April 2014

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

Cameron Lake Bridge Replacement
Structure Site 32-064
Highway 35, Kawartha Lakes
G.W.P. 4045-10-00

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REPORT

Golder Associates

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Golder Associates

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of Cameron Lake bridge (MTO Structure Site No. 32-064) on Highway 35 in the City of Kawartha Lakes, Ontario.

This report addresses the results of the subsurface investigation carried out at the Cameron Lake bridge site. The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Proposal (RFP) for Assignment No. 2008-E-0018 dated February 2012, and in Section 5.8 of the *Technical Proposal* for this assignment.

2.0 SITE DESCRIPTION

The existing Cameron Lake bridge is located on Highway 35 approximately 2.5 km north of Glenarm Road, west of Cameron Lake and Fenelon Falls, in the City of Kawartha Lakes, Ontario. The existing bridge was constructed in 1949 and consists of a single-span structure, approximately 13.5 m long and 11.0 m wide. No design or as-built structural drawings are available, and the foundation type and founding elevation for the existing abutments are not known.

In general, the terrain in the area is relatively flat, with the natural ground surface in the immediate vicinity of the structure site at about Elevation 255.5 m to 256 m. Highway 35 has been constructed on an embankment that is about 2 m to 2.5 m in height immediately adjacent to the existing bridge, with the pavement surface at about Elevation 257.2 m to 257.5 m. The highway embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V). Based on visual observation at the time of the site investigation, the existing embankment slopes appear to be performing satisfactorily.

Cameron Lake (McFarland Bay) is located on the east side of Highway 35 at the structure site; Perrin Creek flows into Cameron Lake from the west. The water level in Cameron Lake is controlled, and the typical operating water level is at approximately Elevation 255 m. The water depth in Perrin Creek under the Highway 35 bridge is typically about 1.4 m to 1.6 m, with the creek channel base at approximately Elevation 253.4 m to 253.6 m. Marsh vegetation was observed at some locations along the edges of Perrin Creek and Cameron Lake/McFarland Bay during site reconnaissance.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in December 2012, at which time two boreholes (Boreholes CL-1 and CL-2) were advanced using a truck-mounted CME-55 drill rig, supplied and operated by Strong Soil Search Inc. of Claremont, Ontario. The borehole locations are shown on Drawing 1. Boreholes CL-1 and CL-2 were advanced from the Highway 35 pavement, north and south of the existing structure respectively.

The boreholes were drilled to total depths of 11.9 m and 12.0 m, using 108 mm inside diameter hollow stem augers through the soil, and triple-tube, NQ-size coring equipment in the bedrock. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth in the boreholes, using a 50 mm outside diameter split-spoon sampler driven with an automatic hammer in accordance with the Standard Penetration Test (SPT) procedure.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. The details of the water level readings are indicated on the borehole records contained in Appendix A. A standpipe piezometer was not installed at this site as the boreholes were drilled through the driving lanes on Highway 35 due to access and space restrictions. However, a falling head permeability test was conducted in both open boreholes prior to abandonment. The boreholes were backfilled with soil cuttings and bentonite pellets upon completion, in accordance with Ontario Regulation 903 (as amended).

The field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples; point load tests and uniaxial compressive strength tests were conducted on selected samples of the bedrock. The geotechnical laboratory testing was completed according to applicable MTO LS procedures.

The borehole locations were measured on-site relative to the existing structure and site features, and the ground surface elevations were obtained from the digital terrain model (DTM) for the site, provided by URS. The borehole locations (referenced to the MTM NAD83 coordinate system) and ground surface elevations (referenced to geodetic datum) are summarized in the following table and are shown on Drawing 1.

Borehole No.	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
CL-1	4,933,756.2	361,420.8	257.2	11.9
CL-2	4,933,732.7	361,435.8	257.4	12.0

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

This area of Highway 35 lies within the Peterborough Drumlin Field physiographic region, as delineated in *The Physiography of Southern Ontario* (Chapman and Putnam, 1984), which extends from Lake Simcoe eastward to the Belleville area. The site is just south of the physiographic region known as the Carden Plain – a limestone plain with limited or no overburden soils.

The surficial soils in the Peterborough Drumlin Field generally consist of drumlinized clayey or sandy till. Localized deposits of silt, clay and peat can be found in the low-lying areas between drumlins. In the Cameron Lake area, a number of spillway and esker features are present, and sand and gravel deposits are present in these features. The overburden soils are underlain by limestone bedrock of the Shadow Lake Formation (Ontario Geological Society, 1991).

4.2 Subsurface Conditions

As part of the current geotechnical/foundations investigation, two boreholes were advanced at the Cameron Lake bridge site. The borehole locations, ground surface elevations and interpreted stratigraphic conditions are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records contained in Appendix A. The results of geotechnical laboratory testing are also presented on Figures B1 and B2 contained in Appendix B. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic section on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions encountered at the site consist of loose to dense silty sand to sand and gravel embankment fill, overlying a deposit of loose to very dense sand and gravel to silty sand and gravel, which is underlain by limestone bedrock at a depth of approximately 8.5 m to 8.7 m below the existing Highway 35 grade. A more detailed description of the soil deposits encountered in the boreholes is provided in the following sections.

4.2.1 Silty Sand to Sand and Gravel Fill

Approximately 0.3 m of asphalt was encountered in the boreholes, overlying 3.4 m to 3.6 m of embankment fill. The base of the fill was encountered at approximately Elevation 253.3 m and 253.7 m in Boreholes CL-1 and CL-2, respectively.

The fill consists of sand and gravel containing trace to some silt, to sand containing trace to some silt, to silty sand. Cobbles were observed within the fill in both boreholes. The results of grain size distribution tests completed on two selected samples of the fill are shown on Figure B1 contained in Appendix B.

The fill has a variable, loose to dense relative density, based on measured Standard Penetration Test (SPT) "N" values that range from 5 blows to 42 blows per 0.3 m of penetration.

4.2.2 Sand and Gravel to Silty Sand and Gravel

An approximately 4.8 m thick deposit of sand and gravel was encountered below the fill in both boreholes. Its surface was encountered at approximately Elevation 253.3 m and 253.7 m in Boreholes CL-1 and CL-2, and its base was encountered at approximately 248.5 m and 248.9 m, respectively.

This deposit consists of sand and gravel containing some silt, to silty sand and gravel. The results of grain size distribution tests completed on two selected samples of the deposit are shown on Figure B2 in Appendix B.

The measured SPT "N" values within the sand and gravel to silty sand and gravel range from 9 blows to 78 blows per 0.3 m of penetration, but are typically between 9 blows and 39 blows per 0.3 m of penetration, indicating a loose to dense relative density. Lower SPT "N" values of 2 blows and 4 blows per 0.3 m of penetration were measured in Borehole CL-2, as shown on the borehole record and Drawing 1; however, these values are considered to have been affected by sample disturbance due to groundwater inflow to the borehole during sampling.

4.2.3 Limestone Bedrock

Grey limestone bedrock was encountered below the sand and gravel deposit at approximately Elevation 248.5 m and 248.9 m in Boreholes CL-1 and CL-2 on the north and south sides of Perrin Creek, respectively. This corresponds to depths of about 8.7 m and 8.5 m below the Highway 35 grade, respectively.

The bedrock consists of fresh, thinly bedded, grey, medium strong to very strong limestone of the Shadow Lake Formation. The Rock Quality Designation (RQD) measured on the recovered bedrock core samples is between 45 and 78 per cent, indicating a rock mass of poor to good quality. The Total Core Recovery (TCR) is between about 93 and 100 per cent, and the Solid Core Recovery (SCR) is between about 73 and 97 per cent.

Point load tests (with axial orientation) were carried out on four core samples of the limestone bedrock. The results are shown on the drillhole records in Appendix A and are summarized in the following table:

Borehole No.	Sample Depth (m)	Approximate Sample Elevation (m)	Axial Is (50 mm) (MPa)	Approximate Unconfined Compressive Strength (MPa)
CL-1	9.9	247.3	7.121	150
CL-1	10.7	246.5	6.087	128
CL-2	9.9	247.5	5.542	116
CL-2	11.4	246.0	6.352	133

The estimated uniaxial compressive strength (UCS) value for each sample tested for point load strength is based on a relationship between Is_{50} and UCS which is given by a correlation factor (C) in accordance with ASTM D5731-08 *(Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classification),* which may vary depending on the size of the core sample and the strength of the rock. A laboratory uniaixial compressive strength test was carried out on two samples of the limestone bedrock, in accordance with ASTM D7012-10 (Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens), and measured uniaxial compressive strengths of approximately 38 MPa and 74 MPa.

Based on visual observation and the point load and uniaxial compressive strength test results, and in accordance with Table 3.5 from CFEM (2006), the limestone bedrock within the depth of exploration is classified as medium strong to very strong (R3 to R5, 25 MPa < UCS < 250 MPa).

4.3 Groundwater Conditions

The water level was measured in the open boreholes during and on completion of drilling. No piezometers were installed in the boreholes drilled in the Highway 35 lanes because of safety and access considerations. The measurements are summarized on the borehole records contained in Appendix A and in the following table:

Borehole No.	Ground Surface Elevation (m)	Depth to Water Level (m)	Water Elevation (m)	Date
CL-1	257.2	3.5	253.7	Dec 6, 2012
CL-2	257.4	2.4	255.0	Dec 3, 2012

The water level in Cameron Lake is controlled, with the typical operating water level at approximately Elevation 255.0 m. It is anticipated that the stabilized groundwater level at the bridge site is at or slightly above Elevation 255.0 m. However, the groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

A falling head permeability test was conducted in both open boreholes prior to abandonment. Based on the test results, the hydraulic conductivity of the site soils has been assessed to be on the order of $1x10^{-2}$ cm/s to $1x10^{-3}$ cm/s.

5.0 CLOSURE

This Preliminary Foundation Investigation Report was prepared by Mr. Billy Murphy, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
CAMERON LAKE BRIDGE REPLACEMENT
STRUCTURE SITE 32-064
HIGHWAY 35, KAWARTHA LAKES
G.W.P. 4045-10-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the existing Cameron Lake bridge on Highway 35. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this preliminary subsurface investigation. The discussion and preliminary recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the foundations for the replacement structure. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

As part of the planning study completed by URS, a number of alignment options were considered for the replacement of the existing Cameron Lake bridge, including permanent realignment of Highway 35 to the west or east of the existing alignment, and temporary detours along either side of the existing highway. However, a longer structure span would be required for such a realignment, due to the increased width of Perrin Creek west of the existing structure, and the presence of Cameron Lake east of the existing structure. Therefore, at the 30% design stage, the following options have been considered for replacement of the existing bridge:

- Replacement of the bridge on the existing Highway 35 alignment, with staged demolition and reconstruction of one-half the existing structure at a time, with a minor vertical profile adjustment.
- Replacement of the bridge on the existing Highway 35 alignment, with staged demolition and reconstruction of one-half the existing structure at a time, with no vertical profile adjustment.

For both alternatives, URS has considered various structure types, with a single-span length of approximately 18 m to avoid interference with the existing structure foundations and to maintain excavation and construction work outside of the creek. Depending on the structure type adopted, a grade raise of up to approximately 250 mm will be required relative to the existing Highway 62 embankment, which has a maximum height of approximately 2.7 m relative to the existing floodplain grade.

6.2 Foundation Options

The existing Cameron Lake bridge is a single-span structure that was constructed in 1949. No design or as-built structural drawings are available, and the foundation type and founding elevation for the existing abutments are not known. According to the ETR plates and contour plans provided by URS, the natural ground surface near the edge of the Perrin Creek channel/Cameron Lake is at approximately Elevation 255 m to 255.5 m, and the typical operating water level in Cameron Lake is at approximately Elevation 255 m.

Based on the subsurface conditions, both shallow and deep foundation options have been considered for the replacement of the existing Cameron Lake bridge. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, risks and approximate costs is provided in Table 1 following the text of this report.

- **Strip footings founded on the loose to dense sand and gravel to silty sand and gravel deposit:** Strip footings could be considered for support of a replacement structure. However, based on the borehole investigation, the founding soils include loose material. Relatively lower geotechnical resistances will apply for this deposit, with potential for approximately 25 mm of settlement of the abutment footings, and such resistances may not be sufficient for design of bridge foundations under the current design codes. The soils are water-bearing, and the use of cofferdams with active dewatering would be required to control the ground and groundwater during excavation and construction. Temporary protection systems would also be required during excavation and construction to facilitate the staged reconstruction of the bridge.

- **Footings "perched" on a compacted granular pad in the Highway 35 approach embankments:** This option would be advantageous in minimizing the depth of excavation and associated groundwater control requirements. However, as the Highway 35 embankment is not very high, the geotechnical resistance will be largely controlled by the underlying loose to dense silty sand and gravel deposit. At this preliminary stage, lower geotechnical resistances are provided; these geotechnical resistances can be revisited at the detail design stage if this option is to be considered further.

- **Steel H-piles:** Steel H-piles driven to refusal on the limestone bedrock are feasible and suitable for support of the bridge replacement, and this option would allow the pile caps to be maintained higher than for a strip footing option, thus minimizing excavation depth, groundwater control requirements, and protection system requirements, while achieving relatively higher geotechnical resistances and minimizing settlement. Steel H-pile foundations would also allow for the construction of integral abutments; based on the depth to bedrock at this site, a minimum pile length of 5 m should be achievable, but depending on the elevation for the underside of the pile cap, rock socketting could be completed to achieve a minimum 5 m pile length. If the piles are driven, the use of driving shoes is recommended to minimize damage while driving through the sand and gravel deposit (which may contain cobbles and boulders) and onto the limestone bedrock. If rock sockets are required to achieve a 5 m pile length, coring or churn drilling would be necessary in the medium strong to very strong limestone bedrock.

- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutments, and this foundation option would have similar advantages to steel H-piles in terms of minimizing excavation depth, protection system requirements and groundwater control requirements. However, pipe piles are considered to have a slightly higher risk than H-piles for "hanging up" or being deflected away from their vertical or battered orientation due to the presence of cobbles and/or boulders within the lower sand and gravel deposit. This foundation type is not compatible with integral abutments, but could be used for a semi-integral abutment configuration.

- **Caissons:** Caissons founded on the limestone bedrock are feasible for this site, but would require the use of temporary or permanent liners to mitigate the potential risks of ground loss during construction through the water-bearing sand and gravel deposit. In addition, the bedrock surface at the site slopes or undulates, and it would likely be necessary to socket the caissons into the bedrock to "seat" the liner properly and avoid the potential for loss of ground at the interface between the soil and bedrock.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective is to support the abutments for the bridge replacement on driven steel H-pile foundations in an integral abutment configuration, or alternatively on spread footings "perched" as high as possible on a granular pad. For an integral abutment structure, the pile cap should be maintained as high as possible to minimize excavation depth and cofferdam/dewatering requirements, and to maximize the pile length. As noted above, depending on the underside elevation of the pile cap, it may be necessary to core or churn drill into the bedrock to achieve a minimum 5 m pile length if integral abutments are selected for the replacement bridge.

6.3 Shallow Foundations

6.3.1 Founding Elevations

Strip or spread footings should be founded below any existing fill and soft or disturbed soils, on the loose to dense sand and gravel to silty sand and gravel deposit. The following table provides the maximum (highest) founding elevations recommended for preliminary design, based on the depth of fill as encountered in Boreholes CL-1 and CL-2 (i.e., extending to approximately Elevation 253.3 m to and 253.7 m on the north and south sides of the bridge, respectively). Dewatering and temporary protection systems/cofferdams will be required to minimize disturbance of the subgrade soils and instability of the excavation side walls, as discussed further in Section 6.7.

Foundation Element	Borehole No.	Founding Stratum	Footing Founding Elevation
North abutment	CL-1	Loose to dense silty sand and gravel	Below 253.1 m
South abutment	CL-2	Loose to compact sand and gravel	Below 253.5 m

These recommended founding elevations are similar to or up to about 0.5 m below the existing creek channel, which has its base at approximately Elevation 253.4 m to 253.6 m at this structure site. Depending on the span length and separation distance between the footing and the creek channel, it may be necessary to construct new footings deeper to provide adequate protection against erosion and scour. In addition, the founding elevation for the new abutment footings should be checked to ensure they are a minimum of 1.6 m below the lowest surrounding grade to provide adequate protection against frost penetration, per Ontario Provincial Standard Drawing (OPSD) 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

It is understood that an 18 m long structure is under consideration to avoid conflicts with the existing abutments, the type and founding elevation for which are not known. The existing superstructure will have to be removed in order to construct the replacement structure on the existing Highway 35 alignment; however, it is recommended that consideration be given to leaving the existing foundations (footings or pile caps) in place in front of the new abutments. This would minimize excavation depth, cofferdam and groundwater control requirements immediately adjacent to the creek channel.

Further assessment of the impact of the existing foundations and their removal will be required at detail design, once the alignment and geometry of the replacement structure is confirmed relative to the existing bridge.

The footing subgrade should be inspected by a Quality Verification Engineer (QVE) following excavation, in accordance with OPSS 902 *(Excavating and Backfilling Structures)* to check that all existing fill, loose or surficial soils, or other unsuitable material have been removed. It is recommended that the founding soils be protected with a concrete working slab (100 mm thick concrete slab with a compressive strength of 20 MPa) if the concrete for the footing is not placed within four hours of the inspection and approval of the subgrade.

6.3.2 Geotechnical Resistance

Strip or spread footings placed on the properly prepared, loose to compact sand and gravel to silty sand and gravel deposit, at or below the design elevations given in the preceding section, should be designed based on the preliminary factored geotechnical resistances at Ultimate Limit States (ULS) and geotechnical resistances at Serviceability Limit States (SLS) given below. Due to the loose relative density of the founding soils, these geotechnical resistances may not be sufficient for design of the replacement bridge under the current code requirements.

Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
3 m	250 kPa	200 kPa
4 m	300 kPa	175 kPa

* For 25 mm of settlement

For spread footings perched on a minimum 2 m thick compacted granular pad, preliminary design can be completed based on a factored geotechnical resistance at ULS of 500 kPa, and a geotechnical resistance at SLS (for 25 mm of settlement) of 250 kPa.

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

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutment locations.

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

6.4.1 Founding Elevations

The abutments for the replacement bridge may be supported on steel H-piles or steel pipe (tube) piles driven to found on the limestone bedrock. Additional borehole investigation will be required at the detail design stage to confirm the bedrock surface variability within the footprint of the proposed north and south abutments. However, based on the borehole results from the preliminary investigation, and assuming about 0.3 m of penetration into the bedrock based on the ability to auger into the upper portion of the bedrock in Borehole CL-2, the following pile tip elevations are recommended for preliminary design:

Foundation Element	Borehole No.	Bedrock Surface Elevation	Design Pile Tip Elevation
North Abutment	CL-1	248.5 m	248.2 m
South Abutment	CL-2	248.9 m	248.6 m

The pile caps should be constructed at a minimum depth of 1.6 m for frost protection purposes, per OPSD 3090.101 *(Foundation Frost Penetration Depths for Southern Ontario).*

For the installation of steel H-piles or steel pipe piles, consideration must be given to the potential presence of cobbles and boulders within the soil deposits. In this regard, steel H-piles are preferred over steel pipe piles as pipe piles are considered to pose a higher risk of "hanging up" or being deflected away from their vertical or battered orientation during installation, due to their larger end area. The piles should be reinforced at the tip with driving shoes or flange plates to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 *(Deep Foundations).* In bouldery soils, as may be encountered at this site, driving shoes (such as Titus Standard "H" Bearing Pile Points) are preferred over flange plates. If steel pipe piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II *(Steel Tube Pile Driving Shoe).*

6.4.2 Axial Geotechnical Resistance

For preliminary design of HP 310x110 piles driven to the estimated tip elevations provided in Section 6.4.1, the factored axial resistance at ULS may be taken as 2,000 kN. Serviceability Limit States (SLS) resistances do not apply to piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS. The same factored axial resistance at ULS may be used in the design of closed-end, concrete filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.); it is noted that such pipe piles have a larger end area, but also (as noted above) a greater risk of "hanging up" above the target elevation. In that case, the capacity of a pipe pile terminated within the very dense sand and gravel would need to be determined in the field by Hiley testing.

Pile installation should be in accordance with OPSS 903 *(Deep Foundations).* The pile termination criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known to ensure that the piles are not overdriven and to avoid possible damage to the piles. When driving to refusal on limestone bedrock, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock, and then to gradually increase the energy over a series of blows to seat the pile on/in the bedrock.

The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the site.

6.5 Caissons

As an alternative to steel H-piles or pipe piles, caissons could be considered for support of the new abutments. Due to the presence of water-bearing granular soils, temporary or permanent liners would be required during caisson construction. If caisson foundations are adopted, it is recommended that an NSSP be included in the Contract Documents regarding the water-bearing soils and potential for disturbance or loss of ground, to facilitate selection of appropriate equipment and procedures for the caisson construction.

6.5.1 Founding Elevations

If caisson foundations are adopted, the pile caps should be constructed with a minimum founding depth of 1.6 m below the lowest surrounding grade, to provide adequate protection against frost penetration.

It is recommended that the caissons be socketed approximately 1 m into the bedrock to allow for some weathering/fracturing of the upper portion of the bedrock, and to minimize the potential for loss of soils at the soil-bedrock interface during caisson construction. Additional borehole investigation will be required at the detail design stage to confirm the bedrock surface variability within the footprint of the proposed north and south abutments. However, based on the borehole results from the preliminary investigation, the following caisson founding levels are recommended for preliminary design:

Foundation Element	Borehole No.	Bedrock Surface Elevation	Design Caisson Founding Elevation
North Abutment	CL-1	248.5 m	247.5 m
South Abutment	CL-2	248.9 m	247.9 m

The limestone bedrock is medium strong to very strong, with unconfined compressive strengths anticipated to be in the range of 50 MPa to 150 MPa. Therefore, the sockets would have to be advanced into the rock by churn drilling or rock coring. If caissons are adopted, it is recommended that an NSSP be included in the Contract Documents to describe to the Contractor the strength and character of the bedrock, to facilitate selection of appropriate equipment and procedures for the caisson construction.

6.5.2 Axial Geotechnical Resistance/Reaction

Caissons socketted approximately 0.3 m or greater into the bedrock should be designed based on end-bearing resistance, using a factored axial geotechnical resistance at ULS of 7 MPa for preliminary design purposes. For a 0.9 m diameter caisson, this would equate to a factored axial geotechnical resistance at ULS of 4,450 kN. Serviceability Limit States (SLS) resistances do not apply to caissons founded within the limestone bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.6 Approach Embankments

6.6.1 Subgrade Preparation and Embankment Construction

The replacement of the Cameron Lake bridge may result in a minor widening of the existing Highway 35 embankment, to allow sufficient width to demolish the existing structure and construct the new structure in two halves. Depending on the option selected, the replacement may also involve a grade raise of less than 1 m.

It is recommended that all topsoil/organic material or existing surficial fill materials be stripped from the footprint of the approaches for the widened/raised Highway 35 embankment. It is noted that the depth of fill in Boreholes CL-1 and CL-2 extends approximately 2 m below the natural ground surface at the site; this may be associated with the depth of the existing abutment foundations, or it may represent removal of poor, near-surface soils prior to the construction of the existing embankments. There is some marsh vegetation present along the edges of Perrin Creek and McFarland Bay/Cameron Lake. The depth and extent of stripping for any embankment widening should be assessed during detail design when additional subsurface information can be obtained for the widened approach embankment areas.

New fill materials for the embankment widening and/or grade raise may consist of earth fill, select subgrade material (SSM), or granular fill; rock fill is not considered to be a practical option for a minor widening or grade raise at this site. Any new fill for the approaches should be placed and compacted in accordance with OPSS PROV 206 *(Grading)* and SP 105S21 *(Amendment to OPSS 501)*. Benching of the existing Highway 35 embankment side slopes should be carried out to "key in" the new fill materials, in accordance with OPSD 208.010 *(Benching of Earth Slopes)*.

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 572 *(Seed and Cover)*.

6.6.2 Global Stability

Preliminary slope stability analyses have been performed for the Highway 35 approach embankments using the commercially available program SLIDE, produced by Rocscience Inc., to check that a minimum factor of safety of 1.3 is achieved for the proposed embankment heights and geometries under static conditions. This minimum factor of safety is considered appropriate for the proposed bridge replacement, considering the design requirements and the available field and laboratory testing data.

The preliminary stability analyses have assumed a nominal widening and a grade raise of approximately 1 m, for a total maximum embankment height of approximately 3.5 m, based on the subsurface conditions as encountered in Boreholes CL-1 and CL-2. The following parameters have been used in the analyses, based on field and laboratory test data as well as accepted correlations:

Soil Conditions	Bulk Unit Weight (kN/m^3)	Effective Friction Angle
Embankment fill	21	32°
Existing silty sand to sand and gravel fill below natural ground surface	20	30°
Loose to dense sand and gravel to silty sand and gravel	21	32°
Limestone bedrock	23	-

The preliminary stability analysis results indicate that an approximately 3.5 m high embankment with side slopes oriented no steeper than 2H:1V will have a factor of safety of 1.3 or better against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials. An example static global stability result is provided on Figure 1. This preliminary assessment of the stability of the approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

6.6.3 Settlement

Preliminary settlement analyses under the widened or realigned approach embankments were carried out using both hand calculations and the commercially available computer program *Settle-3D* from Rocscience, using estimated elastic deformation moduli as given in the table below, based on correlations with the SPT "N" values and engineering judgement from experience with similar soils in this region of Ontario (Bowles, 1984; Kulhawy and Mayne, 1990; Peck et al., 1974).

Soil Deposit	Bulk Unit Weight (kN/m^3)	Elastic Modulus (MPa)
Embankment fill	21	-
Existing silty sand to sand and gravel fill below natural ground surface	20	15
Loose sand and gravel to silty sand and gravel	21	15
Compact to very dense sand and gravel to silty sand and gravel	21	50

Based on this preliminary assessment, the settlement of the foundation soils under a nominal embankment widening and a grade raise of up to approximately 1 m is estimated to be a maximum of approximately 10 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the new approach embankments as determined during the detail design, with emphasis on the thickness and properties of any surficial soil deposits within the embankment widening footprint.

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

6.7 Construction Considerations

The following sections identify future construction issues that should be considered at this stage as they may impact the planning and preliminary design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during detail design for incorporation into the Contract Documents.

6.7.1 Excavation and Temporary Protection Systems

The foundation excavations for spread footings are expected to extend to a depth of about 2 m to 2.5 m below the natural ground surface at the site (up to about 4 m to 4.5 m below the existing Highway 35 grade) into the water-bearing, generally loose to compact sand and gravel to silty sand and gravel deposit. The excavations for pile caps could be maintained higher.

Where space permits, open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fill and loose to compact granular soil (assuming appropriate dewatering is in place) would be classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V), provided that appropriate dewatering is in place to lower the groundwater level to below the excavation base.

At this preliminary stage, it is anticipated that temporary roadway protection would be required to facilitate the excavation to foundation level for the new abutments, and removal of the existing bridge superstructure (along with removal of the existing footings, if required). The temporary excavation support system should be designed and constructed in accordance with OPSS 539 *(Temporary Protection Systems)*. The lateral movement of the temporary protection system should meet Performance Level 2 as specified in OPSS 539. However, where excavations are within the zone of influence of existing or new footings while those footings support structures that are in service, it is recommended that the lateral movement of the protection system meet Performance Level 1b as specified in OPSS 539.

It is considered that an interlocking sheetpile system would aid in groundwater control at this site, although the potential presence of cobbles or boulders may impact on the depth that sheet piling can be driven and therefore on the effectiveness of the system.

6.7.2 Groundwater Control

If it is necessary to contain the Perrin Creek flow during construction, the surface water flow could be passed through the site by means of temporary pipes, or diverted by pumping from behind a temporary cofferdam. Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade.

The excavation for the bridge replacement will extend into the water-bearing sand and gravel to silty sand and gravel deposit. It is anticipated that the use of interlocking sheetpile walls (cofferdams), with dewatering from wells or wellpoints within or outside the cofferdams, will be appropriate to control the excavation sides and groundwater for foundation excavations (whether footings or pile caps). Based on the subsurface soil and groundwater conditions, it is anticipated that the dewatering rate will exceed 50 m^3/day, and therefore a Permit to Take Water (PTTW) will be required for this site. In addition, it is recommended that an NSSP be included in the Contract Documents to warn the contractor of the groundwater conditions at the site and to address the design and construction of the cofferdams.

As discussed in Section 6.5, running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of caissons. If caisson foundations are adopted, temporary or permanent caisson liners would be required to support the soils during construction.

6.7.3 Subgrade Protection

It is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP, which can be developed during the detail design stage.

6.7.4 Vibration Monitoring During Pile Driving

Due to the presence of nearby residential properties and the planned staged reconstruction, vibration monitoring is recommended during pile installation to assist in maintaining vibration levels within tolerable ranges for the residential facilities and for the existing/new portions of the bridge. An NSSP should be developed and included in the Contract Documents during detail design.

6.7.5 Obstructions

The soils at this site should be expected to contain cobbles and boulders, which could affect the installation of deep foundations or protection systems. Further observation is recommended in the next stage of investigation in support of the detail design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

6.7.6 Erosion and Scour Protection

The near-surface soils at the site are expected to be susceptible to erosion and scour under the design flood/flow velocities. The requirements for design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g. rip-rap or granular sheeting) be provided on the creek banks to protect the foundations/pile caps from being exposed. The rip-rap should be consistent with the standard R-10 classification or granular sheeting classification in accordance with OPSS 1004 (Aggregates) but should be approved by the hydraulic design engineer.

6.8 Recommendations for Further Work in Detail Design

Additional boreholes will be required during the future detail design stage of investigation, to further assess and/or confirm the subsurface conditions and the preliminary recommendations provided in this report, as follows:

- Abutments:
  - Assessment of the variability and thickness of any existing fill and surficial soils, and the relative density of the founding soil, to confirm the founding elevation for a spread footing option or for a perched footing option within each abutment area. In this regard, due to the potential for disturbance during sampling due to groundwater inflow to the borehole, it is recommended that future boreholes be drilled using drilling methods and materials to minimize such disturbance, and be accompanied by dynamic cone penetration tests.
  - Confirmation of the bedrock surface elevation within the proposed abutment area, and the thickness of any weathered, fractured or otherwise weakened zone at the top of the bedrock, to confirm the founding elevation for piles or caissons.
  - Observation of the presence of cobbles and/or boulders within the soil deposits, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of driven steel H-pile or pipe pile foundations or sheet piling.
  - Further assessment of the groundwater level and permeability of the site soils to refine dewatering estimates.

- Approach embankments:
  - Assessment of the depth and extent of stripping of topsoil/organics or other weak surficial materials within the footprint of any nominal widening of the approach embankments.
  - Further assessment of the thickness and consolidation/elastic compression properties of any loose or firm/stiff surficial soils within the footprint of the approach embankments, to confirm the settlement estimates.

7.0 CLOSURE

This Preliminary Foundation Design Report was prepared by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Foundations Contact for Golder, conducted an independent review of this report.

GOLDER ASSOCIATES LTD.

Lisa C. Coyne, P.Eng.
Senior Geotechnical Engineer, Principal

Fintan J. Heffernan, P.Eng.
Designated MTO Foundations Contact

BM/LCC/FJH/sm

n:\active\2012\1111\12-1111-0021 urs hwy 62 & hwy 35 hastings county\6 - reports\report 2 - hwy 35 - cameron lake bridge\12-1111-0021 rpt02 14apr cameron lake bridge.docx

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Ontario Provincial Standard Specifications (OPSS)

OPSS PROV 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 572	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavating and Backfilling Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

CDED Special Provisions

105S21	Amendment to OPSS 501 – Water Requirements and Quality Control for Compaction

Ontario Provincial Standard Drawings (OPSD)

OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario

TABLE 1 – COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Strip footings on loose to compact sand and gravel to silty sand and gravel	• May not be feasible for support of new abutments due to low bearing resistance on loose native soils	• Allows for semi-integral abutments	• Significant excavations to a depth of up to 2 m to 2.5 m below the natural ground surface, or about 4 m to 4.5 m below the Highway 35 grade; would require temporary protection systems • Significant groundwater control requirements • Lower geotechnical resistances as compared with deep foundations; potential for approximately 25 mm of settlement • Precludes use of integral abutments; potentially greater maintenance required at abutments	• Conventional excavation and construction techniques, although cofferdams and dewatering will be required	• Less expensive than deep foundations although bridge maintenance costs may be higher due to non-integral abutment configuration • Estimated cost is $600/m^3$ for construction of shallow foundations, excluding deeper excavation and temporary protection system
Strip footings perched on compacted granular pad in approach embankment fill	• A good option from a foundations perspective	• Abutment pile caps could be maintained higher, reducing excavation depth and associated temporary protection system requirements when compared with above-noted footing option	• Greater potential for settlement than for piles due to settlement in loose soils under the approach embankment loading	• Conventional excavation and construction techniques	• Less expensive than footings carried down to the sand and gravel deposit

Foundation Option	Feasibility	Advantages	Disadvantages	Constructability	Estimated Costs
Steel H-piles driven to bedrock	• Feasible for support of bridge replacement • A preferred option from a geotechnical/ foundations perspective	• Abutment pile caps could be maintained higher than for footings, reducing depth of excavation and temporary excavation support requirements • Higher geotechnical resistances and negligible settlement • Allows for integral abutment construction	• Some groundwater control would still be required • Minor potential for encountering obstructions (cobbles and/or boulders) during pile driving that could result in piles "hanging up" and lower geotechnical resistances • Depending on elevation for underside of pile cap, may be necessary to pre-auger and core into bedrock to obtain a minimum pile length of 5 m for integral abutment design	• Conventional construction methods for H-pile foundations	• Lower relative cost compared with caisson option • Estimated cost is \$250/m length for pile installation and \$600/m^3 for pile cap construction
Steel pipe (tube) piles, driven to found in bedrock	• Feasible for support of bridge replacement	• Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system • Higher geotechnical resistances and negligible settlement • Allows for semi-integral abutment configuration	• Slightly greater risk than for steel H-pile foundations if obstructions (cobbles and/or boulders) are encountered during driving; this could result in piles "hanging up" and lower geotechnical resistances	• Conventional construction methods	• Costs for steel pipe (tube) piles slightly higher than for steel H-piles
Caissons	• Feasible for support of bridge replacement	• Abutment pile caps could be maintained higher than footings, reducing depth of excavation and temporary protection system • Higher geotechnical resistances and negligible settlement	• Temporary or permanent liners would be required for construction through water-bearing soils • Socketting of caisson base into medium strong to very strong limestone bedrock would be required, necessitating coring or churn drilling	• Liners and rock socketting would be required	• Costs for caissons higher than for steel H-piles or pipe piles

MINISTRY OF TRANSPORTATION, ONTARIO

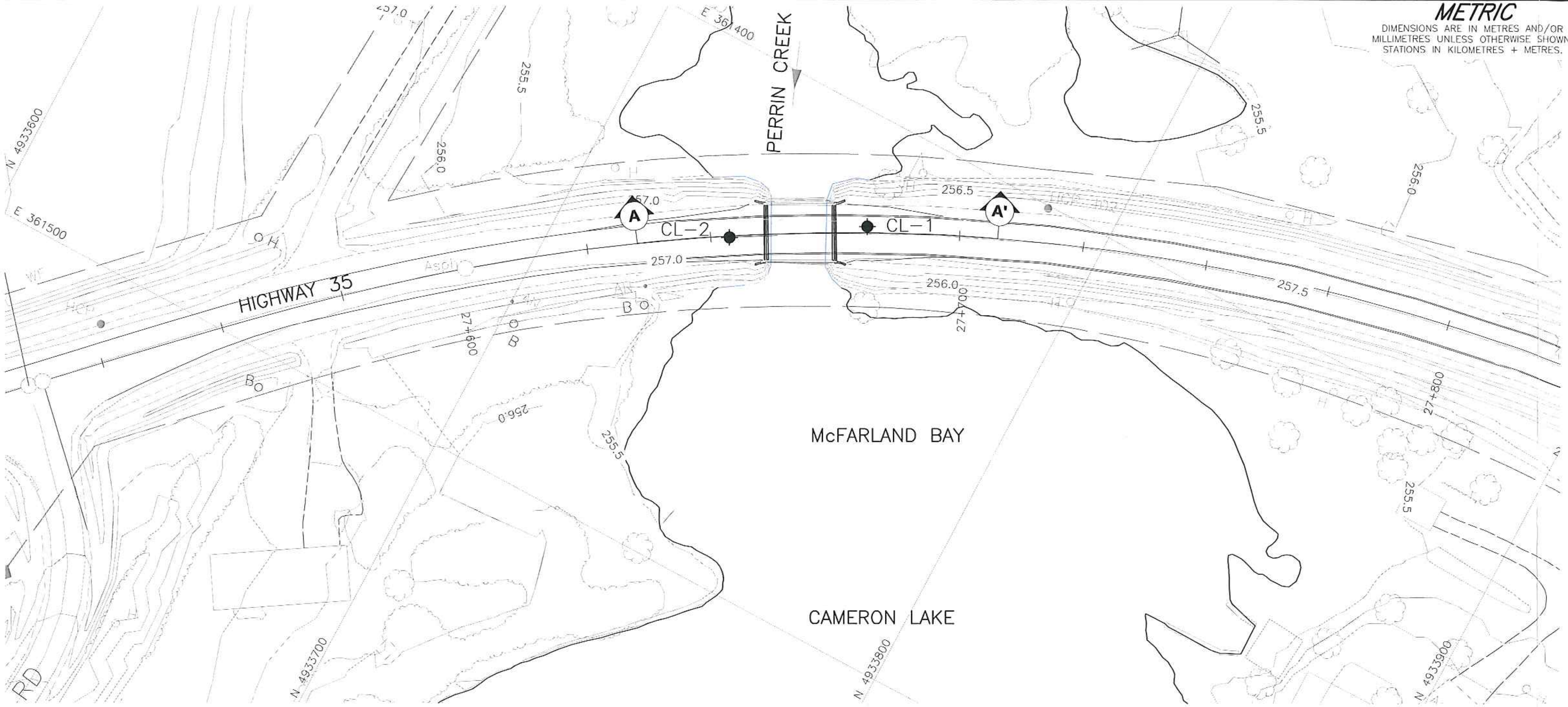
CONT No.
GWP No. 4045-10-00

SHEET

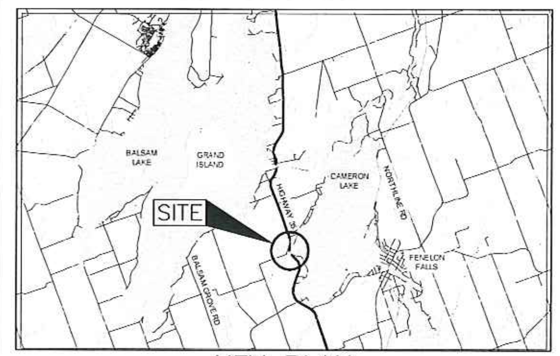
HIGHWAY 35
CAMERON LAKE BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

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



PLAN
SCALE 10 0 10 20 m

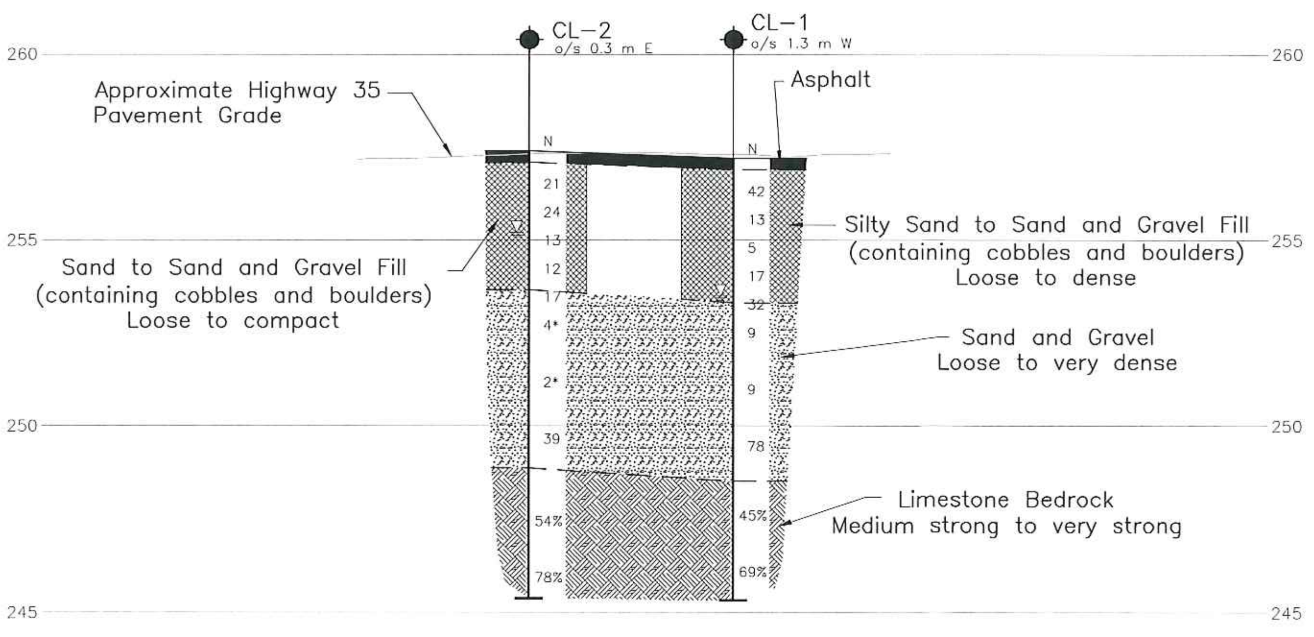


KEY PLAN
SCALE 3 0 3 6 km

LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- * — SPT "N" value considered to have been affected by sample disturbance
- 100% — Rock Quality Designation (RQD)
- WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
CL-1	257.2	4933756.2	361420.8
CL-2	257.4	4933732.7	361435.8



A-A' PROFILE
HORIZONTAL SCALE 10 0 10 20 m
VERTICAL SCALE 2 0 2 4 m

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 preliminary design configuration as shown elsewhere in the Preliminary Design Report.

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

The complete Preliminary 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 report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by URS, drawing file no.s ACAD-Contours_OG-Hwy 35.dwg and ACAD-X-Base_Hwy35.dwg, received December 17, 2012.

LICENSED PROFESSIONAL ENGINEER L.C. COYNE Apr 28, 2014 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER F. J. HEFFERNAN Apr. 28, 2014 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION

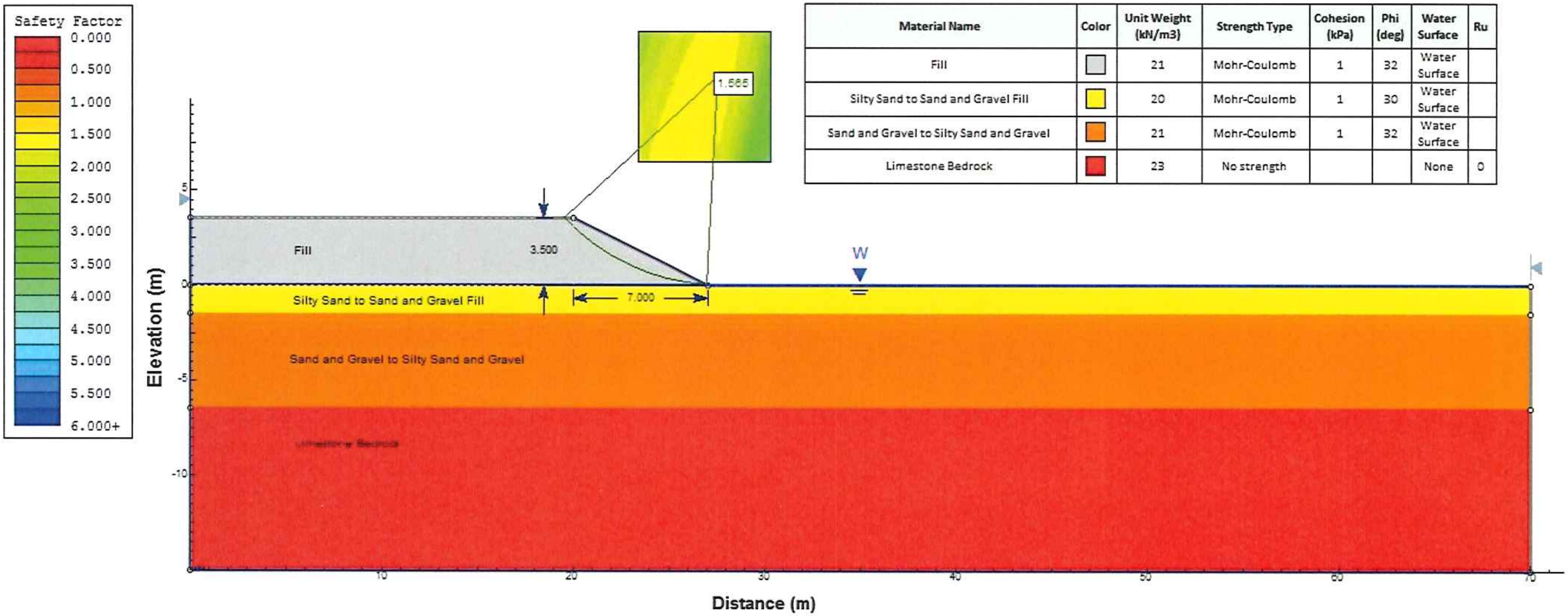
Geocres No. 31D-344

HWY. 35	PROJECT NO. 12-1111-0021		DIST. Eastern
SUBM'D. BM	CHKD. LCC	DATE: 4/28/2014	SITE: 32-064
DRAWN: JFC	CHKD. PKS	APPD. LCC	DWG. 1

PLOT DATE: April 28, 2014

Static Global Stability – Cameron Lake Bridge Approach Embankments

Figure 1



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Water Surface	Ru
Fill		21	Mohr-Coulomb	1	32	Water Surface	
Silty Sand to Sand and Gravel Fill		20	Mohr-Coulomb	1	30	Water Surface	
Sand and Gravel to Silty Sand and Gravel		21	Mohr-Coulomb	1	32	Water Surface	
Limestone Bedrock		23	No strength			None	0

Date: April 2013
Project No: 12-1111-0021

Analysis By: GRL **Reviewed By:** LCC

Golder Associates

APPENDIX A

Borehole Records

LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

(a) Index Properties

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

* 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_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 $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

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

Density Index / **Relative Density**	**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[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	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

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		

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Foundation Design

RECORD OF BOREHOLE No CL-1

SHEET 1 OF 1 **METRIC**

PROJECT 12-1111-0021
W.P. 4045-10-00
LOCATION N 4933756.2 ;E 361420.8
ORIGINATED BY BM
DIST Eastern HWY 35
BOREHOLE TYPE Power Auger, 108 mm I.D. Continuous Flight Hollow Stem
COMPILED BY MAS
DATUM Geodetic
DATE December 6, 2012
CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.2	GROUND SURFACE							
0.0 / 256.9	Asphalt					257		
0.3	Silty sand to sand and gravel, trace to some silt, trace clay, containing cobbles (FILL) Loose to dense Brown Moist	1	SS	42		256		
		2	SS	13			o	31 56 10 3
		3	SS	5		255		
		4	SS	17	∇	254	o	
253.3 / 3.9	Silty SAND and GRAVEL, containing fragments of weathered limestone below 7.6 m Loose to very dense Brown Wet	5	SS	32		253	o	
		6	SS	9		252		
		7	SS	9		251	o	24 51 22 3
		8	SS	78		250, 249		
248.5 / 8.7	Limestone (BEDROCK) Bedrock cored from 8.7 m to 11.9 m For bedrock coring details, refer to Record of Drillhole CL-1	1	RC	REC 93%		248, 247		RQD = 45%
		2	RC	REC 100%		246		RQD = 69%
245.3 / 11.9	END OF BOREHOLE NOTE: 1. Water encountered at a depth of approximately 3.5 m (Elev. 253.7 m) during drilling.							

Plot headings: 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/m³

$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

GTA-MTO 001 12-1111-0021.GPJ GAL-GTA.GDT 4/28/14 DD

PROJECT: 12-1111-0021

RECORD OF DRILLHOLE: CL-1

SHEET 1 OF 1

LOCATION: N 4933756.2 ;E 361420.8

DRILLING DATE:

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: Strong Soil Search

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.

Column headings: DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | PENETRATION RATE min/(m) | FLUSH | COLOUR % RETURN | RECOVERY: TOTAL CORE % (80 60 40 20), SOLID CORE % (80 60 40 20) | R.Q.D. % (80 60 40 20) | FRACT. INDEX PER 0.3 m (5 10 15 20) | DISCONTINUITY DATA: B Angle (0 90 180 270), DIP w.r.t. CORE AXIS (0 30 60 90), TYPE AND SURFACE DESCRIPTION, Jr, Ja, Jn | HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}) | Diametral Point Load Index (MPa) (2 4 6) | RMC -Q' AVG. | NOTES WATER LEVELS INSTRUMENTATION

DEPTH SCALE METRES	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	NOTES WATER LEVELS INSTRUMENTATION
	Continued from Record of Borehole CL-1	248.51		
9	Limestone (BEDROCK), containing shaly interbeds (Shadow Lake Formation) Fresh Thinly bedded Grey Medium strong to very strong	8.69	1	
10				8.5 MPa (Axial)
11			2	(Axial)
		245.32 11.88		
12	END OF DRILLHOLE			
13				
14				
15				
16				
17				
18				

GTA-RCK 004 12-1111-0021.GPJ GAL-MISS.GDT 4/28/14 DD

DEPTH SCALE

1 : 50

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LOGGED: BM

CHECKED: LCC

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RECORD OF BOREHOLE No CL-2

SHEET 1 OF 1

METRIC

PROJECT 12-1111-0021

W.P. 4045-10-00 LOCATION N 4933732.7 ;E 361435.8 ORIGINATED BY BM

DIST Eastern HWY 35 BOREHOLE TYPE Power Auger, 108 mm I.D. Continuous Flight Hollow Stem COMPILED BY MAS

DATUM Geodetic DATE December 3, 2012 CHECKED BY LCC

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.4	GROUND SURFACE						
0.0	Asphalt						
257.1 / 0.3	Sand to sand and gravel, trace to some silt, trace clay, containing cobbles (FILL) Compact Brown Moist becoming wet below a depth of 2.4 m	1	SS	21			
		2	SS	24		o	
		3	SS	13	∇ (Elev. 255.0)		
		4	SS	12		o	44 44 10 2
253.7 / 3.7	SAND and GRAVEL, some silt Loose to compact Brown to grey Wet	5	SS	17			
		6	SS	4*		o	
		7	SS	2*		o	31 54 15 0
		8	SS	39		o	
248.9 / 8.5	Limestone (BEDROCK) Bedrock cored from 9.0 m to 12.0 m For bedrock coring details, refer to Record of Drillhole CL-2	1	RC	REC 95%			RQD = 54%
		2	RC	REC 95%			RQD = 78%
245.4 / 12.0	END OF BOREHOLE						

NOTE:

1. Water encountered at a depth of approximately 2.4 m (Elev. 255.0 m) during drilling.

*SPT "N" values considered to have been affected by disturbance due to groundwater inflow during sampling.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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/m³; ELEVATION SCALE 257–246.

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

PROJECT: 12-1111-0021

RECORD OF DRILLHOLE: CL-2

SHEET 1 OF 1

LOCATION: N 4933732.7 ;E 361435.8

DRILLING DATE:

DATUM: Geodetic

DRILL RIG: CME 55 Track Mount

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Strong Soil Search

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.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH % RETURN	COLOUR	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole CL-2		248.40																		
9		Limestone (BEDROCK), containing shaly interbeds (Shadow Lake Formation) Fresh Thinly bedded Grey Medium strong to very strong		9.00																		
10					1															•		(Axial)
11					2															•		(Axial)
12		END OF DRILLHOLE		245.37 12.03																		
13																						
14																						
15																						
16																						
17																						
18																						
19																						

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: BM

CHECKED: LCC

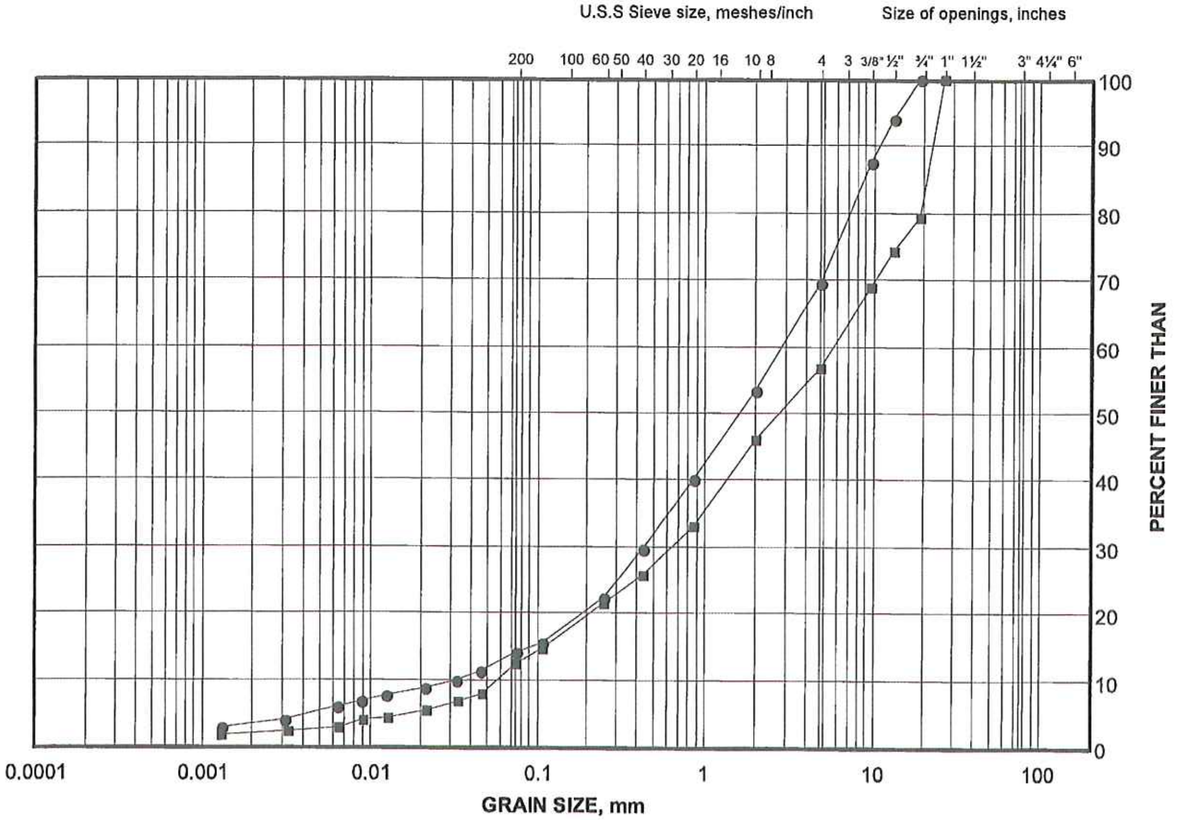
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and 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)
●	CL-1	2	255.4
■	CL-2	4	254.0

Project Number: 12-1111-0021

Checked By: [signature]

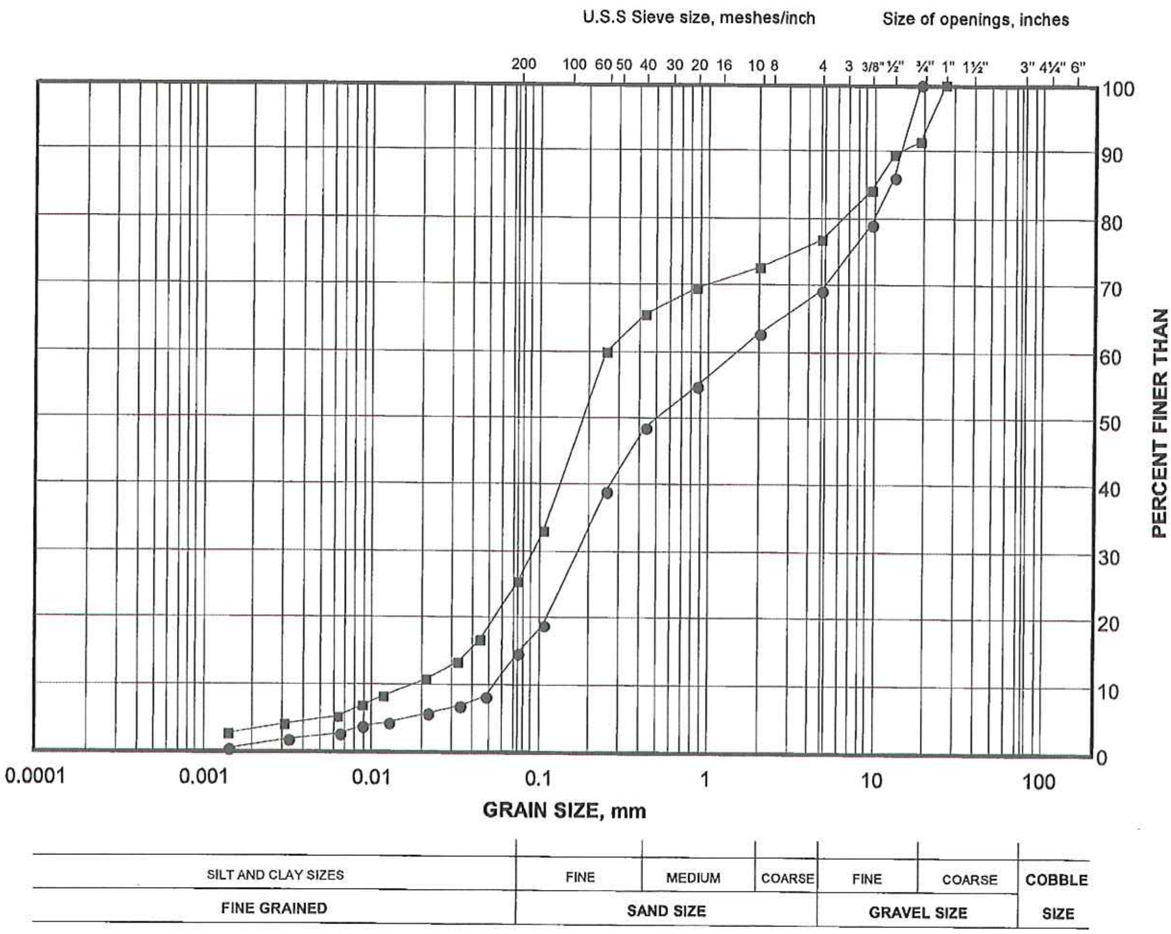
Golder Associates

Date: 03-Apr-13

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Silty Sand and Gravel

FIGURE B2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	CL-2	7	251.0
■	CL-1	7	250.8

Project Number: 12-1111-0021

Checked By: [signature]

Golder Associates

Date: 03-Apr-13