REPORT

Foundation Investigation and Design

Proposed Culvert Replacement, Station 20+385, Highway 60
Chaffey Township, Ontario
GWP 5333-11-00, (DB-2017-5005)

Submitted to:

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Distribution List

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Table of Contents

PART A

Foundation Investigation Report
Proposed Culvert Replacement
Station 20+385, Highway 60
Chaffey Township, Ontario
DB-2017-5005

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by R.W. Tomlinson Limited (Tomlinson) to provide geotechnical investigation and engineering services to the design-build team, relating to the proposed replacement of an existing culvert using trenchless methods in the Township of Chaffey, Ontario (DB-2017-5005).

The purpose of the investigation is to evaluate the subsurface soil and groundwater conditions along the proposed culvert alignment by means of advancing a limited number of test holes. Specifically, the geotechnical investigation consists of borings, test pit excavations, soil and rock sampling, borehole / test pit logging, and field and laboratory testing. Based on our interpretation of the findings of the investigation, this report provides an assessment of potential geotechnical issues and geotechnical engineering recommendations for the trenchless installation of the proposed culvert. It is noted that a Preliminary Foundation Investigation and Design Report was previously completed by others, as referenced in Section 3.

The terms of reference and scope of work for the geotechnical investigation are outlined in MTO's Request for Proposals (RFP) dated November 8, 2017. This report has been completed in general accordance with the RFP.

2.0 SITE DESCRIPTION

It is our understanding that an existing Corrugated Steel Pipe (CSP) culvert (Culvert No. 27) is to be replaced, as identified by the Ministry of Transportation, Ontario (MTO). The existing culvert is located perpendicular to Highway 60, about 10.3 km east of Highway 11 in the Township of Chaffey, Ontario. The center of the existing culvert is located at chainage 20+385 of the Highway 60 alignment as shown on Drawing 1.

The existing culvert is 1520 mm in diameter and is about 43 m in length and the existing inverts are at about Elevations 327 and 326 m, at the inlet (north) and outlet (south) ends, respectively. The date of construction of the culvert is unknown. An unnamed creek flows through the existing culvert perpendicular to Highway 60 in a north to south orientation.

In the site area, Highway 60 is a 2 lane, undivided highway, running in an east-west direction. The maximum height of the existing embankment at the culvert location is about 7 m and the existing pavement grade of Highway 60 at the culvert location is at about Elevation 333.7 m. The existing embankment side slopes are formed at about 1.8 horizontal to 1 vertical (1.8H:1V).

The new culvert is proposed to be a concrete pipe having an inside diameter of approximately 1500 mm. We understand that the new culvert will be installed to the east of and parallel to the existing culvert, while maintaining the same inlet and outlet invert elevations as the existing culvert. Due to the significant depth of cover and the potential for substantial disruption to traffic, trenchless construction methods have been selected by the Contractor for the installation of the culvert.

3.0 INVESTIGATION PROCEDURE

3.1 Previous investigation

A preliminary foundation investigation was previously carried out for this project by LVM-Merlex (now Englobe Corp.) on behalf of the MTO. The results of that investigation were provided in a report titled "*Final Preliminary Foundation Investigation and Design Report, Culvert Replacement, Highway 60, Station 20+385 - Township of Chaffey, GWP 5333-11-00, Ref. No. 14/07/14083-F2, Geocres No. 31E-345*," dated May 5, 2015.

Three boreholes were advanced by LVM-Merlex as part of the preliminary investigation at this site in 2014. The borehole locations including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum are summarized below.

Borehole Number	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)	Piezometer/ Monitoring Well
BH 1	Outlet Side	5025186.9	334414.0	326.1	6.1	None
BH 2	Inlet Side	5025222.2	334434.3	328.5	7.2	Yes
BH 3	Midspan	5025201.7	334426.9	333.5	16.4	None

3.2 Current Investigation

The current geotechnical investigation for the proposed replacement culvert was carried out by Golder between August 14 and August 22, 2018. During this period, one borehole (18-104) and two test pits (18-109A & 18-109B) were advanced to depths ranging from about 1.4 to 9.8 m below ground surface. The borehole and test pit locations are shown on Drawing 1. The Records of Borehole and Test Pit Sheets from the current investigation are provided in Appendix A. Lists of abbreviations and symbols are also provided in this Appendix to assist in the interpretation of the borehole records.

Borehole 18-104 was advanced using a combination of 210 mm outside diameter continuous-flight hollow-stem augering, casing and wash boring and rock coring using a truck-mounted drill rig, supplied and operated by Marathon Drilling Co. Ltd. of Ottawa, Ontario and fitted with capability for Standard Penetration Testing (SPT).

Continuous soil samples were obtained within the tunnel horizon, using a 50 mm outside diameter split spoon sampler. The sampler is driven by an automatic hammer, in accordance with the SPT procedures, as specified in ASTM D1586. Note that the split spoon samplers used in the investigation limit the maximum particle size that can be sampled and tested to about 40 mm. Therefore, particles or objects that may exist within the soils that are larger than this dimension would not be sampled or represented in the grain size distributions (i.e., coarse gravel, cobbles and boulders). The results of the in-situ field tests (i.e. SPT ‘N’ values) as presented on the Record of Borehole sheets and in Section 4.2 are uncorrected.

Where practical refusal to augering was encountered due to the presence of possible cobbles and/or boulders, HQ coring using a wash boring technique was used to advance through the obstructions.

Test pits 18-109A and 18-109B were advanced within the proposed entrance pit (inlet) area and at the toe of the existing embankment slope using a tracked mounted hydraulic excavator supplied and operated by Tomlinson. The test pits were advanced to depths of about 1.4 to 2.0 m below ground surface.

The fieldwork was supervised on a full-time basis by a member of Golder’s technical staff who located the boreholes and test pits in the field, directed the drilling, excavating and sampling operations, logged the boreholes and test pits, and took custody of the samples retrieved. The groundwater conditions and water levels in the open borehole were observed during and immediately following drilling operations. The groundwater seepage conditions in the test pits were also observed during the short time they remained open.

The recovered soil samples were identified visually and select samples were subjected to a laboratory testing program consisting of natural moisture content testing, gradation analyses and Atterberg Limit testing in accordance with MTO LS and/or ASTM Standards, as appropriate. The results of this testing program are shown on the Record of Borehole Sheets in Appendix A as well as graphically in Appendix B.

Upon completion of drilling, the borehole was backfilled with a mixture of bentonite and auger cuttings in general accordance with the Ontario Regulation 903 (as amended). The test pits were backfilled and nominally compacted using excavator bucket upon completion of excavating and sampling. The site conditions were restored following completion of the fieldwork.

The ground surface elevations at the test hole locations were surveyed by McIntosh Perry Consulting Engineers Ltd. using a Trimble GPS unit. The borehole and test pit locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to Geodetic datum, are summarized in the following table and are shown on Drawing 1.

Test hole Number	Type	Borehole Location	Northing (m)	Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)	Piezometer/ Monitoring Well
18-104	Borehole	Mid span	5025207.9	334433.9	333.1	9.8	None
18-109A	Test Pit	Entrance pit	5025222.2	334440.9	327.4	2.0	None
18-109B	Test Pit	Entrance pit	5025219.1	334439.9	328.2	1.4	None

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

As delineated in *The Physiography of Southern Ontario*[1], the site lies within the physiographic region known as the Mount Elgin Ridges. Surficial geologic mapping in the vicinity of the site indicates silt and clay, with minor sand and gravel bordering on fine textured glaciolacustrine deposits[2]. Bedrock in this region consists of siliceous crystalline migmatitic rocks and gneisses.

4.2 Site Stratigraphy

Details of the subsurface soil and groundwater conditions encountered in the test holes advanced as part of the current investigation are shown on the Record of Borehole and Test Pit sheets contained in Appendix A.
The results of the geotechnical laboratory testing carried out as part of the current investigation are included in Appendix B. The Record of Borehole sheets and lab testing results from the previous investigation are presented in Appendices C and D, respectively. Photographs of the test pits from the current investigation are provided in Appendix E.

An interpreted soil stratigraphy section projected along the centreline of the existing trenchless alignment is shown on Drawing 1. Note that the stratigraphic boundaries shown on the Record of Borehole and Test Pit sheets and on the interpreted stratigraphic section are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole and test pit locations.

[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*, Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.
[2] Surficial Geology of Southern Ontario, Ontario Geological Survey, 2003.

The following is a summarized account of the subsurface conditions encountered in the test holes, followed by more detailed descriptions of the major soil strata and groundwater conditions in the following sub-sections.

In general, the subsurface conditions at the site consist of granular embankment fill containing cobbles and boulders or rock fill, overlying interlayered deposits of clayey silt to silty clay and silty sand and sand deposits. The overburden soils are underlain by gneiss bedrock. The groundwater level is at approximately the creek level.

4.2.1 Topsoil

Topsoil was encountered at ground surface in Test Pits 18-109A and 18-109B at Elevation 327.4 m and 328.3 m, respectively. The topsoil measured 200 mm in thickness and consisted of silty sand and/or sandy silt with rootlets.

4.2.2 Pavement Structure

The pavement structure of the highway, encountered at the ground surface at Elevation 333.5 m, was penetrated in Borehole 3 and consisted of approximately 280 mm of asphalt underlain by about 120 mm of crushed gravel.

4.2.3 Embankment Fill

Embankment fill was encountered in Boreholes 18-104 and 3 as well as Test Pit 18-109B. The fill was penetrated to depths ranging from 1.4 to 6.4 m below ground surface. The thickness of the fill ranges from 1.4 m to 6.4 m. The base of the fill ranged from Elevation 326.7 m to 327.9 m.

The fill is variable in composition, consisting of sandy silt, sand and silt, silty sand, sand, gravelly sand and sand and gravel, mixed with rock fill. Three boulders were encountered and cored within the fill in Borehole 18-104 ranging in size from 400 mm to 600 mm. Other instances of augers grinding or SPT sampler that did not penetrate the full sample length are also indicative of the presence of cobbles, boulders or rock fill. In Test Pit 18-109B, cobbles and boulders ranging in size from 100 mm to 150 mm and 400 mm to 500 mm, respectively, were encountered. The test pit was terminated in the embankment fill. The lower portion of the fill in Borehole 18-104 contained wood and organics.

The SPT ‘N’ values measured within the fill range from 12 to 38 blows per 0.3 m of penetration, indicating a compact to dense compactness condition.

The result of a grain size distribution test carried out by Golder on one sample obtained from the embankment fill is provided on Figure B1 in Appendix B. Grain size distribution test results carried out by LVM-Merlex on two samples obtained from the embankment fill are provided on Figure L-1 in Appendix D. The measured water contents of samples from the fill ranged from about 2 to 19 percent.

4.2.4 Silty Sand

A silty sand deposit containing organics was encountered at ground surface and below the embankment fill in Boreholes 1 and 3, respectively. The surface of the deposit was encountered between Elevation 327.9 m and 326.1 m and extended to depths ranging from 1.3 to 7.5 m below ground surface. The presence of cobbles was inferred in Borehole 3 where SPT sampler that did not penetrate the full sample length and the presence of boulder size rocks was noted in Borehole 1.

Two SPT ‘N’ values measured in the silty sand deposit are 14 blows per 0.3 m of penetration, indicating a compact compactness condition.

The result of a grain size distribution test carried out by LVM-Merlex on one sample obtained from this deposit is provided on Figure L-2 in Appendix D. The natural water content measured on samples of the silty sand deposit range from about 28 to 32 percent.

4.2.5 Clayey Silt to Silty Clay

A deposit of clayey silt to silty clay was encountered underlying the topsoil/fill or silty sand deposit in Boreholes 18-104, 2 and 3 as well as Test Pit 18-109A. The deposit is described as having fissures and containing sand seams and rock fragments. The surface of the deposit was encountered between Elevation 326.0 m and 328.5 m and extended to depths ranging from 1.7 m to 8.3 m below ground surface. The thickness of the deposit ranges from 0.8 m to 2.1 m.

The SPT ‘N’ values measured within the clayey silt to silty clay deposit range from 10 to 38 blows per 0.3 m of penetration, indicating a stiff to hard consistency.

The results of a grain size distribution test carried out by LVM-Merlex on one sample obtained from this deposit are provided on Figure L-3 in Appendix D. Atterberg limit testing carried out by Golder and LVM-Merlex on four samples from this deposit and measured liquid limits ranging from about30 to 37 percent, plastic limits ranging from about 20 to 24 percent, and plasticity indices ranging from about 7 to 12 percent. The Atterberg limit test results are shown in Appendices B and D and indicate the deposit is classified as a clayey silt to a silty clay of low plasticity. The natural water content measured on samples of the clayey silt to silty clay deposit range from about 25 to 33 percent.

4.2.6 Sand

A deposit of sand was encountered in all boreholes and in Test Pit 18-109A, ranging in thickness from about 0.2 to 4.4 m, where fully penetrated. The surface of the sand deposit was encountered between Elevation 324.8 m and 326.4 m and generally encountered below the clayey silt to silty clay deposit. The deposit extended to maximum observed depths ranging from 2.0 to 12.7 m below ground surface (Elevations 320.8 m to 325.4 m). Borehole 18-104 and Test Pit 18-109A were terminated within this deposit.

The sand deposit was generally gravelly, and/or contained some gravel. In Test Pit 18-109A, this deposit consisted of silty sand to sandy silt and contained cobbles and boulders ranging in size from 130 mm to 350 mm. Cobble size rock pieces were also noted in Borehole 3.

The SPT ‘N’ values measured within the sand deposit range from 10 to 100 blows per 0.01 m of penetration, indicating a compact to very dense compactness condition. The higher ‘N’ values are likely attributed to the inferred presence of cobbles and boulders.

Grain size distribution test results carried out by LVM-Merlex on two samples obtained from the sand deposit are provided on Figure L-4 in Appendix D. The measured natural water contents of samples of the sand range from about 1 to 24 percent.

4.2.7 Bedrock

Bedrock was encountered beneath the sand deposit in Boreholes 1, 2, and 3 and cored to depths ranging from 3.0 to 3.7 m below bedrock surface. The retrieved core is described as grey/pink gneiss, as presented on the LVM-Merlex Record of Borehole sheets in Appendix C.

The following table summarizes the bedrock surface depths, elevations and Rock Quality Designation (RQD) values for each borehole that encountered bedrock during the previous investigation.

Borehole Number	Existing Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)	Rock Quality Designation (RQD)
1	326.1	3.1	323.0	35% - 41%
2	328.5	3.7	324.8	67% - 89%
3	333.5	12.7	320.8	83% - 97%

The RQD values measured on the recovered core samples range from about 35 to 97 percent, indicating poor to excellent quality rock. The rock quality generally improved with increasing depth. As evident in the table, varying bedrock depths should be expected across the site.

4.2.8 Groundwater Conditions

The groundwater conditions were observed in the open boreholes and test pits during and immediately following the drilling / excavating operations and are shown on the borehole and test pit records. Unstabilized water levels ranging from at ground surface to 5.1 m below ground surface (Elevations 328.0 m to 328.5 m) were measured in Boreholes 18-104, 1 and 2, although these levels may be influenced by addition of water for drilling/coring.

The groundwater depth and elevation measured in the piezometer installed in Borehole 2, and sealed within the bedrock, during the previous investigation is summarized in the table below.

Borehole/ Well ID	Depth (m)	Elevation (m)	Date
BH 2	0.0	328.5	August 21, 2014

At the time of the current investigation, the water level in the creek in August 2018 was measured to be at approximately Elevation 327.3 m, at the inlet (north) side. A creek water level of approximately 326.0 m was measured at the outlet (south) side by LVM-Merlex during the previous investigation in August 2014. It should be noted the unstabilized groundwater levels appear to generally coincide with the creek level.

It should be noted that groundwater levels are subject to seasonal fluctuations and precipitation events and are expected to be higher during wet seasons and sustained periods of precipitation.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Mo’oud Nasr, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. William (Bill) Cavers, P.Eng., Golder’s MTO Designated Tunneling Contact for this, conducted an independent quality control review and audit of the report.

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https://golderassociates.sharepoint.com/sites/21813g/deliverables/rpt-001 culvert 20+385/final/1791525-001-r-rev0-culvert 20+385-september 2018.docx

PART A

Foundation Design Report
Proposed Culvert Replacement
Station 20+385, Highway 60
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6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

This section of the report provides engineering recommendations for the proposed trenchless installation of a new concrete pipe culvert undercrossing Highway 60, located at approximate Station 20+385, as part of the Design Build Contract DB-2017-5005. The following details summarize our current understanding of the proposed works:

- Trenchless construction methods will be used to install the new pipe along an alignment east of and parallel to the existing culvert. This method of installation will minimize traffic disruptions along Highway 60;
- The pipe to be installed is a Class 140-D reinforced concrete jacking pipe, having an inside diameter of approximately 1500 mm and length of 42.5 m; and,
- The invert levels (i.e. inside of the concrete pipe) of the new pipe will be at about elevation 327 m and 326 m, at the inlet and outlet, respectively.

This work is being carried out as part of a design-build assignment, with the team consisting of:

- R.W. Tomlinson Ltd. – General/Prime contractor
- McIntosh Perry Consulting Engineers – Designer
- Marathon Drilling Co. Ltd. – Trenchless contractor

We note that the discussion and recommendations presented in this report are based on our understanding of the project, consultations with the design-build team, and our interpretation of the factual data obtained from the current and previous subsurface investigations at the site.

6.2 Anticipated Ground Behaviour

The anticipated soil conditions within the proposed tunnel horizon consist of gravelly sand, silty sand and sand and silt embankment fill material as well as native stiff to very stiff clayey silt to silty clay and compact silty sand. Cobbles and boulders were encountered within the fill and native soils. Wood was also encountered within the fill at one location.

The stabilized groundwater level as measured in Borehole 2 is likely not representative of the groundwater conditions along the length of the bore. It is expected that the groundwater level in the overburden deposits and embankment fill material will likely be coincident with the creek water levels. This conclusion appears consistent with the unstabilized open borehole groundwater level observations made in the boreholes, at the time of the previous and current investigations.

The Terzaghi 'Tunnelman's Ground Classification System', as reported in Heuer (1974)[3], may be used to classify the anticipated strata that the tunnel would extend through. The System is commonly used in trenchless applications to describe the potential behaviour of an unsupported tunnel face during excavation by using qualitative stand-up time criteria. Stand-up time is defined as the time before the exposed soils will ravel, run or flow. Based on this system, the ground is classified into six principal categories: firm, slow raveling, fast raveling, squeezing, running, flowing and swelling. The ground behaviour above and below the groundwater table is also differentiated in this method.

[3] Heuer, R. 1974. *Important Ground Parameters in Soft Found Tunnelling*. Subsurface Exploration for Underground Excavation in Heavy Construction, New England College, Henniker, NH. New York, ASCE, pp.41-55

The Tunnelman's Ground Classification System has been adopted to provide an objective evaluation of the anticipated ground behaviour at the project site and is described below:

- **Native clayey silt to silty clay**: The native cohesive soils may be classified as 'firm' ground.
- **Native silty sand and non-cohesive embankment fill above the groundwater level**: These materials may exhibit some apparent cohesion due to the weak suction forces which can exist between the soil particles in a moist/unsaturated state (particularly for siltier zones with relatively high 'fines' content and for zones immediately above the groundwater level). The embankment fill would in this case have some limited stand-up time and may be classified as 'slow raveling'. Where more gravelly or sandy zones are encountered, the fill material would behave as 'running' ground.
- **Native silty sand and non-cohesive embankment fill below the groundwater level**: When fully saturated, this material may be classified as 'flowing' ground, potentially having very little stand-up time. This would particularly be the case in areas where sandy/gravelly zones are encountered.

Based on the above assessment, tunneling through mixed-face conditions of firm, slow/fast raveling and running/flowing ground should be expected, depending on the actual water table at the time of boring. It is likely that the groundwater level is coincident with the creek level, likely to be in the lower third of the tunnel horizon. The presence of obstructions (i.e., cobbles and boulders or wood) should also be considered by the contractor, as discussed further in Section 6.11.

In addition, it should be recognized that within embankment fill materials, there can be considerable variation in the fines content, leading to significant variation in the anticipated 'stand-up' time. For this reason, it is considered appropriate to assume that all fill material below the groundwater level, may potentially behave as flowing ground.

6.3 Installation Method and Feasibility

It is understood that the design-build team's preferred method of culvert installation is pipe jacking, using a non-slurry Tunnel Boring Machine (TBM). Installation with this method involves excavating the soil using a rotating cutter head, working within a shield, with the tunnel muck conveyed to the launching pit using a system of auger, conveyor, or buggies.

As a general criterion for tunneling to be feasible, and to reduce the risk of subsidence or heave, tunneling installations require minimum levels of overburden cover. As the overburden cover decreases, the risk of concentrated subsidence or heave increases, as does the risk of extreme events such as sinkholes at the ground surface. In Ontario the general practice is to maintain a depth of cover equivalent to 2 to 3 tunnel diameters. For this project, there is sufficient cover along the proposed culvert alignment for pipe jacking to be feasible from a geometric/cover perspective. An exception exists in the immediate area of the inlet and outlet; however, the cover becomes less critical at these locations. The contractor should be prepared to place temporary additional material over the ditch areas nearest the entry/exit pits to temporarily provide additional cover.

An advantage of pipe jacking with a non-slurry TBM is that it provides for the installation of the rigid and impervious ground support immediately behind the tunneling shield. It also allows direct access to the ground being tunneled, allowing for inspection and manual removal of obstructions, if encountered. However, a particular drawback in tunneling with this method is the risk of flowing ground conditions occurring, if groundwater is present above the tunnel invert level, thereby creating an unsupported tunnel face. Therefore, maintaining face stability is key in controlling ground movements for this method.

Based on the existing cover and in view of the anticipated ground conditions at the site, it is considered that pipe jacking, using a non-slurry TBM is likely to be feasible for the proposed trenchless installation. This assumes that the face stability and tunnel support recommendations provided in this report (Sections 6.4 and 6.5) are fully implemented during the tunneling operations. The settlement instrumentation and monitoring program outlined in Section 6.8 must also be implemented, as per MTO requirements.

We note that the successful completion of any trenchless technology or tunneling project largely depends on the skills and experience of the contractor. The final selection of the trenchless undercrossing technique should be made by the contractor based on their experience and equipment capabilities and the contractor's assessment of the subsurface conditions, provided that all technology-specific undercrossing permit requirements are satisfied.

6.4 Face Stability

The native clayey silt to silty clay soils, if excavated during the tunneling operation are expected to behave as fast raveling to firm ground. However, the non-cohesive native deposits and embankment fill soils may behave as slow/fast raveling ground (above the water table) or flowing ground (below the water table), if exposed. These flowing soils are expected to be encountered across the lower portion of the tunneling face, over the middle section of the tunnel length under the highway. If these soils are encountered above the water table, they are expected to be stable due to dilation in response to stress relief upon initial exposure. However, if left unsupported, they will behave as slow raveling ground.

To maintain face stability and minimize ground movements, the groundwater should be lowered to below the tunnel invert level along the full alignment so that the soils exposed on the working face will have a longer stand-up time. The suitable season for tunneling for this project is considered to be winter, when drier months and a lower groundwater table may be anticipated. Further, creek water should be diverted away from the proposed bore location.

It is also strongly recommended that boring operations continue non-stop once started (24 hours per day, 7 days per week). If it is necessary to stop tunneling operations for any reason, the face should be completely supported by a bulkhead which could consist of breasting boards. Such face support should be pre-cut and assembled prior to the start of tunneling so that it can be readily installed, if required. Further, filter fabric, straw and other packing materials should be available on site to contain any localized occurrences of flowing ground.

6.5 Tunnel Support

The proposed concrete pipe must be designed to accommodate the hoop stress from the weight of the overlying soils plus the jacking forces, which develop as a result of friction along the pipe. The following parameters may be used in assessing the hoop stresses acting upon the pipe:

Soil Type	Unit Weight (kN/m^3)	Drained Shear Strength Parameters	
		Effective Cohesion (kPa)	Effective Friction Angle (degrees)
Non-cohesive Embankment Fill	21	0	32

Since the culvert pipe will be jacked directly into place, it is recommended that the stresses on the pipe, and the adequacy of the proposed pipe to support the vertical/hoop load, be evaluated using the methods described in the Ontario Concrete Pipe Association's 'Concrete Pipe Design Manual' (and the associated software), using a 'bedding factor' appropriate to jacked pipe (for a non-grouted condition).

The jacking forces required to advance the pipe are dependent upon a number of factors directly related to construction equipment and methodology. This includes the following:

- The size of the over-cut between the shield and pipe;
- The alignment maintained during jacking;
- The rate of boring achieved;
- The use of lubricants and the timing of lubricant injection; and,
- The frequency and duration of stoppages during tunneling.

For these reasons and considering the natural variability of the ground, it is not possible to predict actual jacking forces prior to construction. However, assuming that lubrication will be injected through the pipe throughout the jacking operation and that the boring will be carried out on a continuous basis, it is expected that the unit jacking resistance on the surface area of the pipe will be in the range of 5 kPa to 15 kPa. Resistance at the shield face is expected to be within the range of 500 kN to 1,500 kN, assuming that the shield steel is less than 25 mm thick. Higher shield face resistance will be met where boulders are encountered.

Given the uncertainties in predicting jacking forces and the limitations on jacking forces imposed by the pipe strength, the following general recommendations are made:

- The pipe supplier should confirm that the proposed jacking pipe will not be overstressed by the anticipated jacking forces;
- Lubrication should be provided by means of bentonite injection at the shield and along the pipe itself, to reduce the frictional forces to acceptable levels;
- If the jacking is stopped, the frictional resistance to advancement of the pipe could increase significantly. To prevent this, the tunneling and pipe jacking should be a continuous operation. The length of any required stoppages, such as for equipment maintenance and/or removal of obstructions, should be minimized;
- A provision should be made for the installation of intermediate jacking stations along the pipe length; and,
- Jacking forces should be monitored throughout the tunneling operation, in relation to the length of pipe driven. This is to ensure that allowable pipe stresses are not exceeded and to determine if jacking from the intermediate jacking stations is necessary. The operators should be provided with a chart/guideline of the corresponding anticipated jacking force for various lengths of jacked pipe.

6.6 Grouting

The need for grouting around the pipe should be evaluated once jacking is complete. The amount of spoil removed during tunneling should be monitored during jacking to determine whether there is over-excavation occurring. If there is suspicion that over-excavation has occurred, and/or if the settlement monitoring indicates that the ground has settled, then a plan should be in-place for investigating for the presence of gaps/voids around the pipe and for filling them with grout.

6.7 Ground Settlements

The effects of stress relief at the tunnel face and partial closure of the over-cut between shield and pipe may result in settlement of the ground and overlying highway. Given the amount of cover between the crown of the tunnel and the highway, it is anticipated that ground surface settlement can be maintained at 15 mm or less, provided good tunneling procedures are followed, as outlined below:

- The measures to control face stability as described in Section 6.4 are implemented;
- The over-cut between the tunneling shield and the pipe is 12.5 mm (i.e. ½ inch) or less;
- Suitable lubricant is applied directly behind the shield to minimize friction between the pipe and the ground; and,
- The gap created between the soil and the pipe is grouted with cement grout at the completion of jacking. To achieve appropriate grouting, it is recommended that the spacing between grout ports around the circumference of the pipe, be limited to 2 m.

As per the MTO's requirements of this project, it is essential that a settlement monitoring program as described below be implemented during tunneling.

6.8 Instrumentation and Monitoring

As required by Section 7.06 of the 'Pipe Installation by Trenchless Method' NSSP for this project, and in accordance with MTO's "*Guidelines for Foundation Engineering – Tunneling Specialty for Corridor Encroachment Permit Application*", a settlement monitoring program will need to be implemented. This settlement monitoring program will serve to:

- Document the effects of tunneling on the overlying highway;
- Obtain prior warning of ground movements that could occur due to the construction methods and equipment or unforeseen ground conditions;
- Verify the Contractors compliance with the settlement limits imposed in the Contract; and,
- Allow adjustments to be made to the tunneling method such that the settlement limits established are not exceeded, recognizing however, that there is typically some delay between the trenchless construction and the full manifestation of the ground surface settlements.

In brief, the settlement monitoring plan is to include the installation of surface settlement points and in-ground monitoring points. The surface settlement points shall be installed at the pavement/ground surface along the full length of the tunnel alignment as well as shoulders and side slopes. The settlement points shall be installed directly above the centreline at a maximum spacing of 5 m.

The in-ground monitoring points should consist of three sets of sleeved iron bar, with three monitors in each set, encased in a 50-70 mm PVC pipe. The in-ground monitoring points should be set on the highway shoulders, perpendicular to the alignment at a depth of 2.1 m below ground surface. A stick up casing should be used and adequately protected from traffic. A sufficiently high marker should be placed adjacent to the casing if the bore takes place during winter conditions.

Monitoring of settlement instruments on this project is constrained by the potentially high traffic volume and the potentially limited periods during which access to the highway can be obtained. Typically on 2-lane highways, flagging could be used to read the monitoring points with short one-lane closures for this purpose. However, if it is desired to minimize traffic disruptions completely, a non-intrusive system for the surface monitoring points could be considered, such as by the use of a reflectorless total station, however an elevated platform is required to “see” the points, which may be difficult at this high fill embankment location. In both cases, winter road conditions, if applicable, may impact the points themselves and every effort should be made to ensure the points are installed to avoid being damaged by snow clearing equipment and to be made “accessible” for surveying (i.e. clear of ice). The equipment and procedures used to carry out the settlement monitoring must be capable of surveying the settlement point elevations to within ± 1 mm of the actual elevation.

The survey frequency of the monitoring program is to be as follows:

- Baseline readings: Three consecutive readings at least one week prior to tunnel construction.
- Construction readings:
  - Three sets of readings per day, provided settlements are below ‘review’ level.
  - Double that frequency if ‘review’ or ‘alert’ levels are reached.
- Post-construction readings: Weekly for a period of one month.

Additional readings may be required if excessive settlements are measured.

6.9 Shaft Excavations

The shafts for the entrance and exit pits will be constructed in the low-lying areas adjacent to the inlet and outlet of the new culvert alignment. The shaft excavation depths are anticipated to be shallow extending to approximately 0.5 m below the culvert invert depth. The founding soil at this depth will generally consist of compact silty sand and/or dense to very dense sand, below the groundwater level.

A concrete mud slab (working slab) or granular bedding will be required at the base of the shaft excavations to maintain the integrity of the base during construction by providing a trafficable excavation floor. This is provided that adequate groundwater controls are applied, as discussed in Section 6.10.

Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects. According to OHSA, the soil classification and corresponding excavation side slopes for the soils to be removed are provided below. Care must also be taken during excavation to ensure that adequate support is provided for any existing structures, roadways and underground services located adjacent to the excavations.

Soil Description	Above/Below Groundwater	OHSA Soil Type	Maximum Shaft Excavation Side Slope
Compact Silty Sand	Above	Type 2	1 Horizontal :1 Vertical
	Below	Type 3	1 Horizontal :1 Vertical
Stiff to Very Stiff Clayey Silt	Above	Type 2	1 Horizontal :1 Vertical
	Below	Type 3	1 Horizontal :1 Vertical

To maintain temporary excavation stability, excavated materials should be placed away from the edge of the excavation a distance equal to the depth of the excavation or greater. In addition, stockpiling of the material should be prohibited adjacent to the excavation to minimize surcharge loading near the excavation crest. Where sufficient space is not available to stockpile the excavated material at the site, off-site disposal of the excavated material intended for reuse would need to be arranged.

If the open cut excavation side slopes noted in the table above cannot be accommodated (due to the proximity of the embankment and active highway lanes), then a temporary protection system (i.e., temporary excavation shoring) will be required. The shoring system could consist of braced soldier pile and lagging, braced sheet piles or potentially a slide rail system, designed by a Professional Engineer and including an assessment of the potential for basal heave. If shoring is implemented at the site, the requirements of OPSS.PROV 539 should be followed. Design of temporary works will be entirely the responsibility of the contractor.

It is noted that some difficulty may be expected in excavating the pits and/or installation the temporary shoring system due to the likely presence of cobbles and boulders in the site area.

6.10 Groundwater Control

Based on an assessment of the groundwater conditions encountered at the entry and exit pits, the groundwater is generally interpreted to be near the creek water level or essentially at the ground surface.

Based on a shaft excavation depth of approximately 0.5 m below the culvert invert depth, the excavations are anticipated to extend to up to 0.8 m below the groundwater level at the inlet of the culvert and to less than 0.5 m below the groundwater level at the outlet.

As such, it is anticipated that control of seepage for the shaft construction at the culvert outlet can be achieved by conventional pumping from sumps in oversize excavations, if the excavation base is within 0.6 m of the prevailing groundwater level at the time of construction. This dewatering can be achieved by gravity drainage and pumping from strategically placed and properly filtered sumps with side ditches.

The excavation at the inlet may be entirely within the clayey soils at the inlet, based on the investigation results, but the depth of clayey soil above the permeable sand deposit is expected to be very limited and the excavations may extend into the sand deposit. The limited depth of clay and the higher groundwater level is likely to lead to basal heave and failure of the subgrade within the excavation. The design of the dewatering works should therefore consider that the excavation may need to extend to the surface of the sand deposit or into the sand deposit. Dewatering from sumps, within the excavation or prior to excavation, may not be feasible, depending on the groundwater level at the time of construction, and it may be necessary to implement additional dewatering measures such as passive dewatering wells or active dewatering.

The selection of a suitable dewatering system should also take into account water levels and creek flow conditions at the time of construction, as dewatering requirements will be impacted by water levels in the creek. The creek water should be diverted away from the bore, presumably within the existing culvert. The method used should not undermine any existing road embankment or adjacent side slopes. The provision of toe protection at side slopes during drawdown may be required to minimize sloughing and undercutting during dewatering.
All dewatering operations should be carried out in accordance to OPSS.PROV 517, OPSS 518 and the OSHA. Water should not be discharged within 30 m of surface water or watercourses. Care should be taken at all times to ensure trenching operations adhere to OHSA requirements.

The actual rate of groundwater inflow to the excavations will depend on many factors including the contractor's schedule and rate of excavation, the size of the excavation, and the time of year at which the excavation is made. Also, there may be instances where significant volumes of precipitation, surface runoff and/or groundwater may collect in an open excavation and must be pumped out. Surface water runoff should be directed away from open excavations.

The combined rate of pumping from the entrance and exist pits may exceed 50,000 L (50 m^3) per day. An Environment Activity Section Registry (EASR) registration is required for dewatering between 50,000 L/day and 400,000 L/day, and a Permit-to-Take-Water (PTTW) is required for dewatering greater than 400,000 L/day from the Ministry of Environment. A registration or application should be made as soon as possible to avoid construction delays. The design of the dewatering system for the excavations is the responsibility of the Contractor who is expected to retain dewatering specialists for this task, including permit application and supporting documentation.

6.11 Obstructions

Obstructions (i.e. cobbles and boulders) were encountered within the tunnel horizon in both the previous and current boreholes / test pits as well as exposed on the embankment side slopes. Instances of wood were also encountered within the tunnel horizon. Therefore it is anticipated that obstructions will be encountered during construction works for the tunnel and entry/exit pits. These potential obstructions may impact tunneling operations resulting in significant difficulties in advancing the bore, as well as excavations, elements of temporary protection systems and/or dewatering systems. The contractor must be prepared with suitable equipment to remove/penetrate through any obstructions that may be encountered during construction.

6.12 Erosion Protection

Since the new concrete pipe culvert will be jacked directly into the embankment, there will be no new granular bedding or cover material which might be vulnerable to forming a preferential seepage conduit along/around the culvert. There is therefore no particular requirement to provide a cut-off (such as a clay seal) at the inlet.

The need for (and design of) erosion protection at the culvert inlet and outlet (including the slopes and sides of the channel) will depend on the anticipated hydrologic/hydraulic conditions. Typically, rip-rap protection should be provided over these areas. The rip-rap layer should cover all surfaces on the embankment slopes against which creek water is likely to be in contact. If needed, rip-rap treatment should be consistent with the Ontario Provincial Standard Drawing OPSD 810.010.

6.13 Other Considerations

All trenchless works should be carried out by an experienced specialist contractor employing only qualified workers skilled in their trade under the direction of an experienced foreman. Prior to construction, the contractor should be required to submit their proposed construction method and monitoring program for review and approval from the relevant stakeholders. Their work plan should identify the risks and methods of control for possible problems that could cause interference to the county roads or any waterways, such as heaving/settlement, changes of alignment, frac-out, voids, etc. It is recommended that the geotechnical aspects of the contractor's work plan for the undercrossing be reviewed by a geotechnical engineer prior to construction.

Before excavation begins at the proposed undercrossing locations, it is recommended that hand digging or hydro-vacuum methods be used to expose any underground utilities in the vicinity of the proposed undercrossing, if present, to determine the exact locations and depths. The hydro-vacuum holes must be properly backfilled to prevent preferential pathways for fluid migration.

7.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Mo'oud Nasr, P.Eng., and the technical aspects were reviewed by Ms. Sarah E. M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. William (Bill) Cavers, P.Eng., Golder's MTO Designated Tunneling Contact for this project, conducted an independent quality control review and audit of the report.

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https://golderassociates.sharepoint.com/sites/21813g/deliverables/rpt-001 culvert 20+385/final/1791525-001-r-rev0-culvert 20+385-september 2018.docx

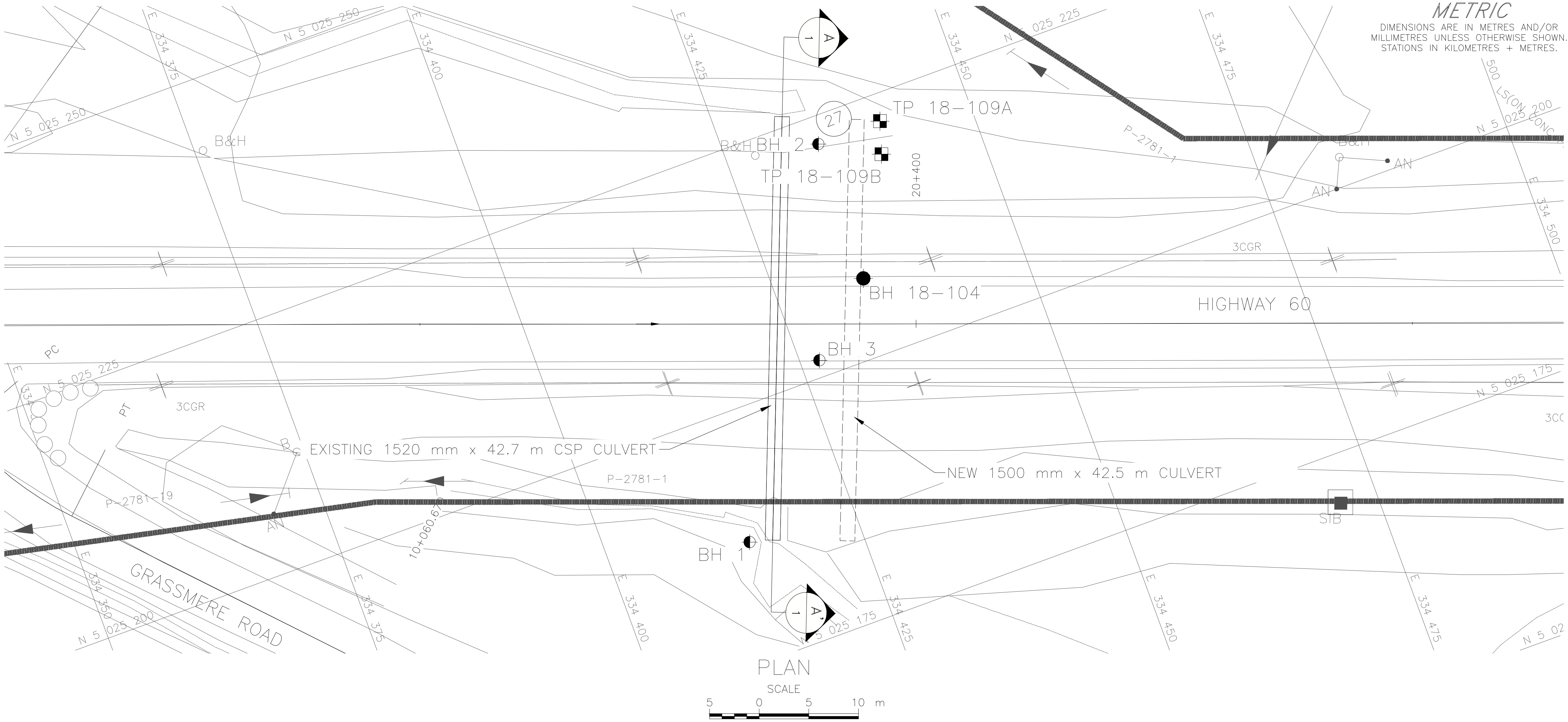
CONT No.
GWP No. 5333-11-00

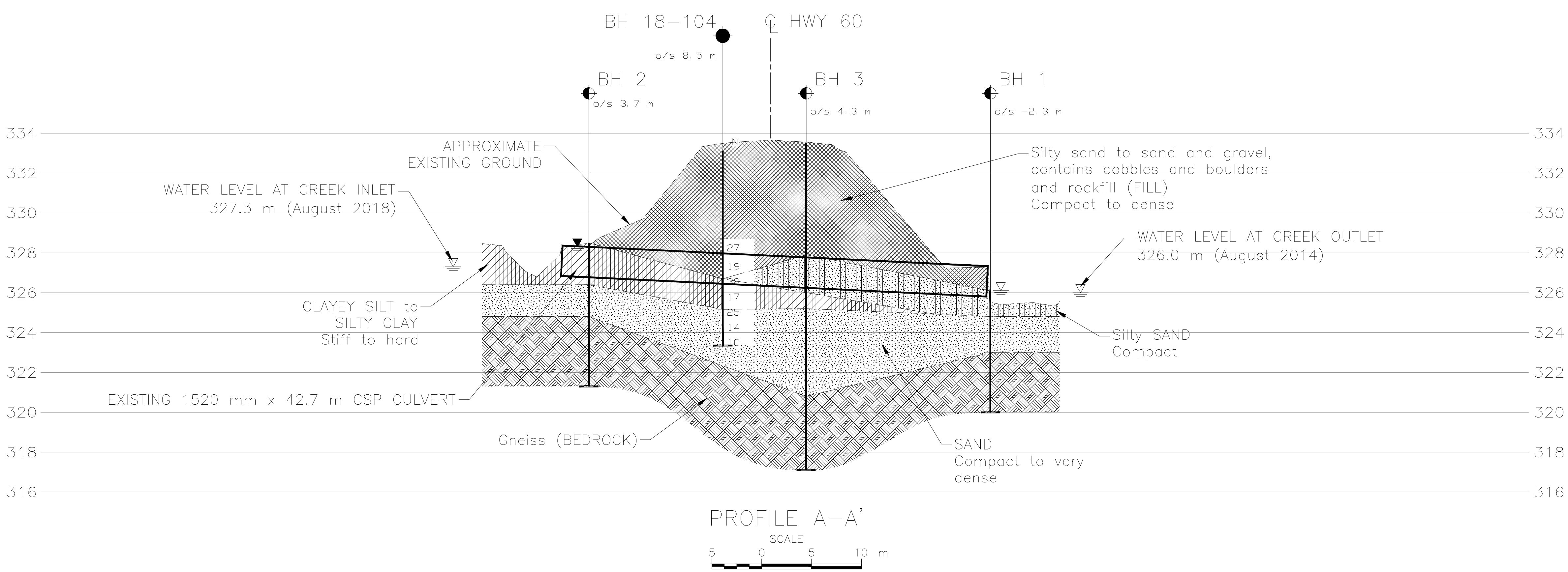
SHEET

HIGHWAY 60 CULVERT
AT STATION 20+385
LAT. 45.366173; LONG. -79.121783
BOREHOLE LOCATIONS AND SOIL STRATA

GOLDER

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





KEY PLAN

SITE
HWY 60
HUNTSVILLE

SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Test Pit – Current Investigation
- Borehole – Previous Investigation (Geocres No. 31E-345)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- Stabilized WL in monitoring well
- WL upon completion of drilling

BOREHOLE/TEST PIT CO-ORDINATES (MTM ZONE 10)

No.	ELEVATION	NORTHING	EASTING
BH 18-104	333.1	5025207.9	334433.9
TP 18-109A	327.4	5025222.3	334440.9
TP 18-109B	328.2	5025219.1	334439.9
BH 1	326.1	5025186.9	334414.0
BH 2	328.5	5025222.2	334434.3
BH 3	333.5	5025201.7	334426.9

NOTES

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

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

REFERENCE

Base plans provided in digital format by McIntosh Perry, drawing file nos. PKP 17-7137-00A_020NEW.dwg and X-KP-17-7137 HWY 60_BASE.dwg, received JUL. 23, 2018.

NO.	DATE	BY	REVISION

Geocres No. 31E-387

HWY. 60	PROJECT NO. 1791525		DIST. NORTHEASTERN
SUBM'D. CK	CHKD. CK	DATE: 8/29/2018	SITE:
DRAWN: JM	CHKD. SEMP	APPD. FJH	DWG. 1

APPENDIX A

List of Abbreviation and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Borehole and Test Pit Sheets - Current Investigation

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

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

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

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

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan\delta$
c'	effective cohesion
c_u, s_u	undrained shear strength (ϕ = 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

Notes:
1. $\tau = c' + \sigma' \tan\phi'$
2. shear strength = (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, 60º 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^2 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

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

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

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

V. MINOR SOIL CONSTITUENTS

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

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^{o} 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		

GOLDER

Foundation Design

RECORD OF BOREHOLE No 18-104

SHEET 1 OF 2

METRIC

PROJECT 1791525
G.W.P. GWP 5333-11-00
DIST Northeastern HWY 60
DATUM Geodetic

LOCATION N 5025207.9; E 334433.9 MTM ZONE 10 (LAT. 45.366198; LONG. -79.121707)
BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Wash Boring/HQ Core
DATE August 22, 2018

ORIGINATED BY CRG
COMPILED BY JM
CHECKED BY CK

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	UNIT WEIGHT kN/m³	GR	SA	SI	CL
333.1	GROUND SURFACE										
0.0	(SM/SP) Silty sand to sand, some gravel to gravelly, contains cobbles and boulders (Fill) Brown Moist - Boulder encountered and cored between 1.4 and 1.8 m - Boulder encountered and cored between 2.4 and 2.9 m - Boulder encountered and cored between 3.8 and 4.4 m										
328.7 4.4	(SM/SP) Silty sand to sand, some gravel to gravelly, contains cobbles and boulders (FILL) Compact Brown Moist	1	SS	27	▽ (approx. 328.0)						
327.9 5.2	(ML/SM) Sand and silt, some gravel, trace to some clay, contains wood pieces and organic matter (FILL) Compact Dark brown to grey Moist	2	SS	19				15	42	32	11
326.7 6.4	(CL/CI) CLAYEY SILT to SILTY CLAY, contains sand seams and rock fragments Very stiff to hard Grey Moist to wet	3	SS	38							
		4	SS	17							
325.2 7.9	(SP) SAND, trace to some silt Compact Grey Wet	5	SS	25							
		6	SS	14							
		7	SS	10							
323.4 9.8											

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%): 10 20 30
ELEVATION SCALE: 333, 332, 331, 330, 329, 328, 327, 326, 325, 324

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

GOLDER

Foundation Design

PROJECT 1791525

RECORD OF BOREHOLE No 18-104

SHEET 2 OF 2

METRIC

G.W.P. GWP 5333-11-00

LOCATION N 5025207.9; E 334433.9 MTM ZONE 10 (LAT. 45.366198; LONG. -79.121707)

ORIGINATED BY CRG

DIST Northeastern HWY 60

BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Wash Boring/HQ Core

COMPILED BY JM

DATUM Geodetic

DATE August 22, 2018

CHECKED BY CK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	END OF BOREHOLE NOTES: 1. Soil description above 4.4 m inferred from soil cuttings and resistance from casing advancement. 2. Water level in open borehole at a depth of 5.1 m below ground surface (Elev. 328.0 m). measured on August 22, 2018, but may be influenced from added water.										

$+^3, \times^3$: Numbers refer to Sensitivity

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTO\HWY60\HUNTSVILLE\02_DATA\GINT\1791525.GPJ GAL-GTA.GDT 9/6/18 JM

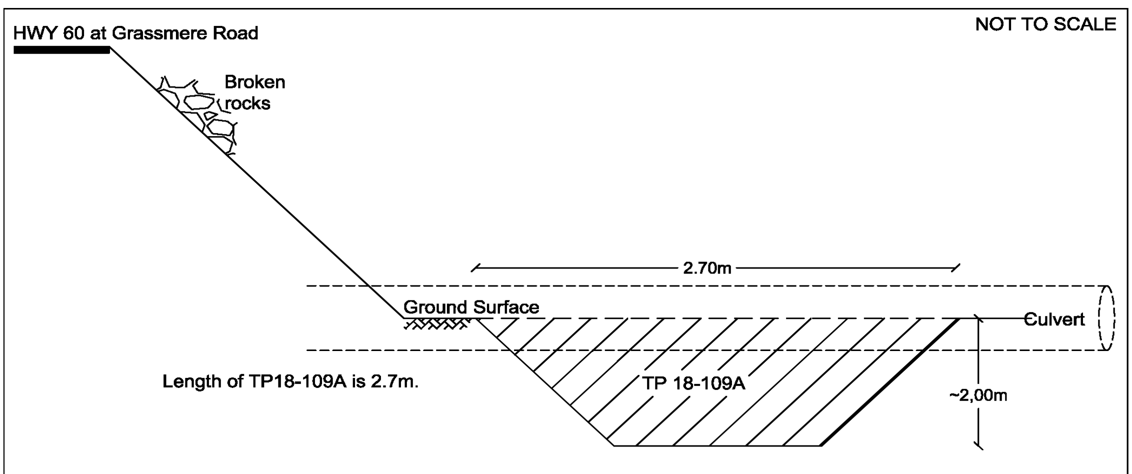
RECORD OF TEST PIT

GOLDER

1931 Robertson Rd,
Nepean, ON K2H 5B7
Téléphone: (613) 592-9600

Project No:	1791525	**Test Pit No:**	TP18-109A
Project name:	MTO HWY 60 Design-Build	**Sub-contractor:**	R. W. Tomlinson Ltd.
Northing/Easting:	5025222.3 / 334440.9 (MTM Zone 10)	**Equipment**	Kubota KX080 Excavator
Latitude/Longitude:	45.3663, -79.1216	**Weather:** **Temperature:**	Sunny ~20°C
Field Technicien/Engineer:	Raj Goyal	**Reviewed by:**	F. Ghobrial
Date:	14/08/2018	**Ground Surface Elevation:**	327.423 m (Geodetic)

Depth (metre)	Elevation (metre)	Visual Description (SYMBOL) GROUP NAME, description, other constituants, coulour, contamination, behaviour, moisture condition	Remarks
0.00	327.42	(SM/ML) SILTY SAND to sandy SILT, contains rootlets (TOPSOIL) brown, wet	* Cobbles and boulders encountered at depth of about 1.8-2.0 m. * Some cobbles ~13cm in size. * Average size of boulders ~33 cm. * Test pit dry after ~20 minutes of digging. * ~5-8 cm of water at bottom of excavation after 30 minutes of digging.
0.15	327.27	(CL/CI) Silty CLAY, fissured Grey Moist	
0.50	326.92		
1.00	326.42	*SAMPLE 1 from 1.0 m to 1.5 m*	
1.50	325.92		
1.75	325.67	(ML/SM) Sandy SILT to SILTY SAND, contains rootlets Brown Wet *SAMPLE 2 from 1.8 m to 2.0 m*	
2.00	325.42	End of Excavation	



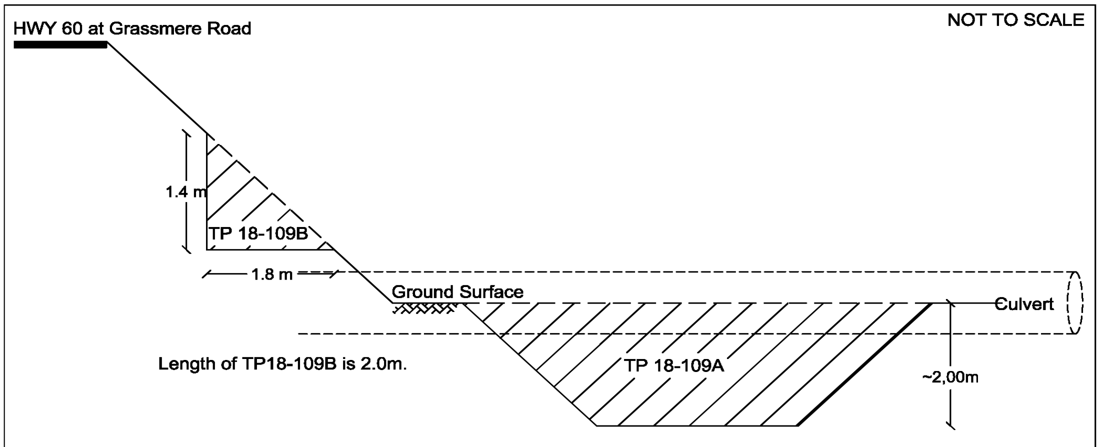
Cross-section of the Test Pit

RECORD OF TEST PIT

GOLDER
1931 Robertson Rd,
Nepean, ON K2H 5B7
Téléphone: (613) 592-9600

Project No:	1791525	**Test Pit No:**	TP18-109B
Project name:	MTO HWY 60 Design-Build	**Sub-contractor:**	R. W. Tomlinson Ltd.
Northing/Easting:	5025219.1 / 334439.9 (MTM Zone 10)	**Equipment**	Kubota KX080 Excavator
Latitude/Longitude:	45.3663, -79.1216	**Weather:** **Temperature:**	Sunny ~20°C
Field Technicien/Engineer:	R. Goyal	**Reviewed by:**	F. Ghobrial
Date:	14/08/2018	**Ground Surface Elevation:**	328.23 m (Geodetic)

Depth (metre)	**Elevation** (metre)	**Visual Description** (SYMBOL) GROUP NAME, description, other constituants, coulour, contamination, behaviour, moisture condition	**Remarks**
0.00	328.23	(SM) SILTY SAND, contains rootlets (TOPSOIL) Dark brown Moist	* Scattered rockfill of cobbles and boulders encouneterd at the surface. * Size of boulders ~40-50 cm. * Some cobbles ~10-15 cm in size. * Test pit dry after 20 minutes of digging.
0.20	328.03	(SM) Silty SAND, contains cobbles, boulders and rootlets, trace of silty clay pockets (FILL) Brown to dark brown Moist	
0.50	327.73		
1.00	327.23	*SAMPLE 1 from 1.0 m to 1.4 m*	
1.40	326.83	End of Excavation	



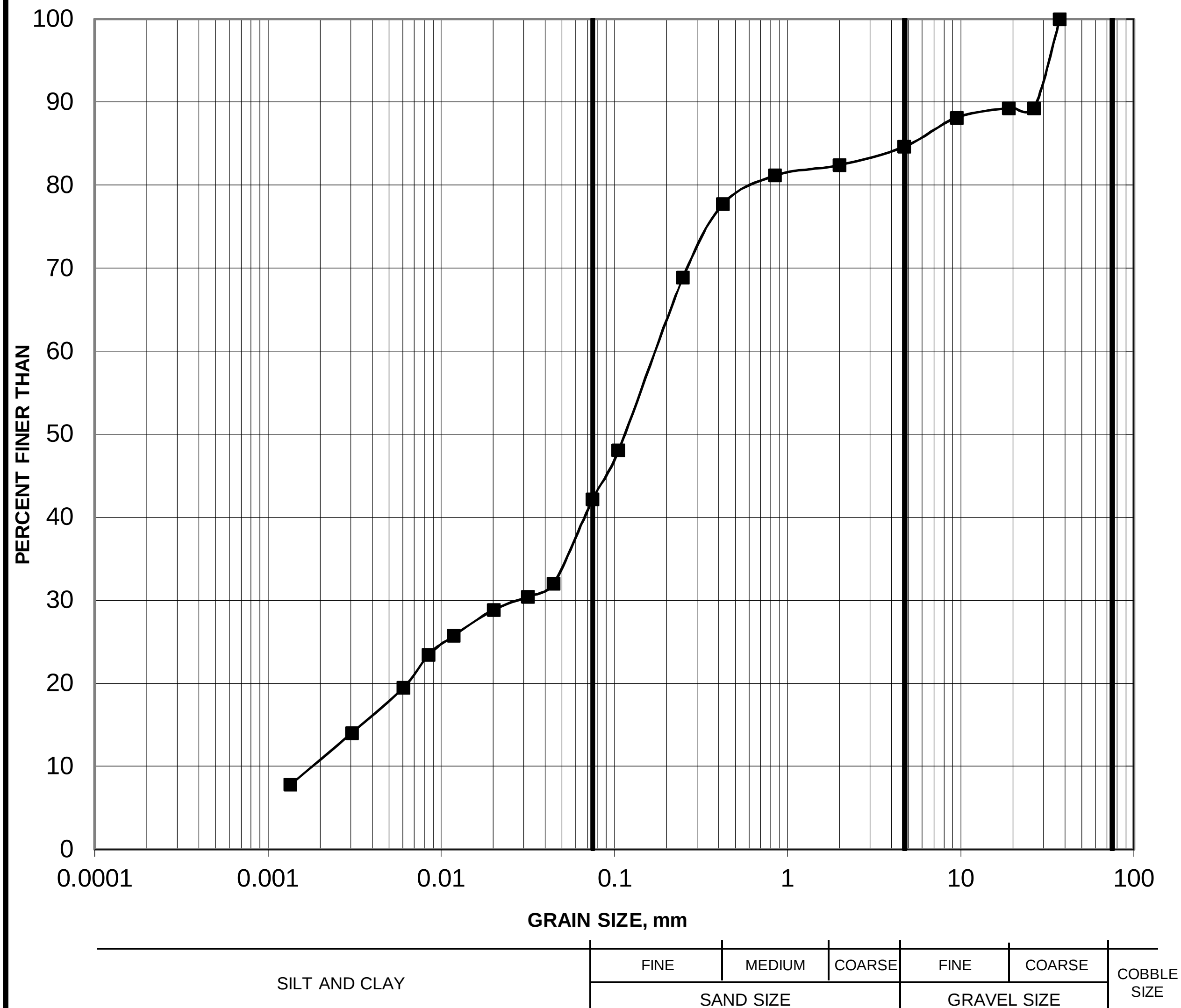
Cross-section of the Test Pit

APPENDIX B

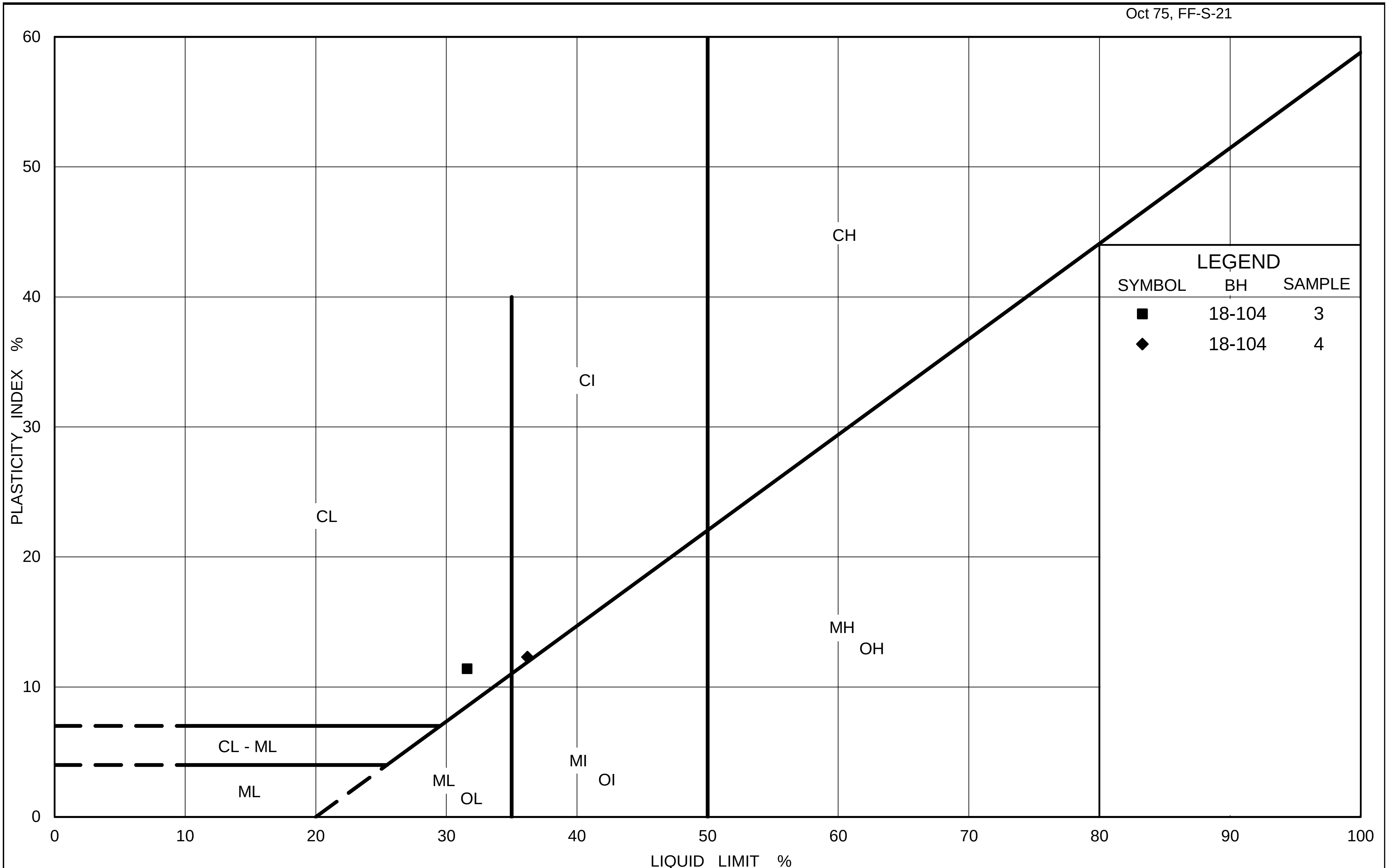
Laboratory Test Results - Current Investigation

GRAIN SIZE DISTRIBUTION	FIGURE B1

SAND AND SILT (FILL)



Borehole	Sample	Depth (m)
18-104	2	5.33-5.94



Ministry of Transportation

Ontario

PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY

FIG No. B2

Project No. 1791525

Compiled By : MI Checked By : CW

APPENDIX C

Record of Borehole Sheets

Previous Investigation by LVM (GEOCRES No. 31E-350)

ENCLOSURE NO.:2 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 1

LVM Merlex

REFERENCE 14/07/14083-F2 DATUM Geodetic LOCATION N 5025186.9 E 334414.0 - Chaffey Twp., Station 20+385 ORIGINATED BY JL

PROJECT GWP 5333-11-00, Highway 60 BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 12 August 2014 DATE (Completed) 13 August 2014 TIME (Completed) 2:30:00 PM CHECKED BY MAM

ELEV / DEPTH	SOIL PROFILE - DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
326.1	Ground Surface					
0.0	SILTY SAND some organic soils boulder size rocks encountered brown/black (compact)	1	SS	14	~32	
324.8 1.3	SAND - some silt with gravel grey	2	SS	60/127 mm	~15	
		3	SS	80/152 mm	~17	23 66 (11)
323.0 3.1	BEDROCK - grey/pink gneiss poor quality	4	SS	20/0 mm		
		5	RC	Rec= 93% RQD= 35%		
		6	RC	Rec= 100% RQD= 41%		
320.0 6.1	End of Borehole					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRATA PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (326, 325, 324, 323, 322, 321, 320); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS		
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 13/8/15 2:30:00 PM	0	-
2)	-	-
3)	-	-

MEL-GEO 14083 - BOREHOL LOGS - F2.GPJ MEL-GEO.GDT 6/5/15

LVM-Merlex, a Division of EnGlobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@lvm.ca

ENCLOSURE NO.:3 (Pg. 1 of 1)

METRIC

RECORD OF BOREHOLE NO. 2

LVM Merlex

REFERENCE 14/07/14083-F2	DATUM Geodetic	LOCATION N 5025222.2 E 334434.3 - Chaffey Twp., Station 20+392	ORIGINATED BY JL
PROJECT GWP 5333-11-00, Highway 60		BOREHOLE TYPE Track Mounted CME 45 - Hollow Stem Augers	COMPILED BY SH
CLIENT AECOM		DATE (Started) 20 August 2014 / DATE (Completed) 20 August 2014 / TIME (Completed) 4:30:00 PM	CHECKED BY MAM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
328.5	Ground Surface						
0.0	50 mm surficial organic soils	1	SS	10	328		
	CLAYEY SILT - trace sand grey (compact)	2	SS	21			0 3 79 18
		3	SS	20	327		
326.4 / 2.1	cobble size rock pieces encountered at depth of 1.8 m SAND - some silt with gravel grey (very dense)	4	SS	45	326		
		5	SS	97	325		
324.8 / 3.7	BEDROCK - grey gneiss fair to good quality	6	RC	Rec= 100% RQD= 67%			
		7	RC	Rec= 100% RQD= 77%	324 / 323		
		8	RC	Rec= 100% RQD= 89%	322		
321.3 / 7.2	End of Borehole						

Column headings: SOIL PROFILE; SAMPLES; STRATA PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

COMMENTS

The stratification lines represent approximate boundaries. The transition may be gradual.

$+^3, \times^3$: Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

WATER LEVEL RECORDS		
Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1) 20/8/15 4:30:00 PM	0.2	-
2) 21/8/15 11:00:00 AM	0	-
3)	-	-

MEL-GEO 14083 - BOREHOL LOGS - F2.GPJ MEL-GEO.GDT 6/5/15

ENCLOSURE NO.:4 (Pg. 1 of 2)

METRIC

RECORD OF BOREHOLE NO. 3

LVM Merlex

REFERENCE	14/07/14083-F2	DATUM	Geodetic	LOCATION	N 5025201.7 E 334426.9 - Chaffey Twp., Station 20+391.5
ORIGINATED BY	TB				
PROJECT	GWP 5333-11-00, Highway 60	BOREHOLE TYPE	Truck Mounted CME 75 - Hollow Stem Augers	COMPILED BY	SH
CLIENT	AECOM	DATE (Started)	19 August 2014	DATE (Completed)	19 August 2014
TIME (Completed)	5:00:00 PM	CHECKED BY	MAM		

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT (approx., %)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
333.5	Ground Surface					
0.0	280 mm Asphalt					
333.2	120 mm Crushed Gravel					
0.3	FILL - gravelly sand asphalt encountered in gravelly sand layer	1	SS	22		
	FILL - sand and gravel to gravelly sand trace silt mixed with rock fill brown, (compact/dense)	2	SS	38		41 52 (8)
		3	SS	12		
		4	SS	23		
		5	SS	0/50 mm		
		6	SS	0/50 mm		
		7	SS	13		18 53 (29)
327.9 5.6	SILTY SAND - with gravel trace clay some organic soils (compact)	8	SS	104/330 mm		
		9	SS	14		23 44 31 2
326.0 7.5	SILTY CLAY (very stiff)	10	SS	25		
325.2 8.3	SAND - with silt with gravel cobble size rock pieces encountered grey (compact/very dense)	11	SS	27		
		12	SS	19		
		13	SS	60		19 51 (30)
		14	SS	100/51 mm		
		15	SS	100/13 mm		
320.8 12.7	BEDROCK - grey gneiss	16	RC	Rec(BDR)= 100% RQD(BDR)=		

Continued Next Page

Column headings: SOIL PROFILE; STRATA PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (333 to 321); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

COMMENTS

+3, ×3 : Numbers on right refer to Sensitivity. Numbers on left refer to values greater than 120 kPa

○ 3% STRAIN AT FAILURE

The stratification lines represent approximate boundaries. The transition may be gradual.

WATER LEVEL RECORDS

Date (dd/mm/yy)/Time	Water Depth (m)	Cave In (m)
1)	-	-
2)	-	-
3)	-	-

MEL-GEO 14083 - BOREHOL LOGS - F2.GPJ MEL-GEO.GDT 6/5/15

LVM-Merlex, a Division of EnGlobe Corp.
120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@lvm.ca

METRIC

RECORD OF BOREHOLE NO. 3

LVM Merlex

REFERENCE	14/07/14083-F2	DATUM	Geodetic	LOCATION	N 5025201.7 E 334426.9 - Chaffey Twp., Station 20+391.5
ORIGINATED BY	TB	PROJECT	GWP 5333-11-00, Highway 60	BOREHOLE TYPE	Truck Mounted CME 75 - Hollow Stem Augers
COMPILED BY	SH	CLIENT	AECOM	DATE (Started)	19 August 2014
DATE (Completed)	19 August 2014	TIME (Completed)	5:00:00 PM	CHECKED BY	MAM

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRATA PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA (SI CL)
	Continued from Previous Page										
	good to excellent quality				83%						
	End of Borehole						320				
			17	RC	Rec= 100% RQD= 97%						
							319				
			18	RC	Rec= 100% RQD= 93%		318				
317.1 16.4											

MEL-GEO 14083 - BOREHOL LOGS - F2.GPJ MEL-GEO.GDT 6/5/15

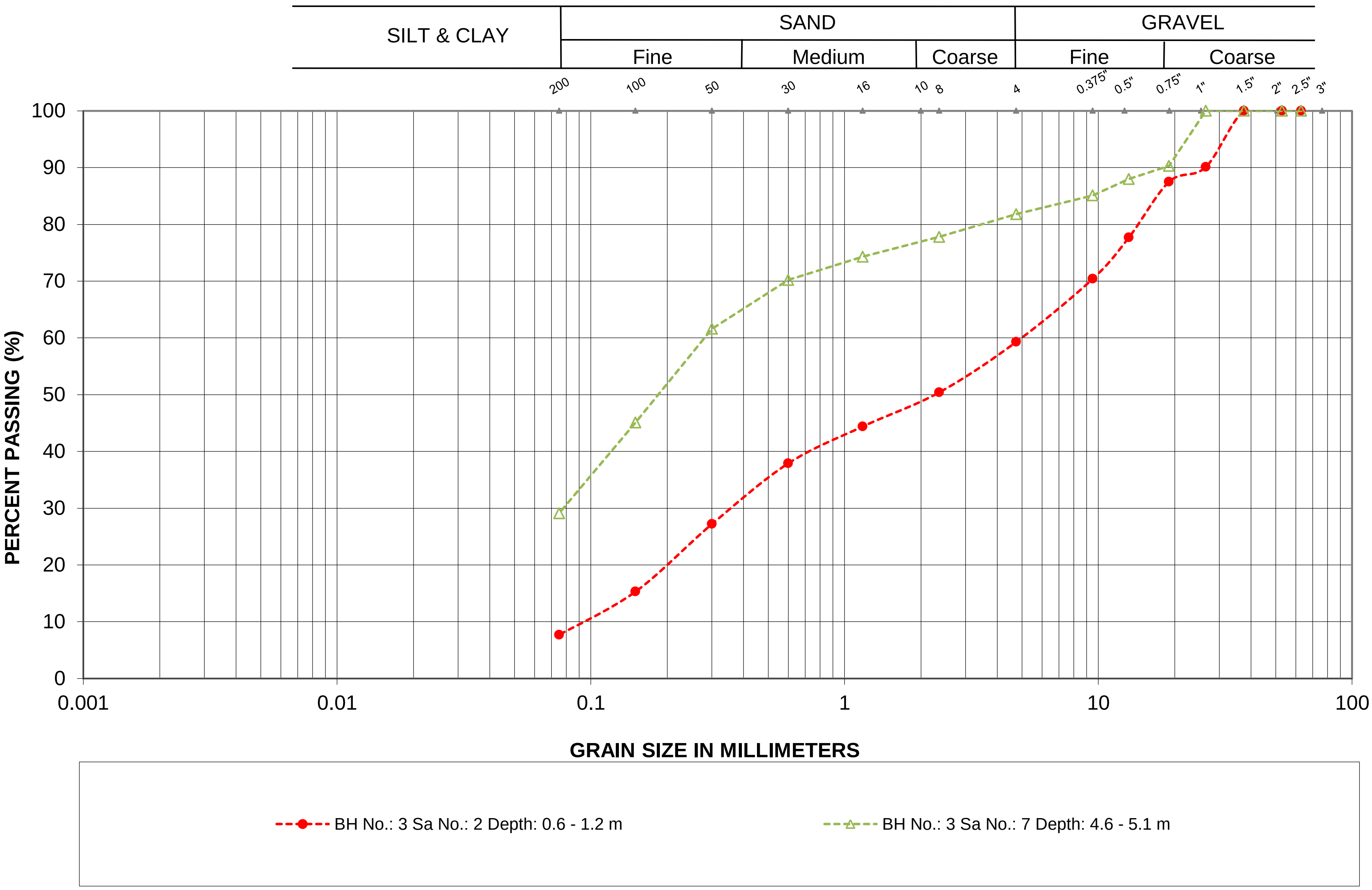
APPENDIX D

Laboratory Test Results

Previous Investigation by LVM (GEOCRES No. 31E-350)

Reference No.: 14/07/14083-F2
Date: May 2015

GRAIN SIZE ANALYSIS



LOCATION: Hwy 60 CSP, Station 20+385
Chaffey TWP, Ontario

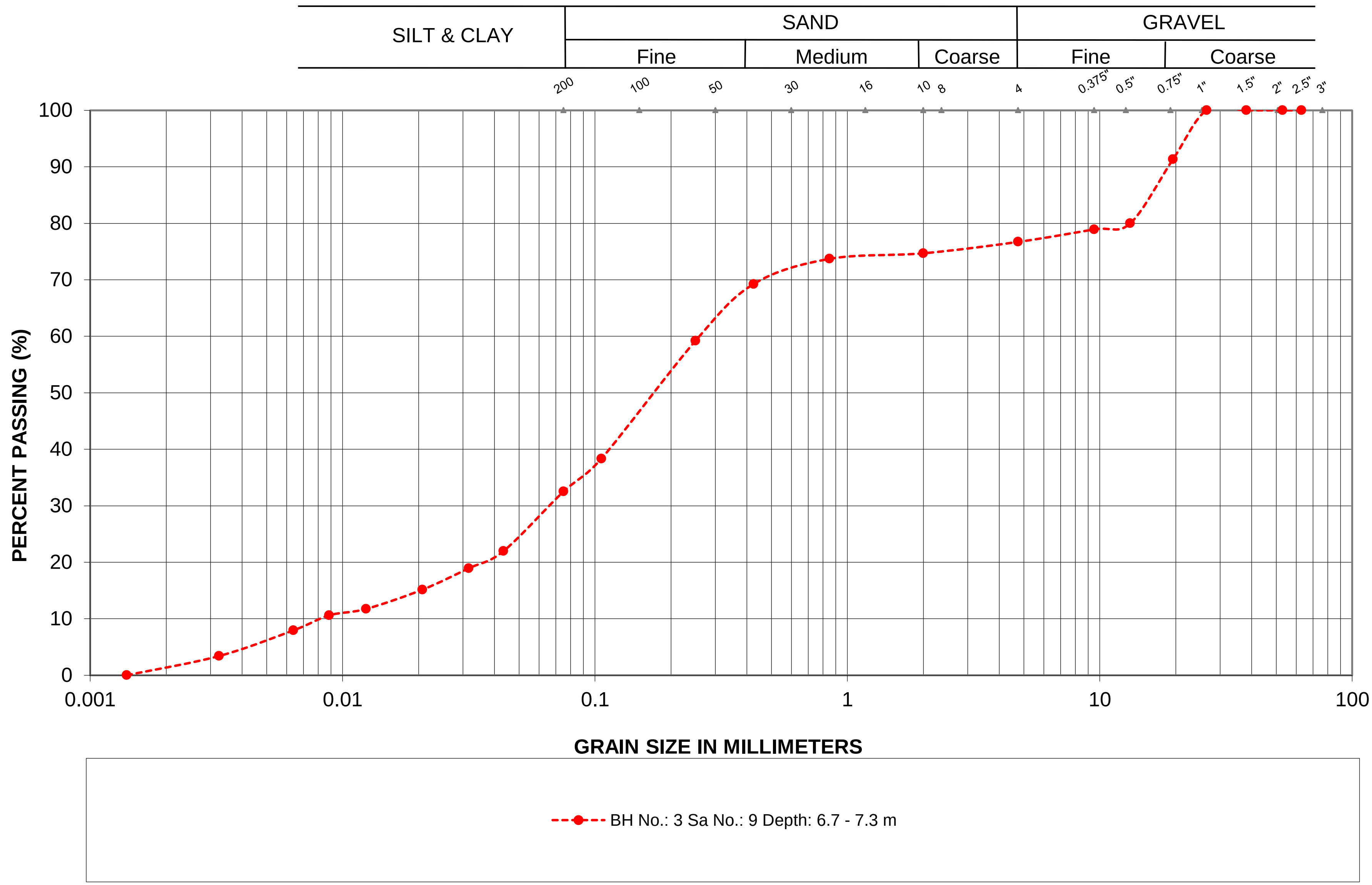
FILL

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-1

Reference No.: 14/07/14083-F2
Date: May 2015

GRAIN SIZE ANALYSIS



LOCATION: Hwy 60 CSP, Station 20+385
Chaffey TWP, Ontario

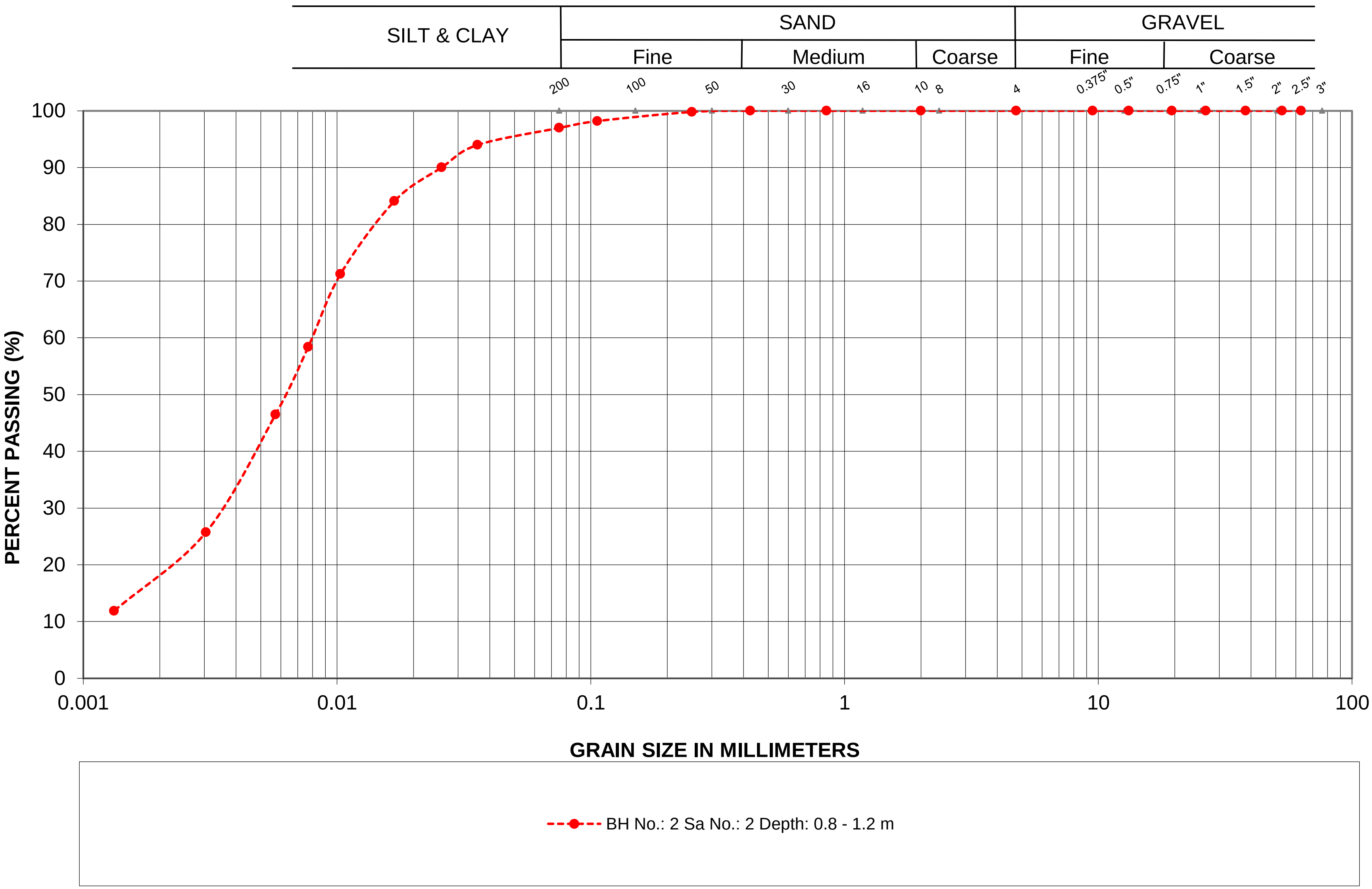
SILTY SAND

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-2

Reference No.: 14/07/14083-F2
Date: May 2015

GRAIN SIZE ANALYSIS



LOCATION: Hwy 60 CSP, Station 20+385
Chaffey TWP, Ontario

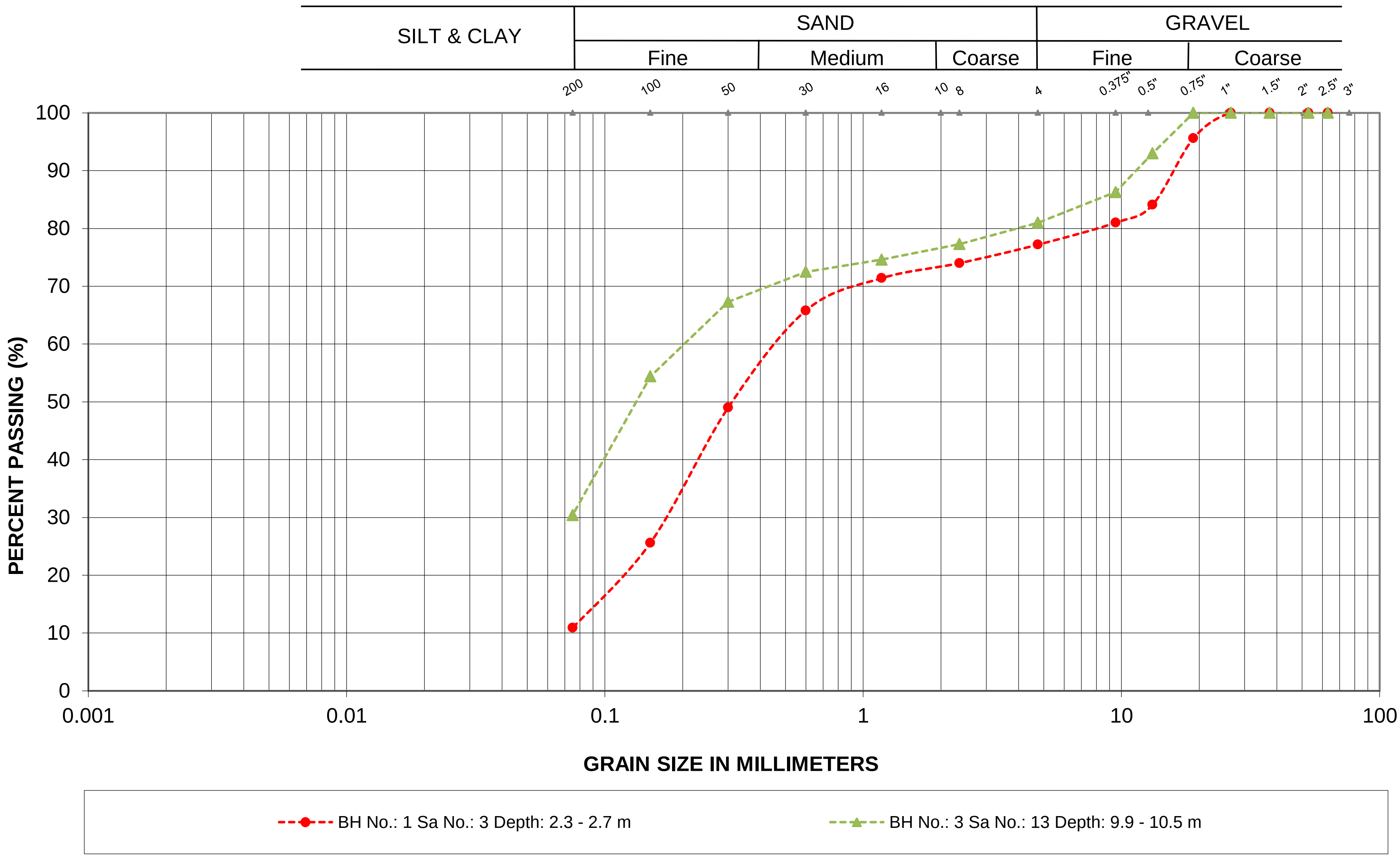
CLAYEY SILT

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-3

Reference No.: 14/07/14083-F2
Date: May 2015

GRAIN SIZE ANALYSIS



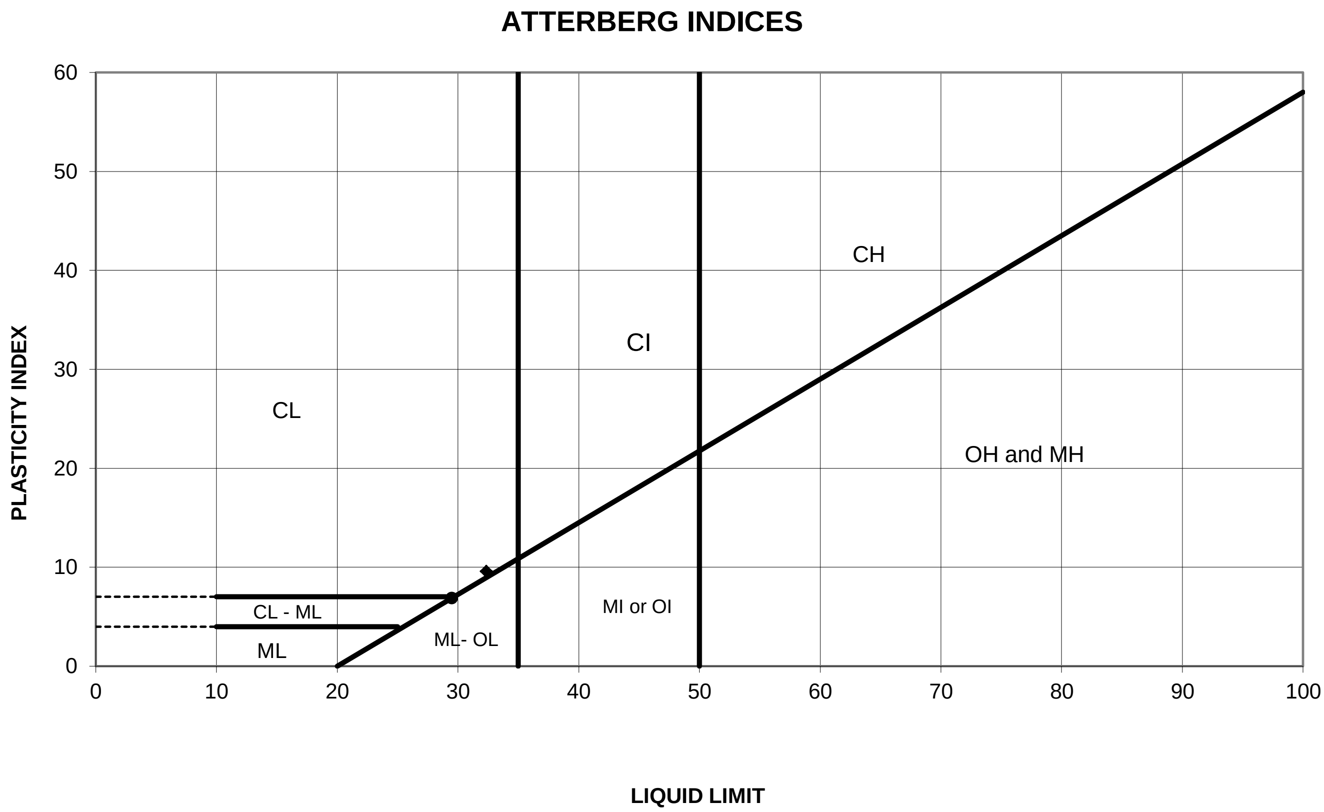
LOCATION: Hwy 60 CSP, Station 20+385
Chaffey TWP, Ontario

SAND

LVM-Merlex, a Division EnGlobe Corp.

FIGURE L-4

ATTERBERG LIMITS CHART	FIGURE L-5



SYMBOL	BH	Sa. No.	Depth(m)	Elev.(m)	Liquid Limit	Plastic Limit	Plasticity Index	NMC %
●	2	2	0.8	327.7	29.5	22.7	6.9	27.1
◆	3	10	7.6	325.9	32.4	22.8	9.6	29.4

Date: May-15
Project: Hwy 60,
Location: CSP Station 20+385, Chaffey TWP

Prep'd: AT
Chkd: MAM
Ref. No.: 14/07/14083-F2

LVM-Merlex, a Division EnGlobe Corp.

Laboratory Tests - Summary Sheet

LVM Merlex

Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.6					32.4				14			
	2	1.5					15.6				60/127 mm			
	3	2.3	23	66	11		17.4				80/152 mm			
	4	3.1									20/0 mm			
	5	3.1												Rec=93%, RQD=35%
	6	4.6												Rec= 100%, RQD=41%
2	1	0.0					28.0				10			
	2	0.8	0	3	79	18	27.1	29.5	22.7	6.9	21			
	3	1.5					27.3				20			
	4	2.3					12.5				45			
	5	3.1					13.5				97			
	6	3.7												Rec=100%, RQD=67%
	7	4.1												Rec=100%, RQD=77%
	8	5.6												Rec=100%, RQD=89%
3	1	0.2					4.8				22			
	2	0.8	41	52	8		3.7				38			
	3	1.5					2.4				12			
	4	2.3					3.8				23			
	5	3.1									0/50 mm			
	6	3.8									0/50 mm			
	7	4.6	18	53	29		10.7				13			
	8	6.1					28.4				104/330 mm			
	9	6.7	23	44	31	2	27.5				14			
	10	7.6					29.4	32.4	22.8	9.6	25			
	11	8.4					22.8				27			

Laboratory Tests - Summary Sheet

LVM Merlex

Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
3	12	9.1					24.1				19			
	13	9.9	19	51	30		12.7				60			
	14	10.67									100/51 mm			
	15	12.19					1.1				100/13 mm			
	16	12.36												Rec(BDR)=100%, RQD=87%
	17	13.36												Rec=100%, RQD=97%
	18	14.86												Rec=100%, RQD=93%

APPENDIX E

Test Pit Photographs

Photograph 1: Looking west (towards HWY 60 embankment) at TP 18-109A

Photograph 2: Boulder encountered while excavating TP 18-109A

Photograph 3: Looking west (towards HWY 60 embankment) at TP 18-109B

Photograph 4: Boulder encountered while excavating TP 18-109B

GOLDER

golder.com