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FOUNDATION INVESTIGATION AND DESIGN REPORT

ABUTMENT REHABILITATION FOR QUINTE SKYWAY BRIDGE HIGHWAY 49, SITE 11-245 HASTINGS COUNTY AND PRINCE EDWARD COUNTY, ONTARIO ASSIGNMENT NO. 9, WP 212-00-02

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GEOCRES No. 31C-265

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REPORT



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PART A

**FOUNDATION INVESTIGATION REPORT
ABUTMENT REHABILITATION FOR QUINTE SKYWAY BRIDGE
HIGHWAY 49, SITE 11-245
HASTING COUNTY AND PRINCE EDWARD COUNTY, ONTARIO
ASSIGNMENT NO. 9, WP 212-00-02**



1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO), to provide foundation engineering services for the abutment rehabilitation of the Quinte Skyway Bridge along Highway 49, spanning the Bay of Quinte from Hasting County to Prince Edward County, Ontario. The Quinte Skyway Bridge abutments are currently supported on spread footings founded on limestone bedrock. It is understood that the superstructure rehabilitation/replacement will result in additional loading at the abutments. Therefore, drilled shafts (caissons) have been proposed by the designer to carry the higher abutment loading.

This work has been carried out under MTO's Eastern Region Retainer Assignment Agreement # 4016-E-0010 – Work Order #9. The Foundations Engineering scope of work includes investigation and recommendations for the design and construction of the proposed abutments.

Two previous geotechnical reports are available for this site as follows:

- MTO, 1962. Foundation Investigation Report for Proposed New Bridge, Hwy 41 and Bay of Quinte, W.P. 210-88-02, Site 11-245, Hwy. 49, District 8, County of Prince Edward and Hastings, Geocres No. 31C-12. Ministry of Transportation and Communications, June 8, 1962.
- CCI Inc., 2014. NPS 6 Picton Lateral Bay of Quinte HDD Replacement Crossing, Report on Geotechnical Evaluation of Site Conditions, Project No. 1108-G0-01-00. CCI Inc., December 8, 2014.

2.0 SITE DESCRIPTION

The Quinte Skyway Bridge runs north-south and carries two lanes of traffic over the Bay of Quinte. The bridge is located approximately 7 km south of the Highway 401/Highway 49 interchange, near Belleville, Ontario as shown on the Key Map on Drawing 1. The bridge is located between Shannonville and Deseronto, with the northern portion of the bridge lying within the Tyendinaga Mohawk Territory.

The high level, sixteen-span bridge was built in 1963 and is about 800 m long with 16 piers and long approach embankments up to about 10 m in height above the surrounding relatively flat agricultural and residential lands.

Signs of moderate to severe pavement distress were observed in close proximity to the abutments, decreasing with distance from the abutments. From the north abutment to approximately 15 m north of the abutment, there is severe transverse cracking, longitudinal cracking and alligator cracking in both the northbound and southbound lanes. North of this point the pavement structure showed few signs of distress with little to no cracking. The side slopes of the north approach embankment are typically vegetated with short grass along with sporadic tree coverage; no visual evidence of instability was observed at the time of the foundation investigation.

The cracking observed at the south approach is less than that at the north approach. From the south abutment to approximately 15 m south of the abutment, there is moderate transverse cracking, longitudinal cracking and alligator cracking in both the northbound and southbound lanes. South of this point, the pavement showed few signs distress with little to no cracking, and this portion of pavement appears to have been recently repaved. The side slopes of the south approach embankment are typically vegetated with short grass and very sparse tree coverage; no visual evidence of slope instability was observed at the time of the foundation investigation.



3.0 INVESTIGATION PROCEDURES

The current investigation for the abutment rehabilitation was carried out between October 16 and 18, 2017, during which time two boreholes (17-1 and 17-2) were advanced to depths of about 14 m below existing ground surface, including about 3 m of rock coring in each. Borehole 17-1 was advanced in the vicinity of the north abutment and Borehole 17-2 was advanced in the vicinity of the south abutment, as shown on Drawing 1. The borehole/drillhole records are provided in Appendix A.

Borehole 17-1 was advanced using hollow-stem augers and HQ rock coring using a track-mounted CME 75 drill rig supplied and operated by Pontil Drilling Ltd. of Mount Albert, Ontario. Borehole 17-2 was advanced using solid stem augers, HQ casing and NQ rock coring equipment using a track-mounted D100 drill rig supplied and operated by MPI Drilling of Picton, Ontario. Traffic protection was required for advancing the boreholes on the roadway and was supplied by Beacon Lite of Kingston, Ontario. Soil sampling was carried out in accordance with the Terms of Reference. In general, soil samples were obtained at depth intervals of 0.75 m and 1.5 m, using a 60 mm outside diameter split-spoon sampler driven by an automatic hammer, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586)¹. The results of geotechnical laboratory testing are contained in Appendix B. The results of the in situ tests (i.e., SPT 'N'-values) as presented on the borehole records and below are uncorrected.

The groundwater conditions were observed in the open boreholes during and immediately following the drilling operations. Standpipe piezometers, equipped with flush-mounted casings, were installed in the boreholes to allow for further monitoring of groundwater levels. The piezometers consist of a 50 mm or 25 mm diameter polyvinyl chloride (PVC) pipe and include a slotted screen sealed within a sand filter pack at a selected depth interval within the boreholes. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was backfilled with bentonite pellets and/or bentonite grout to ground surface. The piezometer installation details and water level readings are indicated on the borehole records.

The field work was supervised by a member of our engineering and technical staff, who observed the drilling, sampling and in situ testing operations, logged the boreholes, and examined and took custody of the soil and bedrock samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Whitby and Mississauga geotechnical laboratories where the samples underwent further visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM standards, as appropriate. Classification testing (water content, grain size distribution and Atterberg limits) was carried out on selected soil samples and unconfined compressive strength (UCS) tests (ASTM D7012)² were carried out on selected bedrock core samples. In addition, select soil samples were submitted to AGAT Laboratories in Mississauga for analytical testing of corrosivity parameters.

The approximate locations of the boreholes were determined based on preliminary drawings provided by MTO. The as-drilled locations and elevations of the boreholes were surveyed using a Trimble Geo7 GPS survey unit. The borehole locations (northing and easting coordinates relative to NAD83 MTM Zone 18, as well as latitude and longitude), Geodetic elevations, and drilling depths are provided on the borehole records and are summarized below and on Drawing 1.

¹ Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils

² Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures



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| Borehole No. | Abutment | Location (MTM NAD 83, Zone9) | | Location (WGS84) | | Ground Surface Elevation (m) | Borehole Depth (m) |
|--------------|----------|------------------------------|----------|------------------|-----------|------------------------------|--------------------|
| | | Northing | Easting | Latitude | Longitude | | |
| 17-1 | North | 4893140.0 | 257886.0 | -77.086717 | 44.176755 | 85.3 | 14.1 |
| 17-2 | South | 4892277.0 | 257908.0 | -77.086362 | 44.168983 | 90.0 | 13.9 |

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The regional geology of the Quinte Skyway Bridge site is divided between the north and south abutments, with each abutment located in a separate physiographic region. The north abutment is located within the western portion of the Napanee Plain physiographic region (Chapman and Putnam, 1984)³. The Napanee Plain features a flat to undulating plain of limestone overlain by a thin layer of overburden. The bedrock located at the north abutment is part of the Gull River and Bobcaygeon Formations. The relatively thin layer of overburden resulted from historic glacial activity stripping away most of the soil in the region. Despite the generally shallow soil deposits, deeper deposits of clay till can be found within valleys in the region. Immediately north and east of the north abutment, several drumlins are visible, and these are longitudinally aligned in a northeast orientation.

The south abutment is located within the northeastern portion of the Prince Edward Peninsula physiographic region (Chapman and Putnam, 1984)³. The Prince Edward Peninsula features a low plateau of limestone, sloping downwards from northeast to southwest, projecting into the northeastern portion of Lake Ontario. The bedrock located at the southern abutment is part of the Lindsay Formation. Originally connected to the mainland by a narrow strip of land in the west, the Prince Edward Peninsula is now entirely separated from the mainland due to the construction of the Murray Canal. Shallow, unconsolidated soils, categorized as Limestone Plains soil, are typically found overlying bedrock within the region. Deposits of deeper clay are found in select areas within the region, such as Big Bay, South Bay and Muscote Bay. Two distinct drumlins are located approximately 1 km south of the south abutment along the western side of Highway 49.

4.2 Subsoil Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are provided on the borehole/drillhole records contained in Appendix A. The results of the laboratory testing are provided in Appendix B and analytical laboratory test results are provided in Appendix C. The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profiles 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.

The general subsurface conditions encountered at the proposed bridge abutment locations consist of asphalt, concrete, and granular fill overlying clayey silt fill, subsequently overlying limestone bedrock at a depth of about 10.5 m below ground surface.

³ 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.



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4.2.1 Asphalt/Concrete

The pavement in Boreholes 17-1 and 17-2 consists of asphalt underlain by the concrete approach slab for the bridge. The depths, elevations and thicknesses of the asphalt and concrete measured at each borehole are provided in the following table.

| Stratigraphy | Borehole 17-1 (North Abutment) | | | Borehole 17-2 (South Abutment) | | |
|--------------|-----------------------------------|------------------|------------------|-----------------------------------|------------------|------------------|
| | Depth (m) | Elevation (m) | Thickness (m) | Depth (m) | Elevation (m) | Thickness (m) |
| Asphalt | 0.0 – 0.2 | 85.3 – 85.15 | 150 mm | 0.0 – 0.2 | 90.0 – 89.85 | 150 mm |
| Concrete | 0.2 – 0.4 | 85.15 – 84.9 | 230 mm | 0.2 – 0.3 | 89.85 – 89.7 | 150 mm |

4.2.2 Gravelly Sand Fill

A deposit of gravelly sand fill was encountered directly below the concrete in both boreholes. This fill is described as brown, dry to moist gravelly sand containing some silt. The surface of the gravelly sand fill was encountered at Elevation 84.9 m and 89.7 m and is 1.8 m and 2.7 m thick in Boreholes 17-1 and 17-2, respectively.

The SPT 'N'-values measured within the gravelly sand fill range from 4 blows to 39 blows per 0.3 m of penetration, indicating a loose to dense relative density. The natural water contents measured on two samples of the gravelly sand fill are about 3 %.

4.2.3 Clayey Silt with Sand Fill

A deposit of clayey silt fill was encountered below the gravelly sand fill in both boreholes. This cohesive fill is described as brown, moist to wet clayey silt with sand, containing some gravel. The surface of the clayey silt fill was encountered at Elevations 83.1 m and 87.0 m, and it is 8.3 m and 7.7 m thick in Boreholes 17-1 and 17-2, respectively.

The SPT 'N'-values measured within the clayey silt fill range from 7 blows to 38 blows per 0.3 m of penetration indicating a firm to hard consistency. The natural water content measured on samples of the clayey silt fill range from about 8 to 10 %.

Atterberg limits testing was carried out on four samples of the clayey silt fill and measured liquid limits ranging from about 16 to 17 %, plastic limits ranging from about 11 to 12 %, and plasticity indices ranging from about 4 to 5 %. The results of the Atterberg limits testing are shown on the plasticity chart on Figure B1 in Appendix B and confirm the fill is classified as, and will behave as, a clayey silt of low plasticity. Additionally, grain size distribution analyses were carried out on five samples of clayey silt fill and the results are shown on Figure B2 in Appendix B.

4.2.4 Limestone Bedrock

The bedrock surface was determined by auger/split-spoon refusal and bedrock coring. Bedrock was encountered at depths of 10.5 m and 10.7 m (Elevation 74.8 m and 79.3 m) in Boreholes 17-1 and 17-2, respectively. The bedrock was cored 3.6 m and 3.2 m in Boreholes 17-1 and 17-2, respectively. Photographs of the bedrock core samples are shown on Figure B3 provided in Appendix B.



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The retrieved bedrock cores are described as slightly weathered to fresh, thin- to medium-bedded, grey, fine- to medium-grained, moderately porous, strong limestone containing shale interbeds. The slightly weathered zone is about 1.5 m and 1.2 m thick in Boreholes 17-1 and 17-2, respectively. The Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), and laboratory UCS test results from the core samples are summarized in the following table.

| Borehole No. | Run | Elevation (m) | TCR | SCR | RQD | Quality Classification (Table 3.10 of CFEM (2006)) ⁴ | UCS (MPa) | Strength Classification (Table 3.5 of CFEM 2006) |
|--------------|-----|---------------|-----|-----|-----|---|-----------|--|
| 17-1 | 1 | 74.8 – 74.5 | 61% | 23% | 0% | Very Poor | - | - |
| | 2 | 74.5 – 72.8 | 84% | 70% | 32% | Poor | 54.6 | (R5) Strong |
| | 3 | 72.8 – 71.2 | 96% | 83% | 83% | Good | 81.7 | (R5) Strong |
| 17-2 | 1 | 79.0 – 78.1 | 49% | 34% | 13% | Very Poor | 69.7 | (R5) Strong |
| | 2 | 78.1 – 77.1 | 91% | 82% | 55% | Fair | 61.2 | (R5) Strong |
| | 3 | 77.1 – 76.1 | 94% | 86% | 60% | Fair | - | - |

As per Table 3.10 of the *Canadian Foundation Engineering Manual* (CFEM, 2006)⁴, the rock mass is classified as very poor to poor quality within the upper 2.0 m and 1.2 m in Boreholes 17-1 and 17-2, respectively, and fair to good quality below these depths. Also, the SCR was above 70% below the first run of core in each borehole. It should be noted that in Borehole 17-2, the upper approximately 0.8 m of core was lost or broken, and difficulty was encountered seating the casing in the upper 0.3 m of the bedrock.

4.2.5 Groundwater Conditions

Unstabilized groundwater levels were measured in the open boreholes upon completion of drilling, and stabilized groundwater levels were measured in the standpipe piezometers on November 24, 2017. The recorded depths to groundwater and corresponding groundwater elevations as measured in the piezometers are summarized in the following table. It should be noted that groundwater levels may vary depending on the time of year and precipitation events, and they should be anticipated to be higher during wet seasons following periods of heavy precipitation and/or snow melt.

| Borehole | Borehole 17-1 (North Abutment) | | Borehole 17-2 (South Abutment) | |
|---|-----------------------------------|---------------|-----------------------------------|---------------|
| | Depth (m) | Elevation (m) | Depth (m) | Elevation (m) |
| Groundwater in Standpipe Piezometer (November 24, 2017) | 9.0 | 76.3 | 10.9 | 79.1 |

Based on these measurements, the groundwater level appears to be within the bedrock. However, there is also potential for some “perched” groundwater to be present near the base of the gravelly sand fill, atop the cohesive fill at the abutment locations.

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



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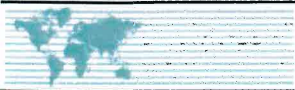
4.2.6 Analytical Results

Two soil samples were submitted to AGAT Laboratories in Mississauga for analysis of chloride, sulphates, conductivity, resistivity, and pH. The following table provides a summary of the sample numbers, soil description, sample depths and elevations, and the resulting parameter concentrations/values. A summary of the soil analytical results is provided on the Laboratory Certificates of Analysis, included in Appendix C.

| Borehole (Sample No.) | | 17-1 (S9) | 17-2 (S9) |
|--------------------------|---------|-------------------------|-------------------------|
| Sample Depth (Elevation) | | 7.6 – 8.2 (77.7 – 77.1) | 7.6 – 8.2 (82.4 – 81.8) |
| Parameter | Units | Clayey Silt Fill | Clayey Silt Fill |
| Chloride (CL) | µg/g | 14 | 77 |
| Sulphate (SO4) | µg/g | 10 | 18 |
| Conductivity (EC) | mS/cm | 0.147 | 0.262 |
| Resistivity | ohms*cm | 6800 | 3820 |
| pH | n/a | 8.35 | 8.15 |

5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Anastasia Poliacik, P.Eng., and reviewed by Ms. Sarah E.M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Ms. Lisa Coyne, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent quality control review of this report.



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Report Signature Page

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PART B

**FOUNDATION DESIGN REPORT
ABUTMENT REHABILITATION FOR QUINTE SKYWAY BRIDGE
HIGHWAY 49, SITE 11-245
HASTING COUNTY AND PRINCE EDWARD COUNTY, ONTARIO
ASSIGNMENT NO. 9, WP 212-00-02**



6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the abutment rehabilitation of the Quinte Skyway Bridge along Highway 49, spanning the Bay of Quinte from Hasting County to Prince Edwards County, Ontario. The recommendations are based on Golder's interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to carry out the design of the proposed caisson abutment rehabilitation. The Foundation Design Report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the contractor or Design-Build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing high level, sixteen-span, bridge was built in 1963 and is about 800 m long with 16 piers and long approach embankments. The existing abutments of the Quinte Skyway Bridge are currently supported on strip footings founded on limestone bedrock. We understand that the superstructure of the bridge is to be rehabilitated, which will result in additional loading at the abutments. In order to minimize the depth of excavation for new abutment foundations, drilled shafts (caissons) have been proposed by the designer to carry the new abutment loading, with four drilled shafts at each abutment location. The centreline of the new abutments will be located approximately 3.7 m behind the centreline of the existing abutments, and less than about 2 m from the back edge of the existing strip footings. It is understood that a retained soil system (RSS) wall is proposed behind the new abutments to separate the approach fill from the abutment walls, thereby minimizing or eliminating the backfill loading on the abutment walls. It is also understood that there is very limited construction access space for this project.

6.2 Site Classification

6.2.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* and its *Commentary* (CHBDC, 2014), the proposed abutment rehabilitation is considered to be classified as having a "typical consequence level" associated with exceeding limits states design. In addition, given the level of foundation investigation completed to date at this location in comparison to the degree of site understanding in Section 6.5 of CHBDC (2014), the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, ψ , from Table 6.1 and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Table 6.2 of the CHBDC (2014) have been used for design.

6.2.2 Seismic Site Classification

The 2012 Ontario Building Code (2012 OBC) came into effect on January 1, 2014 and contains updated seismic analysis and design methodology. Seismic hazard is defined for an earthquake with a 2 percent probability of exceedance in 50 years (i.e. a return period of 2,400 years) which encompasses a larger earthquake hazard than in prior editions of the OBC. Design earthquakes are commonly defined by an earthquake magnitude, distance, and peak ground acceleration (PGA). The 2012 OBC uses the uniform hazard spectra (UHS) to define the



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response of the structure to the design earthquake and also considers the effects of the localized site conditions on the structural response. The 2012 OBC also uses a refined site classification system defined by the average soil/bedrock properties in the top 30 metres of the subsurface profile beneath the structure(s). There are six site classes designated as A to F related to decreasing ground stiffness from A for hard rock to E for soft soil and site class F for problematic soils (e.g. sites underlain by thick peat deposits and/or liquefiable soils). The site class is then used to obtain acceleration- and velocity-based site coefficients, F_a and F_v , respectively, used to modify the reference UHS to account for the effects of site-specific soil conditions in design.

A Site Class C may be used for the site due to the presence of shallow bedrock. Since more than 3 m of fill is present between the underside of pile cap and the bedrock surface, Site Classes A and B are not to be utilized.

6.2.2.1 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC, the peak ground acceleration (PGA) values and design spectral acceleration values for Site Class C are presented below.

Seismic Hazard Values for Reference Ground Condition Site Class C

| Seismic Hazard Values | 10% Exceedance in 50 years (475-year return period) | 5% Exceedance in 50 years (975-year return period) | 2% Exceedance in 50 years (2,475 return period) |
|------------------------|---|--|---|
| PGA (g) | 0.039 | 0.059 | 0.097 |
| Sa (0.2) (g) | 0.068 | 0.101 | 0.159 |
| Sa (0.5) (g) | 0.048 | 0.070 | 0.106 |
| Sa (1.0) (g) | 0.028 | 0.041 | 0.062 |
| Sa (2.0) (g) | 0.014 | 0.021 | 0.032 |
| Sa (≥ 10.0) (g) | 0.0013 | 0.0021 | 0.0035 |

The fundamental period of the structure is expected to be greater than 0.5 s and, as indicated by MTO, the bridge is classified as a *Major-Route Bridge*. Therefore, in accordance with Table 4.10 of the CHBDC, the bridge structure falls under Seismic Performance Category 1. Based on this Seismic Performance Category, it is understood that no seismic analysis is required.

6.3 Drilled Shafts (Caissons)

6.3.1 General

The new abutments can be supported on drilled shafts (caissons) founded on or socketed into the limestone bedrock. Forming large diameter sockets within the strong limestone bedrock may be difficult at this site and could also pose issues from an equipment/constructability perspective. In addition, due to the potential presence of water perched within the fill, a temporary steel liner or casing should be utilized to support the overburden soils during construction to minimize disturbance to the side walls and to prevent materials from sloughing into the hole and affecting the drilled shaft base. Based on discussions with the design team, the drilled shaft resistance has been assessed based on end-bearing conditions and will not rely on shaft resistance within the variable consistency fill materials nor within the upper fractured portion of rock, and therefore the liners can be temporary or permanent. If temporary liners are utilized, they should be installed and/or removed in accordance OPSS.PROV 903 (Deep Foundations).



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Although the upper 2.0 m of bedrock at the north abutment (Borehole 17-1) and the upper 1.2 m of bedrock at the south abutment (Borehole 17-2) measured Rock Quality Designation (RQD) values of less than 50%, the highly fractured portion of the bedrock is limited to the upper 0.3 m in Borehole 17-1 and upper 0.8 m in Borehole 17-2. The solid core recovery below these depths was also high. Therefore, consideration could be given to founding the caissons on the surface of the bedrock, or alternatively socketed into the bedrock to achieve a higher resistance. The following table provides the elevation of the ground surface, proposed underside of pile cap and bedrock surface as encountered in the boreholes.

| Borehole | Borehole 17-1 (North Abutment) | Borehole 17-2 (South Abutment) |
|-------------------------------------|-----------------------------------|-----------------------------------|
| Ground Surface Elevation (m) | 85.3 | 90.0 |
| Underside of Pile Cap Elevation (m) | 80.8 | 85.5 |
| Top of Bedrock Elevation (m) | 74.8 | 79.3 |

It is recommended that all pile caps be provided with a minimum of 1.5 m of soil cover for frost protection as per OPSD 3090.101 (Foundation, Frost Penetration Depths for Southern Ontario). Alternatively, where lesser cover is available in areas where there exists a risk of frost action (i.e., frost susceptible soils and water, subjected to freezing temperatures), insulation may be incorporated to provide protection.

6.3.2 Axial Geotechnical Resistance

The caissons founded on/socketed into the bedrock should be designed based on end-bearing resistance only, as any shaft resistance from the upper fractured bedrock is not likely to be mobilized without sufficient socket length. The resistance will depend on the strength and quality of the bedrock.

For a “typical” consequence factor of 1.0 and a “typical” degree of understanding, a geotechnical resistance factor of 0.4 was applied to the geotechnical resistances provided below.

Based on the bedrock core retrieved at the north abutment, founding the drilled shafts on the surface of the bedrock can be considered, but due to the fracturing and low RQD in the upper portion, a reduced resistance will be available as compared to having a minimum 1.0 m long socket. At the south abutment, approximately 0.8 m of core was lost immediately below the surface of the bedrock, which is likely indicative of fracturing; thus, the bedrock at this location is considered to be more fractured than the north abutment and as such, a further reduced resistance will be available. The reduced resistance is also a factor of the ability to clean the base (see details below). The factored ultimate axial geotechnical resistance at each abutment is provided in the following table. Serviceability Limit States (SLS) resistances do not apply to drilled shafts founded on or socketed into the limestone bedrock since the factored serviceability geotechnical resistance for 25 mm of settlement is greater than the factored ultimate geotechnical resistance. The resistances are based on the strength and fracture profile of the bedrock and the equations provided in the CFEM and amended by engineering judgement.

| Socket Length (m) | Borehole 17-1 (North Abutment) | Borehole 17-2 (South Abutment) |
|----------------------|---|-----------------------------------|
| | Factored Ultimate Axial Geotechnical Resistance (MPa) | |
| 0 | 7 MPa | 5 MPa |
| 1.0 | 10 MPa | 10 MPa |



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The above resistances should be divided by the base area of the drilled shafts for the total capacity (MN). If the above-noted resistances are not sufficient for support of the new abutments, higher geotechnical resistances may be able to be provided if in situ load testing could be carried out as part of the construction contract. However, it is understood based on discussions with the design team that the above-noted resistances are workable for design.

Commercially available casing/liner sizes for drilled shafts are 0.6 m, 0.9 m and 1.2 m in diameter. Rock socketing equipment such as downhole hammer, churn drill, etc. are also available to fit within or be consistent with these sizes. The larger the socket, the more difficult to install and less likely to be commonly available. However, casing/socket sizes up to 1.5 m in diameter are possible, albeit at a cost premium relative to smaller diameter drilled shafts.

The bedrock surface must be sufficiently cleaned prior to drilled shaft construction. A clean base is essential to achieving the resistances provided above, as loose material left in place could lead to settlement of the drilled shaft prior to mobilizing the design resistance. For non-socketted drilled shafts founded on the bedrock surface, this is typically accomplished by using air lifting/jetting techniques. For socketed caissons, this is also achieved using the equipment that forms the socket. Ontario Provincial Standard Specification (OPSS).PROV 903 (Deep Foundations) should be included in the Contract Documents requiring inspection and approval of the drilled shaft hole and bearing area by the Quality Verification Engineer (QVE) prior to drilled shaft construction. In addition, since the wording in the OPSS regarding inspection is vague, a Non-Standard Special Provision (NSSP) for inspection of the base using a video camera should be included in the Contract Documents, as well as for proper cleaning of the base; an example NSSP is included in Appendix D. This NSSP also warns the contractor about the strength and abrasivity of the bedrock.

Due to the presence of highly fractured bedrock in the upper 0.3 m and 0.8 m of core from Boreholes 17-1 and 17-2, respectively, cleaning of the base may result in advancing the drilled shaft base deeper than the measured bedrock surface elevation. Therefore, as steel reinforcing cages are typically prefabricated, they should be made at least 1.0 m longer and then cut to the final length on site as the final cleaning and inspection will dictate the ultimate length of the drilled shaft. Alternatively, any additional depth required for bedrock depth / cleaning below the design elevation can be accommodated by the placement of a mass concrete plug of minimum 30 MPa compressive strength. This requirement is also captured in the NSSP.

Due to the potential presence of groundwater above the bedrock surface, the drilled shaft concrete will need to be installed using tremie methods. In this regard, the discharge pipe must be maintained at least 1 m below the surface of the concrete at all times during concrete placement. The water level in the drilled shaft liner should be allowed to stabilize prior to the start of concrete placement to minimize the potential adverse impacts of upward water flow through the concrete.

Temporary liners are typically installed using vibratory techniques. Further, vibrations are also generated during socketing operations in bedrock (if required). Section 6.6.1 provides additional recommendations regarding vibrations.



6.3.3 Resistance to Lateral Loads

It is understood that an RSS wall will be constructed behind the caissons in order to minimize/eliminate transfer of lateral loading from the backfill onto the new abutment walls/pile caps, within the upper 4.5 m below ground surface at the new abutments. Below 4.5 m, the drilled shafts will provide resistance to lateral loading. The resistance to lateral loading would be derived from the soil surrounding the drilled shafts. The evaluation of the drilled shaft behavior subjected to lateral loads should take into account factors such as the rigidity of the structure relative to the surrounding soil, the structural capacity of the structure to withstand the bending moment and shear, the soil resistance that can be mobilized, and the group effects.

The lateral load response may be calculated using subgrade reaction theory suggested in CHBDC (2014) Commentary (Section C6.11.2.2), where the coefficient of horizontal subgrade reaction, k_h , (kPa/m) is based on the equations given below, as described by Terzaghi (1955) and the *Canadian Foundation Engineering Manual* (CFEM 1992).

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where: n_h is the constant of horizontal subgrade reaction (kPa/m);
 z is the depth (m); and,
 B is the caisson diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where: s_u is the undrained shear strength of the soil (kPa); and,
 B is the caisson diameter/width (m).

The values of n_h and s_u to be used to calculate the coefficient of horizontal subgrade reaction (k_h) in the structural analysis are given below. The estimated angles of internal friction, ϕ' , and passive lateral earth pressure coefficient, K_p , are also provided below.

| Soil Deposit | Depth (Elevation) at North Abutment (m) | Depth (Elevation) at South Abutment (m) | s_u (kPa) | ϕ' (degrees) | K_p * |
|---------------------------|---|---|-------------|-------------------|---------|
| Existing Clayey Silt Fill | 4.5 – 10.5 (74.8 – 80.8) | 4.5 – 10.7 (79.3 – 85.5) | 100 | 30 | 3.0 |

* The total passive resistance may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

Lateral resistance calculations of the drilled shafts should neglect the soil below the pile cap to about 1.5 times the drilled shaft diameter, to account for disturbance effects during installation (Broms, 1964). In addition, the upper 1.5 m of soil below the pile cap should be neglected as it may be within the frost penetration zone (depending on the configuration of the pile cap with respect to the embankment). The greater depth of these two cases should be used in assessing the depth of lateral resistance to be neglected in the upper portion of the drilled shaft.

Additional lateral resistance, if required, can be provided by socketing the drilled shafts into the bedrock. For example, for drilled shafts socketed at least 1.0 m into bedrock within a 900 mm diameter concrete-filled socket (i.e., socketed for a depth of at least equal to the socket diameter), the ultimate (unfactored) lateral resistance of the limestone may be taken as the lesser of 30 MPa or the compressive strength of the Portland cement concrete placed in the bedrock socket.



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

| Drilled Shaft Spacing in Direction of Loading (d = Drilled Shaft Diameter) | Subgrade Reaction Reduction Factor, R |
|--|---|
| 8d | 1.00 |
| 6d | 0.70 |
| 4d | 0.40 |
| 3d | 0.25 |

Where a caisson group is oriented perpendicular to the direction of loading, group action may be considered by reducing the coefficient of horizontal subgrade reaction (NAVFAC, 1986) by a reduction factor, R_P , as follows:

| Drilled Shaft Spacing Perpendicular to Direction of Loading d = Drilled Shaft Diameter | Horizontal Subgrade Reaction Reduction Factor, R_P |
|--|--|
| 4d | 1.00 |
| 1d | 0.50 |

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

6.4 Retaining Walls

It is understood that an RSS wall ("RSS abutment wall", for clarity in this section of the report) will be constructed behind the drilled shafts/new abutment wall to retain the approach fill and eliminate/reduce the lateral loading on the drilled shafts/abutment wall. Further, RSS walls will be implemented as wingwalls ("RSS wingwalls" in this section of the report) at the new abutments and these will not be structurally connected to the abutment wall. Both the RSS abutment wall and the RSS wingwalls should be designed such that the reinforcing strips are not in conflict and constructed such that the reinforced soil mass of all the walls behind the abutments can be raised/placed concurrently. The following sections provide foundation recommendations for the retaining walls.

The RSS abutment walls will extend across the entire width of the abutment (approximately 10.5 m) and will be located immediately behind the pile cap/abutment wall and be separated by a layer of polystyrene. The RSS abutment walls will be approximately 4.5 m in height (i.e. the same as the height of the pile cap/abutment wall). The reinforced soil mass will extend outwards from the abutment walls. The details of the reinforcing strips, drainage and separator layer should be provided by the proprietary wall designer/supplier.

The RSS wingwalls will extend approximately 6 m behind the abutment wall and be stepped from the maximum height behind the abutment wall upwards with distance away from the abutment, within the approach slab area. The RSS walls will consist of front facing panels which are typically supported on a concrete footing/alignment element, constructed on a compacted granular levelling pad, and a reinforced soil mass behind the wall.



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The reinforced soil masses for both walls may be founded on the existing clayey silt fill. Prior to the placement of the reinforced soil mass, all founding surfaces must be inspected in accordance with MTO Special Provision (SP) 599S22 (Retained Soil System) to ensure that the existing fill has been adequately cleaned of ponded water and disturbed, loosened, softened, organic and other deleterious material. If encountered, such deleterious materials should be subexcavated and replaced with compacted granular fill meeting the requirements of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II and OPSS.PROV 501 (Compacting).

For the RSS wingwalls, the facing panel footings should be founded at a minimum depth of 0.5 m below the backfilled grade, on a minimum 0.5 m thick levelling pad. The levelling pad should consist of compacted OPSS.PROV 1010 (Aggregates) Granular 'A' material that should extend at least 0.5 m beyond the outside edge of the facing panels, then outward/downward at 1 horizontal to 1 vertical (1H:1V).

The RSS abutment wall performance criteria shall be designated as "high", as the wall must be designed to resist / retain the soils loads from the approach fills. As the barrier walls will be constructed on top of the RSS wingwalls, they shall be designed for "high" performance in accordance with SP 599S22 (Retained Soil System). Final design details are to be provided by the proprietary wall designer.

6.4.1 Geotechnical Resistance

Assuming that each wall's (RSS abutment wall and RSS wingwalls) reinforced soil mass act as an individual unit and uses the full width of the reinforced soil mass (which can be taken as 0.8 times the wall height, or a width of 3.6 m for the maximum 4.5 m high wall), a factored ultimate geotechnical resistance of 300 kPa and a factored serviceability geotechnical resistance (for 25 mm of settlement) of 200 kPa may be used for design of the reinforced soil mass founded on the properly prepared existing fill. For the RSS wingwall facing panels supported on a concrete footing constructed on a compacted granular pad, the wall design may be completed based on a factored ultimate geotechnical resistance of 150 kPa and a factored serviceability geotechnical resistance (for 25 mm of settlement) of 100 kPa.

For a "typical" consequence factor of 1.0 and a "typical" degree of understanding, a geotechnical resistance factor of 0.5 was applied to the geotechnical resistances provided above.

6.4.2 Resistance to Lateral Loads / Sliding Resistance

Resistance to lateral forces / sliding resistance between the compacted fill of the RSS wall and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC (2014). The coefficient of friction, $\tan \delta'$, between the compacted granular fill of the RSS wall and the properly prepared fill subgrade may be taken as 0.58. This represents an unfactored value. The actual values used should be reviewed and revised, if necessary, by the proprietary RSS wall designer during detail design.

6.4.3 Global Stability

The RSS abutment walls and wingwalls at this site will have a factor of safety of greater than 1.3 in short-term (undrained) conditions, and greater than 1.5 in long-term (effective stress) conditions, provided that the reinforcing strip length is at least 0.8 times the height of the wall, and that the fill placement in front of the RSS wingwall panels is as shown on the design drawings (i.e. the fill extends to at least 1.5 m above the base of the wall). These factors of safety are calculated based on a "typical" consequence factor of 1.0 and a "typical" degree of understanding geotechnical resistance factors of 0.75 and 0.65, respectively.



6.5 Lateral Earth Pressures for Design

As RSS abutment walls will be constructed behind the abutment walls to resist lateral loads instead of the abutment wall, lateral earth pressures will not be transmitted to the back of the abutment walls. Therefore, conventional lateral earth pressure recommendations do not apply for the abutment walls at this site. However, for completeness, recommendations for the design of conventional concrete walls are provided below:

- Select, free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular B Type II, should be used for construction of the RSS abutment walls and RSS wingwalls, and as backfill behind conventional concrete walls if adopted; however, the RSS backfill will ultimately be as specified by the proprietary RSS designer. Additional backfill outside of the RSS backfill locations, shall be free-draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II. Longitudinal drains or weep holes should be installed in the abutment walls to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) for the backfill within the RSS walls should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement), OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement), and 3190.100 (Walls, Retaining and Abutment, Wall Drain).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with *CHBDC* (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.5 m behind the back of the wall (Case (a) on Figure C6.20 of the Commentary to the *CHBDC* (2014)). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or pile cap (Case (b) on Figure C6.20 of the Commentary to the *CHBDC* (2014)).

6.5.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.



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- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

| Material | Granular 'A' | Granular 'B' Type II |
|--|----------------------|----------------------|
| Soil Unit Weight: | 22 kN/m ³ | 21 kN/m ³ |
| Coefficients of static lateral earth pressure: | | |
| Active, K_a | 0.27 | 0.27 |
| At rest, K_o | 0.43 | 0.43 |
| Passive, K_p | 3.7 | 3.7 |

- If the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:
 - Rotation of approximately 0.002 about the base of a vertical wall (where the rotation is calculated as the horizontal displacement divided by the height of the wall);
 - Horizontal translation of 0.001 times the height of the wall; or,
 - A combination of both.
- If the wall does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.

6.5.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its *Commentary*, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.



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Seismic Active Pressure Coefficients, K_{AE}

| | Design Earthquake | Site PGA | SSM | Granular A | Granular B Type II |
|-------------------|-------------------|----------|------|------------|--------------------|
| Yielding wall | 2,475 Yr | 0.097 g | 0.46 | 0.39 | 0.39 |
| Non-yielding wall | 2,475 Yr | 0.097 g | 0.66 | 0.55 | 0.55 |

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250k_h$ mm, where k_h is the site adjusted PGA estimated at the ground surface. At the site, the ground surface PGA (Site Class C) is 0.097 g. This corresponds to displacements of up to approximately 25 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e., an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d , (kPa);

K_a is the static active earth pressure coefficient;

K_o is the static at-rest earth pressure coefficient;

K_{AE} is the seismic active earth pressure coefficient;

γ is the unit weight of the backfill soil (kN/m^3), as given previously;

d is the depth below the top of the wall (m); and,

H is the total height of the wall (m).

6.6 Construction Considerations

6.6.1 Excavations and Temporary Protection Systems

Excavations for the pile caps and the retaining wall structures are anticipated to extend to approximately 5 m below road grade and will therefore extend through the asphalt, approach slab concrete, granular fill, and clayey silt fill.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing embankment fill is classified as Type 3 soil above the groundwater table and Type 4 soil below the groundwater table, 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 1H:1V in Type 3 soils and 3H:1V in Type 4 soils. However, due to spatial constraints, these excavation slopes are not achievable at this site and therefore, temporary protection systems will be required to maintain stability of excavation walls.



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The complete detailed design of the temporary protection systems will be the responsibility of the contractor. The temporary protection system will have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support. The temporary protection system could consist of either soldier piles and lagging or steel sheet piling. Conceptually, steel sheet piles may work if sufficient toe resistance is available between the base of excavation and bedrock, as the sheets will not be able to be seated into bedrock. If soldier piles are selected, they would likely need to be socketed into the bedrock. Due to the proposed construction staging and limited space, standard bracing such as tiebacks, deadmans, rakers, etc. may not be feasible and a sufficiently heavy pile section and wall rigidity would be required.

Recommended values of the parameters for use in design of temporary protection systems are provided below. The loading from adjacent structures and construction equipment as well as any material stockpiles within a distance defined by a 1.5 horizontal to 1 vertical line drawn from the bottom of the excavation to the existing ground surface should be included as a surcharge. The geotechnical engineering parameters provided below are considered appropriate for design of the temporary protection systems with respect to the ultimate conditions and do not account for control of ground displacements. If control of ground displacements is critical it may be necessary to use parameter values that result in higher design loads or undertake an iterative evaluation of assumed ground and water pressure and structural displacement analyses to arrive at an appropriately stiff ground support system.

| Soil Type | Coefficient of Earth Pressure ¹ | | | | Internal Angle of Friction ϕ' (degrees) | Undrained Shear Strength s _u (kPa) | Unit Weight γ (kN/m³) |
|------------------------------------|--|---------------------------|-------------------------------------|-------|---|---|-----------------------------|
| | Active K _a | At Rest K _o | Passive K _p ² | | | | |
| | | | Level | 2H:1V | | | |
| Gravelly Sand Fill | 0.29 | 0.46 | 3.39 | 1.29 | 33 | - | 20 |
| Clayey Silt to Silt with Sand Fill | 0.35 | 0.52 | 2.88 | 1.06 | 29 | 100 | 19 |
| Granular 'A' | 0.31 | 0.47 | 3.25 | 1.23 | 32 | - | 21 |
| Granular 'B' Type II | 0.27 | 0.43 | 3.69 | 1.39 | 35 | - | 22 |

1. The lateral earth pressure coefficients presented above under "No Sloping Ground in Front of Wall" are based on a horizontal surface adjacent to the excavation. If sloped surfaces are expected behind or in front of the wall, the coefficients should be corrected accordingly, as per the CFEM. Recommendations for 2H:1V sloping ground in front of the wall (i.e. K_p) have been provided and may be required depending on the planned geometry at this site to be determined by the contractor.
2. The total passive resistance below the base of the excavation (i.e. adjacent to the temporary protection system) may be calculated based on the values of K_p indicated above but reduced by an appropriate factor that considers the allowable wall movement in accordance with Figure C6.16 of the CHBDC (2014) to account for the fact that a large strain would be required for mobilization of the full passive resistance.

The design groundwater level can be assumed to be Elevation 76.3 m and 79.1 m at the north and south abutments, respectively. The temporary protection system design should be assessed for both the drained (ϕ') and undrained (s_u) cases and the design should be based on the more conservative earth pressure conditions. The active earth pressure coefficients are based on the assumption that the ground surface behind the temporary protection system is horizontal. Where the retained ground is sloping, the lateral earth pressure coefficients must be adjusted to account for the slope and these earth pressure coefficients can be estimated from the equations provided on Figures 24.1 and 24.3 in the Canadian Foundation Engineering Manual, 4th Edition (CFEM, 2006).



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The temporary protection system should be in accordance with OPSS.PROV 539 (Temporary Protection Systems) and should be designed to Performance Level 2, provided that any buried utilities that may be present adjacent to the excavations can tolerate this magnitude of deformation. Design of the temporary protection system shall include an evaluation of base stability (“base heave” or soil squeezing stability) and hydraulic uplift stability as defined in the *Canadian Foundation Engineering Manual* (CFEM 2006).

For the excavation depths proposed, the height of the embankment above the original ground and the groundwater level within the bedrock at depth, basal heave and hydraulic uplift is not considered to be an issue and an NSSP is not warranted. Perched water within the fill, if present, will not impact basal heave or hydraulic uplift.

6.6.2 Control of Groundwater and Surface Water

Excavations for the construction of the pile caps and retaining walls are anticipated to extend to approximately 5 m below existing road grade which is above the groundwater level at this site. The groundwater level measured immediately after rock coring (i.e. introduction of water) was 3.0 m and 4.6 m below ground surface; however, the stabilized groundwater level was measured at 9.0 m and 10.9 m below ground surface at the north and south abutments, respectively. Therefore, only minor groundwater seepage is anticipated within the excavation and this should be able to be handled using properly filtered sump pumps within the excavation and positive drainage away from the excavation. Dewatering should be in accordance with OPSS 902 (Excavation for Structure) and the MTO NSSP FOUND003., reference to this NSSP is provided in Appendix D.

Water takings in excess of 50,000 L/day are regulated by the Ontario Ministry of Environment and Climate Change (MOECC). Certain takings of groundwater and stormwater for construction dewatering purposes with a combined total less than 400,000 L/day qualify for self-registration on the MOECC's Environmental Activity and Sector Registry (EASR). Registry on the EASR replaces the need to obtain a PTTW for water taking and a Section 53 approval for discharge of water to the environment. A “Water Taking Plan” and a “Discharge Plan” are required by the MOECC if water is taken in accordance with an EASR. In all cases, discharge under the EASR must be in accordance with a Discharge Plan (to be developed by a qualified professional). The contractor will be responsible for obtaining any required discharge approvals. A Category 3 PTTW would be required for water takings in excess of 400,000 L/day. At this site, neither an EASR nor a PTTW will be required.

As the groundwater level will be encountered within the length of the drilled shafts, the existing fill should be considered susceptible to disturbance and sloughing during drilled shafts installation both above and below the groundwater level, and therefore a temporary liner will be required. The NSSP for drilled shafts also contains a clause alerting the contractor to the presence of these soils; an example is included in Appendix D.

6.6.3 Vibration Monitoring

The drilled shafts and/or sheet piles for protection systems will be located immediately adjacent to the existing bridge footings, the travelled roadway and any utilities present within the roadways. As discussed in Section 6.2.2, permanent/temporary liners will be required for the drilled shafts, and these are typically installed using vibratory techniques. Similarly, sheet piles, if utilized for temporary protection systems, are also typically installed using vibratory methods. Further vibrations are also generated during rock socketing (if utilized) or for rock-socketed soldier piles.



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Given that the existing footings are founded on the bedrock, there is a low risk of vibrations from the drilled shaft construction causing disturbance to the founding stratum. Further, the existing spread footings will be taken out of service once the drilled shafts are installed and the superstructure replaced. It is our understanding that there are no utilities present within the approach slab area of the bridge, and therefore there is negligible risk of vibrations impacting utilities at this site. It should be noted that the time that vibrations will be induced is limited to only a few hours per drilled shaft or sheet pile.

It should also be noted that the radial influence of vibrations is generally quite limited (less than a few to several metres) and dissipates very rapidly with distance from the source. Therefore, it is considered that the risk of vibrations impacting the homes located closest to the bridge will be low to negligible, as it is understood that the nearest dwelling or well is some 70 m away from the bridge abutments. However, it is our experience that some homeowners may have the “perception” that the vibrations are being felt, and that they are having adverse effects on their persons or property.

If MTO considers the risk of this “perception” of vibrations by adjacent property owners is high, then it is suggested that the contract contain a requirement for pre- and post-construction condition surveys on the adjacent property structures/wells, and an NSSP to address vibration monitoring requirements during sheet pile and drilled shaft installation.

Assuming that vibration monitoring is adopted, it should be carried out during drilled shaft and driven sheet pile installation and also during other activities that may induce vibrations. Vibrations up to a maximum peak particle velocity (PPV) of 100 mm/s are generally considered applicable for bridge structures in good condition. However, given the sensitivity of the overall site, the bridge designers should assess and establish an acceptable, lower PPV threshold for the existing structure. Monitoring stations should be set up on the ground and bridge deck adjacent to the abutments, as well as at select ground surface locations at set distances away from the bridge in all (non-water) directions depending on the specified overall radius.

It is understood that the MTO and the bridge designers consider vibration monitoring (and pre- and post-condition surveys) is merited based on the above-noted discussion and risks. Therefore, and although not strictly required from a foundations perspective, a sample NSSP for vibration monitoring, including qualifications, positioning of the monitoring equipment, tolerances, and frequency is provided in Appendix D.

6.6.4 Obstructions

Although auger grinding and/or hard drilling was not observed during borehole advancement through the existing fill at the two borehole locations, it is possible that cobbles, boulders, or other obstructions are present within the fill. An NSSP should be included in the Contract Documents to identify to the contractor the possible presence of cobbles, boulders or other obstructions within the embankment fill; an example is included in Appendix D.

6.6.5 Analytical Testing for Construction Materials

Two samples of the embankment fill were submitted to AGAT Laboratories in Mississauga for analytical testing of soil corrosivity parameters. The potential for sulphate attack and corrosion are discussed in the following paragraphs; however, it is ultimately up to the designer to determine the appropriate construction materials, including the exposure class and ensuring that all aspects of CSA A23.1-14 Section 4.1.1 “*Durability Requirements*” are followed when designing concrete elements.



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The analytical test results were compared to CSA A23.1-14 Table 3 ("Additional requirements for concrete subjected to sulphate attack") for the potential sulphate attack on concrete. The water soluble-sulphate concentration measured in the soil samples tested are less than 0.01 percent, indicating the degree of exposure is less than moderate. Therefore, based on the samples tested, when the designer is selecting the exposure class for the structure, the effects of sulphates from within the soil stratum tested may not need to be considered. However, given the location of structures within the highway right-of-way, they may be exposed to de-icing salts and selection of the exposure class should consider this.

The analytical test results were also compared to Table 7.1 of FWHA NHI-09-087, November 2009 which correlates the relative level of corrosion potential to soil resistivity. The measured resistivity values indicate the fill is classified as mildly corrosive at Borehole 17-1 (north abutment) and moderately corrosive at Borehole 17-2 (south abutment).

The analytical test results were also compared to Table 7.2 Criteria of the U.S. Criteria for Assessing Ground Corrosion Potential (as outlined by Lazarte *et al.* 2015 in FHWA) for the potential corrosion of buried steel, based on the measured values of pH, resistivity, chlorides, and sulfates in the soil samples tested. Based on the measured values, the soil samples are classified as non-aggressive.

However, as noted above, given the location of the drilled shafts within the highway right-of-way, it is expected that the drilled shafts could experience exposure to de-icing salts and therefore consideration should be given to providing corrosion protection to reinforcing elements.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Anastasia Poliacik, P.Eng., and the technical aspects were reviewed by Ms. Sarah E.M. Poot, P.Eng., a senior geotechnical engineer and Associate of Golder. Ms. Lisa Coyne, P.Eng., Golder's Designated MTO Foundations Contact for this project, conducted an independent quality review of the report.



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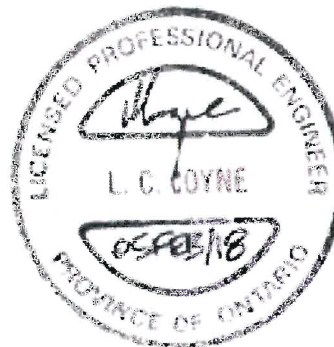
Report Signature Page

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AMP/SEMP/LC/amp/nh

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FOUNDATION REPORT - ABUTMENT REHABILITATION QUINTE SKYWAY BRIDGE, HIGHWAY 49

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Foundation Reports

- MTO, 1962. Foundation Investigation Report for Proposed New Bridge, Hwy 41 and Bay of Quinte, W.P. 210-88-02, Site 11-245, Hwy. 49, District 8, County of Prince Edward and Hastings, Geocres No. 31C-12. Ministry of Transportation and Communications, June 8, 1962.
- CCI Inc., 2014. NPS 6 Picton Lateral Bay of Quinte HDD Replacement Crossing, Report on Geotechnical Evaluation of Site Conditions, Project No. 1108-G0-01-00. CCI Inc., December 8, 2014.

ASTM International

- | | |
|------------|---|
| ASTM D1586 | Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils |
| ASTM D7012 | Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures |

Ontario Occupational Health and Safety Act

- Ontario Regulation 213/91 Construction Projects

Ontario Provincial Standard Drawings

- | | |
|---------------|---|
| OPSD 3090.101 | Foundation, Frost Penetration Depths for Southern Ontario |
| OPSD 3101.150 | Walls Abutment, Backfill Minimum Granular Requirement |
| OPSD 3121.150 | Walls Retaining, Backfill Minimum Granular Requirement |



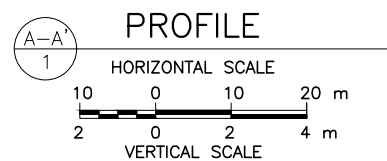
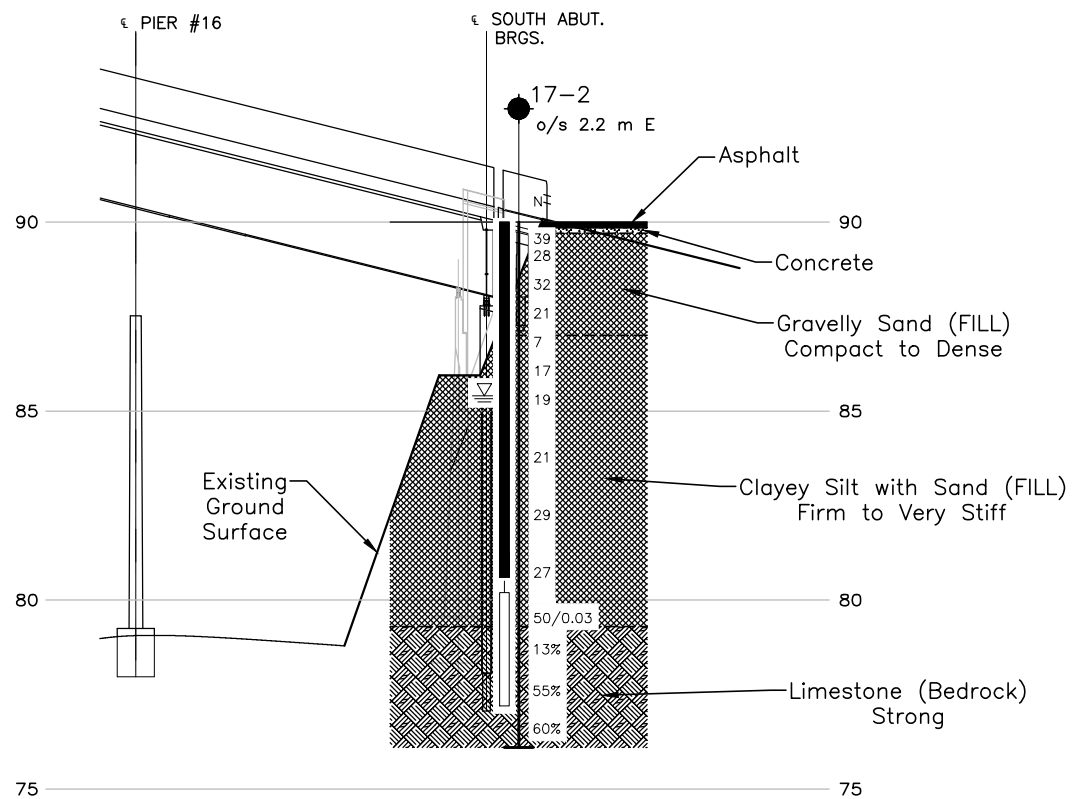
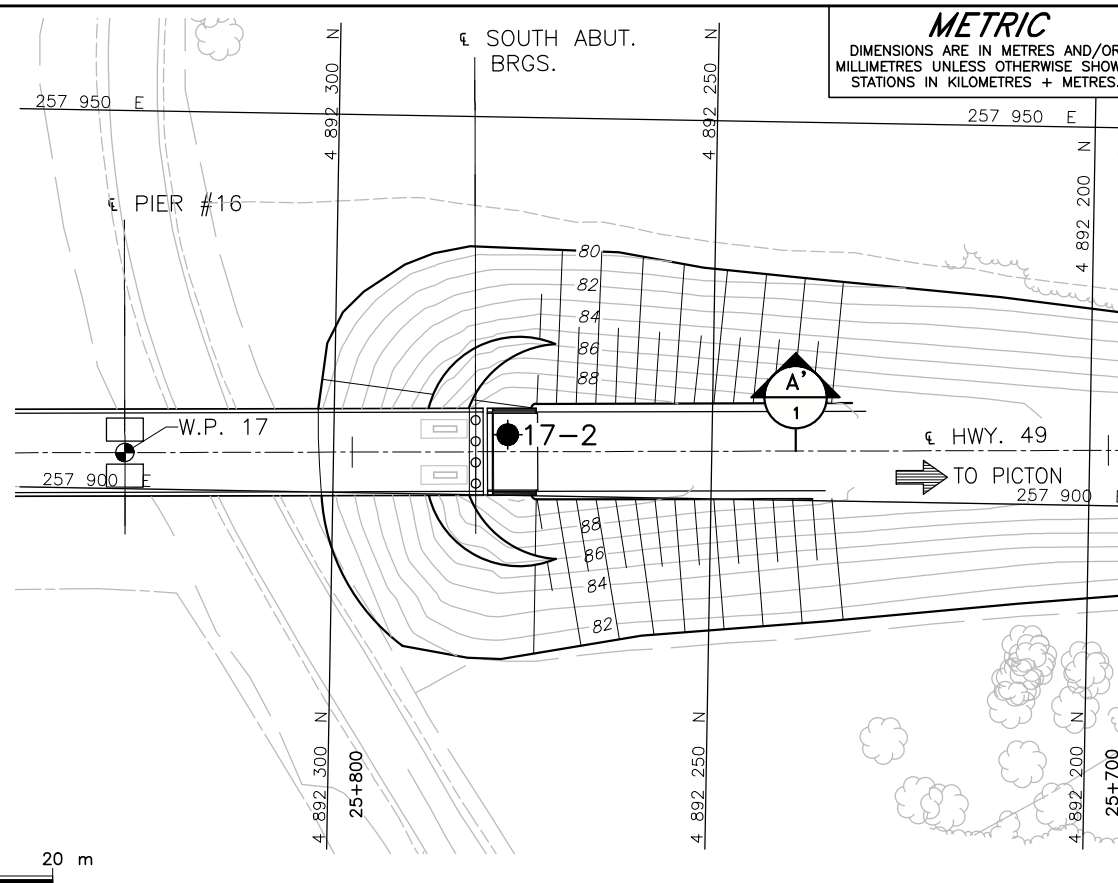
FOUNDATION REPORT - ABUTMENT REHABILITATION QUINTE SKYWAY BRIDGE, HIGHWAY 49

Ontario Provincial Standard Specifications

| | |
|----------------|--|
| OPSS.PROV 501 | Construction Specification for Compacting |
| OPSS.PROV 539 | Construction Specification for Temporary Protection Systems |
| OPSS.PROV 903 | Construction Specification for Deep Foundations |
| OPSS.PROV 1010 | Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material |

Ontario Water Resources Act

| | |
|---------------------------|---|
| Ontario Regulation 903/90 | Wells: O.Reg. 468/10 Amendment to Ontario Regulation 90 |
|---------------------------|---|

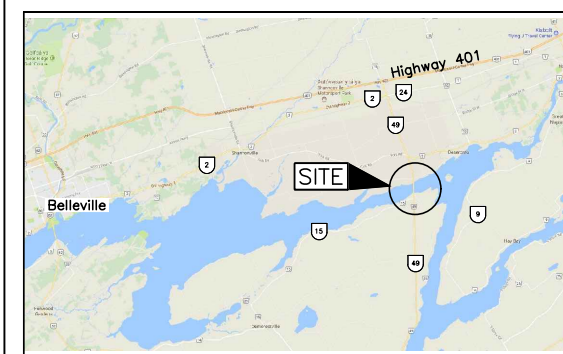


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN
STATIONS IN KILOMETRES + METRES.

CONT No. 2017-4047
WP No. 212-00-02







HIGHWAY 49 QUINTE SKYWAY BRIDGE BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN
N.T.S



LEGEND

- | | |
|---|--|
|  | Borehole |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
|  | WL upon completion of drilling |

BOREHOLE CO-ORDINATES

| No. | ELEVATION | NORTHING | EASTING |
|------|-----------|-----------|----------|
| 17-1 | 85.3 | 4893140.0 | 257886.0 |
| 17-2 | 90.0 | 4892277.0 | 257908.0 |

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 MTO, drawing file no. PGA.DWG,
received OCT 24, 2017.



| | | | |
|---------------------|------------|--------------------------|----------------|
| | | | |
| | | | |
| NO. | DATE | BY | REVISION |
| Geocres No. 31C-265 | | | |
| HWY. 49 | | PROJECT NO. 1663816/9004 | DIST. KINGSTON |
| SUBM'D. | CHKD. AP | DATE: 1/3/2018 | SITE: 11-245 |
| DRAWN: TB | CHKD. SEMP | APPD. LCC | DWG. 1 |



APPENDIX A

Borehole Records



LIST OF SYMBOLS

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

I. GENERAL

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

II. STRESS AND STRAIN

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

III. SOIL PROPERTIES

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

(a) Index Properties (continued)

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

(b) Hydraulic Properties

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

(c) Consolidation (one-dimensional)

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

(d) Shear Strength

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

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

Notes: 1
2

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

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



LIST OF ABBREVIATIONS

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

I. SAMPLE TYPE

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

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

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

Dynamic Cone Penetration Resistance; N_d :

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

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

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

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

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

(b) Cohesive Soils Consistency

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

IV. SOIL TESTS

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

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

V. MINOR SOIL CONSTITUENTS

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



LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERINGS STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

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

Description and Notes

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

Abbreviations

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

\\GOLDER.GDS\GAL\MIS\S\S\GA\S\M\C\CLIENTS\MT\O\HWY 49\02 DATA\GIN\T\1663816 SKYWAY.GPJ GAL-MISS.GDT 11/29/17 TB

| PROJECT <u>1663816/9004</u> | | RECORD OF BOREHOLE No 17-1 | | | | 2 OF 3 METRIC | | | | | | | | | | | |
|------------------------------------|---|---|--------|------|----------------------------|-------------------------|---|--------------------|----|----|-----|---|-------------------|----------------|---|--|-----------|
| W.P. <u>212-00-02</u> | | LOCATION <u>N 4893140.0; E 257886.0 MTM ZONE 9 (LAT. 44.176755; LONG. -77.086717)</u> | | | | ORIGINATED BY <u>MB</u> | | | | | | | | | | | |
| DIST <u>KINGSTON</u> HWY <u>49</u> | | BOREHOLE TYPE <u>108 mm I.D. Hollow Stem Augers, HQ Coring</u> | | | | COMPILED BY <u>AP</u> | | | | | | | | | | | |
| DATUM <u>GEODETIC</u> | | DATE <u>October 18, 2017</u> | | | | CHECKED BY <u>SEMP</u> | | | | | | | | | | | |
| SOIL PROFILE | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | | | "N" VALUES | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | 20 | 40 | 60 | 80 | 100 | W _p | W | W _L | | | |
| | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | | | | | | | | | | |
| | | | | | | | 20 | 40 | 60 | 80 | 100 | 10 | 20 | 30 | | | |
| 71.2 | LIMESTONE (BEDROCK) | | 2 | RC | REC 84% | | 73 | | | | | | | | | | RQD = 32% |
| | For coring details see Record of Drillhole 17-1. | | 3 | RC | REC 96% | | 72 | | | | | | | | | | |
| 14.1 | END OF BOREHOLE | | | | | | | | | | | | | | | | |
| | Note(s): 1. Water level at a depth of 3.0 m below ground surface (Elev. 82.3 m) upon completion of drilling. 2. Groundwater measured in piezometer at a depth of 9.0 m below ground surface (Elev. 76.3 m), on November 24, 2017. | | | | | | | | | | | | | | | | |

PROJECT: 1663816/9004

RECORD OF DRILLHOLE: 17-1

SHEET 3 OF 3

LOCATION: N 4893140.00 ;E 257886.00

DRILLING DATE: October 18, 2017

DATUM: GEODETIC

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 75

DRILLING CONTRACTOR: Pontil Drilling Inc.

| DEPTH SCALE METRES | DRILLING RECORD | DESCRIPTION | SYMBOLIC LOG | ELEV. DEPTH (m) | RUN No. | NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY | | | | | | | | | | | | | | | | | FEATURES | PIEZOMETER | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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| | | | | | | | TOTAL CORE % | SOLID CORE % | | | DIP w.r.t. CORE AXIS | TYPE AND SURFACE DESCRIPTION | Jr | Ja | Jzon | W1 | W2 | | W3 | W4 | W5 | W6 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
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| 11 | CME 75 HQ Coring | REFER TO PREVIOUS PAGE | | 74.8 | 1 | Brown | 100 | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |

DEPTH SCALE

1 : 60



LOGGED:

CHECKED:

GTA-RCK 046_SUPBURY \GOLDER\GDS\GAL\MISSAUGA\MISSAUGA\IMTOHWY_49102_DATA\GINT\1663816_SKYWAY.GPJ GAL-MISS.GDT 11/24/17 TB

RECORD OF BOREHOLE No 17-2

1 OF 3 **METRIC**

PROJECT 1663816/9004

W.P. 212-00-02

LOCATION N 4892277.0; E 257908.0 MTM ZONE 9 (LAT. 44.168983; LONG. -77.086362)

ORIGINATED BY MB

DIST KINGSTON HWY 49

BOREHOLE TYPE 150 mm Solid Stem Augers, HQ Casing and NQ Coring

COMPILED BY AP

DATUM GEODETIC

DATE October 16 and 17, 2017



CHECKED BY SEMP

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---------------|--|------------|---------|------|------------|----------------------------|--------------------|---|--|--|--------------|--|------------------------------------|-------------------------------------|-----------------------------------|--|---|-------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | | WATER CONTENT (%) | | |
| | | | | | | | | ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED | | | | | | | | | | | | |
| | | | | | | 20 40 60 80 100 | | | | | 10 20 30 | | | GR SA SI CL | | | | | | |
| 90.0 | GROUND SURFACE | | | | | | | | | | | | | | | | | | | |
| 0.0 | ASPHALT (150 mm) | | | | | | | | | | | | | | | | | | | |
| | CONCRETE (150 mm) | | | | | | | | | | | | | | | | | | | |
| 0.3 | Gravelly sand, some silt (FILL) Compact to dense Brown Dry to moist | | 1 | SS | 39 | | | | | | | | | | | | | | | |
| | | | 2 | SS | 28 | | 89 | | | | | | | | | | | | | |
| | | | 3 | SS | 32 | | 88 | | | | | | | | | | | | | |
| | | | 4 | SS | 21 | | | | | | | | | | | | | | | |
| 87.0 | | | | | | | 87 | | | | | | | | | | | | | |
| 3.0 | Clayey silt with sand, some gravel (FILL) Firm to very stiff Brown Moist to wet | | 5 | SS | 7 | | | | | | | | | | | | | | | |
| | | | 6 | SS | 17 | | 86 | | | | | | | | | 10 42 35 13 | | | | |
| | | | 7 | SS | 19 | | 85 | | | | | | | | | | | | | |
| | | | | | | | 84 | | | | | | | | | | | | | |
| | | | 8 | SS | 21 | | 83 | | | | | | | | | | | | | |
| | | | | | | | 82 | | | | | | | | | | | | | |
| | | | 9 | SS | 29 | | 81 | | | | | | | | | | | | | |
| | | | | | | | 80 | | | | | | | | | | | | | |
| | | | 10 | SS | 27 | | | | | | | | | | | 12 42 31 15 | | | | |
| 79.3 | Split-spoon bouncing/refusal at 10.7 m depth. | | 11 | SS | 50/0.03 | | | | | | | | | | | | | | | |
| 10.7 | LIMESTONE (BEDROCK) For coring details see Record of Drillhole 17-2. | | 1 | RC | REC 49% | | 79 | | | | | | | | | RQD = 13% | | | | |
| | | | 2 | RC | | | | | | | | | | | | RQD = 55% | | | | |

Continued Next Page

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

SUD-MTO 001 MTM ZN INC LAT/LONG \GOLDER.GDS\GALMISSAUSCASIM\CLIENTS\MT01HWY_49\02_DATA\GINT\1663816_SKYWAY.GPJ GAL-MISS.GDT 11/29/17 TB

| PROJECT <u>1663816/9004</u> | | | | RECORD OF BOREHOLE No 17-2 | | | | 2 OF 3 METRIC | | | | | | | | | | | | |
|------------------------------------|--|---|---------|---|------------|---|-----------------|---|----|----|----|-----|---|--|--|---|--|--|-----------|-----------|
| W.P. <u>212-00-02</u> | | | | LOCATION <u>N 4892277.0; E 257908.0 MTM ZONE 9 (LAT. 44.168983; LONG. -77.086362)</u> | | | | ORIGINATED BY <u>MB</u> | | | | | | | | | | | | |
| DIST <u>KINGSTON</u> HWY <u>49</u> | | | | BOREHOLE TYPE <u>150 mm Solid Stem Augers, HQ Casing and NQ Coring</u> | | | | COMPILED BY <u>AP</u> | | | | | | | | | | | | |
| DATUM <u>GEODETIC</u> | | | | DATE <u>October 16 and 17, 2017</u> | | | | CHECKED BY <u>SEMP</u> | | | | | | | | | | | | |
| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL | | | |
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | WATER CONTENT (%) | | | | | | | |
| | --- CONTINUED FROM PREVIOUS PAGE --- | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | | | |
| | LIMESTONE (BEDROCK) For coring details see Record of Drillhole 17-2. |  | 2 | RC | REC 91% |  | 77 | | | | | | | | | | | | | RQD = 55% |
| | | | 3 | RC | REC 94% | | | | | | | | | | | | | | RQD = 60% | |
| 76.1 13.9 | END OF BOREHOLE Note(s): 1. Water level at a depth of 4.6 m below ground surface (Elev. 85.4 m) upon completion of drilling. 2. Groundwater measured in piezometer at a depth of 10.9 m below ground surface (Elev. 79.1 m) on November 24, 2017. | | | | | | | | | | | | | | | | | | | |

SUD-MTO 001 MTM ZN INC LAT/LONG \GOLDER.GDS\GALMISSISSAUGASIM\CLIENTS\MTOH\HWY_49\02_DATA\GINT\1663816_SKYWAY.GPJ GAL-MISS.GDT 11/24/17 TB

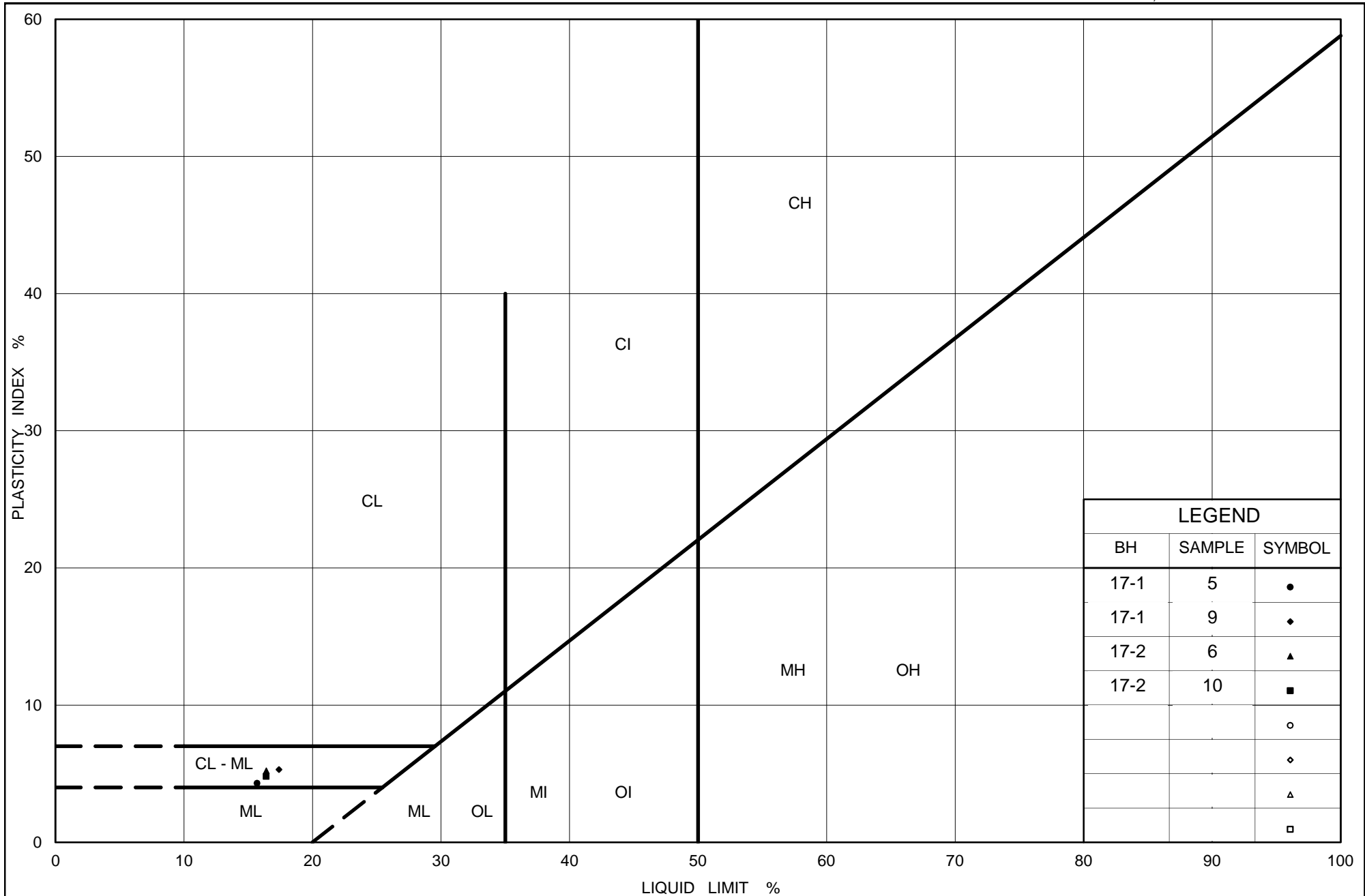
[illegible]

GTA-RCK 046 SUDBURY \GOLDER.GDS\GAL\MISSAUGA\SIMCLIENTS\IMTO\HWY 49\02 DATA\GINT\1663816 SKYWAY.GPJ GAL-MISS.GDT 11/24/17 TB



APPENDIX B

Geotechnical Laboratory Test Results



Ministry of Transportation

Ontario

PLASTICITY CHART CLAYEY SILT with Sand (FILL)

Figure No. B1

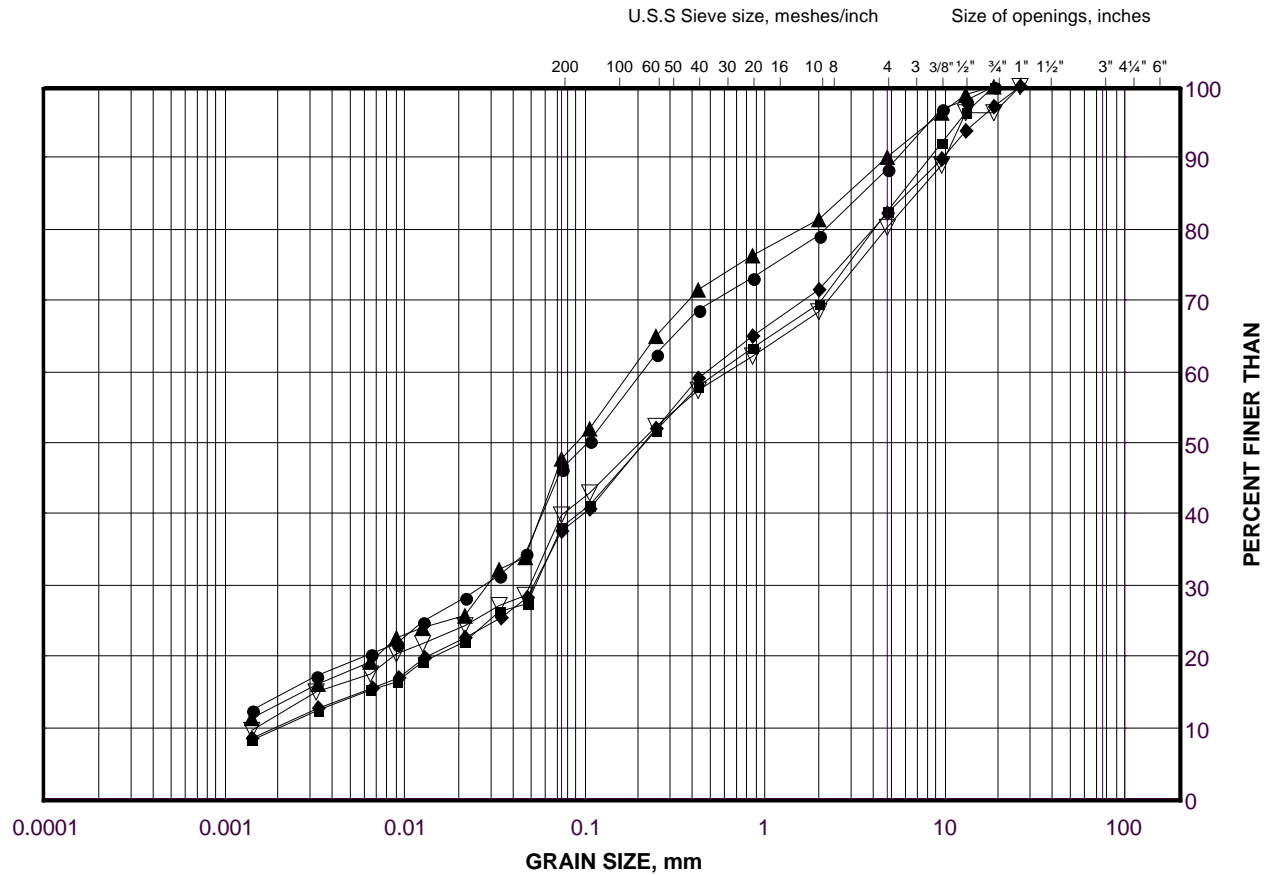
Project No. 1663816 (9003)

Checked By: AMP

GRAIN SIZE DISTRIBUTION

CLAYEY SILT with Sand (FILL)

FIGURE B2



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

LEGEND

| SYMBOL | BOREHOLE | SAMPLE | ELEVATION(m) |
|--------|----------|--------|--------------|
| ● | 17-2 | 10 | 80.6 |
| ■ | 17-1 | 4 | 82.7 |
| ◆ | 17-1 | 5 | 82.0 |
| ▲ | 17-2 | 6 | 85.9 |
| ▽ | 17-1 | 9 | 77.4 |

Project Number: 1663816 (9003)

Checked By: AMP

Golder Associates

Date: 23-Nov-17

Borehole 17-1 (HQ Core)



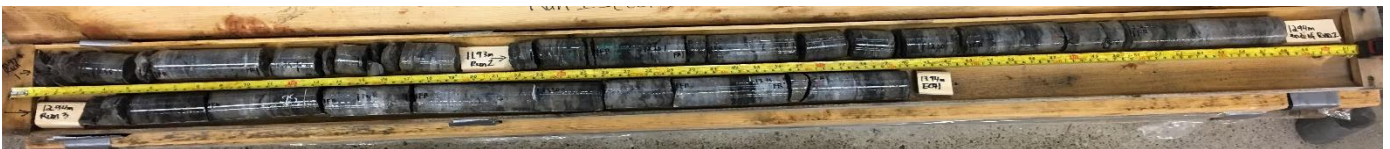
Box 1: Run 1 and Run 2 (10.5 m to 12.5 m)

Borehole 17-1 (HQ Core)



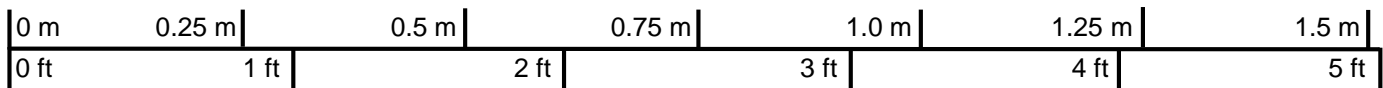
Box 2: Run 3 (12.5 m to 14.1 m)

Borehole 17-2 (NQ Core)



Box 1: Run 1 to Run 3 (10.9 m to 13.9 m)

Scale



| | | | | | |
|---|------|--------|---------------|-----|------|
| PROJECT | | | | | |
| Highway 49 Quinte Skyway Bridge | | | | | |
| TITLE | | | | | |
| Bedrock Core Photographs Boreholes 17-1 and 17-2 | | | | | |
| PROJECT No. 1663816 / 9003 | | | FILE No. ---- | | |
| DESIGN | NH | Nov 17 | SCALE | NTS | REV. |
| CADD | -- | | FIGURE B3 | | |
| CHECK | AMP | Nov 17 | | | |
| REVIEW | SEMP | Nov 17 | | | |





APPENDIX C

Analytical Laboratory Test Results

CLIENT NAME: GOLDER ASSOCIATES LTD.
100 SCOTIA COURT
WHITBY, ON L1N8Y6
(905) 723-2727

ATTENTION TO: Anastasia Poliacik

PROJECT: 1663816 Assignment #9

AGAT WORK ORDER: 17T275591

SOIL ANALYSIS REVIEWED BY: Amanjot Bhela, Inorganic Coordinator

DATE REPORTED: Oct 31, 2017

PAGES (INCLUDING COVER): 5

VERSION*: 1

Should you require any information regarding this analysis please contact your client services representative at (905) 712-5100

*NOTES

All samples will be disposed of within 30 days following analysis. Please contact the lab if you require additional sample storage time.



AGAT Laboratories

Certificate of Analysis

AGAT WORK ORDER: 17T275591

PROJECT: 1663816 Assignment #9

5835 COOPERS AVENUE
MISSISSAUGA, ONTARIO
CANADA L4Z 1Y2
TEL (905)712-5100
FAX (905)712-5122
<http://www.agatlabs.com>

CLIENT NAME: GOLDER ASSOCIATES LTD.

SAMPLING SITE:

ATTENTION TO: Anastasia Poliacik

SAMPLED BY:

Corrosivity Package

DATE RECEIVED: 2017-10-23

DATE REPORTED: 2017-10-31

| | | SAMPLE DESCRIPTION: | | 17-1 S9 | 17-2 S9 |
|-------------------------------|----------|---------------------|-------|------------|------------|
| | | SAMPLE TYPE: | | Soil | Soil |
| | | DATE SAMPLED: | | 2017-10-18 | 2017-10-18 |
| Parameter | Unit | G / S | RDL | 8847472 | 8847474 |
| Sulfide (S2-) | % | | 0.05 | <0.05 | <0.05 |
| Chloride (2:1) | µg/g | | 2 | 14 | 77 |
| Sulphate (2:1) | µg/g | | 2 | 10 | 18 |
| pH (2:1) | pH Units | | NA | 8.35 | 8.15 |
| Electrical Conductivity (2:1) | mS/cm | | 0.005 | 0.147 | 0.262 |
| Resistivity (2:1) | ohm.cm | | 1 | 6800 | 3820 |
| Redox Potential (2:1) | mV | | 5 | 200 | 185 |

Comments: RDL - Reported Detection Limit; G / S - Guideline / Standard

8847472-8847474 EC/Resistivity, pH, Chloride, Sulphate and Redox Potential were determined on the extract obtained from the 2:1 leaching procedure (2 parts DI water: 1 part soil).

*Sulphide analyzed at AGAT 5623 McAdam

Certified By:

Amanjot Bhela



Quality Assurance

CLIENT NAME: GOLDER ASSOCIATES LTD.

PROJECT: 1663816 Assignment #9

SAMPLING SITE:

AGAT WORK ORDER: 17T275591

ATTENTION TO: Anastasia Poliacik

SAMPLED BY:

Soil Analysis

| RPT Date: Oct 31, 2017 | | | DUPLICATE | | | Method Blank | REFERENCE MATERIAL | | | METHOD BLANK SPIKE | | | MATRIX SPIKE | | |
|-------------------------------|---------|-----------|-----------|--------|------|--------------|--------------------|-------------------|-------|--------------------|-------------------|-------|--------------|-------------------|-------|
| PARAMETER | Batch | Sample Id | Dup #1 | Dup #2 | RPD | | Measured Value | Acceptable Limits | | Recovery | Acceptable Limits | | Recovery | Acceptable Limits | |
| | | | | | | | | Lower | Upper | | Lower | Upper | | Lower | Upper |
| Corrosivity Package | | | | | | | | | | | | | | | |
| Sulfide (S2-) | 8847474 | 8847474 | < 0.05 | < 0.05 | NA | < 0.05 | 98% | 80% | 120% | | | | | | |
| Chloride (2:1) | 8847472 | 8847472 | 14 | 14 | 0.0% | < 2 | 109% | 80% | 120% | 100% | 80% | 120% | 96% | 70% | 130% |
| Sulphate (2:1) | 8847472 | 8847472 | 10 | 10 | 0.0% | < 2 | 95% | 80% | 120% | 101% | 80% | 120% | 102% | 70% | 130% |
| pH (2:1) | 8847472 | 8847472 | 8.35 | 8.42 | 0.8% | NA | 101% | 90% | 110% | NA | | | NA | | |
| Electrical Conductivity (2:1) | 8847472 | 8847472 | 0.147 | 0.152 | 3.3% | < 0.005 | 96% | 90% | 110% | NA | | | NA | | |
| Redox Potential (2:1) | 8847472 | 8847472 | 200 | 192 | 4.1% | < 5 | 103% | 70% | 130% | NA | | | NA | | |

Comments: NA signifies Not Applicable.

Duplicate Qualifier: As the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Certified By:

Amanjot Bhela

Method Summary

CLIENT NAME: GOLDER ASSOCIATES LTD.

AGAT WORK ORDER: 17T275591

PROJECT: 1663816 Assignment #9

ATTENTION TO: Anastasia Poliacik

SAMPLING SITE:

SAMPLED BY:

| PARAMETER | AGAT S.O.P | LITERATURE REFERENCE | ANALYTICAL TECHNIQUE |
|-------------------------------|---------------|---|---------------------------|
| Soil Analysis | | | |
| Sulfide (S ²⁻) | MIN-200-12025 | ASTM E1915-09 | GRAVIMETRIC |
| Chloride (2:1) | INOR-93-6004 | McKeague 4.12 & SM 4110 B | ION CHROMATOGRAPH |
| Sulphate (2:1) | INOR-93-6004 | McKeague 4.12 & SM 4110 B | ION CHROMATOGRAPH |
| pH (2:1) | INOR 93-6031 | MSA part 3 & SM 4500-H+ B | PH METER |
| Electrical Conductivity (2:1) | INOR-93-6036 | McKeague 4.12, SM 2510 B | EC METER |
| Resistivity (2:1) | INOR-93-6036 | McKeague 4.12, SM 2510 B, SSA #5 Part 3 | CALCULATION |
| Redox Potential (2:1) | | McKeague 4.12 & SM 2510 B | REDOX POTENTIAL ELECTRODE |



APPENDIX D

Non-Standard Special Provisions (NSSPs)

CAISSONS – Item No.

Non-Standard Special Provision

Amendment to OPSS.PROV 903, April 2016

Deep Foundations

903.07 CONSTRUCTION

Section 903.07.03.02 of OPSS.PROV 903 shall be amended by the addition of the following:

The Contactor shall be alerted to the potential presence of cobbles and boulders within the fill. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations, installation of caissons.

The contractor is advised that the limestone bedrock at this site is strong, and moderately abrasive. Appropriate equipment and procedures shall be used to reach the bedrock surface, for socket formation (if required), and for removal of loose/fractured rock and preparation of the base.

Section 903.07.03.03 of OPSS.PROV 903 shall be amended by the addition of the following:

The contractor shall clean the base of the caisson prior to placement of concrete.

Inspection of the base by the QVE shall be conducted using a video camera, after sufficient time has elapsed to allow sediment in the water within the caisson hole to settle to allow clear visibility and verification of the condition of the base.

The Contractor is advised that water may be perched within the gravelly sand fill, and that the clayey silt fill may contain zones of water-bearing soil, including at/above the bedrock interface. The construction methods and techniques shall be the responsibility of the Contractor, but temporary liners shall be used to support the sidewalls to permit camera inspection of the cleaned base, and consideration shall be given to using tremie techniques to place the concrete where conditions warrant.

Section 903.07.03.06 of OPSS.PROV 903 shall be amended by the addition of the following:

If the steel reinforcement cage is prefabricated, it shall be prefabricated with an additional 1 m length beyond that shown on the contract drawings, in order to account for variations in the depth to the bedrock surface, and variations related to cleaning fractured bedrock at/below the bedrock surface. The top of the steel reinforcement cage shall then be cut to the required height after installation and prior to placement of concrete. Alternatively, any additional depth required for bedrock depth/cleaning below the design elevation can be accommodated by the placement of mass concrete of minimum 30 MPa compressive strength.

EXCAVATION FOR STRUCTURE – Item No.

Non-Standard Special Provision

Amendment to OPSS 902, November 2010

Excavating and Backfilling - Structures

902.07 CONSTRUCTION

Section 902.07 of OPSS 902 shall be amended by the addition of the following:

The Contactor shall be alerted to the potential presence of cobbles and boulders within the fill. Considerations of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for excavations and temporary protection systems.

DEWATERING STRUCTURE EXCAVATIONS - Item No.

Special Provision

Amendment to OPSS 902, November 2010

902.02 REFERENCES

Section 902.02 of OPSS 902 is amended by the addition of the following:

Ontario Provincial Standard Specifications, Construction

OPSS 517 Dewatering
OPSS 805 Temporary Erosion and Sediment Control Measures

902.03 DEFINITIONS

Section 903.03 of OPSS 902 is amended by the addition of the following:

Automatic Transfer Switch means as defined in OPSS 517.

Cofferdam means as defined in OPSS 539.

Cut-Off Wall means as defined in OPSS 517.

Design Storm Return Period means as defined in OPSS 517.

Dewatering System means as defined in OPSS 517.

Groundwater Control System means as defined in OPSS 517.

Plug means as defined in OPSS 517.

Sediment means as defined in OPSS 517.

Sediment Control Measure means as defined in OPSS 517.

Temporary Flow Passage System means as defined in OPSS 517.

Unwatering means as defined in OPSS 517.

Vegetated Discharge Area means as defined in OPSS 517.

Waterbody means as defined in OPSS 517.

Watercourse means as defined in OPSS 517.

902.04 DESIGN AND SUBMISSION REQUIREMENTS

902.04.01 Design Requirements

902.04.01.01 Dewatering

Clause 902.04.01.01 of OPSS 902 is deleted in its entirety and replaced with the following:

A dewatering system shall be designed to control water and the flow of water into the excavation, prevent disturbance of the foundation, permit the placing of concrete in the dry, and complete the excavating and backfilling for structures work.

When the system includes temporary flow passage system, the system shall be designed, as a minimum, for a 2 year design storm return period, and groundwater discharge. A longer return period shall be used when determined appropriate for the work.

The dewatering system shall be according to the design requirements specified in OPSS 517.

902.04.02 Submission Requirements

Subsection 902.04.02 of OPSS 902 is deleted in its entirety and replaced with the following:

902.04.02.01 Working Drawings

Working Drawings for the dewatering system shall be according to OPSS 517.

902.04.02.02 Preconstruction Survey

When a groundwater control system by wells or a well point system will be used, a condition survey of property and structures that may be affected by the work shall be carried out. The condition survey shall include the location and condition of adjacent properties, buildings, underground structures, water wells, Utilities, and structures, within a distance of 300 metres from the groundwater control system. In addition, all water wells used as a supply of drinking water and located within this distance shall be tested for compliance with Ontario Drinking Water Quality Standards.

Water wells within the preconstruction survey distance can be located using the website <https://www.ontario.ca/environment-and-energy/map-well-records> or its successor site.

Copies of the condition survey and water quality test results shall be submitted to the Contract Administrator prior to the operation of the groundwater control system.

902.04.02.03 Milestone Inspections

The Quality Verification Engineer shall witness the following Interim Inspections of the work:

- a) Dewatering of excavation for structure.
- b) Completion of excavation for foundation.
- c) Excavation for backfill and frost tapers.

d) Backfilling.

A copy of the written permission to proceed shall be submitted to the Contract Administrator prior to commencement of the successive operation.

902.07 CONSTRUCTION

Subsection 902.07.04 of OPSS 902 is deleted in its entirety and replaced with the following:

902.07.04 Dewatering Structure Excavation

902.07.04.01 General

The dewatering systems shall be constructed and operated according to the Working Drawings.

Activation and deactivation of a temporary flow passage system, if applicable, shall be according to OPSS 517.

The dewatering system shall be continuously operational to control buoyancy forces until such forces can be resisted by backfill and structure self-weight, to keep excavations stable, to avoid erosion impacts from the release of accumulated water, and to keep the work area in the condition required to complete the associated work as specified in the Contract Documents.

When a temporary flow passage system is to remain operational through a seasonal shutdown period, the Contractor shall be responsible for any maintenance or repair costs due to the system during the seasonal shutdown period.

Temporary erosion and sediment control measures, including controlling the discharge of water, shall be according to OPSS 805. Measures not specified in OPSS 805 shall be according to the Working Drawings. Temporary erosion and sediment control measures and cover material to protect exposed soils, as required by the Working Drawings, shall be installed as soon as is practical.

Stranded fish shall be managed as specified in the Contract Documents.

Unwatering shall be carried out as necessary.

Water suspected of being contaminated as indicated by visual or olfactory observations shall be reported to the Contract Administrator.

Dewatering and temporary flow passage systems shall be discontinued in a manner that does not disturb any structure, pipeline, or flow channel. Operation of the dewatering system shall be shut down according to the procedures specified in the Working Drawings, where applicable.

902.07.04.02 Discharge of Water

The discharge of water shall be according to OPSS 517.

902.07.04.03 Monitoring

Monitoring shall be according to OPSS 517.

902.07.04.04 System Amendments

Amendments to stop any displacement, damage, soil loss or erosion due to the operation of the dewatering system shall be according to OPSS 517.

902.07.04.05 Removal

Removal of dewatering system and temporary flow passage system components shall be according to OPSS 517.

NOTES TO DESIGNER:

Designer Fill-Ins

- * Fill in the design storm return period according to MTO Drainage Design Standard TW-1.
- ** Fill in the preconstruction survey distance as recommended by the foundation engineer.

WARRANT: Include with this standard tender item **only** on the recommendation of a foundation engineer.

CUSTODIAN: Tony Sangiuliano, MERO - Foundation Group.

VIBRATION MONITORING - Item No.

Special Provision

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- 3.0 DEFINITIONS**
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- 5.0 MATERIALS - Not Used**
- 6.0 EQUIPMENT**
- 7.0 CONSTRUCTION**
- 8.0 QUALITY ASSURANCE - Not Used**
- 9.0 MEASUREMENT FOR PAYMENT - Not Used**
- 10.0 BASIS OF PAYMENT**

1.0 SCOPE

This special provision describes requirements for vibration monitoring during caisson installation and temporary protection system installation for the abutment rehabilitation of the Bay of Quinte Skyway Bridge.

2.0 REFERENCES

The subsurface conditions at the site are described in the following Foundation Investigation Report for GWP 4071-10-00/WP 4063-10-01:

Foundation Investigation Report, Abutment Rehabilitation for Quinte Skyway Bridge, Highway 49, Site 11-245, Hastings County and Prince Edward County, Ontario, Assignment No. 9, WP 212-00-02.

3.0 DEFINITIONS

For the purpose of this specification, the following definitions apply:

Quality Verification Engineer (QVE) means an Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope

to the Contract. The QVE shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

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

- a) Equipment and methods used by the Contractor to perform the work, that may cause undue vibration.
- b) Qualifications of vibration monitoring specialist.
- c) Details regarding proposed instrumentation.
- d) Proposed location of instruments on the existing bridge deck, existing abutment footings, residences, utilities, wells, or other vibration sensitive structures within a 300 m radius from the north and south abutment areas, as applicable.
- e) Proposed frequency of readings.
- f) Action plan to be taken to adjust deep foundation and protection system installation methods if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

The contractor shall provide up to 30 individual vibration monitoring instruments, with adequate protection to prevent damage to the monitoring instruments throughout the duration of their use .

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, docks, and facilities within 300 m of each abutment location.

7.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each of the north and south sides of the bridge, shall be completed a minimum of two (2) weeks prior to commencement of installation of deep foundations and temporary protection systems. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation or temporary protection system installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

- a) Type of structure, including type of construction and if possible, the date when built.
- b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
- c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

7.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey is required within two (2) months of completion of the installation of deep foundations and temporary protection systems at each of the north and south sides of the bridge.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

- a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
- b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
- c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there has been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations and temporary protection systems.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the existing bridge deck at each abutment location, on the ground surface in front of each abutment, and at the ground surface at select locations within a 300 m radius of each abutment as shown on the Contract Drawings. The Contractor shall take readings continuously during caisson installation and during installation of temporary protection systems, as applicable at the abutment locations, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on the existing bridge structure shall not exceed 100 mm/s (Peak Particle Velocity). Those measured on the private structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities shall not exceed 10 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

7.3 Records

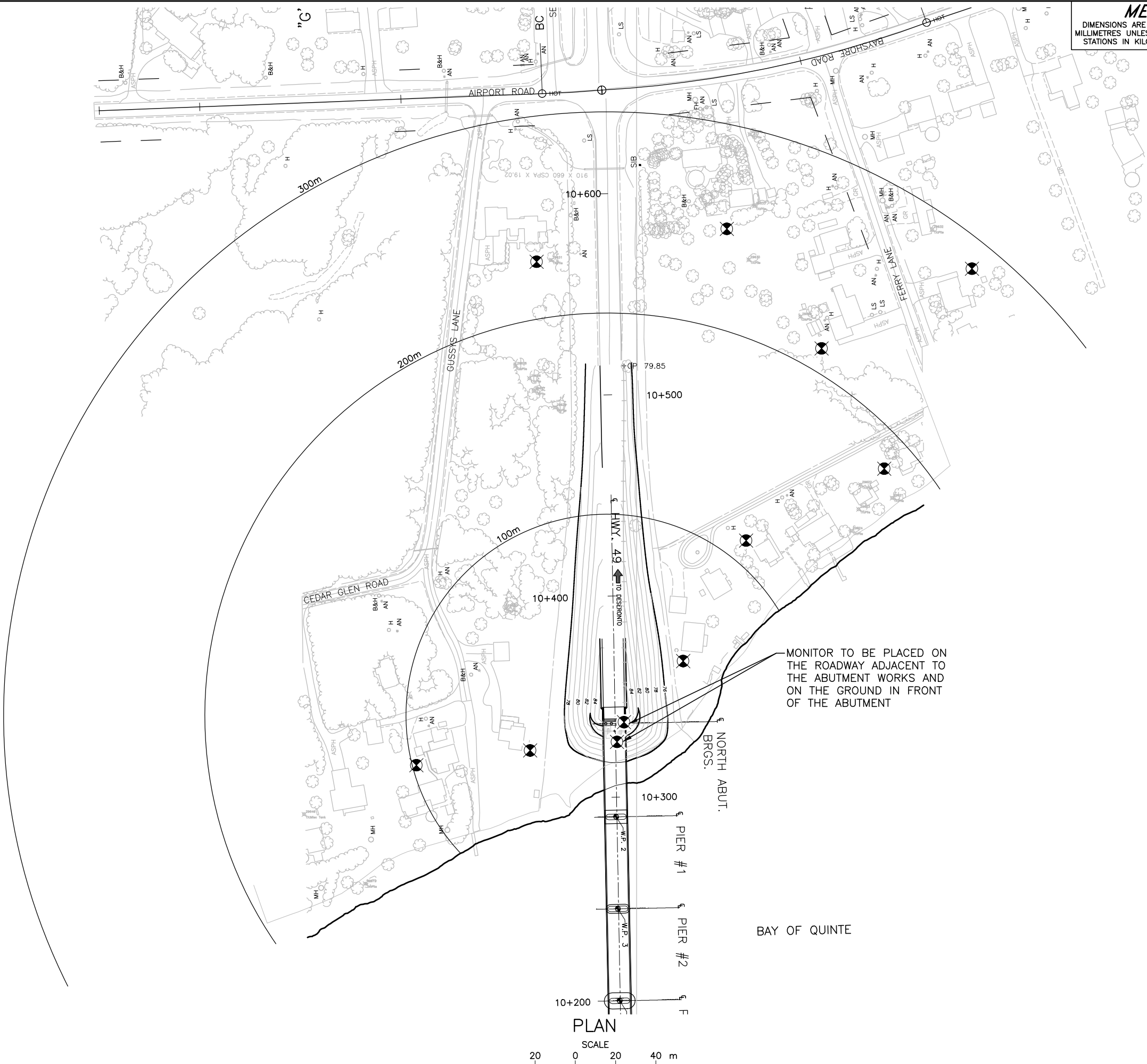
The Contractor shall submit details of the vibration monitoring as follows:

- a) The time/duration of each reading.
- b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.
- c) Details of exceedances and modifications to operations.
- d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

10.0 BASIS OF PAYMENT

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

The Contractor shall be responsible for the management of all claims and payment arising from all effects, directly or indirectly related to the installation of deep foundations and temporary protection systems.

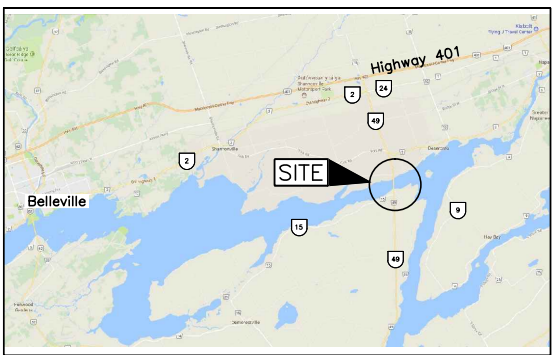


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No. 2017-4047
WP No. 212-00-02

HIGHWAY 49
QUINTE SKYWAY BRIDGE
MONITORING LOCATION PLAN
NORTH ABUTMENT

SHEET



KEY PLAN
N.T.S

LEGEND

Vibration Monitor (Approximate Location)

NOTES

1. Vibration monitor locations are shown for illustration purposes only. The final locations must be agreed upon by the Contractor and Contract Administrator prior to commencement of monitoring.
2. Vibration monitors shall be strategically located near vibration sensitive structures.
3. Additional monitors to be added if required for coverage.
4. Vibration monitoring shall be in accordance with non-standard special provision.
5. Contractor is responsible for obtaining permission to enter private property.

REFERENCE

Base plans provided in digital format by Morrison Hershfield, drawing file no. WP4063-10-01 Hwy 49 Skyway Bridge.dwg, received OCT 24, 2017.



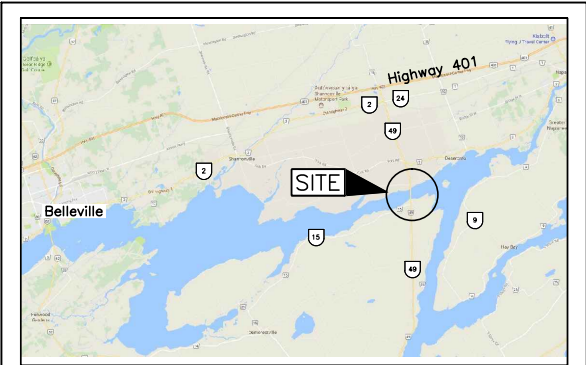
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| | | | |
| NO. | DATE | BY | REVISION |
| Geocres No. 31C-265 | | | |
| HWY. 49 | PROJECT NO. 1663816/9004 | | DIST. KINGSTON |
| SUBM'D. | CHKD. AP | DATE: 1/8/2018 | SITE: 11-245 |
| DRAWN: TB | CHKD. SEMP | APPD. LCC | DWG. M1 |

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

CONT No. 2017-4047
WP No. 212-00-02

HIGHWAY 49
QUINTE SKYWAY BRIDGE
MONITORING LOCATION PLAN
SOUTH ABUTMENT

SHEET



KEY PLAN
N.T.S.



LEGEND

- Vibration Monitor (Approximate Location)

NOTES

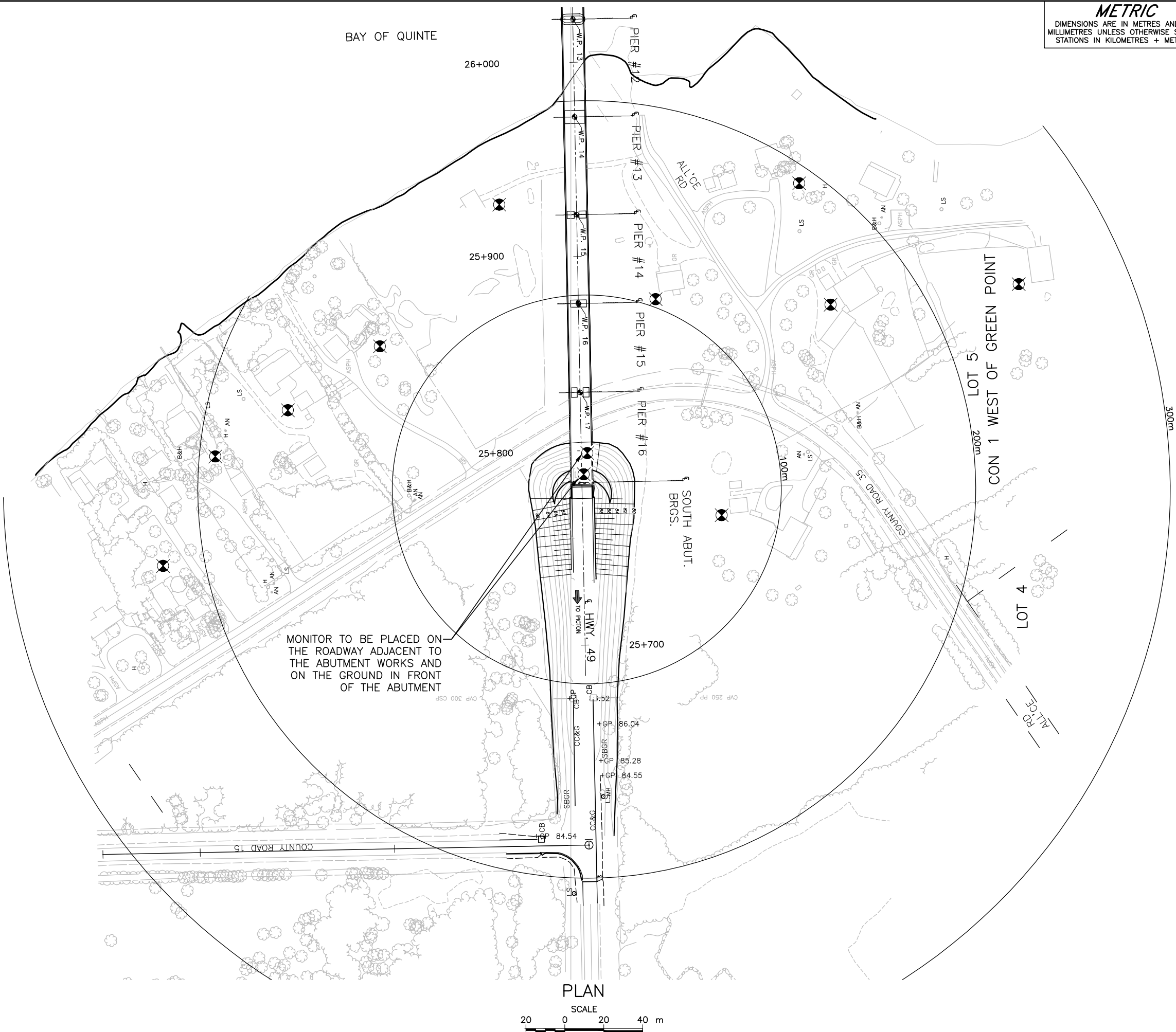
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|---------------------|------------|--------------------------|----------------|
| | | | |
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| HWY. 49 | | PROJECT NO. 1663816/9004 | DIST. KINGSTON |
| SUBM'D. | CHKD. AP | DATE: 1/8/2018 | SITE: 11-245 |
| DRAWN: TB | CHKD. SEMP | APPD. LCC | DWG. M2 |



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