended as mitigation for the current seepage volumes, due to canal configuration, long-term pumping requirements and potential drawdown impacts</li> </ul>	<ul style="list-style-type: none"> <li>• Minimizes any water inflow through cracks in the tunnel</li> </ul>	<ul style="list-style-type: none"> <li>• Some risk of settlement of nearby structures that are supported on the overburden</li> </ul>	<ul style="list-style-type: none"> <li>• Large scale de-watering effort and equipment,</li> </ul>	<ul style="list-style-type: none"> <li>• Lower initial costs, but long-term pumping costs apply</li> </ul>
Waterproofing membrane on the interior of the wall with a layer of shotcrete (Passive drainage system)	<ul style="list-style-type: none"> <li>• Not recommended.</li> </ul>	<ul style="list-style-type: none"> <li>• Would be less intrusive, faster and potentially have lower long-term impact on traffic than backwall grouting option.</li> </ul>	<ul style="list-style-type: none"> <li>• Does not stop water from migrating through the existing concrete walls, floor and ceiling slab leading to further deterioration of the concrete structure.</li> </ul>	<ul style="list-style-type: none"> <li>• Potentially shorter/less impact on traffic than some other options</li> </ul>	<ul style="list-style-type: none"> <li>• Lower to moderate cost</li> </ul>



## FOUNDATION REPORT THOROLD TUNNEL REHABILITATION

**Table 2: Comparison of Alternatives, Rehabilitation of Rock Face East of the East Portal and on the North and South Sides of Highway 58**

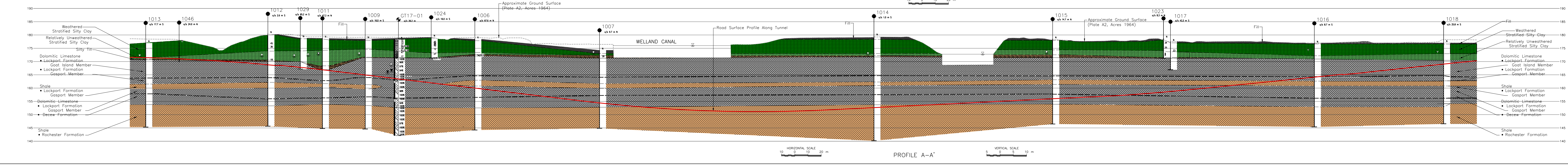
Rock Face Rehabilitation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Do nothing (without or with period inspection/monitoring)	<ul style="list-style-type: none"> <li>Feasible, but does not address the problem</li> </ul>	<ul style="list-style-type: none"> <li>Low cost</li> <li>Does not require closure of the highway, which would be required for concrete wall options.</li> </ul>	<ul style="list-style-type: none"> <li>Requires removal of ice that builds up on sidewalk from water seeping from rock face.</li> <li>Ice on sidewalk is a safety hazard and a liability.</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>Lowest cost; aside from the daily/weekly maintenance of sidewalk.</li> </ul>
Shotcrete facing, MiraDRAIN and rock bolts (MiraDRAIN outlets into trench to the subdrain at base of wall which connects to an existing catch basin)	<ul style="list-style-type: none"> <li>Feasible and preferred from a geotechnical perspective</li> </ul>	<ul style="list-style-type: none"> <li>Lower cost compared to concrete wall.</li> <li>Easier to apply compared to cast-in-place or precast concrete wall.</li> <li>Shotcrete can be coloured with pigments with a minimal increase in cost and/or it can be sculpted but this would increase the cost significantly.</li> <li>Least amount of protrusion of face treatment from the rock face.</li> </ul>	<ul style="list-style-type: none"> <li>Application of shotcrete limited to non-winter months as it cannot be applied due to low temperatures in winter months.</li> <li>MiraDRAIN is required to be placed in strips (to allow the shotcrete to bond to the rock face); therefore low potential for water to become trapped and freeze and not find its way to the MiraDRAIN.</li> <li>Poor aesthetic impact;</li> <li>Potential for graffiti on shotcrete facing</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction techniques.</li> <li>Shorter construction duration compared to cast-in-place.</li> <li>Does not require formwork.</li> <li>Previously used with success on an MTO project for the construction access road at QEW Credit River.</li> </ul>	<ul style="list-style-type: none"> <li>Lower cost compared to precast or cast-in-place concrete wall.</li> </ul>
Cast-in-place panel wall, MiraDRAIN and rock bolts (MiraDRAIN outlets into trench to the subdrain at base of wall which connects to an existing catch basin)	<ul style="list-style-type: none"> <li>Feasible</li> </ul>	<ul style="list-style-type: none"> <li>Panel wall can have architectural detail; however, it is more costly compared to standard construction.</li> <li>Cast-in-place allows for any field adjustments to the bolt locations.</li> <li>Cast in place concrete would have a longer design life compared to shotcrete</li> </ul>	<ul style="list-style-type: none"> <li>Construction of the cast-in-place concrete wall is limited to non-winter months, unless the formwork is wrapped with tarps and heating units are used between the wall and the tarps to allow the concrete to cure properly.</li> <li>Greater protrusion of face treatment from the rock face compared to shotcrete option.</li> <li>Potential for graffiti on concrete wall.</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction techniques.</li> <li>Requires formwork.</li> <li>Limited to non-winter months.</li> </ul>	<ul style="list-style-type: none"> <li>Higher than the shotcrete options</li> </ul>
Precast panel wall with MiraDRAIN behind wall (ie similar to a noise wall panel with vertical steel members bolted to the rock) (MiraDRAIN outlets into trench to the subdrain at base of wall which connects to an existing catch basin)	<ul style="list-style-type: none"> <li>Feasible</li> </ul>	<ul style="list-style-type: none"> <li>Panel can be constructed off-site thereby reducing the closure window.</li> <li>Wall thickness can be custom made</li> <li>MiraDRAIN can be applied over the full surface area of the rock face.</li> </ul>	<ul style="list-style-type: none"> <li>Posts have pre-formed holes for the rock bolts; therefore if there are any adjustments/changes based on the field conditions, these are more difficult to accommodate.</li> <li>Potential for graffiti on concrete wall.</li> <li>Will require more space as the panels will need to be placed some distance from the face in order to maintain a straight wall and allow for</li> </ul>	<ul style="list-style-type: none"> <li>Conventional construction techniques.</li> <li>Shorter construction duration compared to cast-in-place, and lesser restrictions related to temperature.</li> <li>Does not require formwork.</li> </ul>	<ul style="list-style-type: none"> <li>Higher cost than the shotcrete option</li> </ul>



## FOUNDATION REPORT THOROLD TUNNEL REHABILITATION

Rock Face Rehabilitation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
		<ul style="list-style-type: none"> <li>Panel wall can be constructed and applied at any time of the year.</li> <li>Precast panel wall would have a longer design life compared to shotcrete.</li> <li>Panel can be constructed with a recess detail for the bolt, which is more aesthetically pleasing.</li> <li>Panel wall can have architectural detail</li> </ul>	<ul style="list-style-type: none"> <li>irregularities (protrusions) in the rock face.</li> <li>Will require a level bearing surface between posts to support the bottom panels.</li> <li>Greater protrusion of face treatment from the rock face compared to shotcrete option.</li> </ul>		
Grout curtain behind wall	<ul style="list-style-type: none"> <li>Feasible</li> </ul>	<ul style="list-style-type: none"> <li>Does not require closure or reduction of lanes of the highway to implement solution</li> </ul>	<ul style="list-style-type: none"> <li>Potential for water to still leak through if there are any deficiencies in the construction of the grout curtain (which cannot be inspected).</li> <li>Requires permission for construction equipment on the St. Lawrence Seaway Authority property during construction</li> <li>Although the grouting will minimize the seepage of water it is not address the falling rocks from the rock face</li> </ul>	<ul style="list-style-type: none"> <li>Requires specialized equipment to carry out line drilling behind the rock face.</li> </ul>	<ul style="list-style-type: none"> <li>Highest cost</li> </ul>





**METRIC**

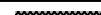





DIMENSIONS ARE IN METRES AND  
MILLIMETRES UNLESS OTHERWISE  
STATIONS IN KILOMETRES + METRE

### NOTES

1. The boreholes locations as provided on Plate A1 in Appendix A of the Thorvald Tunnel Feasibility Report were plotted using available imagery of the site and the borehole locations were identified from the coordinate system superimposed on the imagery. The boreholes locations and nothing and easting are therefore approximate.
2. BH1025 to BH1028 are not shown as they are not consistent with stratigraphy in adjacent boreholes nor as shown on plates A2 and A3 of the site and the Thorvald Tunnel Feasibility Report, October 1964, and the Acres Company Limited.
3. The ground surface elevations as depicted on this drawing have been adjusted from the SLA datum to be consistent with Geodetic datum. This adjustment is  $-0.35\text{ m}$  relative to the SLA datum as referred to the Borehole Records from the Acres 1964 report.

1. D.H.O Thorold Tunnel Feasibility Report Appendix A, prepared by H.I. Acres and Company Limited, dated October 1964, GEORES 30M-
2. Road labels – MNRF UO, obtained 2017.
3. Imagery – Region of Niagara, 2013.
4. Base plan, key plan and road surface in profile obtained from Morrison Hershfield, drawing file no. BC133581.dwg by Tulloch with file no. 2016-E-0001 Deliverables 2017-08-14.zip, downloaded April 15, 2017.




STRATIGRAPHY LEGEND	
	Fill
	Weathered Stratified Silty Clay Crust
	Relatively Unweathered Stratified Silty Clay
	Silty Till
	Limestone
	Shale

**GENERAL NOTES**

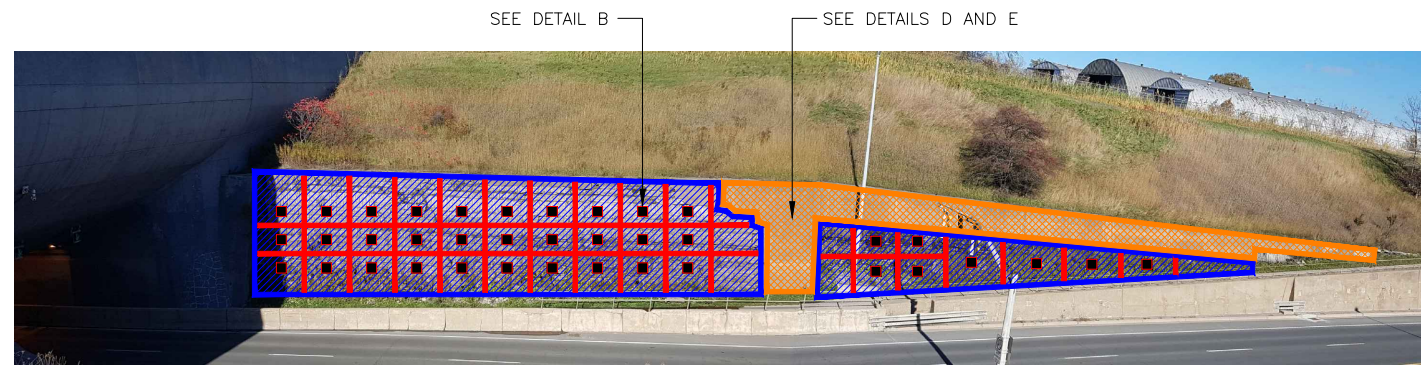
This drawing is for subsurface information only. The proposed structure and soil profiles are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this

Section 06 207 01-01 General Conditions									
-	-	-	-	-	-	-	-	-	-
NO.	DATE	BY	REVISION						
Geocres NO. 30M.3-302									
HWY. 58			PROJECT NO. 1668652				DIST. CENTRAL		
SUBM'D. KN		CHKD. DH		DATE: 2/27/2018			SITE: 34-177		
DRAWN: MR		CHKD. SMM		APPD. LCC			DWG. 1		

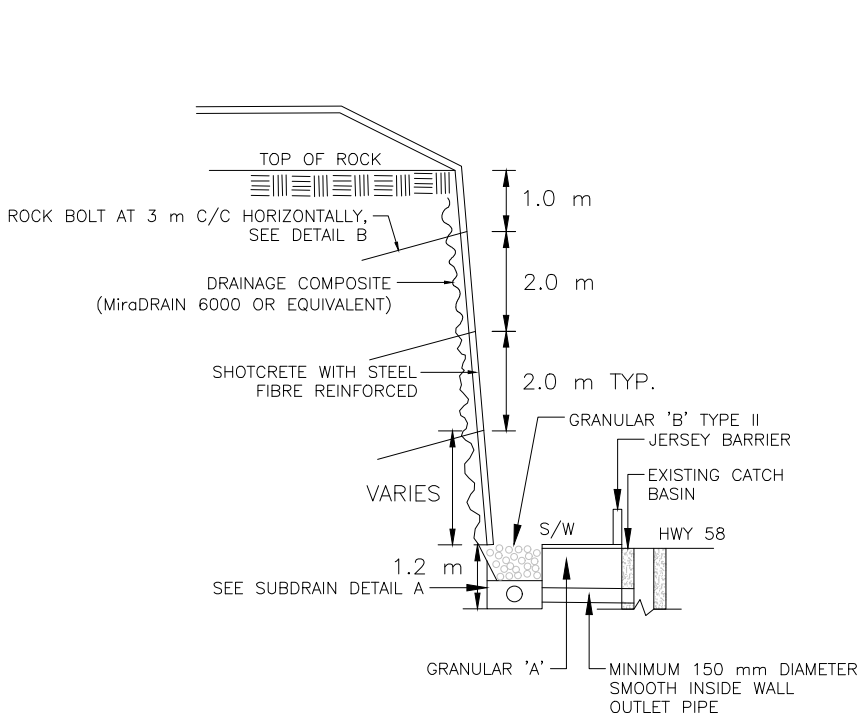
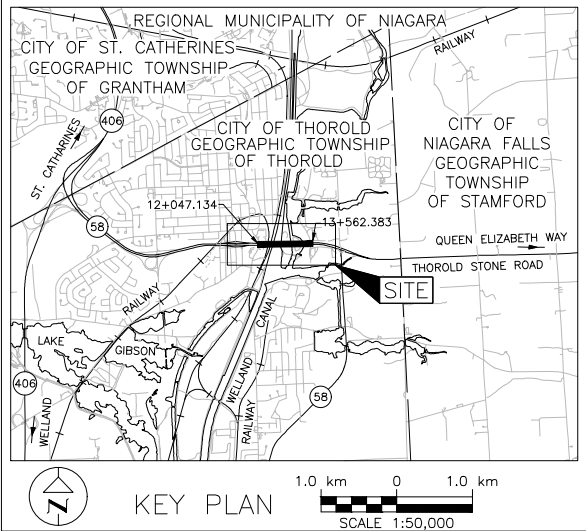




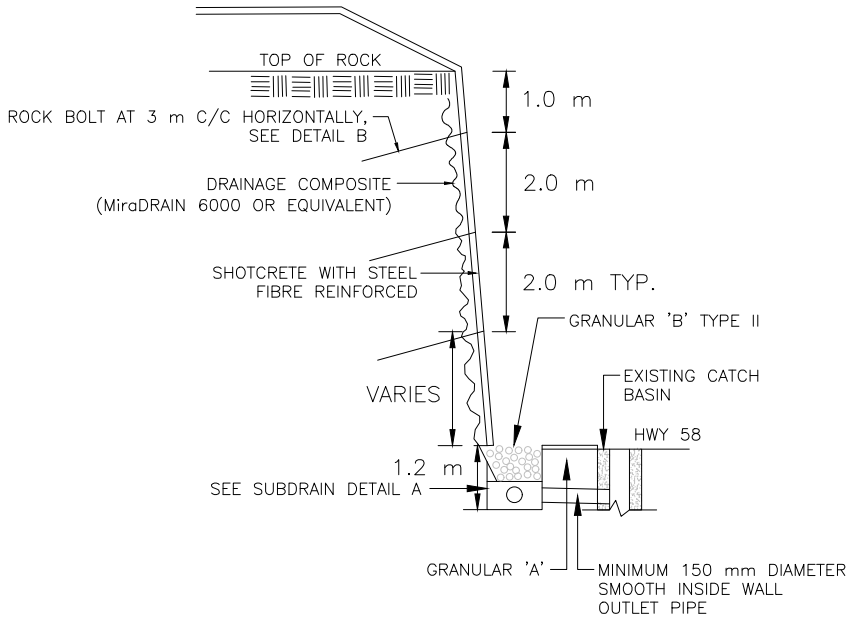
SEE DETAIL B

SEE DETAILS D AND E

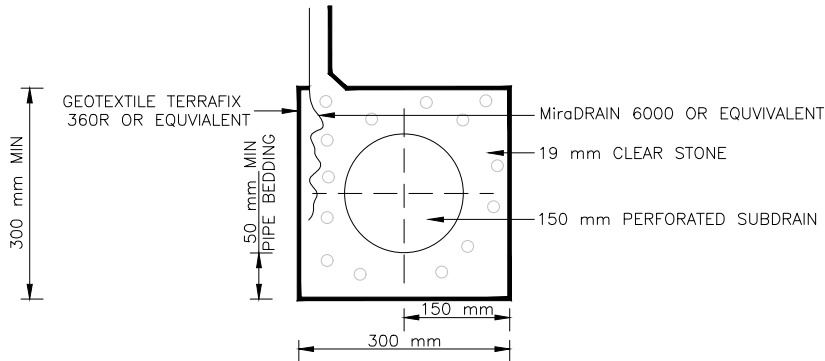
NO.		DATE		BY		REVISION	
Geocres No. 30M3-302							
HWY. 58				PROJECT NO. 1668652		DIST. CENTRAL	
SUBM'D. AC		CHKD. AC		DATE: 3/9/2018		SITE: 34-177	
DRAWN: DD		CHKD. SMM		APPD. MJT		DWG. 2	



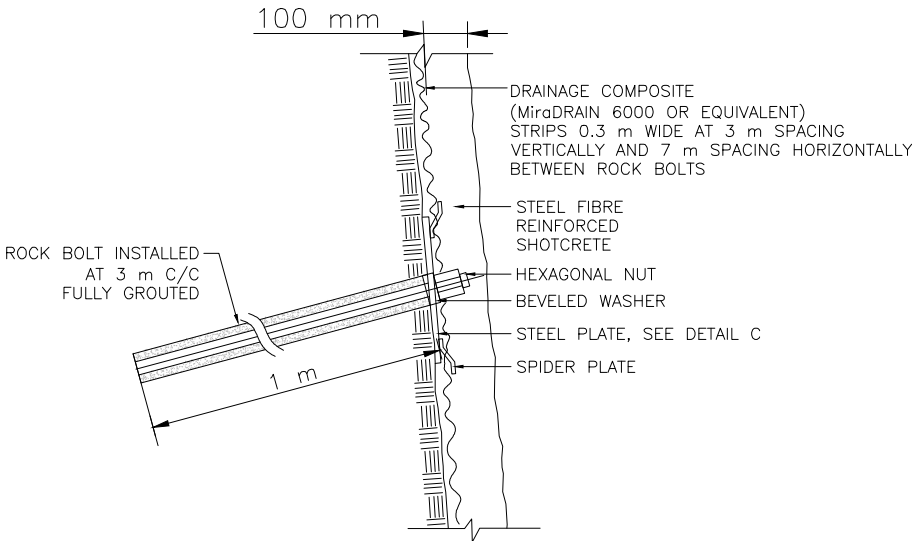
CROSS SECTION C - C' (13+140)  
NOT TO SCALE



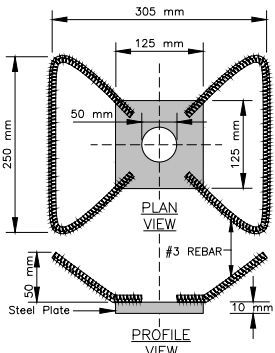
CROSS SECTION D - D' (13+140)  
NOT TO SCALE



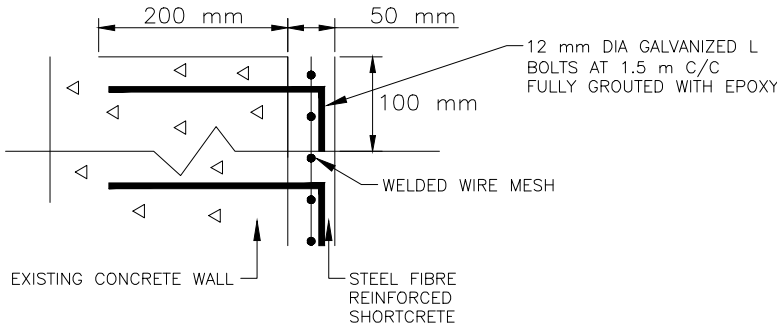
SUBDRAIN DETAIL A  
NOT TO SCALE



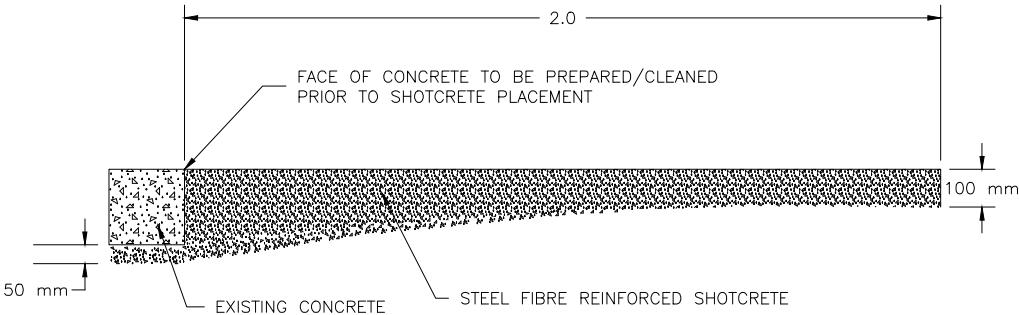
ROCK BOLT DETAIL B  
NOT TO SCALE



SPIDER PLATE DETAIL C  
NOT TO SCALE



SHOTCRETE ON EXISTING CONCRETE WALL DETAIL D  
NOT TO SCALE



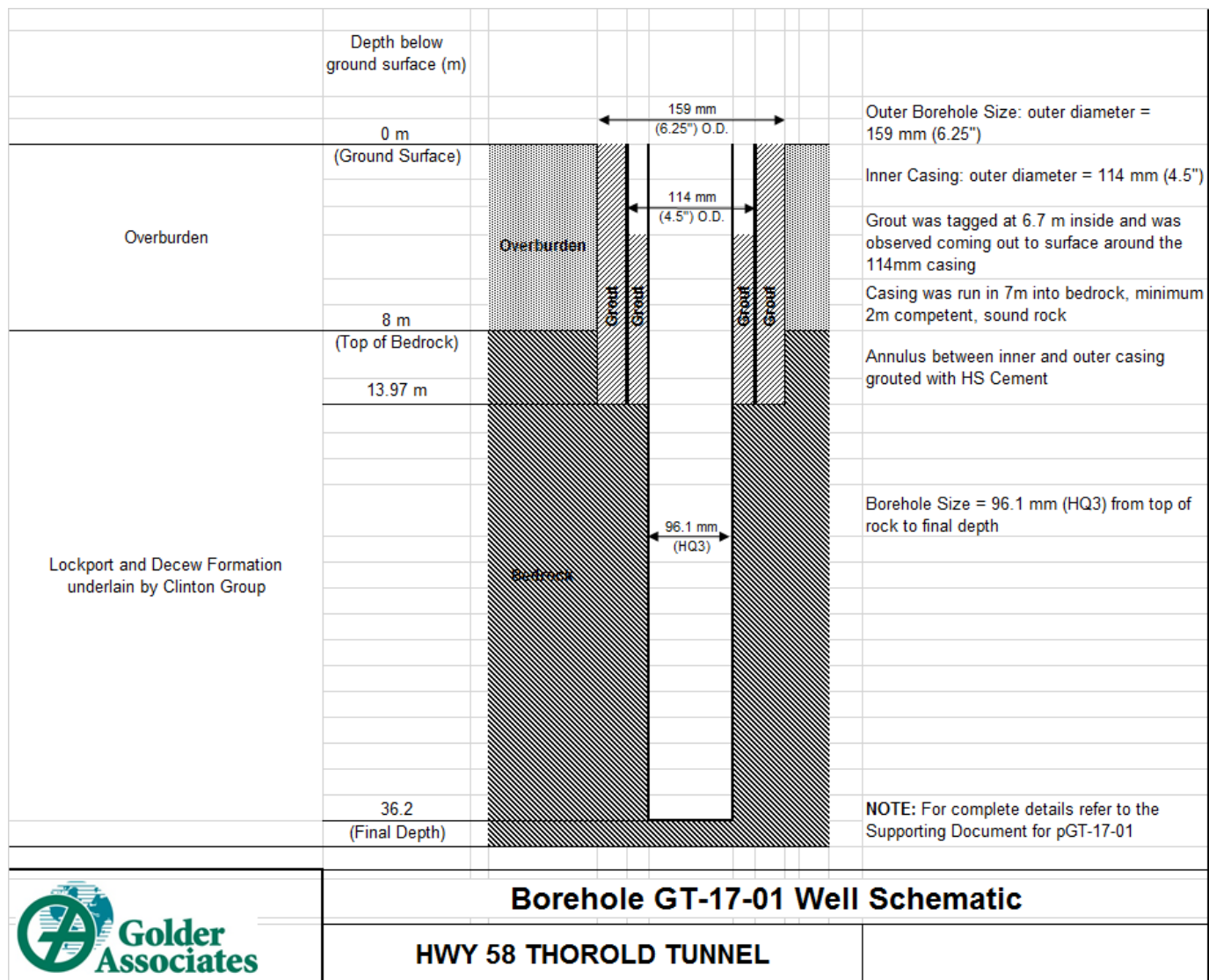
TYPICAL SHOTCRETE DETAIL AT CONCRETE STRUCTURE DETAIL E  
NOT TO SCALE



NO.	DATE	BY	REVISION
1			
Geocres No. 30M3-302			
HWY. 58		PROJECT NO. 1668652	
SUBM'D. AC		CHKD. AC	
DRAWN: DD		APPD. MJT	
		DIST. CENTRAL	
		SITE: 34-177/T	
		DWG. 3	



Last Edited By: ddoganovic Date: 2018-03-15 Time: 2:33:31 PM | Printed By: Ddoganovic Date: 2018-03-23 Time: 12:44:28 PM  
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CLIENT  
MTO

PROJECT  
THOROLD TUNNEL  
HIGHWAY 58  
PARTIAL WALL GROUTING

TITLE  
**BOREHOLE SCHEMATIC FOR GT17-01**

CONSULTANT

YYYY-MM-DD 2018-03-23

DESIGNED

PREPARED DD/AS

REVIEWED SMM

APPROVED LCC



PROJECT NO.  
1668652

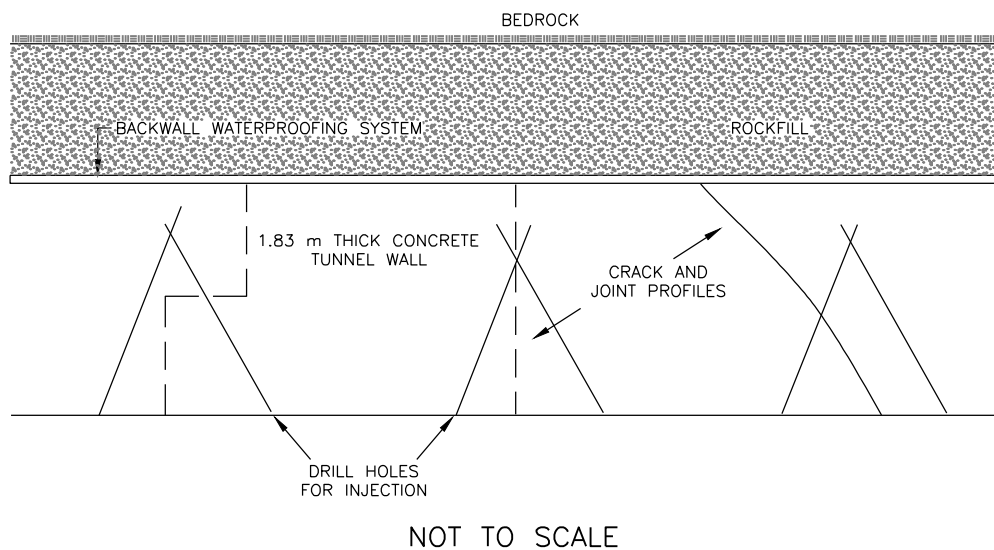
CONTROL

REV.  
A

FIGURE  
1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI A

25 mm

FIGURE  
2





# **APPENDIX A**

## **Borehole and Drillhole Records**



## 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
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	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
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

<b>(a)</b>	<b>Index Properties</b>
$\rho(\gamma)$	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
$w_l$ 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
$C_{\alpha}$	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
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

### (d) Shear Strength

$\tau_p, \tau_r$	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  
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 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) Non-Cohesive (Cohesionless) Soils

Condition	N 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

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\MTOWHWY 58 THOROLDTUNNEL\02 DATA\GINT\HWY 58 THOROLD.GPJ GAL-GTA.GDT 04/20/18

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



PROJECT <u>1668652</u>		<b>RECORD OF BOREHOLE No GT17-01</b>		SHEET 2 OF 3		<b>METRIC</b>	
G.W.P. <u>2370-16-00</u>		LOCATION <u>N 4775177.9; E 329361.1 MTM NAD ZONE 10 (LAT. 43.116049; LONG. -79.198193)</u>		ORIGINATED BY <u>AC</u>			
DIST <u>Central</u> HWY <u>58</u>		BOREHOLE TYPE <u>CME 75 - 210 mm O.D Hollow Stem Augers</u>		COMPILED BY <u>AC</u>			
DATUM <u>Geodetic</u>		DATE <u>October 17 to November 7, 2017</u>		CHECKED BY <u>SMM</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL LIMIT   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					
	--- CONTINUED FROM PREVIOUS PAGE ---																			
163.0	DOLOSTONE (BEDROCK) with shale partings (Gasport Member, Lockport Formation)		7	HQ	REC 100%														RQD = 100%	
15.8			8	HQ	REC 100%														RQD = 100%	
			9	HQ	REC 100%														RQD = 100%	
			10	HQ	REC 100%														RQD = 98%	
			11	HQ	REC 100%														RQD = 100%	
			12	HQ	REC 100%														RQD = 100%	
156.6	DOLOSTONE (BEDROCK) (Decew Formation)		13	HQ	REC 100%													RQD = 94%		
22.1			14	HQ	REC 100%														RQD = 100%	
			15	HQ	REC 100%														RQD = 100%	
153.0	SHALE (BEDROCK) (Rochester Formation)		16	HQ	REC 100%													RQD = 100%		
25.7			17	HQ	REC 100%														RQD = 100%	
			18	HQ	REC 100%														RQD = 100%	

Continued Next Page

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

GTA-MTO 001 S:\CLIENTS\MTOWHY\_58\_THOROLDTUNNEL\02\_DATA\GINT\HWY\_58\_THOROLD.GPJ GAL-GTA.GDT 04/20/18



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

PROJECT: 1668652

## RECORD OF DRILLHOLE: GT17-01

SHEET 1 OF 4

LOCATION: N 4775177.9 ; E 329361.1

DRILLING DATE: October 17 to November 7, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	JN - Joint FLT - Fault SHR- 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 MB- Mechanical Break BR - Broken Rock <b>NOTE:</b> For additional abbreviations refer to list of abbreviations & symbols.																FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
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## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: AC

CHECKED: MT

GTA-RCK 054 S:\CLIENTS\MT\HWY\_58\_THOROLDTUNNEL\02\_DATA\GINT\HWY\_58\_THOROLD.GPJ GAL-MISS.GDT 04/20/18

SHEET 2 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: Aardvark

[illegible]

## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



# GOLDER

LOGGED: AC

CHECKED: MT

STA-RCK 054 S:\CLIENTS\TOIHWY 58 THOROLDTUNNEL\02 DATA\GIN\THWY 58 THOROLD.GPJ GAL-MISS.GDT 04/20/18

SHEET 3 OF 4

DATUM: Geodetic

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	CORING LOG														FEATURES	R0/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



# GOLDER

LOGGED: AC

CHECKED: MT

\\GTA-RCK054\S\CLIENTS\TOI\HWY 58 THOROLDTUNNEL\02 DATA\GIN\THWY 58 THOROLD.GPJ GAL-MISS.GDT 04/20/18

PROJECT: 1668652

## RECORD OF DRILLHOLE: GT17-01

SHEET 4 OF 4

LOCATION: N 4775177.9 ; E 329361.1

DRILLING DATE: October 17 to November 7, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Aardvark

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	JN - Joint FLT - Fault SHR- 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 MB- Mechanical Break BR - Broken Rock  NOTE: For additional abbreviations refer to list of abbreviations & symbols.																FEATURES	RO/R1 ZONES	NOTES WATER LEVELS INSTRUMENTATION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
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## FEATURES LEGEND



BROKEN CORE



CLAY SEAM



LIMESTONE



LOST CORE

DEPTH SCALE

1 : 50



GOLDER

LOGGED: AC

CHECKED: MT

GTA-RCK 054 S:\CLIENTS\MT\Hwy\_58\_THOROLDTUNNEL\02\_DATA\GINT\Hwy\_58\_THOROLD.GPJ GAL-MISS.GDT 04/20/18





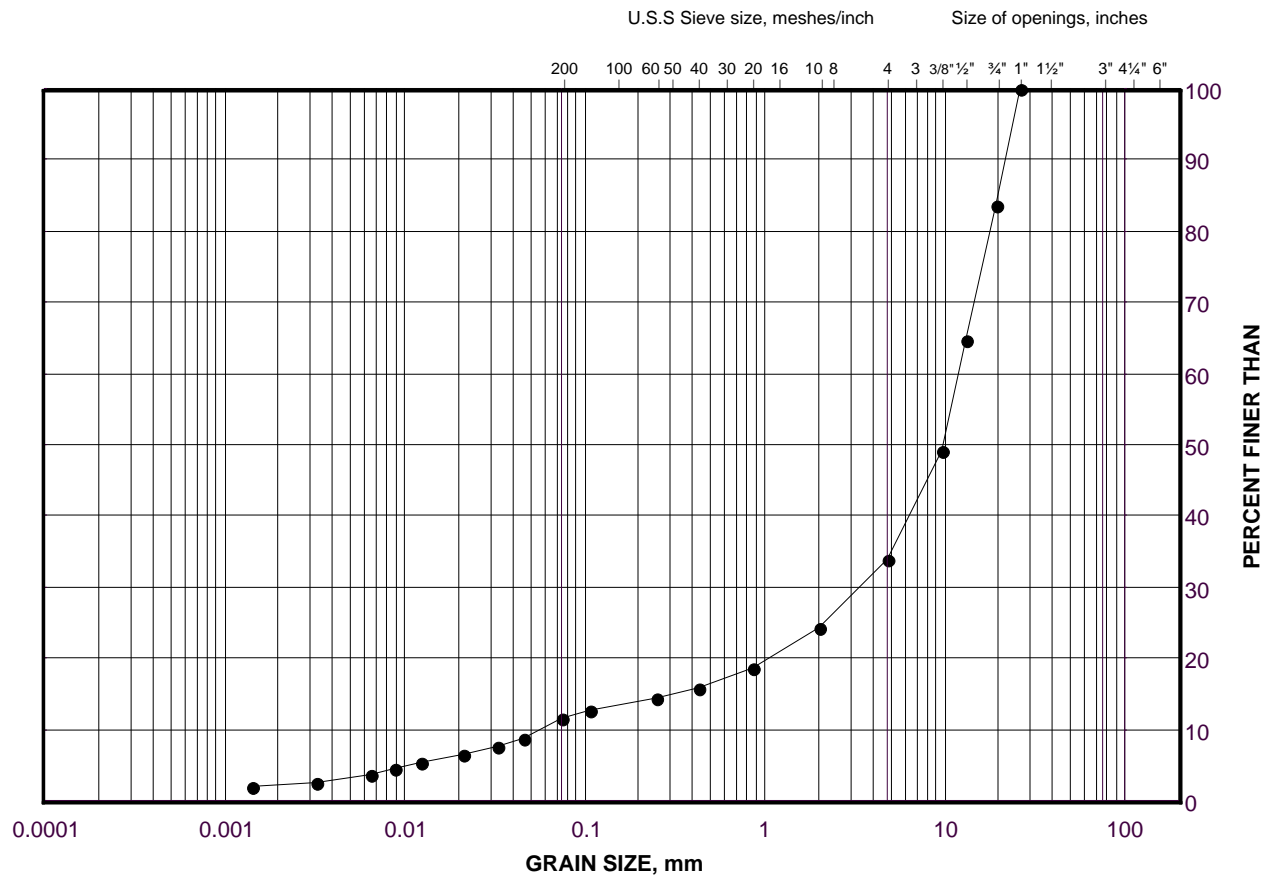
# **APPENDIX B**

## **Geotechnical Laboratory Test Results**

# GRAIN SIZE DISTRIBUTION

Sandy Gravel Fill

FIGURE B1



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

## LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	GT17-01	7	173.8

Project Number: 1668652

Checked By: SMM

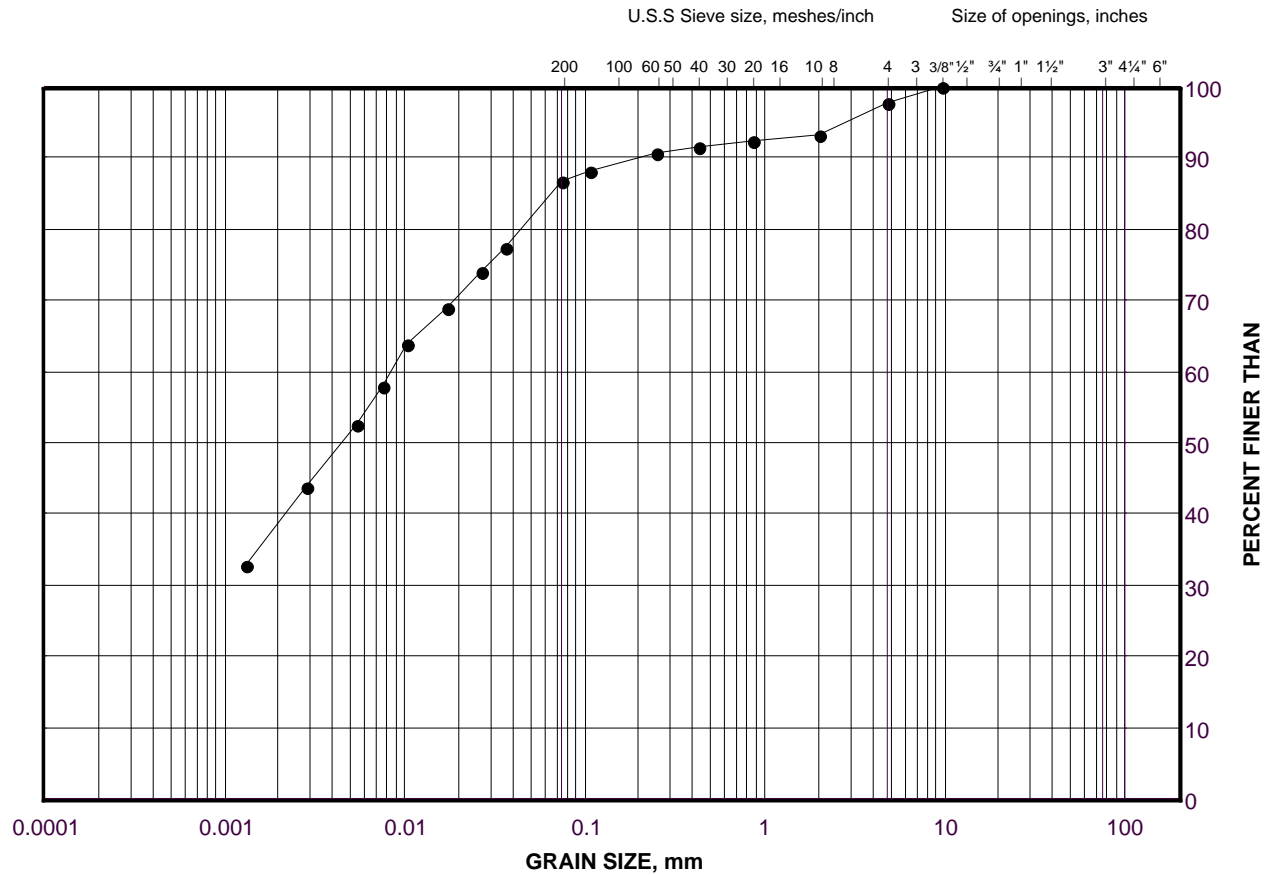
**Golder Associates**

Date: 21-Feb-18

# GRAIN SIZE DISTRIBUTION

Silty Clay Fill

FIGURE B2



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

## LEGEND

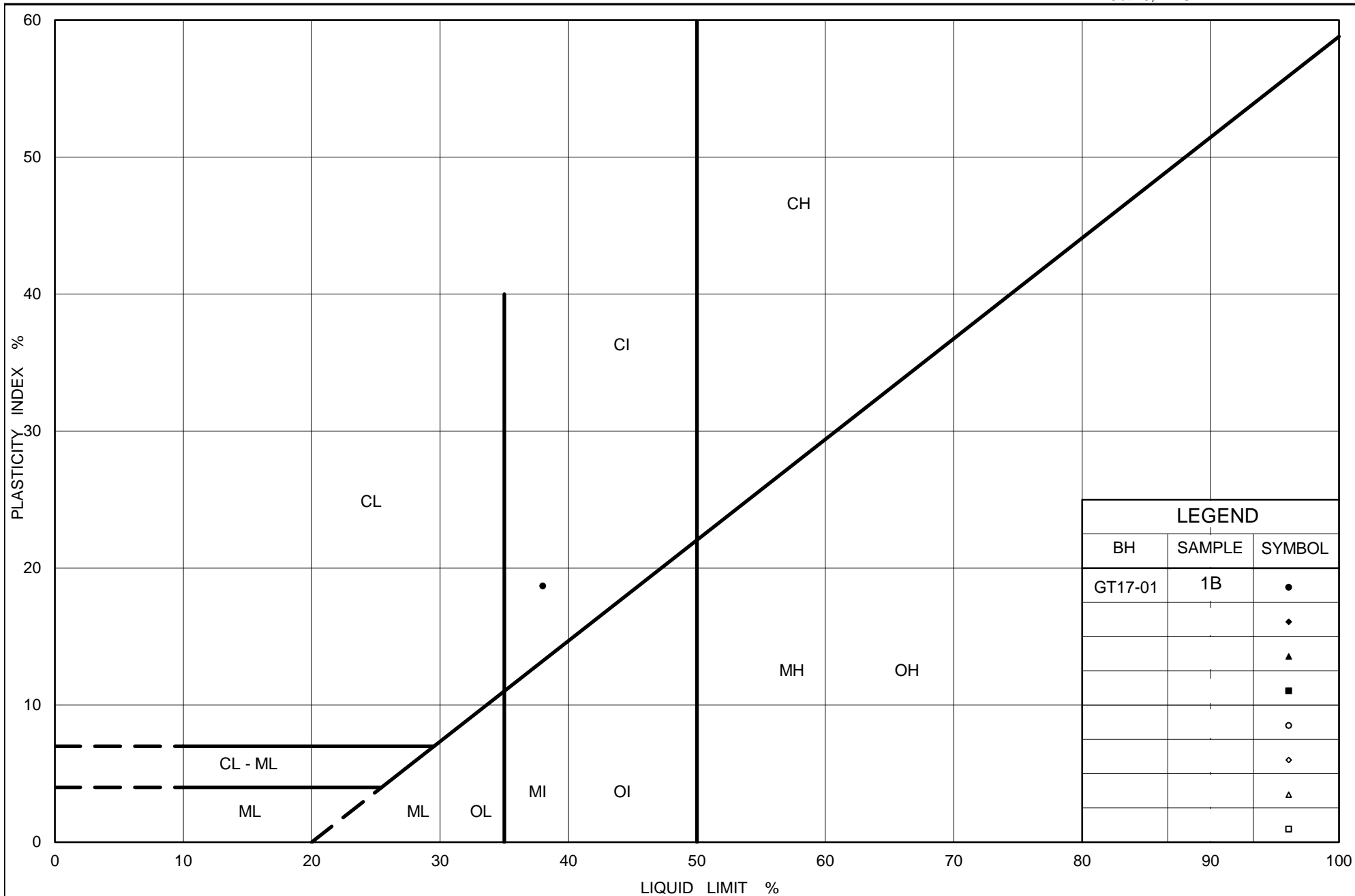
SYMBOL	Borehole	SAMPLE	ELEVATION(m)
•	GT17-01	1B	178.4

Project Number: 1668652

Checked By: SMM

**Golder Associates**

Date: 21-Feb-18



Ministry of Transportation

Ontario

## PLASTICITY CHART

### Silty Clay Fill

Figure No. B3

Project No. 1668652

Checked By: SMM

Start of Run No. 1 (7.09 m)

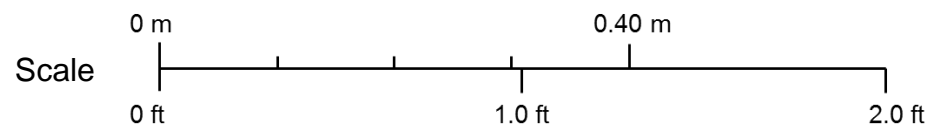
Start of Run No. 2 (7.83 m)


Start of Run No. 3 (9.39 m)

Start of Run No. 4 (10.92 m)

Start of Run No. 5 (12.44 m)

Box 1-3: 7.09 m to 13.97 m

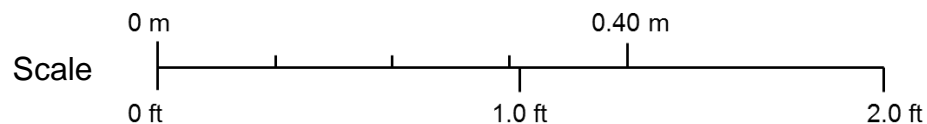



PROJECT		Thorold Highway 58 Tunnel Improvements			
TITLE		Bedrock Core Photographs Borehole GT17-01 (7.09 m to 13.97 m)			
	PROJECT No. 1668652		FILE No. ----		
	DESIGN	AC	171117	SCALE	NTS
	CADD	--		FIGURE B4- A	
	CHECK	SMM	20170208		
	REVIEW		20170208		



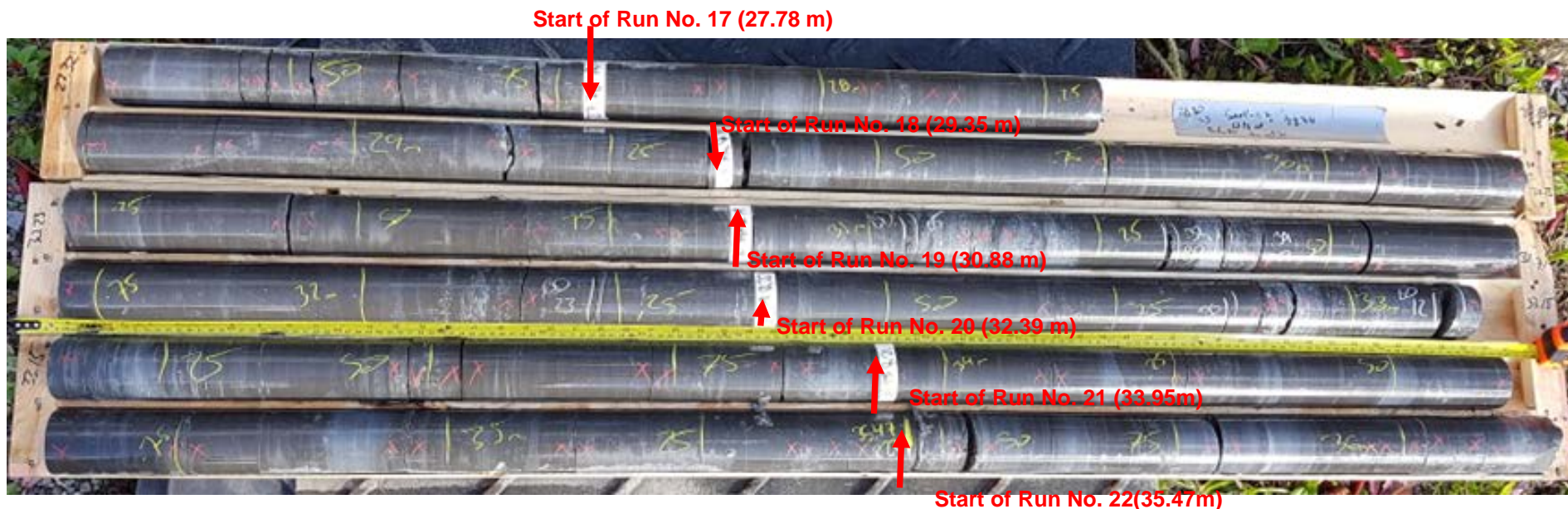


Box 4-8: 13.97 m to 27.32 m

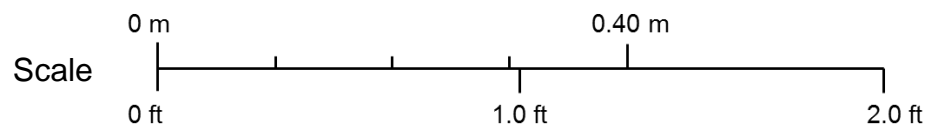



PROJECT		Thorold Highway 58 Tunnel Improvements			
TITLE		Bedrock Core Photographs Borehole GT17-01 (13.97 m to 27.32 m)			
	PROJECT No. 1668652		FILE No. ----		
	DESIGN	AC	171117	SCALE	NTS
	CADD	--		FIGURE B4- B	
	CHECK	SMM	20170208		
	REVIEW		20170208		





Box 9-11: 27.32 m to 36.18 m



PROJECT					
Thorold Highway 58 Tunnel Improvements					
TITLE					
Bedrock Core Photographs Borehole GT17-01 (27.32 m to 36.18 m)					
	PROJECT No. 1668652			FILE No. ----	
	DESIGN	AC	171117	SCALE	NTS
	CADD	--		FIGURE B4- C	
	CHECK	SMM	20170208		
	REVIEW		20170208		



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

## **SAMPLE IDENTIFICATION**

PROJECT NUMBER	1668652 (1008)	SAMPLE NUMBER	SA-1-1
PROJECT NAME	MTO/2016-E-0001/Thorold Tunnel	SAMPLE DEPTH, m	8.64-8.87
BOREHOLE NUMBER	GT17-01	DATE:	2018-01-17

## **TEST CONDITIONS**

MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.43

## **SPECIMEN INFORMATION**

SAMPLE HEIGHT, cm	14.81	WATER CONTENT, (specimen) %	0.30
SAMPLE DIAMETER, cm	6.10	UNIT WEIGHT, kN/m <sup>3</sup>	26.78
SAMPLE AREA, cm <sup>2</sup>	29.18	DRY UNIT WT., kN/m <sup>3</sup>	26.70
SAMPLE VOLUME, cm <sup>3</sup>	432.02	SPECIFIC GRAVITY	-
WET WEIGHT, g	1180.12	VOID RATIO	-
DRY WEIGHT, g	1176.59		

## **VISUAL INSPECTION**

## **FAILURE SKETCH**



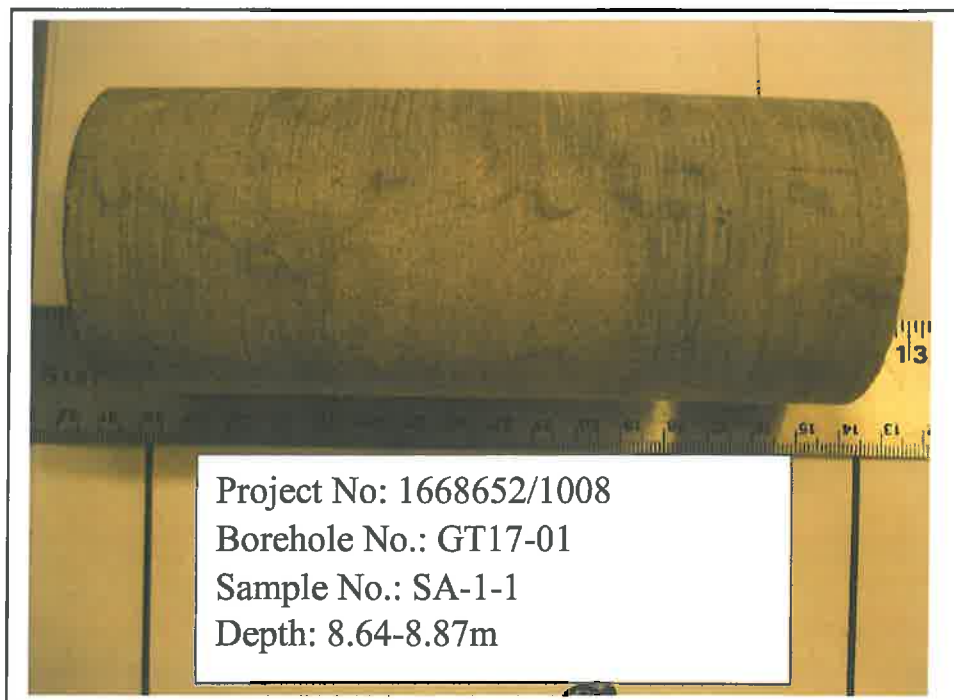
## **TEST RESULTS**

STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	101.4
----------------------	-----	---------------------------	-------

REMARKS:

Checked By: SMM

**Golder Associates**



BEFORE COMPRESSION



AFTER COMPRESSION

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

SAMPLE IDENTIFICATION			
PROJECT NUMBER	1668652 (1008)	SAMPLE NUMBER	SA-1-4
PROJECT NAME	MTO/2016-E-0001/Thorold Tunnel	SAMPLE DEPTH, m	11.78-12.07
BOREHOLE NUMBER	GT17-01	DATE:	2018-01-17

TEST CONDITIONS			
MACHINE SPEED, mm/min	N/A	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.49

SPECIMEN INFORMATION			
SAMPLE HEIGHT, cm	15.20	WATER CONTENT, (specimen) %	0.60
SAMPLE DIAMETER, cm	6.10	UNIT WEIGHT, kN/m <sup>3</sup>	26.63
SAMPLE AREA, cm <sup>2</sup>	29.19	DRY UNIT WT., kN/m <sup>3</sup>	26.47
SAMPLE VOLUME, cm <sup>3</sup>	443.72	SPECIFIC GRAVITY	-
WET WEIGHT, g	1205.41	VOID RATIO	-
DRY WEIGHT, g	1198.22		

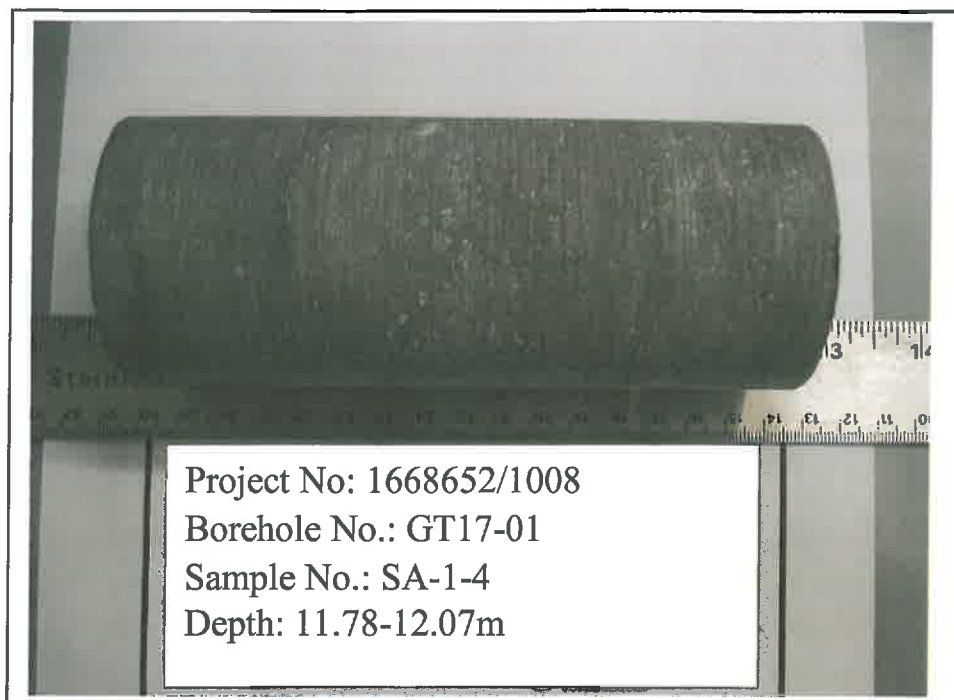
### VISUAL INSPECTION

### FAILURE SKETCH

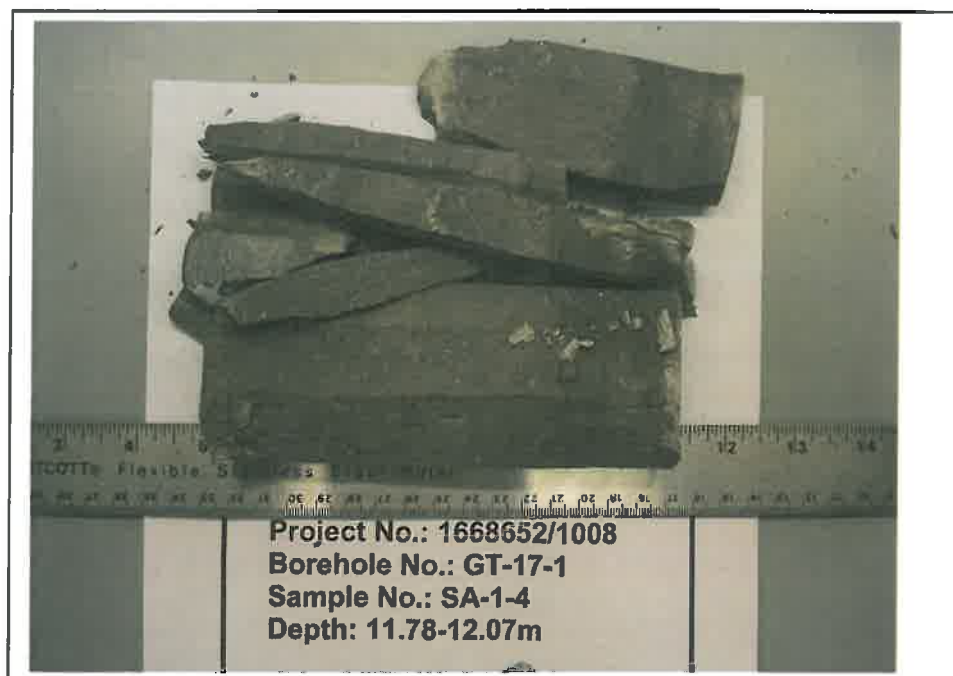


TEST RESULTS			
STRAIN AT FAILURE, %	N/A	COMPRESSIVE STRENGTH, MPa	70.9

REMARKS:



BEFORE COMPRESSION



AFTER COMPRESSION

**FACTUAL REPORT**

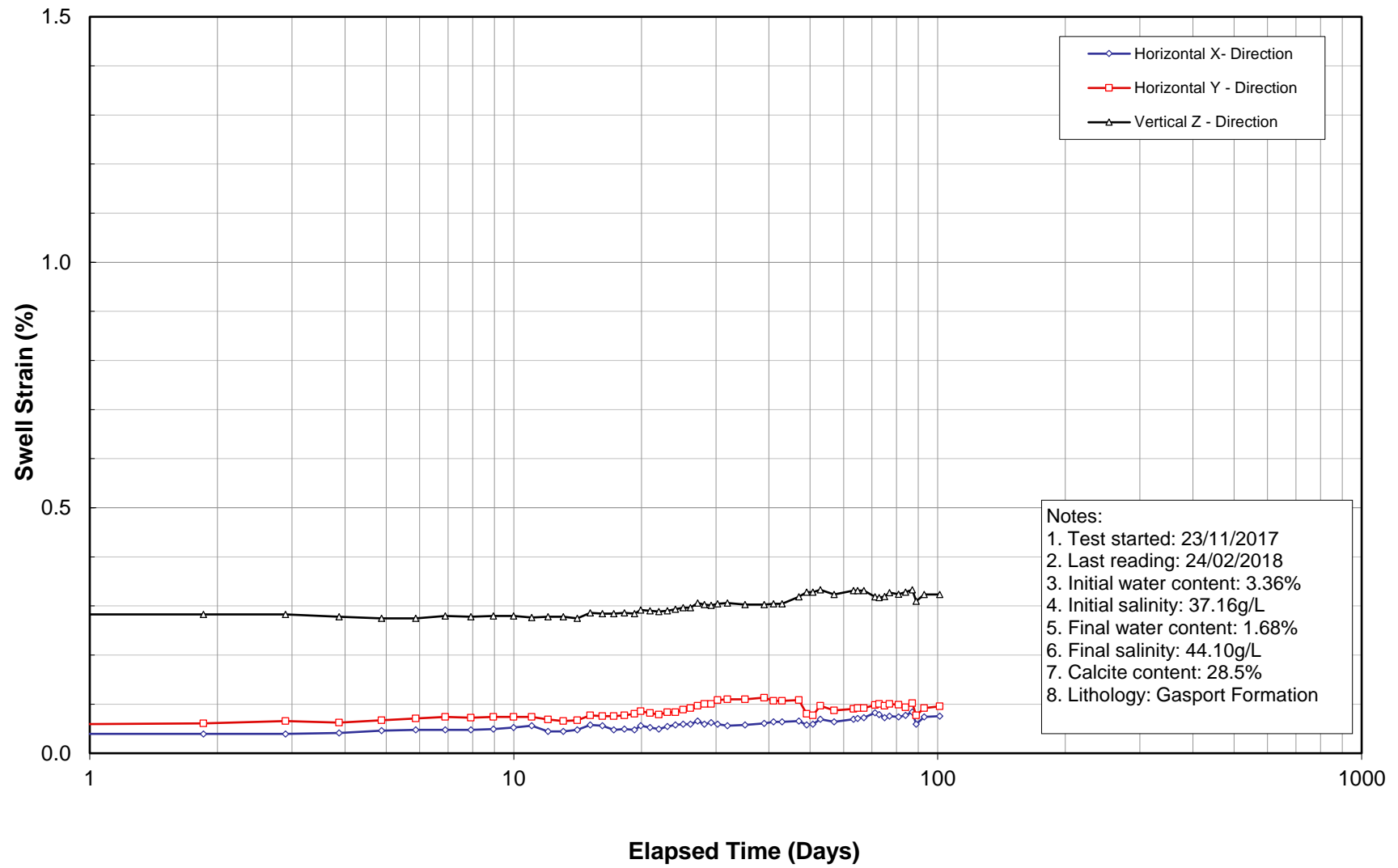
**Factual Results of Laboratory Swell Tests  
on Rock Samples  
*Thorold Tunnel***

Prepared for:  
*Golder Associates Ltd.*

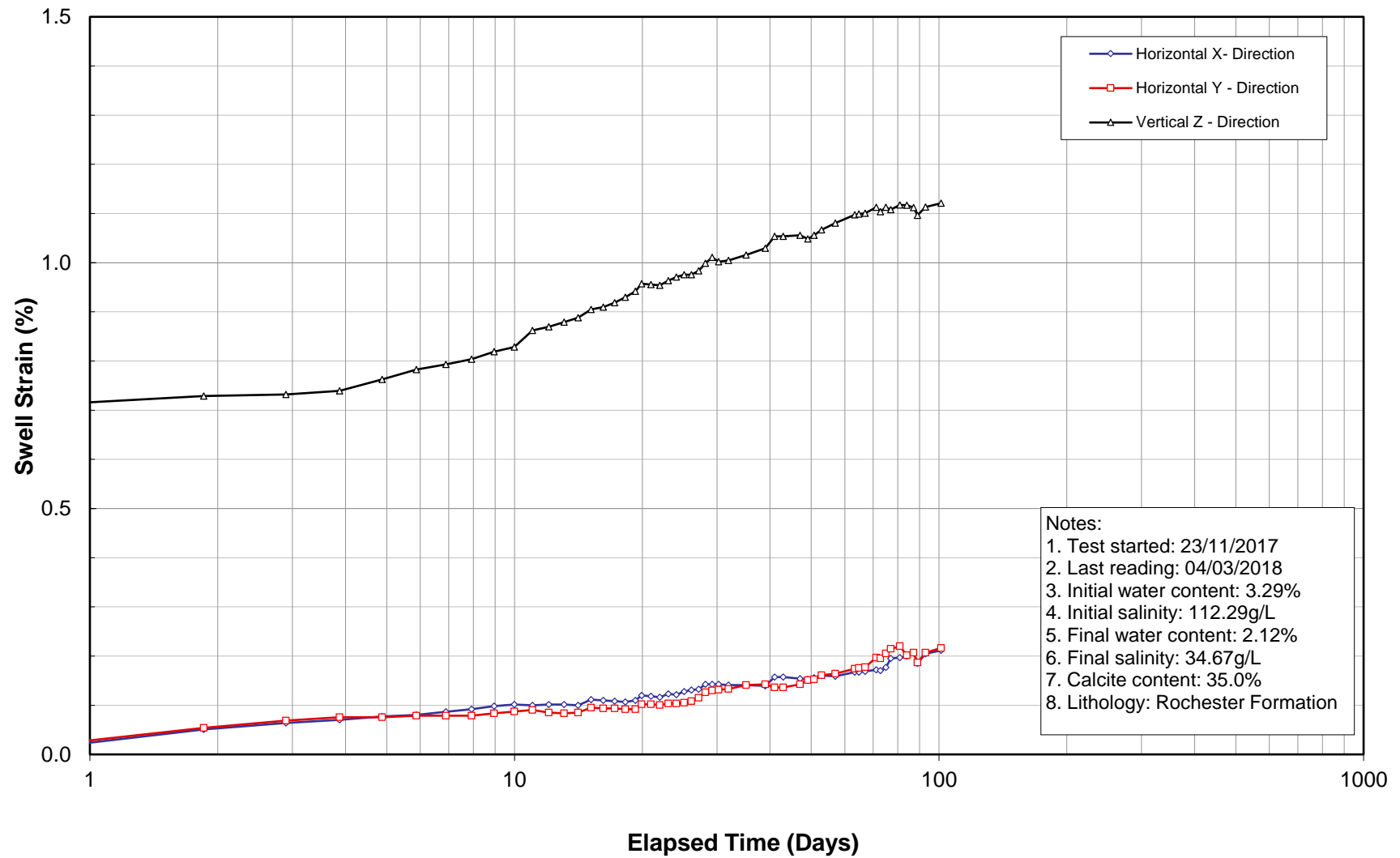
**K. Y. Lo Inc.**  
March 9, 2018



Free Swell Test  
Geotechnical Investigation at Thorold Tunnel  
**SA-1-1-FST-1**  
BH: SA-1-1, Depth: 17.19 m - 17.25 m



Free Swell Test  
Geotechnical Investigation at Thorold Tunnel  
**SA-1-2-FST-2**  
BH: SA-1-2, Depth: 28.33 m - 28.39 m





# **APPENDIX C**

## **Non-Standard Special Provisions**

## **COMPOSITE DRAINAGE PANELS - Item No.**

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### **Special Provision**

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#### **1.0 SCOPE**

This Special Provision covers the requirements for the supply and placement of Composite Drainage Panels along the rock faces at the locations shown on the Contract Drawings or as directed by the Contract Administrator. The installation of the Composite Drainage Panels shall be coordinated with the rock scaling and drainage improvement works at the site.

#### **2.0 REFERENCES – Not Used**

#### **3.0 DEFINITIONS**

**Composite Drainage Panels** are defined as high strength, dimpled, plastic drainage systems installed on the excavated rock face to allow groundwater to be collected and drained from the rock face.

#### **4.0 DESIGN AND SUBMISSION REQUIREMENTS**

The Contractor shall submit manufacturer's product data certificates of compliance for Composite Drainage Panels to the Contract Administrator at least seven (7) days prior to the use of the products.

No products shall be used in the works without prior submittal of sufficient acceptable information.

#### **5.0 MATERIALS**

Composite Drainage Panels shall be prefabricated drainage composites consisting of a three-dimensional, dimpled, polystyrene core with a non-woven filter fabric (MiraDrain 6000 or equivalent).

Composite Drainage Panels shall have a minimum compressive strength of 700 kPa and a minimum vertical flow rate of 100 litres/minute per metre.

#### **6.0 EQUIPMENT - Not Used**

#### **7.0 CONSTRUCTION**

Composite Drainage Panels shall be installed as shown on the Contract Drawings and as specified by the manufacturer or as directed by the Contract Administrator.

Composite Drainage Panels shall be installed at the locations shown on the Contract Drawings in strips over the full height of the excavated rock faces. All rock surfaces shall be machine scaled of loose rock and rubble prior to placement of drainage composite strips.

Composite Drainage Panels shall be positioned with the dimples towards the rock/drainage side, and attached to the rock surface as specified by the manufacturer.

If required, adjacent panel ends shall be connected as specified by the manufacturer. The panel edge shall be butted to the edge of the adjacent panel, dimple-to-dimple, or the edge of the next panel shall be placed over

two dimples and interlocked. Connections shall be completed in shingle fashion so that moisture will flow within the overlap and not against/through it.

Composite Drainage Panels shall extend to the toe of the rock faces, where they shall be connected to the site drainage system in such a way that the flow of water to the drainage system is unimpeded.

**8.0 QUALITY ASSURANCE - Not Used**

**9.0 MEASUREMENT FOR PAYMENT - Not Used**

**10.0 BASIS OF PAYMENT**

Payment at the Contract price for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

**ROCK SHOTCRETING – Item No.**  
**ROCK BOLTING – Item No.**

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Special Provision

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**Amendment to OPSS.PROV 203, November 2014**

**Rock Stabilization**

**203.01 SCOPE**

Section 203.01 of OPSS.PROV 203 is deleted in its entirety and replaced with the following:

This specification covers the requirements for the stabilization of the existing rock face on the north and south sides of Highway 58 east of the east portal of the Thorold Tunnel using a combination of Composite Drainage Panels (specified elsewhere in the Contract Documents), rock bolts, spider plates, welded wire mesh (where noted) and the application of steel fibre reinforced shotcrete, all as shown on the Contract Drawings.

**203.02 REFERENCES**

Section 203.02 of OPSS.PROV 203 is amended by the addition of the following:

**Ontario Provincial Standard Specifications, Construction**

OPSS 202      Rock Removal by Manual Scaling, Machine Scaling, Trim Blasting, or Controlled Blasting

**ASTM International**

A 108            Standard Specification for Steel Bar, Carbon and Alloy, Cold-Finished  
A 536            Standard Specification for Ductile Iron Castings

**Others**

CSA            General Requirements for Rolled or Welded Structural Quality Steel / Structural Quality Steel

**203.03 DEFINITIONS**

Section 203.03 of OPSS.PROV 203 is amended by the addition of the following:

**Composite Drainage Panels** are defined as high strength, dimpled, plastic drainage systems installed on the excavated rock face to allow groundwater to be collected and drained from the rock face.

**Spider Plates** are defined as a steel plate with two #3 rebar in the form of a circular shape from the steel plate. The spider plates shall be affixed to the rock bolt for the purpose of assisting the shotcrete to adhere to the rock bolt. For the purposes of this specification, any reference to the term “face plate” shall be deemed to be a reference to Spider Plate.

**Galvanized “L” Bolts** are defined as 12 mm diameter galvanized bolts with a 90-degree bend that are 200 mm long in one direction and 100 mm long in the perpendicular direction.

**Welded Wire Mesh** are defined as the welded wire mesh that is applied to the existing concrete wall within the shotcrete limits as shown on the Drawings.

## **203.04 DESIGN AND SUBMISSION REQUIREMENTS**

### **203.04.01 Submission Requirements**

Section 203.04 of OPSS.PROV 203 is amended by the addition of the following:

#### **203.04.01.03 Spider Plates**

The Contractor shall submit manufacturer’s product data certificates of compliance for Spider Plates to the Contract Administrator at least seven (7) days prior to the use of the products.

No products shall be used in the works without prior submittal of sufficient acceptable information.

#### **203.04.01.05 Galvanized “L” Bolts**

The Contractor shall submit mill certificates for Galvanized “L” Bolts to the Contract Administrator at least seven (7) days prior to the use of the products.

No products shall be used in the works without prior submittal of sufficient acceptable information.

#### **203.04.01.06 Welded Wire Mesh**

The Contractor shall submit mill certificates for Welded Wire Mesh to the Contract Administrator at least seven (7) days prior to the use of the products.

No products shall be used in the works without prior submittal of sufficient acceptable information.

## **203.05 MATERIALS**

Section 203.05 of OPSS.PROV 203 is amended by the addition of the following:

### **203.05.03 Galvanized “L” Bolts**

The “L” bolts shall have a minimum 12 mm diameter, galvanized and fully grouted with epoxy. The “L” bolts shall have a length of 200 mm in one direction and 100 mm in the perpendicular direction.

### **203.05.04 Welded Wire Mesh**

Welded wire mesh shall have 100 mm x 100 mm openings fabricated from 4 mm diameter galvanized wires conforming to CSA G30.5. The wires shall be welded at the joints.

## **203.05 MATERIALS**

### **203.05.01 Rock Bolts**



Section 203.05.01 of OPSS.PROV 203 is deleted in its entirety and replaced with the following:

Rock bolts shall be a minimum 25M reinforcing steel bars according to OPSS 1440 with a minimum yield strength of 400 MPa and a minimum length of 1.0 m, unless specified elsewhere in the Contract Documents. Rock bolts shall be fully threaded bars or bars threaded at one end and provided with a Spider Plate of at least 125 mm x 125 mm x 10 mm in area as shown on the Contract Drawings with a nut and beveled or spherical washers as recommended by the manufacturer. The steel for nuts shall conform to ASTM A108, washers shall conform to ASTM A536 and bearing plates shall conform to CSA G40.21, Grade 44 G or equal. The Spider Plate shall consist of #3 rebar. All rock bolt components shall be hot-dip galvanized according to ASTM A123.

Section 203.05.01.02 of OPSS.PROV 203 is amended by the addition of the following:

#### **203.05.01.02                      Grout**

Cement grout shall be pre-mixed, non-metallic, shrinkage compensating grout placed according to the manufacturer's specifications.

Water for use in grout mixes shall be clean and free of deleterious substances. The water shall be filtered if necessary to reduce the suspended solids to less than 500 mg/litre.

Cubes shall be cast and bleed tests performed as required by the Engineer to check the performance of the grout.

#### **203.07                                      CONSTRUCTION**

Section 203.07 of OPSS.PROV 203 is amended by the addition of the following:

#### **203.07.06                                      Galvanized "L" Bolts**

Holes for the galvanized "L" bolts shall be drilled at the diameter recommended by the supplier for the size of the "L" bolt as specified on the Drawings. The 200 mm long portion of the bolt shall be drilled into the existing concrete wall and epoxy grouted into the concrete. The 100 mm long portion shall be tied with wire to the welded wire mesh for the purpose of securing the welded wire mesh to the existing concrete wall. The "L" bolts shall be installed at 1.5 m centre-to-centre as specified on the Drawings.

The drill holes shall be thoroughly cleaned with compressed air prior to the installation of an epoxy.

The 100 mm long portion of the "L" bolt shall be tied to the welded wire mesh.

#### **203.07.01                                      Rock Bolting**

Section 203.07.01 of OPSS.PROV 203 is amended by the addition of the following:

The existing concrete wall that is to be shotcreted shall be protected with Welded Wire Mesh which shall be secured with Galvanized "L" Bolts to the concrete wall so that shotcrete can be applied in behind and on top of the welded wire mesh. The 200 mm long portion of the bolt shall be drilled into the existing concrete wall and epoxy grouted into the concrete. The 100 mm long portion shall be tied with wire to the welded wire mesh for the purpose of securing the welded wire mesh to the existing concrete wall.

All concrete surfaces shall be roughened prior to placement of Welded Wire Mesh and Galvanized “L” Bolts.

**203.07.02                      Rock Shotcreting**

**203.07.02.01                      Operational Constraint**

Section 203.07.02.01 of OPSS.PROV 203 is amended by the deletion of the second paragraph and replacement with the following:

The application of shotcrete to the rock surface shall not proceed until the rock surface has been properly prepared in accordance with OPSS 202 and verified by the Contract Administrator and the Composite Drainage Panels are fully installed and verified by the Contract Administrator. Furthermore, the application of shotcrete to the existing concrete wall shall not proceed until the existing concrete wall has been roughened and verified by the Contract Administrator.

**203.07.02.03                      Placing**

Section 203.07.02.03 of OPSS.PROV 203 is amended by the deletion of the fifth paragraph and replacement with the following:

Shotcrete shall be placed to create a minimum 100 mm thick layer to stabilize fractured rock and support rock overhangs at the locations as specified on the Contract Drawings. Shotcrete shall be placed to create a minimum 50 mm thick layer over the existing concrete wall at the locations as specified on the Drawings. At the juncture between the rock and concrete wall, the shotcrete shall be tapered over a distance of 2 m to provide a smooth transition between the 100 mm thick and 50 mm thick shotcrete. Where an undercut is present on the existing rock face, the shotcrete shall have a minimum thickness of 100 mm and shall not be built out to match the rock face above.

Maintain applied shotcrete moist by water spraying at the start, middle and end of each working day for the first three days after placement.

**203.09                      MEASUREMENT FOR PAYMENT**

**203.09.01                      Actual Measurement**

**203.09.01.02                      Rock Shotcreting**

Section 203.09 of OPSS.PROV 203 is amended by the deletion of the following:

Measurement shall be by area in square metres of shotcrete and shall include those areas that have a minimum thickness of 100 mm on the existing rock face and a minimum thickness of 50 mm on the existing concrete wall and shall also include the taper between the two thicknesses.

**203.10                      BASIS OF PAYMENT**

**203.10.02                      Rock Shotcreting – Item  
Rock Bolting – Item**

Section 203.10 of OPSS.PROV 203 is amended by deletion of the first paragraph and replacement with the following:

Payment at the Contract price for the above tender items shall be full compensation for all labour, Equipment and Material to do the work, including Spider Plates, Galvanized “L” Bolts, Welded Wire Mesh, roughening of the existing concrete wall and tapering the shotcrete.

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