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**FOUNDATION INVESTIGATION
AND DESIGN REPORT
HIGHWAY 401-TRENT RIVER BRIDGE
REHABILITATION AND WIDENING
TRENTON, ONTARIO
G.W.P. 196-99-00**

Submitted to:

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PART A

FOUNDATION INVESTIGATION REPORT

**HIGHWAY 401-TRENT RIVER BRIDGE
REHABILITATION AND WIDENING
TRENTON, ONTARIO
G.W.P. 196-99-00**

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by LEA Consulting Ltd. (LEA) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the detailed design for the rehabilitation of Highway 401 from 1.0 km west of Quinte West Road 33 to 0.7 km east of Glen Miller Road, in Trenton, Ontario. Foundation engineering services are required for the rehabilitation and widening of the Trent River bridge, and for three culverts at the Highway 401-Glen Miller Road interchange.

This report addresses the foundation engineering services for the rehabilitation and superstructure widening of the Trent River bridge, including widening of the existing wing walls and immediate approach embankments. A foundation investigation was completed at the site in 2007; in addition, use has been made of boreholes drilled at the site in 1954 as part of a foundation investigation by Department of Highways, Ontario for the original bridge structure (MTO GEOCREs No. 31C-42, *Report of Foundation Investigation, Highway 401 at Trent River*, dated 1954).

The terms of reference and scope of work for the foundation investigation are outlined in MTO's Request for Proposal for GWP No. 196-99-00, Purchase Order No. 4006-E-0016, dated November 2006, and in Section 6.8 of LEA's *Technical Proposal* for G.W.P. 196-99-00.

2.0 SITE DESCRIPTION

The Trent River bridge (MTO Structure Site No. 11-185) is located on Highway 401 near Trenton, in the City of Quinte West, in the County of Hastings. The existing bridge, which was constructed in 1956, is a 188.3 m long, five-span structure consisting of a reinforced concrete deck on variable depth steel girders for the four westerly spans, and a steel rigid frame for the easterly span.

The natural ground surface on the west side of the bridge is at approximately at Elevation 90.5 m to 91 m, while the natural ground surface on the east side of the bridge is at about Elevation 81 m. The Highway 401 grade is at about Elevation 91.3 m to 89.7 m, declining eastward, with an approximately 0.5 m to 1 m high approach embankment on the west side of the bridge, and an approximately 8.7 m high approach embankment on the east side of the bridge. At the west abutment, the bedrock was cut near-vertically to approximately Elevation 84 m in front of the abutment; Quinte West Road 33 runs at this grade along the west edge of the Trent River, between the west abutment and Pier A. Pier A is located at the west edge of the Trent River, Piers B and C are located within the river, and Pier D is located at the east edge of the Trent River.

Based on the available design drawings for the existing bridge (Sidney Township Bridge No. 1 and 2 Over the Trent Canal, dated October 1955), the existing abutments and Piers A to D are supported on spread footings that are founded on “solid limestone bedrock”, at a depth of approximately 0.7 m to 5.8 m below the bedrock surface. The following table presents the approximate bedrock elevation and the founding elevation for each foundation element, based on the 1955 design drawings for the existing bridge.

<i>Foundation Element</i>	<i>Bedrock Surface Elevation</i>	<i>Footing Founding Elevation</i>
West Abutment	89.2 m	83.4 m
Pier A	77.7 m	74.7 m
Pier B	75.9 m	74.7 m
Pier C	77.1 m	75.2 m
Pier D	77.8 m	76.8 m
East Abutment	78.9 m (North End) 77.7 m (South End)	77.0 m

3.0 INVESTIGATION PROCEDURES

3.1 Borehole Investigation

The field work for the Trent River bridge site was carried out between August 9 and 14, 2007, at which time four boreholes (Boreholes 07-1 to 07-4) were advanced to investigate the subsurface conditions at the east and west abutments and associated wing walls, along with the area around the east pier. The borehole locations are shown on Drawing 1, along with the locations of Boreholes 54-3, 54-4, 54-7, 54-10 and 54-11, which were advanced at the site as part of the 1954 investigation by DHO.

Boreholes 07-1 to 07-4 were advanced using a track-mounted CME-55 drill rig, supplied and operated by Walker Drilling Ltd. of Utopia, Ontario. The boreholes were advanced through the overburden using 114 mm outside diameter continuous flight solid stem augers, and into the bedrock by NQ coring. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outer diameter split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedure. Samples from the bedrock were obtained using an 'NQ' size rock core barrel. Boreholes 07-1 to 07-3 were advanced to depths of between 7.5 m and 9.5 m, whereas Borehole 07-4 was advanced to a depth of 17.2 m.

The groundwater conditions in the open boreholes were observed during the drilling operations. A standpipe piezometer was installed in Boreholes 07-1 and 07-3, in order to monitor the groundwater level at the site. The standpipe piezometer consists of a 1.5 m long, 50 mm diameter slotted screen installed within a sand pack, and sealed into the overlying soils, then backfilled to ground surface with bentonite pellets. The piezometer installation details and recorded water level readings are included in the Record of Borehole sheets. For boreholes in which no piezometer was installed, the boreholes were backfilled to ground surface using bentonite pellets as per Ontario Regulation 128 (amendment to Ontario Regulation 903).

The field work was supervised throughout by a member of Golder's technical staff, who located the boreholes in the field, arranged for the clearance of underground services, supervised the drilling, sampling and in situ testing operations, logged the boreholes, and examined and cared for the samples obtained. The samples were identified in the field, placed in appropriate containers, labelled and transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and geotechnical classification testing (water content determinations, Atterberg limit tests, and grain size distribution analyses on soil samples, and point load testing and unconfined compressive strength testing on bedrock samples). All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The northing and easting coordinates and ground surface elevations of the as-drilled borehole locations were surveyed in the field by J.D. Barnes, Ontario Land Surveyors. The borehole

locations, based on the MTM NAD87 coordinate system, and the ground surface elevations at the borehole locations referenced to the geodetic datum, are summarized below.

<i>Borehole Number</i>	<i>MTM NAD83 Northing (m)</i>	<i>MTM NAD83 Easting (m)</i>	<i>Ground Surface Elevation (m)</i>
07-01	4,888,064.5	217,225.4	90.7
07-02	4,888,078.2	217,245.9	83.8
07-03	4,888,165.7	217,393.0	81.3
07-04	4,888,183.6	217,410.2	89.8

3.2 Underwater Inspection of Bridge Piers

An underwater inspection of the Trent River bridge piers was completed on July 31 and August 1, 2007 by ASI Group Ltd. (ASI) of St. Catharines, Ontario; this included inspection of the east face of Pier A (located at the west edge of the Trent River), Piers B and C, and the west face of Pier D (located at the east edge of the Trent River).

ASI's four-person commercial dive crew performed and videotaped the underwater inspections, while in two-way voice and video contact with Golder personnel on ASI's diving support vessel. During the inspection, ASI's crew examined the river bottom around each pier and, where bedrock was not exposed, probed the thickness of river sediment/rip-rap above the top of the footing and observed scour conditions where present. The ASI crew also observed the condition of the concrete below the waterline. Golder's on-site representative examined the concrete condition at and immediately above the waterline.

All diving operations were carried out in accordance with the Ontario Ministry of Labour Diving Regulation (Ontario Regulation 629/94, amended to Ontario Regulation 115/04).

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The Highway 401-Trent River area lies within the Iroquois Plain physiographic region, as delineated in *The Physiography of Southern Ontario*¹; the Iroquois Plain extends around the western and northern shores of Lake Ontario from the Niagara Escarpment to the Trent River.

The soils within this physiographic region represent the flat to undulating lake bed and beaches of the former glacial Lake Iroquois, which occupied the Lake Ontario basin during the last glacial recession. The soils in the Iroquois Plain are typically comprised of glaciolacustrine clays and silts, though deposits of sand to sand and gravel are also known to be present; in particular, sand to sand and gravel soils are often present within river valleys, such as the Trent River valley. The overburden soils are underlain by limestone bedrock of the Trenton Group.

4.2 Subsurface Soil and Bedrock Conditions

Four boreholes were advanced adjacent to the Trent River bridge abutments and retaining walls, at the locations shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of laboratory tests are given on the Record of Borehole sheets; the results of the geotechnical laboratory testing are also presented on Figures 1 to 5. The stratigraphic boundaries shown on the borehole records, and on the interpreted stratigraphic profile 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.

On the west side of the river, Borehole 07-1 (which was advanced to the west of the west abutment) encountered approximately 1.5 m of compact to dense sand and gravel overlying bedrock; the bedrock surface was encountered in this borehole at approximately Elevation 89.2 m. Borehole 07-2, which was advanced to the east of the west abutment from the Quinte West Road 33 cut grade, encountered about 1.4 m of compact to very dense sand and gravel fill overlying bedrock, which has been cut at this location down to Elevation 82.5 m.

On the east side of the river, Borehole 07-3 was advanced from the original ground surface and encountered 4.3 m of compact to very dense sand and gravel to gravel, overlying bedrock; Borehole 07-4 was advanced from the Highway 401 grade and encountered 10.7 m of compact to dense sand and gravel fill, overlying a thin layer of hard silty clay, in turn underlain by bedrock.

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

The surface of the bedrock was encountered in these two boreholes at Elevation 77.0 m and 78.5 m, rising eastward.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Asphalt / Fill

Borehole 07-4 was drilled through the existing Highway 401 pavement structure, east of the east abutment, and encountered a 300 mm thick layer of asphalt. At this location, the asphalt is underlain by 10.4 m of sand and gravel fill, comprising the east approach embankment fill (including fill for the original excavation for the east abutment). The base of the fill was encountered in this borehole at a depth of 10.7 m (Elevation 79.1 m).

In Borehole 07-2, which was drilled from the Quinte West Road 33 grade, about 1.4 m of sand and gravel fill was encountered immediately below the ground surface; the base of the fill was encountered at Elevation 82.5 m.

The sand and gravel fill contains trace silt and clay; the results of grain size distribution tests on three selected samples of the sand and gravel fill are shown on Figure 1. Measured Standard Penetration Test (SPT) “N” values of 10 to 56 blows per 0.3 m of penetration indicate that the fill has a compact to very dense relative density.

4.2.2 Sand and Gravel to Gravel

A 1.5 m thick layer of sand and gravel was encountered immediately below the ground surface in Borehole 07-1, which was located south of the Highway 401 embankment, west of the west abutment. A 4.3 m thick layer of sand and gravel to gravel was encountered immediately below the ground surface in Borehole 07-3, which was advanced from the original ground surface on the east side of the river. These deposits directly overlie the bedrock; the base of the deposits was encountered at Elevation 89.2 m west of the west abutment, and at Elevation 77.0 m east of the river.

The deposit consists of sand and gravel containing some silt and trace clay, to gravel containing some sand and trace silt and clay; cobbles were encountered within the lower portion of the gravel layer in Borehole 07-3, as noted on the record for this borehole. The results of grain size distribution tests completed on two samples of sand and gravel and one sample of gravel are shown on Figure 2.

The measured SPT “N” values within the sand and gravel to gravel deposits range from 17 to 55 blows per 0.3 m of penetration, indicating a compact to very dense relative density; one SPT “N”

value of 50 blows per 0.05 m of penetration was measured in the lower portion of the gravel layer in Borehole 07-3, and is attributed to the presence of cobbles within this portion of the deposit.

4.2.3 Silty Clay

A 0.6 m thick layer of silty clay with sand and gravel was encountered below the fill in Borehole 07-4. The surface of the silty clay was encountered at a depth of about 10.7 m (Elevation 79.1 m), and its base was encountered at Elevation 78.5 m.

This deposit consists of silty clay with sand and gravel; the result of a grain size distribution test on the recovered sample of this deposit is shown on Figure 3. Atterberg limits testing was conducted on the recovered sample, and measured a plastic limit of 19 per cent, a liquid limit of 43 per cent, and a plasticity index of 24 per cent. These results, which are plotted on a plasticity chart on Figure 4, confirm that this material is a silty clay of intermediate plasticity.

One SPT “N” value of 50 blows per 0.03 m of penetration was measured. This value is considered to result from the presence of gravel within the sample; however, the silty clay soil does have a hard consistency.

4.2.4 Limestone / Calcareous Shale Bedrock

Argillaceous (shaley) limestone to calcareous shale bedrock underlies the soil deposits at this site. On the west side of the river, west of the west abutment, the bedrock surface was encountered at Elevation 89.2 m, and this was cut to approximately Elevation 82.5 m in front of the west abutment; through the river, the bedrock surface was encountered between Elevations 76.0 m and 77.8 m; and east of the river, the bedrock surface was encountered between Elevation 77.0 m and 78.5 m. The following table summarizes the bedrock surface elevation as encountered in the boreholes; the bedrock was confirmed by coring in all of these boreholes.

<i>Borehole Location</i>	<i>Borehole Number</i>	<i>Bedrock Surface Elevation</i>
West of West Abutment	07-1	89.2 m
East of West Abutment	07-2	82.5 m
Between West Abutment and Pier A	3	84.7 m*
Between Pier A and B	4	76.0 m
Between Pier B and C	7	77.0 m
Pier D	10	77.8 m
Pier D	07-3	77.0 m
East Abutment	11	77.8 m
East of East Abutment	07-4	78.5 m

* In 1954, prior to formation of rock cut on west side of Trent River

A description of some of the terms used in the description of the bedrock is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet which precedes the Record of Borehole sheets included with this report.

The bedrock consists of light grey to black, slightly weathered to fresh, thinly to medium bedded, weak to extremely strong shaley limestone to calcareous shale of the Trenton formation. Rock Quality Designation (RQD) values of 0 to 25 per cent were measured in the upper approximately 1.0 m of the bedrock, except in Borehole 07-1 where the upper 3.0 m of bedrock had RQD values of 0 to 17 per cent, indicative of very poor quality. Below this level in all of the 2007 boreholes, the RQD was generally between 50 and 100 per cent, indicative of fair to excellent quality. The discontinuities observed in the rock core are typically horizontal, and generally associated with the bedding planes.

Unconfined compressive strength testing was performed on four bedrock core samples (one from each borehole), and measured unconfined compressive strengths ranging from 37 MPa to 52 MPa. Diametral and axial point load strength tests were performed on selected samples of the rock core from Boreholes 07-1 to 07-4; the measured point load strength index values are shown on Table 1 following the text of this report, and are summarized in the following table:

<i>Rock Type</i>	<i>Point Load Test Type</i>	<i>Point Load Strength Index</i>	<i>Correlated Unconfined Compressive Strength</i>	<i>Intact Rock Strength Classification</i>
Argillaceous limestone	Diametral	0.3 – 12.4 MPa	6 – 261 MPa	Weak to extremely strong
Argillaceous limestone	Axial	3.6 – 6.7 MPa	82 – 155 MPa	Strong to very strong
Calcareous shale	Diametral	0.2 – 2.1 MPa	5 – 45 MPa	Weak to medium strong
Calcareous shale	Axial	4.2 MPa	97 MPa	Strong

4.3 Groundwater Conditions

A standpipe piezometer was installed in each of Boreholes 07-1 and 07-3, sealed within the bedrock or overlying sand and gravel; details of the piezometer installations are shown on the borehole records. The water levels measured in the piezometers are summarized below.

<i>Date</i>	<i>Groundwater Elevation</i>	
	<i>Borehole 07-1</i>	<i>Borehole 07-3</i>
August 14, 2007	Dry at 88.6 m	80.0 m
October 16, 2007	Dry at 88.6 m	Not Measured

The measured water level in Borehole 07-3 is similar to the Trent River water level (Elevation 80.0 m) at the time of the measurements. The groundwater level in the area will be subject to seasonal fluctuations, and should be expected to rise during wet periods of the year, and with changes in the Trent River water level.

4.4 Underwater Inspection of Piers


Details of the observations made during the underwater pier inspection by ASI Group Ltd. (ASI) are provided in the factual report prepared by ASI, which is contained in Appendix B of this report.


As noted in the ASI report, the top of the existing pier footing could be observed only at the north end of Pier B, with a vertical exposure of approximately 0.3 m; the base of the footing was not visible at this location, and the river bottom composition adjacent to this area consisted of a thin layer of silt, sand and cobbles overlying bedrock. Over the southern portion of Pier B, and on all in-water faces of Piers A, C and D, the top of the existing pier footings was not exposed at the time of the ASI inspection. With the exception of the north end of Pier B as noted above, the pier footings were generally observed to be surrounded by placed stone and rip-rap overlying a mixture of silt, sand, gravel and cobbles. Between the pier footings, exposed bedrock was observed at the river bottom.

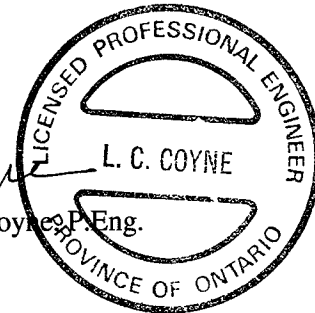
5.0 CLOSURE

This Foundation Investigation Report was prepared by Ms. Sandra McGaghran, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Designated MTO Contact for Golder, conducted an independent quality review of the report.

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JB/SMM/LCC/FJH/jb/smm/lcc

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PART B

FOUNDATION DESIGN REPORT

**HIGHWAY 401-TRENT RIVER BRIDGE
REHABILITATION AND WIDENING
TRENTON, ONTARIO
G.W.P. 196-99-00**

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides geotechnical/foundation design recommendations for the design and construction of the proposed rehabilitation of the Highway 401 bridge over Trent River, immediately to the east of Quinte West Road 33 in Trenton, Ontario. As part of the rehabilitation, the bridge superstructure is to be widened to the north and the south, including widening of both the west and east abutments and associated wing walls; the in-water pier foundations will not be widened under this assignment. The existing wing walls adjacent to both the west and east abutments and the existing retaining walls (stems and footings) adjacent to the west abutment will be removed before construction of the abutment widening.

The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during a subsurface investigation at this site. The interpretation and recommendations are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the foundations for the proposed rehabilitation/widening. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Foundation Options

It is understood that the widening of Highway 401 will involve widening of the abutments and replacement of the associated concrete wing walls. No widening will be carried out for the foundations for Piers A to D; the bridge superstructure only will be widened.

Based on the available design drawings for the existing bridge (Sidney Township Bridge No. 1 and 2 Over the Trent Canal, dated October 1955), the existing abutments and Piers A to D are supported on spread footings that are founded on “sound limestone bedrock” at a depth of about 0.7 m to 5.8 m below the bedrock surface. The following table presents the approximate bedrock elevation and the design founding elevation for each foundation element, taken from the 1955 design drawings for the existing bridge; the original bedrock surface elevation at the pier locations has been interpolated based on the 1954 borehole investigation results.

<i>Foundation Element</i>	<i>Bedrock Surface Elevation</i>	<i>Design Footing Founding Elevation</i>
West Abutment	89.2 m	83.4 m**
Pier A	77.7 m	74.7 m
Pier B	75.9 m	74.7 m
Pier C	77.1 m	75.2 m
Pier D	77.8 m	76.8 m
East Abutment	78.9 m (North End) 77.7 m (South End)	77.0 m

** A hand-dug test excavation was advanced by LEA adjacent to the south end of the west abutment, and determined that the west abutment footing was actually constructed with founding level below Elevation 82.9 m; this is consistent with the bedrock surface (Elevation 82.5 m) encountered in Borehole 07-2.

An assessment of the geotechnical resistance for the existing abutment and pier foundations is provided in Section 6.3.

Various alternatives for the widening of the abutment foundations and new wing walls/retaining walls have been considered; a summary of these alternatives is presented in Table 2 following the text of this report, and geotechnical recommendations for these alternatives are presented in the sections that follow.

- **West Abutment and Wing Walls/Retaining Walls:** On the west side of the river, where the bedrock surface is near the ground surface, shallow foundations founded on the bedrock are the preferred option for the support of the abutment widening and new wing walls/retaining walls. The abutment footings for the widening may be founded on the surface of the bedrock at the same level as the existing foundation; as noted in the table above, this is expected to be at approximately Elevation 82.5 m. Deep foundations are not suitable at this location due to the shallow depth to bedrock.
- **East Abutment and Wing Walls:** Shallow foundations supported on bedrock can be considered for widening of the east abutment and associated wing walls, although excavation to a depth of about 3 m below original ground surface (and deeper below the embankment fill) will be required, and this excavation will extend about 1 m to 2.5 m below the groundwater and river water level at the site, within sand and gravel to gravel soils that contain cobbles. Consideration could also be given to deep foundations (driven steel H-piles or caissons supported on/within the bedrock) for the abutment widening, wing wall replacement and new retaining walls. Since the deep foundation options would minimize excavation and groundwater control adjacent to the river, they would be preferable from a construction perspective; however, based on the structural assessment, deep foundations are not feasible for the widening and, therefore, spread footings supported on the bedrock are preferred.
- **East Abutment Retaining Walls:** If retaining walls are required adjacent to the widened east abutment/wing walls, new concrete retaining walls or RSS walls could be founded within the compact to very dense sand and gravel fill. These retaining wall foundation options are preferable, from a foundations perspective, to those presented above for the east abutment/wing walls, since they minimize excavation and groundwater control requirements.

6.3 Assessment of Existing Abutment and Pier Foundations

It is understood that the superstructure for the rehabilitated Trent River bridge will be widened by cantilevering, with no in-water widening of the piers. The following sections provide an assessment of the capacity of the existing piers.

6.3.1 Founding Elevations

Based on the available design drawings for the existing bridge (Sidney Township Bridge No. 1 and 2 Over the Trent Canal, dated October 1955), Piers A to D are supported on spread footings that are founded on “solid limestone bedrock”, at a depth of approximately 0.7 m to 5.8 m below the bedrock surface. The following table presents the approximate bedrock elevation and the founding elevation for each foundation element, taken from the 1955 design drawings for the existing bridge; the original bedrock surface elevation at the pier locations has been interpolated based on the 1954 borehole investigation results.

<i>Foundation Element</i>	<i>Bedrock Surface Elevation</i>	<i>Design Footing Founding Elevation</i>
Pier A	77.7 m	74.7 m
Pier B	75.9 m	74.7 m
Pier C	77.1 m	75.2 m
Pier D	77.8 m	76.8 m

6.3.2 Geotechnical Resistance

As discussed in Section 4.2.4, the measured unconfined compressive strength of the argillaceous limestone to calcareous shale bedrock ranged from 37 MPa to 52 MPa, while the correlated unconfined compressive strength (from point load testing) ranged from 5 MPa to 261 MPa. For the existing spread footings founded on bedrock, a factored geotechnical resistance of 3 MPa at Ultimate Limit States (ULS) may be used for design.

LEA has undertaken structural modelling of the piers to confirm that the superstructure widening (with no foundation widening) is feasible. For structural modelling of the piers, the pier loading was taken as 1,020 kN at Serviceability Limit States, acting over a 3.9 m (footing width) by 0.86 m (length of pier between “springs”) area. Based on this estimated SLS load and loading area and an estimated rock mass modulus of 20 GPa, an average spring constant value of 3,000 MPa/m is recommended for the pier modelling. However, since the subgrade reaction modulus is sensitive to the bedrock conditions under the piers, and since a detailed borehole investigation has not been completed in the water adjacent to the piers, it is recommended that the structural modelling also be checked for a range in spring constant from approximately 1,500 MPa/m to 5,000 MPa/m.

6.3.3 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the cast-in-place concrete footings and the bedrock should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the concrete and the bedrock may be taken as 0.70. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.4 Shallow Foundations on Bedrock

On the west side of the river, where the bedrock surface is near the ground surface, shallow foundations founded on the surface of the bedrock are considered the most appropriate option, from a foundations perspective, for the support of the abutment widenings and new wing walls/retaining walls.

On the east side of the river, the bedrock surface is at a depth of approximately 2.5 m to 4 m below the original ground surface, and about 11 m below the Highway 401 grade. Excavations in this area to construct spread footings on the bedrock surface will extend to about 1 m to 2.5 m below the groundwater level and the Trent River water level at the site, and dewatering/unwatering will be required; additional discussion regarding coffer dam construction and excavation dewatering/unwatering is provided in Section 6.11.3 (*Construction Considerations – Groundwater Control*).

6.4.1 Founding Elevations

The founding elevations given in the table below are recommended for design of spread footings supported on the surface of the bedrock, based on the bedrock surface elevations as encountered in the boreholes. Excavation into the bedrock is not recommended (except where required for the wing walls/retaining walls adjacent to the existing rock cut at the west abutment); however, subexcavation of any loose fractured bedrock will be required before construction of the footings. MTO's Special Provision SP902S01 should be included in the Contract Documents requiring inspection and approval of the foundation area by the Quality Verification Engineer prior to footing construction, to ensure that all loose fractured rock has been removed from the foundation areas before construction of the footings.

<i>Foundation Element</i>	<i>Design Founding Elevation</i>
West Abutment – North Widening	82.5 m
West Abutment – South Widening	82.5 m
East Abutment – North Widening	78.9 m
East Abutment – South Widening	77.7 m

<i>Foundation Element</i>	<i>Design Founding Elevation</i>
Northwest Wing Wall/Retaining Wall	West End – 89.2 m East End – As required to fit rock cut
Southwest Wing Wall/Retaining Wall	West End – 89.2 m East End – As required to fit rock cut
Northeast Wing Wall/Retaining Wall	78.9 m
Southeast Wing Wall/Retaining Wall	West End – 77.7 m East End – 78.5 m

As discussed above, excavation into the bedrock has generally not been recommended for the new footings, which will be constructed in close proximity to the existing foundations. However, some bedrock excavation may be required to accommodate stepped footings for wing walls or retaining walls adjacent to the existing west abutment and rock cut along the west side of Quinte West Road 33, if such walls are adopted. If “stepped” footings are required, those portions of the footing at lower levels should be constructed first. Discussion regarding bedrock excavation has been provided in Section 6.11.2 (*Construction Considerations – Bedrock Excavation*).

Higher founding elevations may also be used, with the placement of mass concrete or tremie concrete above the cleaned and inspected bedrock surface. At the east abutment, in particular, based on assessment of the excavation and dewatering/unwatering requirements (as discussed further in Section 6.11.3 (*Construction Considerations – Groundwater Control*)), the use of tremie concrete is recommended as part of the ground and groundwater control scheme; a sample NSSP to address the placement of tremie concrete is included in Appendix C.

Since the bedrock and mass/tremie concrete are not frost-susceptible, no frost protection is required for these foundations.

6.4.2 Geotechnical Resistance

Spread footings placed on the properly prepared bedrock or on mass concrete may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 3 MPa. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock and mass concrete are considered to be unyielding materials; as such, ULS conditions will govern the design.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination and eccentricity of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings for the abutment widenings or wing wall replacements and the argillaceous limestone/calcareous shale bedrock should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \phi'$, may be taken as 0.70 for cast-in-place concrete footings constructed on the bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance for the widened footings can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels will be dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded. For uplift of the dowels, a factored value of 700 kPa may be assumed for the grout-to-rock bond stress for ULS design. The actual bond stress along the rock-grout interface may vary from this design value and it should, therefore, be verified in the field by pull-out testing. If dowelling into bedrock is adopted at this site, an NSSP should be included in the Contract Documents to specify the installation, materials and testing of the dowels; as summarized in Section 6.11 (Construction Considerations), a sample NSSP has been provided in Appendix C.

6.5 Shallow Foundations Within Embankment Fill

On the east side of the river, the bedrock surface is at a depth of approximately 2.5 m to 4 m below the original ground surface, and about 11 m below the Highway 401 grade. To minimize excavation and groundwater control requirements in this area, consideration could be given to supporting the east abutment widening/wing wall replacement on the compact to very dense sand and gravel, or to supporting new retaining walls on the compact to very dense sand and gravel fill. However, some differential settlement would occur between the new, soil-supported elements and the existing bedrock-supported structure; therefore, from a foundations perspective, this option is not considered to be appropriate for the east abutment widening/wing wall replacement, although it may be feasible for support of new retaining walls adjacent to the east abutment/wing wall. Recommendations for strip footings founded within the compact to very dense sand and gravel fill, to support new retaining walls if adopted, are provided in the following sub-sections.

6.5.1 Founding Elevations

For retaining walls adjacent to the east abutment/wing walls, strip footings supported within the compact to very dense sand and gravel fill may be founded above Elevation 82.0 m; the footing and embankment geometry should be checked to ensure a minimum of 1.6 m of soil cover (or equivalent) to provide adequate protection against frost penetration.

If stepped strip footings are constructed, the difference in elevation between individual footings should not be greater than one-half the clear distance between the footings. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevations of the upper footings can be adjusted accordingly.

6.5.2 Geotechnical Resistance

For retaining walls adjacent to the east abutment/wing walls, strip footings placed on the properly prepared, compact to very dense sand and gravel fill, at the design elevations given in the preceding section, should be designed based on the following factored geotechnical resistances at ULS and geotechnical resistances at SLS:

<i>Footing Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS*</i>
2 m	500 kPa	300 kPa
3 m	550 kPa	300 kPa

* For 25 mm of settlement.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs from those given above.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination and eccentricity of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

6.5.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings for the retaining wall replacements and the compact to very dense sand and gravel fill should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \phi'$, may be taken as 0.60 for cast-in-place concrete footings constructed on the properly prepared subgrade. This

represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.6 Steel H-Pile Foundations

Steel H-piles driven to found on the bedrock may be used for support of the east abutment widening, or for the replacement of the wing walls/retaining walls adjacent to the east abutment. In order to achieve a minimum pile length of 5 m, the new pile caps would have to be perched within the approach embankment fill.

In the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the overburden soils (based on the observed presence of cobbles within the gravelly soils in this area), and to the potential for deflection of the piles along the bedrock surface. Based on these considerations, vertically driven piles should be equipped with flange reinforcement (driving shoes) as per SS103-12. Any battered piles should be equipped with suitable driving points (such as Titus standard bearing points or equivalent) to ensure adequate seating of the piles on the bedrock. If there are few vertical piles, standard bearing points can be used for all of the piles since this may be more practicable for contractual purposes.

6.6.1 Founding Elevations

For design of pile-supported widenings at the east abutment and associated wing walls/retaining walls, the following pile tip levels may be assumed based on consideration of the elevation of the bedrock surface as encountered in Boreholes 07-3 and 07-4, together with the bedrock surface information from the 1954 investigation at the site:

<i>Foundation Element</i>	<i>Design Pile Tip Elevation</i>
East Abutment – North Widening and Associated Wing Wall/Retaining Wall	78.9 m
East Abutment – South Widening	77.7 m
Southeast Wing Wall/Retaining Wall	77.7 m (west end) 78.5 m (east end)

6.6.2 Axial Geotechnical Resistance

For HP 310x110 piles driven to found on the argillaceous limestone and calcareous shale bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value represents a structural limitation for the pile rather than a geotechnical limitation. The geotechnical resistance at Serviceability Limit States (SLS) for 25 mm of settlement will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

The Contract Drawings should indicate that pile installation should be in accordance with SP903S01, include the note “Piles to be driven to bedrock”, and indicate that the piles should be equipped with driving shoes or rock points, as discussed above.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; all of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

6.6.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. If vertical piles are used, the resistance to lateral loading will have to be derived from the soil in front of the piles.

The resistance to lateral loading in front of the pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction is determined based on the equations given below (CFEM, 1992, as noted in Section 6.8.7.3 of the *CHBDC*).

For cohesionless soils:

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (MPa/m);
 n_h is the constant of subgrade reaction (MPa/m);
 z is the depth (m); and
 B is the pile diameter (m).

For cohesive soils:

$$k_h = \frac{6Zs_u}{B} \quad \text{where}$$

k_h is the coefficient of horizontal subgrade reaction (kPa/m);
 s_u is the undrained shear strength of the soil (kPa); and
 B is the pile diameter (m).

The following ranges for the value of n_h and s_u may be assumed in the structural analyses:

<i>Soil Unit</i>	<i>n_h</i>	<i>s_u</i>
East Abutment – North Widening and Associated Wing Wall/Retaining Wall Replacement		
Compact to dense sand and gravel fill above Elevation 80 m	10 MPa/m	–
Compact, wet sand and gravel fill between Elevation 80 m and 79 m	5 MPa/m	–
Bedrock below Elevation 179 m		

<i>Soil Unit</i>	<i>n_h</i>	<i>s_u</i>
East Abutment – South Widening and Associated Wing Wall/Retaining Wall Replacement		
Compact to dense sand and gravel fill above Elevation 80 m	10 MPa/m	–
Compact, wet sand and gravel fill between Elevation 80 m and 79 m	5 MPa/m	–
Hard silty clay between Elevation 79 m and Elevation 77.7 m to 78.5 m	–	500 kPa
Bedrock below Elevation 77.7 m to 78.5 m		

A maximum factored lateral resistance at ULS of 110 kN, and a maximum lateral resistance at SLS (for 10 mm of horizontal deflection at pile cap level) of 40 kN are recommended for HP 310x110 piles, based on the “Assessed Horizontal Passive Resistance and Geotechnical Reaction at SLS” values provided under Clause C6.8.7.1, Table C6.4 of the *Commentary on CHBDC*:

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

<i>Pile Spacing in direction of Loading (d = Pile Diameter)</i>	<i>Subgrade Reaction Reduction Factor</i>
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2.
Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those provided in the above table.

6.7 Caissons

As an alternative to steel H-piles, caissons could be considered for support of the east abutment widenings and associated replacement of the wing walls/retaining walls. However, caissons will be more difficult to construct since it will be necessary to use a temporary liner during construction to minimize running or flowing of the water-bearing soils into the caisson hole, and to socket the caissons at least 0.5 m into the bedrock. The argillaceous limestone to calcareous shale bedrock is typically moderately strong to strong, so the sockets would have to be advanced into the rock by churn drilling or rock coring supplemented by down-hole hammer. If caisson foundations are adopted for this site, it is recommended that an NSSP be included in the Contract Documents to address the need for control of the ground and groundwater during caisson construction, and to warn the Contractor of the bedrock strength as it may affect the caisson socketting operation; these recommendations are summarized under *Construction Considerations* in Section 6.11.

6.7.1 Founding Elevations

For design of caisson-supported widenings at the east abutment and associated wing walls/retaining walls, the following caisson base levels may be assumed, based on consideration of the elevation of the bedrock surface as encountered in Boreholes 07-3 and 07-4, together with the bedrock surface information from the 1954 investigation at the site. These design founding elevations assume penetration of 0.5 m into the bedrock.

<i>Foundation Element</i>	<i>Design Caisson Base Elevation</i>
East Abutment – North Widening and Associated Wing Wall/Retaining Wall	78.4 m
East Abutment – South Widening	77.2 m
Southeast Wing Wall/Retaining Wall	77.2 m (west end) 78.0 m (east end)

6.7.2 Axial Geotechnical Resistance

Caissons socketted approximately 0.5 m into the bedrock should be designed based on end-bearing resistance using a factored axial resistance at ULS of 5 MPa; for a 1.5 m diameter caisson, this would equate to a factored axial resistance at ULS of 8,800 kN. Serviceability Limit States (SLS) resistances do not apply to caissons founded on the argillaceous limestone to calcareous shale bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.7.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons (based on subgrade reaction theory), and the reductions due to group effects, may be determined as per Section 6.6.3.

6.8 Retained Soil System (RSS) Walls

Mechanically-reinforced soil retaining systems (retained soils system or RSS walls) are considered, from a foundations perspective, to be a cost-effective and practicable solution for new retaining walls adjacent to the east abutment, since it will not be necessary to extend foundation excavations below the groundwater and river water level at the site. At the west approach embankment, the new wing walls/retaining walls will have to be constructed against the existing bedrock cut and extend westward from this point; therefore, RSS walls are not considered to be an appropriate option adjacent to the west abutment.

6.8.1 Founding Elevations

A typical RSS wall has a front facing supported on a strip footing placed at shallow depth below the ground surface in front of the wall; this footing, and the RSS mass, should be founded below any topsoil, loose fill or unsuitable native soils. For new retaining walls adjacent to the east abutment, it is recommended that the RSS mass be founded at Elevation 81 m, and that the facing footing be founded at Elevation 80.5 m; this would maintain the earthworks above the groundwater level at the site.

If stepped footings are constructed for the facing panel, the difference in elevation between individual footings should not be greater than one-half the clear distance between the footings. In addition, the lower footings should be constructed first so that if it is necessary to construct the lower footings at a greater depth than anticipated, the elevations of the upper footings can be adjusted accordingly.

6.8.2 Geotechnical Resistance

Assuming that the RSS walls act as a unit and utilizes the full width of the reinforced soil mass, taken as two-thirds of the height of the wall, the factored geotechnical resistances at ULS given below may be used for assessment of the reinforced mass founded on the properly prepared embankment fill materials.

<i>Wall Height</i>	<i>Assumed Reinforced Width</i>	<i>Factored Geotechnical Resistance at ULS</i>	<i>Geotechnical Resistance at SLS</i>
3 m	2 m	200 kPa	200 kPa
6 m	4 m	300 kPa	200 kPa

For RSS wall heights of 3 m to 6 m, the settlement of new RSS walls adjacent to the east abutment will be approximately 10 mm to 15 mm.

6.8.3 Resistance to Lateral Loads

The resistance to lateral forces / sliding resistance between the compacted backfill and the subgrade should be calculated in accordance with Section 6.7.5 of the CHBDC. The coefficient of friction, $\tan \phi'$, between the compacted granular fills of the RSS wall and the properly prepared subgrade may be taken as 0.6. This represents an unfactored value; in accordance with the CHBDC, a factor of 0.8 is to be applied in calculating the horizontal resistance.

6.8.4 Global Stability

The static global stability of RSS walls adjacent to the east abutment has been analyzed using the commercially available program SLOPE/W produced by Geo-Slope International Ltd.,

employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A target factor of safety of 1.5 against global failure of the RSS walls is normally used for the design under static conditions. This factor of safety is considered appropriate for the RSS walls at this site, considering the design requirements and the field data available.

Based on the analysis results, the factor of safety against global instability of RSS walls adjacent to the east abutment is greater than 1.5, assuming that the reinforcing strips have a length equal to at least two-thirds of the height of the wall.

6.9 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and associated wing walls/retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of OPSS 1010 Granular A or Granular B Type II (but with less than 5 per cent passing the 200 sieve) should be used as backfill behind the walls. This fill should be placed and compacted in accordance with SP 105S10. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 and 3121.150.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with MTO's Special Provision 105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the wall stem (Case I, Figure C6.20(a) of the *Commentary on CHBDC*) or within a 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 (Case II, Figure C6.20(b) of the *Commentary on CHBDC*).

- For Case I, the pressures are based on the existing and new embankment fill materials and the following parameters (unfactored) may be used, assuming the use of Select Subgrade material for the widened portions of the approach embankments:

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case II, the pressures are based on the granular fill and the following parameters (unfactored) may be assumed:

	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

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows in accordance with Section C6.9.1 of the *Commentary to CHBDC*:
 - rotation (i.e. ratio of wall movement to wall height) of approximately 0.002 about the base of a vertical wall;
 - horizontal translation of 0.001 times the height of the wall; or
 - a combination of both.

Seismic (earthquake) loading must be considered in the design in accordance with Section 4.6.4 of *CHBDC*, as seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure.

$$K \gamma' d + (K_{AE} - K) \gamma' (H-d)$$

- where
- K = either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
 - K_{AE} = the seismic active earth pressure coefficient determined in accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*;
 - γ' = the effective unit weight of the soil (kN/m³), as given above for the fill materials, and taken as 21 kN/m³ for the native soils;
 - d = the investigated depth below the top of the wall (m); and

H = the total height of the wall above the underside of footing or toe (m).

The peak zonal acceleration used for the Trent River bridge site (Trenton) is 0.06g, which is based on a zonal acceleration of 0.05g multiplied by an amplification factor of 1.2 for the types of soils encountered at the east abutment. Using the amplified zonal acceleration ratio of 0.06g obtained for this site, the seismic lateral earth pressure coefficients (K_{AE}) for both yielding and non-yielding walls, considering earth and granular fills, were determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary*, and these are presented below. These seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is essentially flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
	Earth Fill	Granular A	Granular B Type II
Yielding wall ¹	0.32	0.26	0.26
Non-yielding wall	0.37	0.30	0.30

¹ The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.06.

6.10 Approach Embankment Design and Construction

The west approach embankment is less than 1 m in height and the east approach embankment is about 8.7 m in height, relative to the surrounding natural grade. The widening of Highway 401 will require placement of a vertical thickness of less than 1 m of fill on top of the existing west approach embankment side slopes, and a vertical thickness of up to about 2 m of fill on top of the existing east approach embankment side slopes.

6.10.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil and softened/loosened soils be stripped from below the widened approach embankment areas, to minimize differential settlement between the existing and widened portions of the approach embankments. Embankment fill should be placed and compacted in accordance with MTO's Special Provision 105S10. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

In accordance with MTO requirements for all embankments greater than 8 m in height, a 2 m wide mid-height bench should be incorporated into the widened east approach embankment side slopes.

To reduce surface water erosion on the widened embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

6.10.2 Approach Embankment Stability

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the 1 m to 8.7 m high approach embankments with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have a factor of safety of greater than 1.3 against deep-seated slope instability.

The static slope stability analyses for these embankment configurations were carried out based on the following parameters, derived from field and laboratory testing and accepted correlations, using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis.

<i>Soil Type</i>	<i>Bulk Unit Weight</i>	<i>Effective Angle of Friction</i>
Existing embankment fill	21 kN/m ³	32°
New embankment fill for widening (range of parameters for earth fill and granular fill)	20 – 22 kN/m ³	32° to 35°
Compact to very dense sand and gravel to gravel	21 kN/m ³	35°
Hard silty clay	21 kN/m ³	32°

6.10.3 Approach Embankment Settlement

Settlement of Existing Fill / Founding Soils

Settlement analyses have been carried out to estimate the total magnitude of settlement that will occur under the approach embankment widening. The settlement analyses were carried out using the commercially-available program UNISETTLE (Version 3.0).

The compression of the existing compact to dense embankment fill, the compact to very dense sand and gravel to gravel and the hard silty clay were modelled by estimating an elastic modulus of deformation based on the SPT “N” values and correlations proposed by Bowles (1984)² and Kulhawy and Mayne (1990)³. The parameters used in the settlement analyses are presented in the following table:

² Bowles, J.E. 1984. *Physical and Geotechnical Properties of Soils*, 2nd Edition, Ed. McGraw-Hill Book Company.

³ Kulhawy, F.H. and P.W. Mayne. 1990. *Manual on Estimating Soil Properties for Foundation Design*. Final Report 1493-6, EL-6800, Electric Power Research Institute, Palo Alto, California.

<i>Soil Unit</i>	<i>Bulk Unit Weight</i>	<i>Elastic Modulus</i>
Fill for embankment widening	21 kN/m ³	–
Existing compact to dense embankment fill	21 kN/m ³	30 MPa
Compact to very dense sand and gravel to gravel	21 kN/m ³	35 MPa
Hard silty clay	21 kN/m ³	50 MPa

Based on the methods of settlement analysis and parameters identified above and the subsurface conditions as encountered in the boreholes, the following magnitudes of settlement of the founding soils are predicted:

- **West Approach Embankment:** It is estimated that less than 5 mm of elastic compression of the founding soils will occur under the widening of the west approach embankment, based on placement of a vertical thickness of about 1 m of fill on the existing embankment side slopes.
- **East Approach Embankment:** It is estimated that approximately 15 mm of elastic compression of the existing embankment fill and native founding soils will occur under the east approach embankment widening, based on placement of a vertical thickness of up to 2 m of new fill on the existing embankment side slopes.

The elastic settlement of the founding soils as predicted above will occur relatively rapidly during and immediately following (within approximately one month) construction of the embankment widening.

Settlement of New Fill

In addition to the settlements predicted above, self-weight compression will occur within the new embankment fill that is used for the widenings. Provided that clean earth fill (cohesive or cohesionless soil) or granular fill is used, the settlement of the approximately 1 m to 2 m vertical thickness of new fill is expected to be less than approximately 5 mm to 10 mm. The majority of the settlement of this new fill will occur during construction.

6.11 Construction Considerations

6.11.1 Excavations and Temporary Excavation Support

It is anticipated that excavations for footings or pile caps for the widened abutments and replacement wing walls/retaining walls will be advanced through the existing fill and compact to very dense sand and gravel to gravel soils, to or into bedrock. Where space permits, open-cut 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 fill materials and the sand and gravel to gravel soils at the site are classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period)

should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V) through these materials, provided that appropriate dewatering or groundwater control measures are in place.

It is expected that temporary excavation protection will be required along the north and south sides of Highway 401 adjacent to the abutments to facilitate construction of the abutment widening and replacement of the wing walls/retaining walls. These temporary excavation support systems should be designed and constructed in accordance with MTO's Special Provision SP105S19. The lateral movement of temporary shoring systems on Highway 401 should meet Performance Level 2 as specified in SP 105S19, provided that any utilities that may be present adjacent to the temporary shoring systems can tolerate this level of deformation.

Based on the subsurface conditions at the site and the likely excavation geometry, it is anticipated that a soldier pile and lagging system using rakers to provide lateral support would be suitable. Due to the shallow depth to bedrock at the west abutment area, it may also be necessary to socket the soldier piles into the bedrock to provide sufficient lateral resistance at the pile toe once the excavation is advanced.

6.11.2 Bedrock Excavation

The bedrock at the site is weak to very strong (corresponding to unconfined compressive strengths in the range of 5 MPa to 200 MPa), which could make excavation difficult, particularly where relatively shallow depths are needed. Bedrock excavation could be carried out using hoe-ramming techniques; however, line drilling and pre-shearing techniques, if properly executed and inspected, could provide better control over the final configuration of the founding surface.

It is recommended that an NSSP be included in the Contract Documents to warn the contractor that the shaley limestone to calcareous shale bedrock at the site is weak to very strong, that excavation into the bedrock will require appropriate equipment and construction procedures, and that the bedrock excavation shall not disturb the existing bridge footings. An example NSSP is provided in Appendix C.

6.11.3 Groundwater Control

The groundwater level at the site was measured at Elevation 80.0 m (similar to the Trent River water level) in August 2007. Foundation excavations that are advanced to bedrock for the construction of the east abutment will extend approximately 1 m to 2.5 m below the water level at the site. Appropriate groundwater control measures will be required to allow excavation through the water-bearing sand and gravel to gravel fill and native soils.

Consideration has been given to two different approaches to groundwater control for the east abutment area: the use of a coffer dam with dewatering to allow foundation construction on the surface of the bedrock in dry conditions; and the use of a coffer dam with in-water inspection of the prepared bedrock surface, placement of tremie concrete and excavation unwatering to allow foundation construction at a higher elevation (on top of the tremie concrete) in dry conditions. The advantages, disadvantages and risks associated with these two options are summarized in Table 3 following the text of this report.

Based on this comparison, it is considered that the risks are relatively high for creation of a “dry” excavation to allow construction directly on the bedrock surface, given the proximity to the Trent River, the high permeability of the coarse-grained soils in the area, and the presence of discontinuities in the bedrock (through which groundwater will flow). Therefore, the best option for groundwater control at the east abutment is considered to be the use of a coffer dam with the water level maintained at or similar to the groundwater and river water level during excavation, and in-water cleaning and inspection of the bedrock surface prior to placement of tremie concrete. This will allow construction of the footings for the east abutment widening at a higher founding elevation in “dry” conditions.

Non-Standard Special Provisions have been developed by LEA/Golder for inclusion in the Contract Documents that address coffer dam construction, in-water cleaning and inspection of the bedrock surface and placement and testing of tremie concrete, and unwatering of the coffer dams. These NSSPs are provided in Appendix C.

6.11.4 Cobbles and Boulders in Overburden Soils

The sand and gravel to gravel soils overlying the bedrock contain cobbles and boulders, as noted on the borehole records. Appropriate equipment and construction procedures will be required where cobbles and/or boulders are encountered during the installation of coffer dams for the east abutment widening. The NSSP that has been developed for coffer dam construction includes a warning regarding the presence of cobbles and boulders within the native soils at this site.

6.11.5 Dowels into Bedrock

If dowelling into bedrock is adopted at this site, a Special Provision should be included in the Contract Documents to specify the installation, materials and testing of the dowels. A sample Special Provision is included in Appendix C.

6.11.6 Ground and Groundwater Control for Caisson Installation

As discussed in Section 6.7, running or flowing of water-bearing cohesionless soil strata could occur during or after drilling of the caissons. If caisson foundations are adopted for support of the widening of the east abutment and replacement of its associated wing walls/retaining walls, temporary caisson liners would be required to support the soils during construction and permit inspection and cleaning of the caisson base. It is recommended that an NSSP be included in the Contract Documents to warn the contractor of these conditions and the need to control the ground and groundwater during caisson construction; an example NSSP is included in Appendix C.



6.11.7 Caisson Socket Formation in Bedrock


If caissons are adopted for the east abutment widening, nominal socketting into the bedrock will be required. It is recommended that an NSSP be included in the Contract Documents to warn the contractor that the argillaceous limestone to calcareous shale bedrock at the site is generally medium strong to strong, and will require socket formation using appropriate construction equipment and procedures (coring or churn drilling) to advance the hole. An example NSSP is provided in Appendix C.

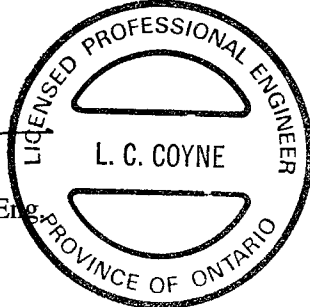
7.0 CLOSURE

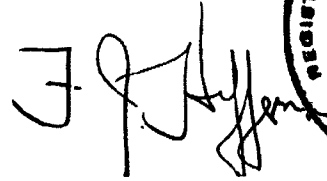
This Foundation Design Report was prepared by Ms. Sandra McGaghran, P.Eng., and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder. Mr. Fin Heffernan, P.Eng., Designated MTO Contact for Golder, conducted an independent quality control review of the report.


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TABLE 1
SUMMARY OF POINT LOAD INDEX TEST RESULTS
ON ROCK CORE SAMPLES

<i>Borehole Number</i>	<i>Elevation (m)</i>	<i>Bedrock Type</i>	<i>Test Type</i>	<i>Is Axial (MPa)</i>	<i>Is Diametral (MPa)</i>	<i>Is₅₀ (MPa)</i>	<i>UCS* (MPa)</i>
07-1	90.1	Shale	Diametral		2.774	2.606	55
	89.9	Limestone	Diametral		7.023	8.494	178
	87.0	Limestone	Diametral		6.125	6.875	144
	86.5	Limestone	Axial	6.969		6.719	155
	85.7	Limestone	Diametral		5.068	6.224	131
	85.2	Limestone	Diametral		5.825	7.121	150
	84.8	Shale	Diametral		0.273	0.326	7
	84.8	Shale	UCS				52
	84.0	Shale	Diametral		0.176	0.219	5
	83.8	Shale	Diametral		0.198	0.254	6
	83.4	Mudstone	Diametral		0.201	0.249	6
	82.6	Limestone	Diametral		0.336	0.427	9
	82.3	Shale	Diametral		0.320	0.372	8
07-2	82.2	Shale	Diametral		1.653	2.138	45
	82.0	Shale	Diametral		0.805	0.935	22
	81.5	Limestone	UCS				48
	81.3	Limestone	Axial	3.471		3.572	82
	80.8	Limestone	Diametral		6.179	7.402	155
	79.9	Shale	Diametral		0.170	0.216	5
	79.6	Shale	Diametral		0.301	0.367	8
	79.3	Limestone	Diametral		6.374	5.751	121
	78.6	Limestone	Diametral		10.936	12.437	261
	78.3	Limestone	Diametral		0.257	0.296	6
	77.7	Shale	Diametral		0.284	0.360	8
	77.2	Shale	Diametral		0.595	0.795	18
	76.6	Shale	Axial	4.798		4.202	97
07-3	77.0	Shale	Diametral		0.510	0.623	13
	76.3	Shale	Diametral		1.179	1.354	28
	75.9	Shale	Diametral		0.857	1.154	27
	75.2	Limestone	Diametral		1.484	1.794	41
	74.8	Shale	Diametral		0.370	0.463	11
	74.4	Limestone	Diametral		1.369	1.663	35
	74.1	Shale	Diametral		0.343	0.418	10
	73.8	Limestone	UCS				37
	73.8	Limestone	Axial	6.182		6.004	138
	72.9	Limestone	Diametral		3.467	4.238	89
	72.4	Limestone	Diametral		6.939	8.548	180
	72.1	Limestone	Diametral		3.709	4.274	90

Checked: J. BrownReviewed: L.C. Coyne

* The UCS values have been approximated using $Is_{50} \times 23$, from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

TABLE 1 (Continued)
SUMMARY OF POINT LOAD INDEX TEST RESULTS
ON ROCK CORE SAMPLES

<i>Borehole Number</i>	<i>Elevation (m)</i>	<i>Bedrock Type</i>	<i>Test Type</i>	<i>Is Axial (MPa)</i>	<i>Is Diametral (MPa)</i>	<i>Is₅₀ (MPa)</i>	<i>UCS* (MPa)</i>
07-4	77.8	Limestone	Diametral		6.882	8.509	179
	77.5	Shale	Diametral		1.110	1.256	29
	77.1	Shale	Diametral		1.057	0.987	21
	76.3	Limestone	Diametral		4.692	5.496	115
	76.2	Shale/Limestone	UCS				42
	75.9	Limestone	Diametral		5.147	6.149	129
	75.4	Shale/Limestone	Axial	5.060		4.605	106
	74.9	Limestone	Diametral		0.347	0.430	9
	74.5	Shale	Diametral		0.476	0.569	13
	74.2	Limestone	Diametral		1.384	1.909	40
	73.6	Shale	Diametral		0.259	0.312	7
	73.1	Limestone	Diametral		5.744	6.467	136
	72.7	Shale/Limestone	Diametral		0.160	0.201	5

Checked: J. BrownReviewed: L.C. Coyne

* The UCS values have been approximated using $Is_{50} \times 23$, from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.

TABLE 2
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
ABUTMENT WIDENING AND WING WALL/RETAINING WALL REPLACEMENT
TRENT RIVER BRIDGE, HIGHWAY 401, TRENTON, ONTARIO
G.W.P. 196-99-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
Shallow foundations supported on bedrock	<ul style="list-style-type: none"> • Feasible for support of the widened west abutment and new wing walls/ retaining walls adjacent to the west abutment, as the depth to bedrock is very shallow • Feasible for support of the widened east abutment and new wing walls/retaining walls adjacent to the east abutment 	<ul style="list-style-type: none"> • On the west side of the river, the depth to bedrock is very shallow • Bedrock surface will provide adequate bearing resistance and minimal differential settlement relative to existing structure 	<ul style="list-style-type: none"> • On the east side of the river, the bedrock surface is at a depth of about 2.5 m to 4 m below the original ground surface, and about 11 m below Highway 401 grade; excavations would extend about 1 m to 2.5 m below the groundwater level and Trent River water level, and groundwater control would be required • Nominal subexcavation of bedrock may be required to remove loose, fractured bedrock; deeper bedrock excavation may be required to accommodate stepped footings for the northwest and southwest wing walls/retaining walls adjacent to the west abutment and existing rock cut 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques • Groundwater control will be required for excavations extended to bedrock at east abutment area 	<ul style="list-style-type: none"> • Foundation costs likely less expensive than deep foundations • However, costs will be higher at east abutment area due to excavation depth, temporary shoring and groundwater control requirements
Shallow foundations supported on compact to very dense sand and gravel or compact to very dense sand and gravel fill	<ul style="list-style-type: none"> • Not feasible at west abutment and associated wing walls/retaining walls, where bedrock is very shallow • Not recommended for support of widened east abutment due to potential for differential settlement relative to existing bedrock-supported structure • Feasible for support of new retaining walls adjacent to east abutment and wing walls, if adopted 	<ul style="list-style-type: none"> • Minimizes excavation and groundwater control for retaining walls east of east abutment/wing walls 	<ul style="list-style-type: none"> • Potential for differential settlement relative to existing bedrock-supported foundation elements 	<ul style="list-style-type: none"> • Conventional excavation and construction techniques 	<ul style="list-style-type: none"> • Least expensive foundation costs

TABLE 2 (Continued)
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
ABUTMENT WIDENING AND WING WALL/RETAINING WALL REPLACEMENT
TRENT RIVER BRIDGE, HIGHWAY 401, TRENTON, ONTARIO
G.W.P. 196-99-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
Driven steel H-piles	<ul style="list-style-type: none"> • Not feasible for support of widened west abutment and new wing wall/retaining walls adjacent to west abutment, as the depth to bedrock is very shallow • Feasible for support of the widened east abutment and new wing walls/retaining walls adjacent to the east abutment 	<ul style="list-style-type: none"> • Minimizes excavation depth and groundwater control requirements at east abutment area, as compared to shallow foundations supported on bedrock 	<ul style="list-style-type: none"> • Depending on pile cap location (depending on connection to existing structure), steel H-piles may be very short and therefore may not be practical • Potential for obstructions from cobbles and boulders in the coarse glaciofluvial soils overlying the bedrock 	<ul style="list-style-type: none"> • Conventional construction methods for H-pile foundations for east abutment widening or eastern wing wall/retaining wall replacement 	<ul style="list-style-type: none"> • Less expensive than caissons
Caissons cored to or into bedrock	<ul style="list-style-type: none"> • Not practical for support of widened west abutment and new wing wall/retaining walls adjacent to west abutment, as the depth to bedrock is very shallow • Feasible for support of the widened east abutment and new wing walls/retaining walls adjacent to the east abutment 	<ul style="list-style-type: none"> • Minimizes excavation depth and groundwater control requirements at east abutment area, as compared to shallow foundations supported on bedrock 	<ul style="list-style-type: none"> • Temporary liner will be required during caisson construction to minimize running or flowing of water-bearing cohesionless soils into caisson hole • Caisson will have to be socketted nominally into bedrock to cut off groundwater and to penetrate below loose, fractured layers of rock; the bedrock is typically moderately strong to strong, which will affect the installation method and rate 	<ul style="list-style-type: none"> • Conventional construction methods for caisson foundations for east abutment widening or eastern wing wall/retaining wall replacement 	<ul style="list-style-type: none"> • More expensive than steel H-piles

TABLE 2 (Continued)
COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES
ABUTMENT WIDENING AND WING WALL/RETAINING WALL REPLACEMENT
TRENT RIVER BRIDGE, HIGHWAY 401, TRENTON, ONTARIO
G.W.P. 196-99-00

<i>Foundation Option</i>	<i>Feasibility</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Constructability / Practicability</i>	<i>Relative Costs</i>
RSS walls	<ul style="list-style-type: none"> • Not practical for replacement of retaining walls adjacent to west abutment, as excavation into existing near-vertical bedrock cut would be required • Feasible for replacement of retaining walls adjacent to east abutment 	<ul style="list-style-type: none"> • Conventional construction methods for eastern wing walls/retaining walls • Existing pile caps/footings for existing retaining walls could remain in place below reinforced soil mass following removal of existing retaining wall stems, minimizing depth of excavation 	<ul style="list-style-type: none"> • None 	<ul style="list-style-type: none"> • Conventional construction methods 	<ul style="list-style-type: none"> • Less expensive than all other foundation/retaining wall options for eastern wing walls/retaining walls

TABLE 3
COMPARISON OF DEWATERING/UNWATERING ALTERNATIVES
FOR CONSTRUCTION OF EAST ABUTMENT WIDENING
TRENT RIVER BRIDGE, HIGHWAY 401, TRENTON, ONTARIO
G.W.P. 196-99-00

<i>Option</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Risks / Consequences</i>
Coffer dam (full cut-off or supplemented with dewatering system to maintain coffer dam in dry conditions)	<ul style="list-style-type: none"> Allows for excavation, subgrade preparation and inspection, and foundation construction in “dry” conditions. 	<ul style="list-style-type: none"> The bedrock surface is variable, including natural undulations/steps and excavation into bedrock for construction of original east abutment footings. The sheet piles for the coffer dam will not be able to penetrate the typically medium strong to strong bedrock, and “gaps” will be left between the base of the sheet piles and the surface of the bedrock. Groundwater inflow will still occur through the “gaps” below the sheet pile walls and through the bedrock, and this groundwater inflow will be difficult to control with typical dewatering systems; additional measures will be required. Groundwater inflow through the “gaps” below the sheet pile walls also has the potential to carry soil particles into the coffer dam, resulting in loosening and/or settlement of soils surrounding the coffer dam. 	<ul style="list-style-type: none"> High risk that sheet piles will not create a full cut-off, since the bedrock surface is variable (natural variability plus the excavation into the bedrock for the original footing construction); groundwater will still be able to flow through these “gaps”. Even with mitigation (sand bags applied at base of coffer dam walls and other measures), it is anticipated that groundwater flow cannot be fully cut-off due to proximity to the Trent River and the relatively high permeability of the coarse granular soils in this area. Medium to high risk that soil adjacent to coffer dam walls will be affected by groundwater flow through “gaps” at base of coffer dam walls, causing loosening and/or settlement of the soils surrounding the coffer dams. Medium risk that it will be difficult to remove all fill/native soil from the surface of the bedrock, given problems with groundwater inflow (and carrying of soil particles into excavation).
Coffer dam with in-water inspection of bedrock subgrade, followed by placement of tremie concrete and construction of footings at a higher elevation	<ul style="list-style-type: none"> Allows for water level within coffer dams to remain consistent with groundwater and river water levels during excavation and bedrock surface preparation, thereby minimizing risks associated with groundwater inflow and soil loss (as identified under “disadvantages” for the option above). Allows for foundation construction in dry conditions at a higher founding elevation, on the surface of the tremie concrete. 	<ul style="list-style-type: none"> In-water cleaning and inspection of the bedrock surface will be required before placement of tremie concrete up to the design founding level; such in-water cleaning and inspection can be more difficult than similar operations in dry conditions. 	<ul style="list-style-type: none"> Low to medium risk related to difficulties with underwater cleaning of all fill/native soil from the surface of the bedrock, cleaning of all loose highly fractured material from the surface of the bedrock, and inspection of the prepared bedrock surface. Low risk associated with placement of tremie concrete. Low risk associated with development of “dry” excavation at proposed higher founding level (surface of tremie concrete at Elevation 79.5 m).

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

III. SOIL DESCRIPTION

(a) Cohesionless Soils

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

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.)

Consistency

	<u>kPa</u>	<u>psf</u>
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

(b) Cohesive Soils

c_u, s_u

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.

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.

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
F	factor of safety
V	volume
W	weight

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

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	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_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

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

- Notes:** 1 $\tau = c' + \sigma' \tan \phi'$
2 Shear strength = (Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING 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 texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
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	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

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

CORE CONDITION

Total Core Recovery

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 varies 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 (W.R.T.) 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 abbreviated 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

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/ Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

RECORD OF BOREHOLE No 07-1

1 OF 1 **METRIC**

PROJECT 07-1111-0019 LOCATION N 4888064.5 ; E 217225.4 ORIGINATED BY PKS
 W.P. 196-99-00 DIST Eastern HWY 401 BOREHOLE TYPE Track-Mounted CME 55, 108 mm Solid Stem Augers, Automatic Hammer COMPILED BY JB
 DATUM Geodetic DATE August 13, 2007 CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
90.7	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	10 20 30					
0.0	SAND and GRAVEL, some silt, trace clay, containing organics Compact to dense Brown Moist		1	SS	38								41 40 14 5	
			2	SS	17					o				
89.2														
1.5	Shaley LIMESTONE to calcareous SHALE (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		1	RC	REC 50%								RQD = 0%	
	Bedrock cored between 1.5 m and 8.4 m depth. For additional bedrock coring details, refer to Record of Drillhole 07-1.		2	RC	REC 74%								RQD = 0%	
			3	RC	REC 67%								RQD = 0%	
			4	RC	REC 95%								RQD = 17%	
			5	RC	REC 98%								RQD = 36%	
			6	RC	REC 100%								RQD = 71%	
82.3			7	RC	REC 100%								RQD = 100%	
8.4	END OF BOREHOLE													
	Notes: 1. Borehole dry prior to start of rock coring; 2. Piezometer dry on August 14, 2007. 3. Piezometer dry on October 16, 2007.													

MIS-MTO 001 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ

PROJECT: 07-1111-0019

RECORD OF DRILLHOLE: 07-1

SHEET 1 OF 1

LOCATION: N 4888064.5 ;E 217225.4

DRILLING DATE: August 13, 2007

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-55

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/min	FLUSH % RETURN	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	SD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	RECOVERY TOTAL CORE % SOLID CORE %	R.Q.D. % FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA B Angle DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr Ja Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole		89.20																			
2		Shaley LIMESTONE to calcareous SHALE (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		1.50	1																		Screen
3					2																		
4					3																		
5					4																		
6					5																		Sand
7					6																		
8					7																		
9		END OF DRILLHOLE		82.30																			
10				8.40																			
11																							

DEPTH SCALE

1 : 50



LOGGED: PKS

CHECKED: LCC

MIS-RCK 004 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ

RECORD OF BOREHOLE No 07-2

1 OF 1 **METRIC**

PROJECT 07-1111-0019

W.P. 196-99-00

LOCATION N 4888078.2 ; E 217245.9

ORIGINATED BY PKS

DIST Eastern HWY 401

BOREHOLE TYPE Track-Mounted CME 55, 108 mm Solid Stem Augers, Automatic Hammer

COMPILED BY JB

DATUM Geodetic

DATE August 10, 2007

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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 (%)
								20 40 60 80 100							
								20 40 60 80 100							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L								
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED								
83.8	GROUND SURFACE														
0.0	Sand and gravel (FILL) Compact to very dense Brown Moist to wet		1	SS	18										
82.5			2	SS	56										
1.4	Shaley LIMESTONE to calcareous SHALES (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		1	RC	REC 100%									RQD = 25%	
	Bedrock cored between 1.4 m and 7.5 m depth. For additional bedrock coring details, refer to Record of Drillhole 07-2.		2	RC	REC 100%									RQD = 85%	
			3	RC	REC 100%									RQD = 66%	
			4	RC	REC 100%									RQD = 58%	
			5	RC	REC 100%									RQD = 87%	
76.4	END OF BOREHOLE														
7.5	Note: 1. Water level in open borehole measured at a depth of 1.4 m (Elevation 82.4 m) prior to start of rock coring.														

MS-MTO 001 07-1111-0019.GPJ GAL-MISS.GDT 1/21/08 DD/HJ

+ 3, X 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

PROJECT: 07-1111-0019

RECORD OF DRILLHOLE: 07-2

SHEET 1 OF 1

LOCATION: N 4888078.2 ; E 217245.9

DRILLING DATE: August 10, 2007

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME-55

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/min	COLLOID % RETURN	FLUSH	UN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	RECOVERY TOTAL CORE % SOLID CORE %	R.O.D. %	FRACT. INDEX PER 0.3 m	B Angle DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q- AVG	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole		82.48																						
2		Shaley LIMESTONE to calcareous SHALE (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		1.37	1																					
3					2																					
4					3																					
5					4																					
6					5																					
7																										
8		END OF DRILLHOLE		76.36																						
9				7.49																						
10																										
11																										

DEPTH SCALE

1 : 50



LOGGED: PKS

CHECKED: LCC

MIS-RCK 004 07-1111-0019.GPJ GAL-MISS GDT 1/21/08 DD/RJ

RECORD OF BOREHOLE No 07-3

1 OF 1 **METRIC**

PROJECT 07-1111-0019

W.P. 196-99-00

LOCATION N 4888165.7 ; E 217393.0

ORIGINATED BY PKS

DIST Eastern HWY 401

BOREHOLE TYPE Track-Mounted CME 55, 108 mm Solid Stem Augers, Automatic Hammer

COMPILED BY JB

DATUM Geodetic

DATE August 9, 2007

CHECKED BY LCC

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								
81.3	GROUND SURFACE							20	40	60	80	100				
0.0	SAND and GRAVEL, some silt, trace clay Compact to very dense Brown Moist to wet		1	SS	55		81									
			2	SS	44		80									41 39 14 6
79.3			3	SS	18											
2.0	GRAVEL, some sand, trace silt and clay Compact to very dense Brown to grey Wet		4	SS	27		79									78 19 2 1
			5	SS	27		78									
	Containing cobbles below 3.7 m depth		6	SS	80/0.08											
77.0							77									
4.3	Shaley LIMESTONE to calcareous SHALE (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		1	RC	REC 100%		76									RQD = 50%
	Bedrock cored between 4.3 m and 9.5 m depth. For additional bedrock coring details, refer to Record of Drillhole 07-3.		2	RC	REC 100%		75									RQD = 96%
			3	RC	REC 100%		74									RQD = 85%
			4	RC	REC 100%		73									RQD = 44%
71.9							72									
9.5	END OF BOREHOLE															
	Notes: 1. Water level in open borehole at a depth of 2.4 m (Elevation 78.9 m) prior to start of rock coring; 2. Water level in piezometer measured at a depth of 1.3 m (Elevation 80.0 m) on August 14, 2007.															

MIS-MTO 001 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ

+ 3, X 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PROJECT: 07-1111-0019

RECORD OF DRILLHOLE: 07-3

SHEET 1 OF 1

LOCATION: N 4888165.7 ; E 217393.0

DRILLING DATE: August 9, 2007

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME-55

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE mm/rev	FLUSH	RECOVERY				FRACT. INDEX PER 0.3 m	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY				Diametral Index (MPa)	RMC -Q- AVG.	NOTES WATER LEVELS INSTRUMENTATION
								TOTAL CORE %	SOLID CORE %	R.Q.D. %	B Angle		DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	K, cm/sec	10"	10"			
								UN - Joint	BD - Bedding	PL - Planar	PO - Polished	BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.										
								FLT - Fault	CO - Contact	OR - Orthogonal	CU - Curved	UN - Undulating	SM - Smooth	Ro - Rough	MB - Mechanical Break								
		Continued from Record of Borehole		77.05																			
5		Shaley LIMESTONE to calcareous SHALE (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		4.27	1																		
6					2																		
7					3																		
8					4																		
9																							
		END OF DRILLHOLE		71.87 9.45																			
10																							
11																							
12																							
13																							
14																							

DEPTH SCALE

1 : 50



LOGGED: PKS

CHECKED: LCC

MIS-RCK 004 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ

RECORD OF BOREHOLE No 07-4

1 OF 2 **METRIC**

PROJECT 07-1111-0019

W.P. 196-99-00

LOCATION N 4888183.6 ; E 217410.2

ORIGINATED BY PKS

DIST Eastern HWY 401

BOREHOLE TYPE Track-Mounted CME 55, 108 mm Solid Stem Augers, Automatic Hammer

COMPILED BY JB

DATUM Geodetic

DATE August 14, 2007

CHECKED BY LCC

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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
89.8	GROUND SURFACE							20 40 60 80 100						
0.0	ASPHALT							20 40 60 80 100						
89.5								20 40 60 80 100						
0.3	Sand and gravel, trace to some silt, trace clay (FILL) Compact to dense Brown Moist		1	SS	30		89							
			2	SS	39		88							40 46 10 4
			3	SS	36		87							
			4	SS	34		86							
			5	SS	26		85							
			6	SS	23		84							
			7	SS	37		83							
			8	SS	45		82							41 46 9 4
			9	SS	10		81							
							80							
79.1							79							
10.7	SILTY CLAY with sand and gravel Hard Brown Moist to wet		10	SS	50/0.03		79						43	33 26 14 24
78.5							78							RQD = 0%
11.3	Shaley LIMESTONE to calcareous SHALE (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		1	RC	REC 58%		78							RQD = 83%
			2	RC	REC 83%		77							RQD = 50%
			3	RC	REC 100%		76							
			4	RC	REC 100%		75							RQD = 97%

Continued Next Page

+ 3, X 3: Numbers refer to
Sensitivity

O 3% STRAIN AT FAILURE

MIS-MTO 001 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ

RECORD OF BOREHOLE No 07-4

2 OF 2 **METRIC**

PROJECT 07-1111-0019

W.P. 196-99-00

LOCATION N 4888183.6 ; E 217410.2

ORIGINATED BY PKS

DIST Eastern HWY 401


BOREHOLE TYPE Track-Mounted CME 55, 108 mm Solid Stem Augers, Automatic Hammer

COMPILED BY JB

DATUM Geodetic

DATE August 14, 2007

CHECKED BY LCC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L			
--- CONTINUED FROM PREVIOUS PAGE ---																		
	Shaley LIMESTONE to calcareous SHALES (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black Bedrock cored between 11.3 m and 17.2 m depth. For additional bedrock coring details, refer to Record of Drillhole 07-4.		4	RC	REC 100%												RQD = 97%	
			5	RC	REC 100%													RQD = 40%
72.6			6	RC	REC 100%													RQD = 50%
17.2	END OF BOREHOLE																	
	Note: 1. Borehole dry prior to start of rock coring.																	

MIS-MTO 001 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ

PROJECT: 07-1111-0019

RECORD OF DRILLHOLE: 07-4

SHEET 1 OF 1

LOCATION: N 4888183.6 ,E 217410.2

DRILLING DATE: August 14, 2007

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME-55

DRILLING CONTRACTOR: Walker Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE min/(m)	FLUSH	COLOUR % RETURN	JN - Joint FLT - Fault SHR - Shear VN - Vein CJ - Conjugate	BD - Bedding FO - Foliation CO - Contact OR - Orthogonal CL - Cleavage	PL - Planar CU - Curved UN - Undulating ST - Stepped IR - Irregular	PO - Polished K - Slickensided SM - Smooth Ro - Rough MB - Mechanical Break	BR - Broken Rock	NOTE: For additional abbreviations refer to list of abbreviations & symbols.	RECOVERY TOTAL CORE % SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3 m	B Angle DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	Jr	Ja	Jn	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -G' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continued from Record of Borehole		78.52																						
		Shaley LIMESTONE to calcareous SHALES (BEDROCK) Fresh to slightly weathered Weak to extremely strong Thinly to medium bedded Light grey to black		11.28																						
12					1																					
					2																					
13					3																					
					4																					
14					5																					
15					6																					
16																										
17		END OF DRILLHOLE		72.58 17.22																						
18																										
19																										
20																										
21																										

DEPTH SCALE

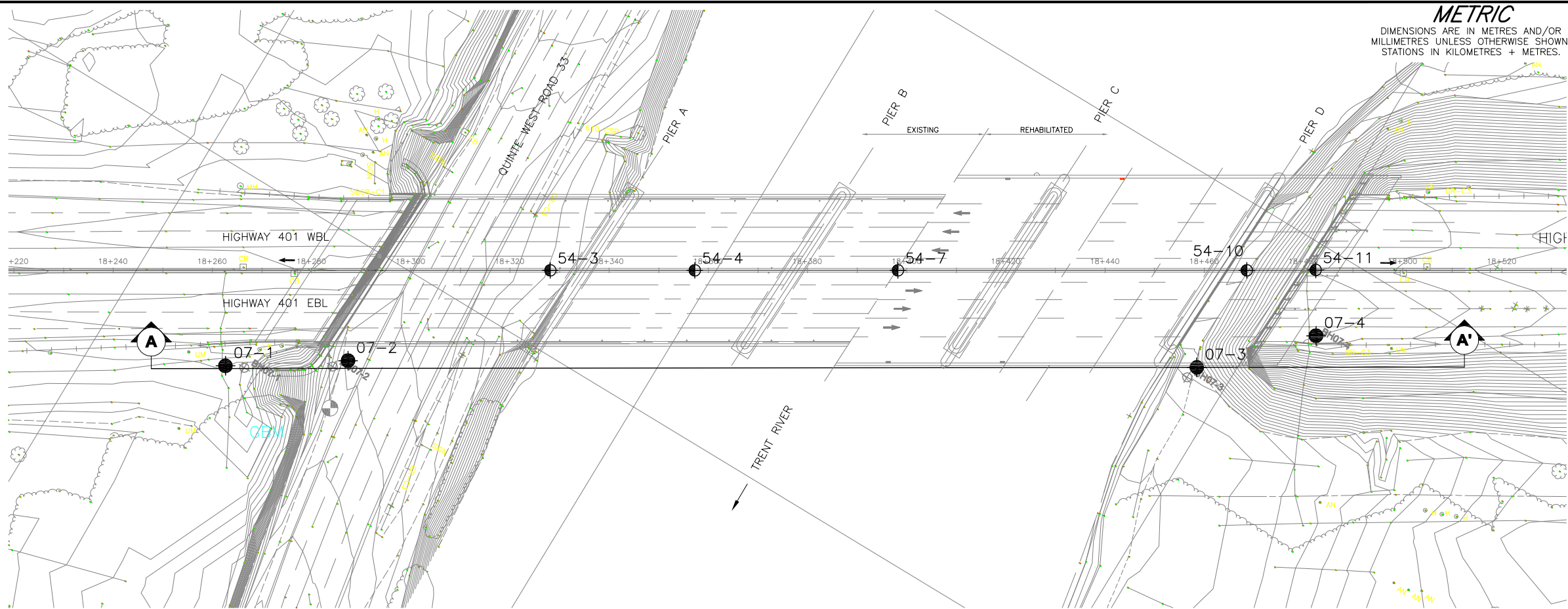
1 : 50



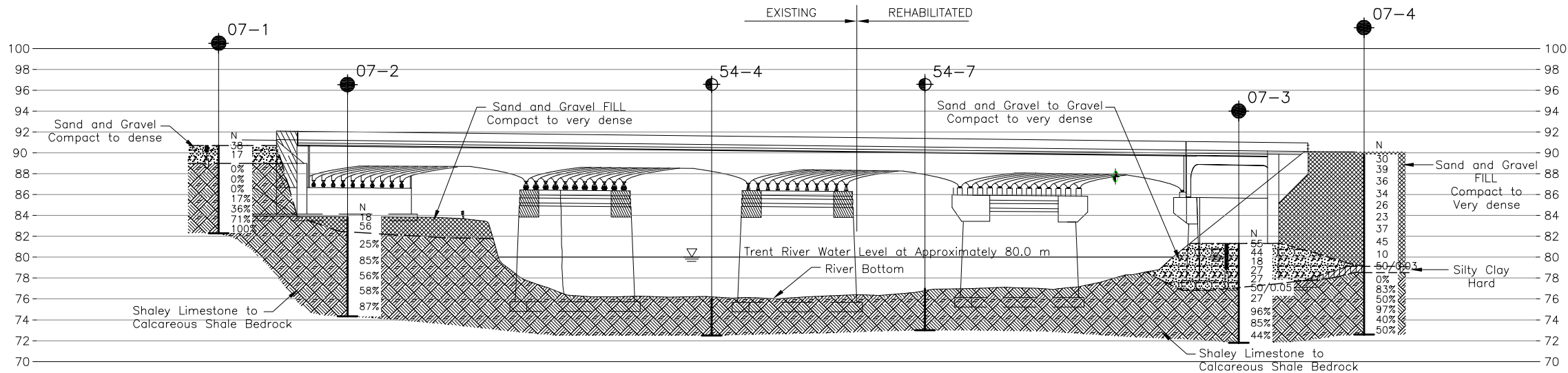
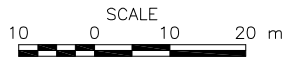
LOGGED: PKS

CHECKED: LCC

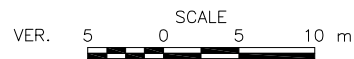
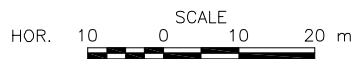
MIS-RCK 004 07-1111-0019.GPJ GAL-MISS.GDT 1/18/08 DD/RJ



PLAN



PROFILE A-A'



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

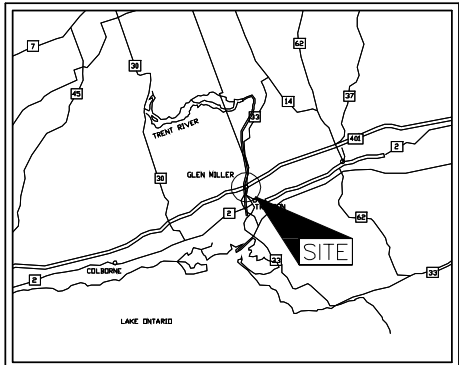
CONT No.
WP No.196-99-00

HIGHWAY 401
Trent River Bridge

Borehole Locations and Soil Strata



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

LEGEND

- Borehole - Current Investigation
- Approximate Borehole Location - 1954 Investigation
- 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 in piezometer, measured on MMM DD, YYYY
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
07-1	90.7	4888064.5	217225.4
07-2	83.8	4888078.2	217245.9
07-3	81.3	4888165.7	217393.0
07-4	89.8	4888183.6	217410.2
54-3	84.7	4888114.9	217271.4
54-4	76.0	4888130.0	217296.3
54-7	77.0	4888151.2	217331.3
54-10	81.0	4888187.7	217391.5
54-11	82.0	4888195.0	217403.3

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.

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

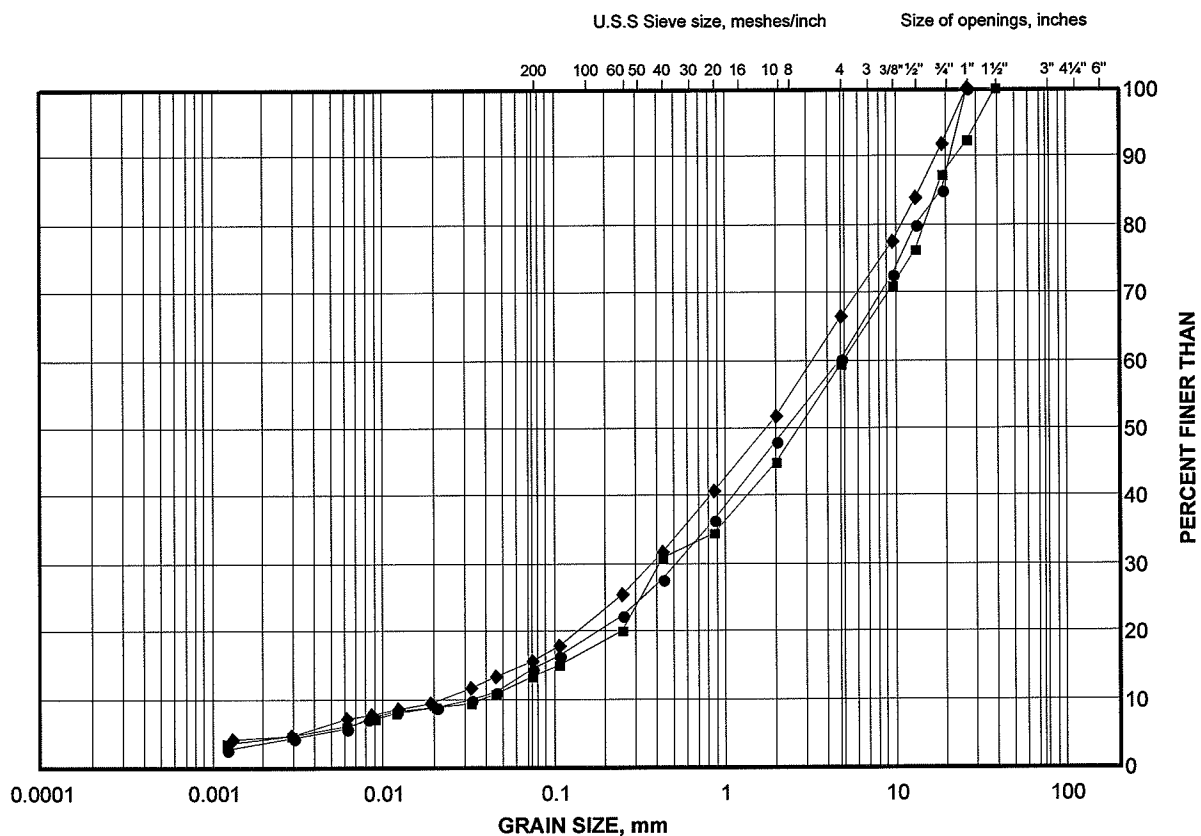
Base plans provided in digital format by LEA Consulting Ltd, drawing file no. "wp1969900.dwg", received August 20, 2007 and 8378-S01.dwg, received December 5, 2007.

NO.	DATE	BY	REVISION
Geocres No.31C-183			
HWY. 401		PROJECT NO. 07-1111-0019	
SUBM'D. JB		DIST.	
CHKD. PKS		DATE: 3/11/2008	
DRAWN: DD		SITE:	
CHKD. PKS		APPD. LCC	
		DWG.	

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel Fill

FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	07-4	2	88.0
■	07-4	6	84.9
◆	07-4	8	81.9

Project Number: 07-1111-0019-1

Checked By: Ulogre

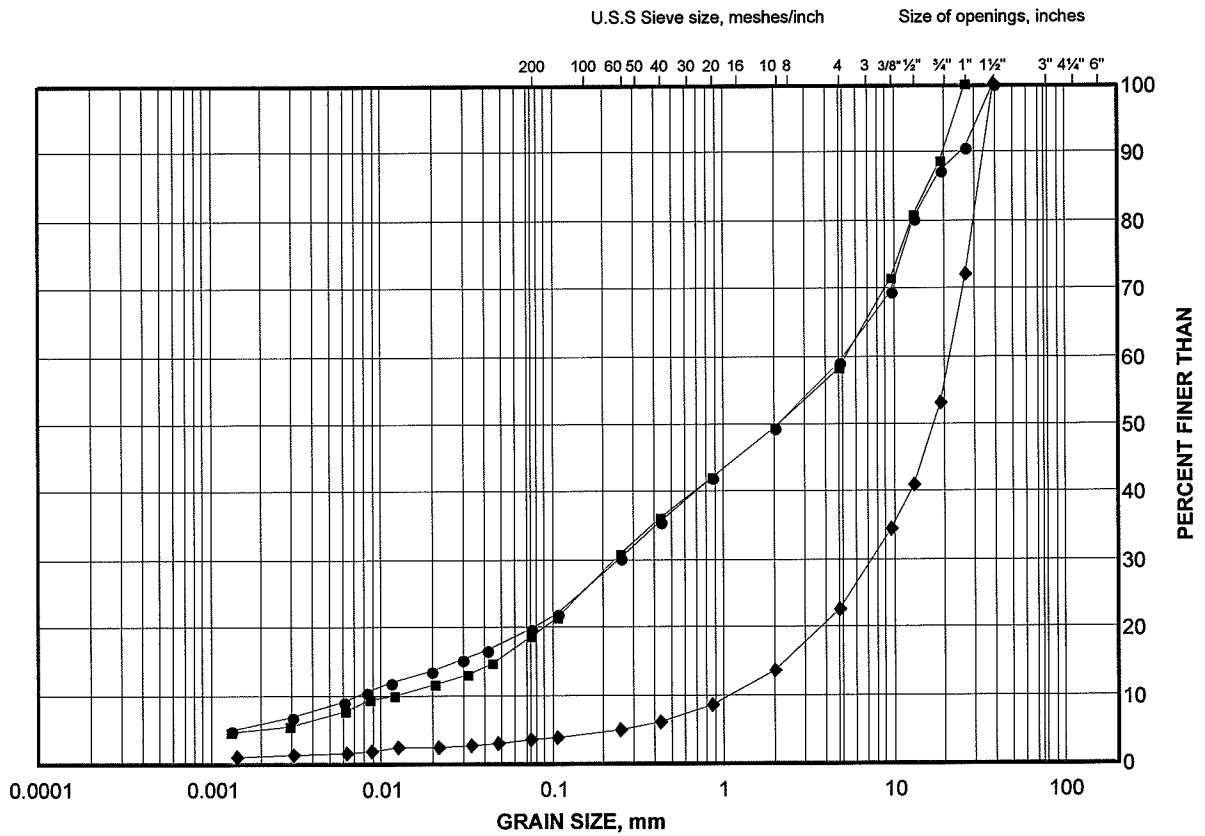
Golder Associates

Date: 23-Jan-08

GRAIN SIZE DISTRIBUTION TEST RESULTS

Sand and Gravel to Gravel

FIGURE 2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	07-3	2	80.3
■	07-1	2	89.7
◆	07-3	4	78.7

Project Number: 07-1111-0019-1

Checked By: *[Signature]*

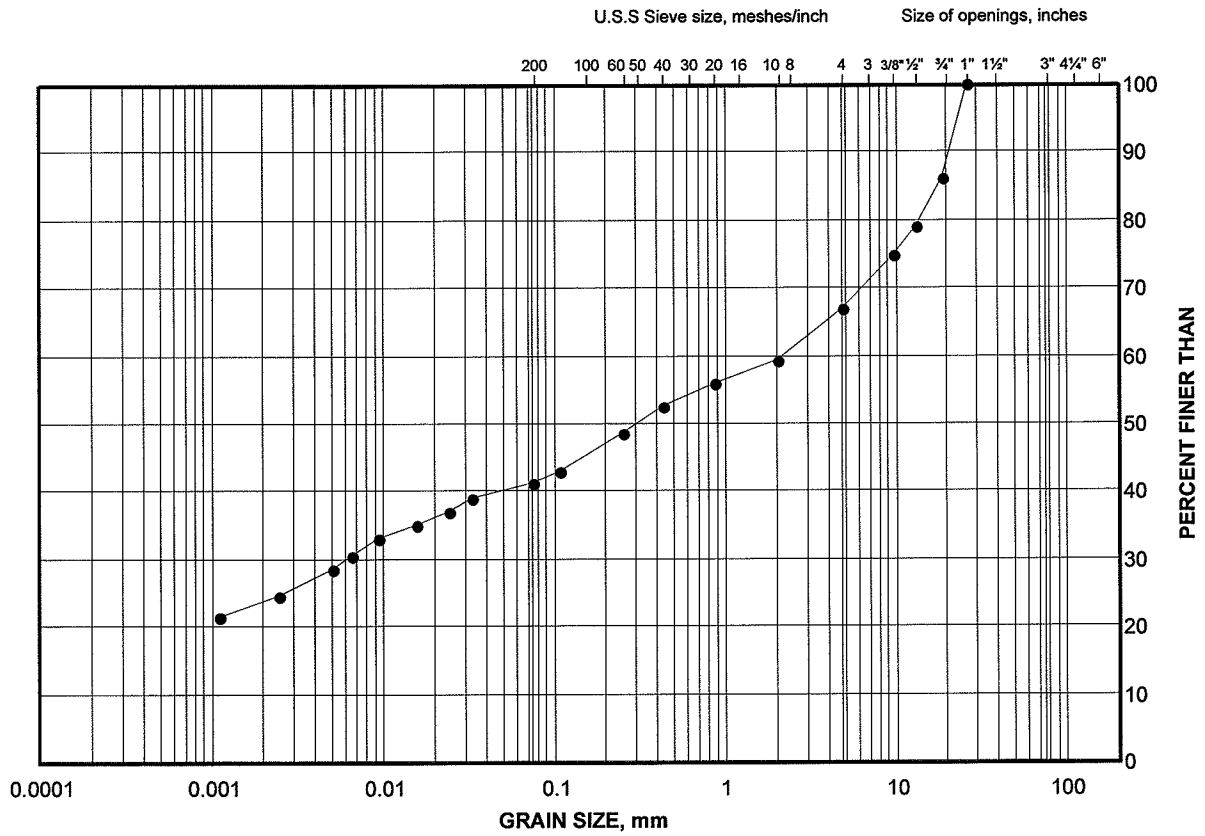
Golder Associates

Date: 23-Jan-08

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Clay

FIGURE 3



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

LEGEND

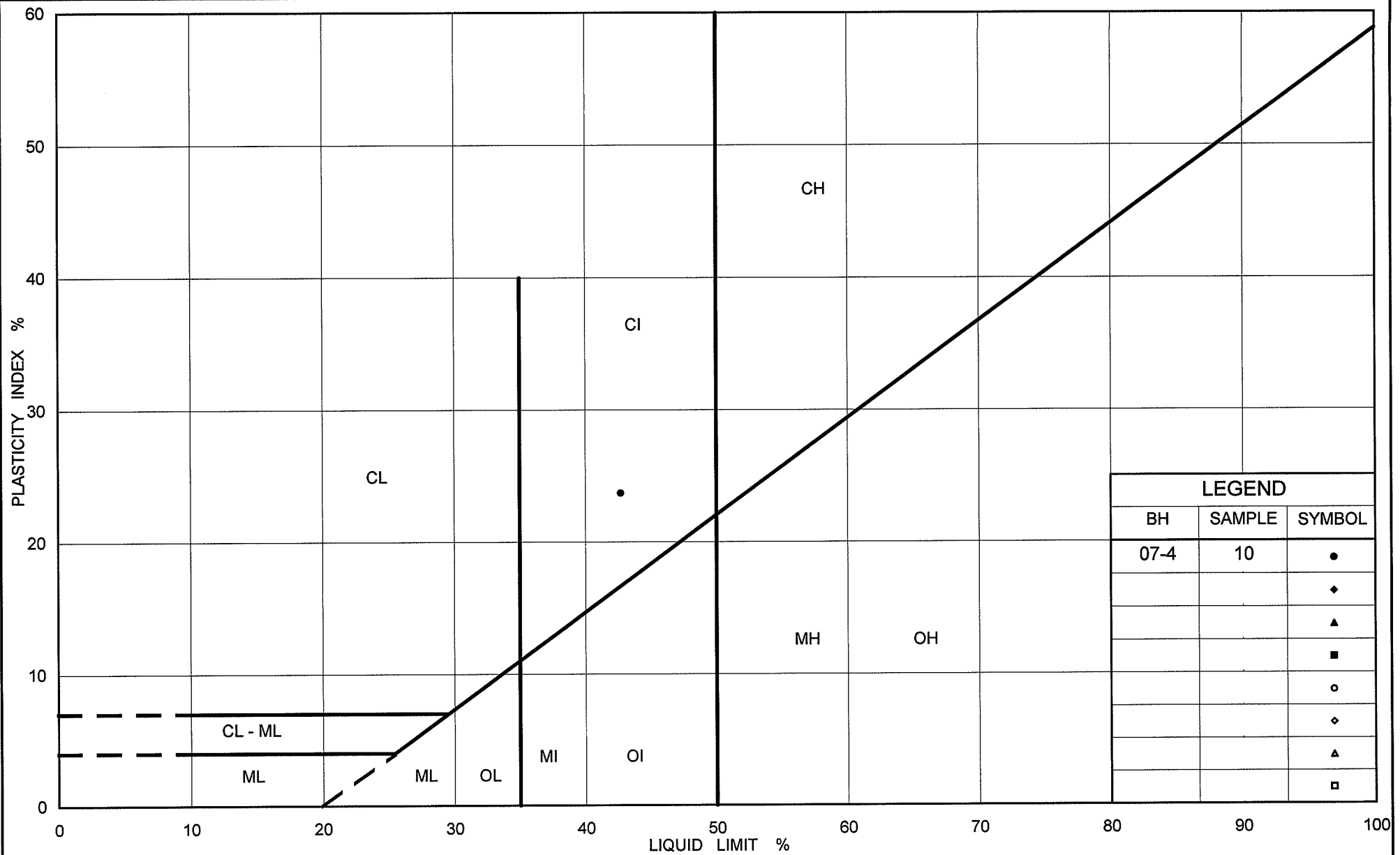
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
•	07-4	10	79.0

Project Number: 07-1111-0019-1

Checked By: *[Signature]*

Golder Associates

Date: 23-Jan-08



Ministry of Transportation

Ontario

PLASTICITY CHART Silty Clay

Figure No. 4

Project No. 07-1111-0019-1

Checked By: *Ullayee*

APPENDIX A

**RECORDS OF BOREHOLES 54-3, 54-4, 54-7, 54-10 AND 54-11
1954 INVESTIGATION BY DEPARTMENT OF HIGHWAYS, ONTARIO**

JOB F54-2 BORING NO. 3
 DATUM Good 2N-9 5TH 2477+16.7 f. DATE REPORT 21ST JUNR 1954
 COMPILED BY W.W. CHECKED BY G.M.F. BORING DATE 28TH MAY 1954

ABBREVIATIONS



V-INSITU VANE SHEAR TEST	Z-UNIT WEIGHT
U-MECHANICAL ANALYSIS	K-PERMEABILITY
M-UNCONFINED COMPRESSION	C-CONSOLIDATION
Q _u -TRIAXIAL CONSOLIDATED QUICK	CA-CASING
Q-TRIAXIAL QUICK	WL-WATER LEVEL IN CASING
S-TRIAXIAL SLOW	WT-WATER TABLE IN SOIL

SAMPLES

[illegible]

TL 119
54-90

MATERIALS LABORATORY-DEPARTMENT OF HIGHWAYS - ONTARIO
OFFICE REPORT ON SOIL EXPLORATION
BOREHOLE 54-4

31C-42
GEOCRESS No.

DRILL RIG 1 JOB F54-2 BORING NO. 4
CASING BX (STANDARD SAMPLERS TO FIT UNLESS NOTED) DATUM STN. 2478+06 (2' E.) DATE REPORT 21st June 1964
SAMPLER HAMMER WT. 6 DROP INCHES COMPILED BY N.W. CHECKED BY G.M.A. BORING DATE

SAMPLE CONDITION



DISTURBED
GOOD
LOST

SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN

W.S. - WASHED SAMPLE
R.C. - ROCK CORE

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Qc - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW
γ - UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION
CA - CASING
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

SOIL PROFILE				SHEAR STRENGTH TONS/SQ.FT. OR $Q_{u/2}$		WATER CONTENT W %		SAMPLES				
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	PENETRATION TEST RESISTANCE BLOWS PER FOOT	Δ P.W.	Δ L.V.	OTHER TESTS	CONDITION	TYPE	№	ELEV. RECOV.
266.8		Top of STAKE		0								
262.8		Water Level of River		2								
				4								
				6								
				8								
				10								
				12								
				14								
249.3		Bottom of River		16								
14.8		SHALEY ROCK		18								
247.9				20								
16.3		SOLID BEDROCK		22								
				24								
				26								
227.9		END OF BOREHOLE		28								
24.3												

247.9
100%

R.C. 1

MATERIALS LABORATORY-DEPARTMENT OF HIGHWAYS-ONTARIO
OFFICE REPORT ON SOIL EXPLORATION
BOREHOLE 54-7.

31C-42
ENCLOSURE No.

DRILL RIG _____
CASING _____ (STANDARD SAMPLERS TO FIT UNLESS NOTED)
SAMPLER HAMMER WT _____ DROP _____ INCHES

Joe E. E. E.

DATUM

COMPILED BY_

STANDARD

DRING N°

DIST. REPON

TRÓFICA DA

REVIATIO

SAMPLE CONDITION



SAMPLE TYPES

C.O - CHUNK
O.O - DRIVE OPEN
D.F - DRIVE FOOT VALVE
T.O - THIN WALLED OPEN

WS - WASHED SAMPLE
RC - ROCK CORE

ABBREVIATIONS

V-IN SITU VANE SHEAR TEST	γ - UNIT WEIGHT
W-MECHANICAL ANALYSIS	K - PERMEABILITY
U-UNCONFINED COMPRESSION	C - CONSOLIDATION
Q _u - TRIAXIAL CONSOLIDATED QUICK	CA - CASING
Q - TRIAXIAL QUICK	WL - WATER LEVEL IN CASING
S - TRIAXIAL SLOW	WT - WATER TABLE IN SOIL

[illegible]

MATERIALS LABORATORY-DEPARTMENT OF HIGHWAYS - ONTARIO
OFFICE REPORT ON SOIL EXPLORATION

BOREHOLE 54-10

31C-42

• **Google** Inc.

DRILL PIPE _____
CASING _____ STANDARD SAMPLERS TO FIT UNLESS NOTED
SAMPLER HAMMER WT _____ * DROP _____ INCHES

100 F 84-2 BORING NO. 10
 DATUM SEA LEVEL DATE REPORT 21 June 1954
 COMPILED BY N.Y.L. CHECKED BY G.H.F. BORING DATE 7-8 May 1954

SAMPLE CONDITION



SAMPLE TYPES

C.S. - CHUNK
DQ - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
TQ - THIN WALLED OPEN

WA-WASHED SAMPLE
R.C.-ROCK CORE






ABBREVIATIONS

V-INSITU VANE SHEAR TEST	P-UNIT WEIGHT
U-MECHANICAL ANALYSIS	K-PERMEABILITY
M-UNCONFINED COMPRESSION	C-CONSOLIDATION
Q-TRIAXIAL CONSOLIDATED QUICK	CA-CASING
R-TRIAXIAL QUICK	WL-WATER LEVEL IN CASING
S-TRIAXIAL SLOW	WT-WATER TABLE IN SOIL

[illegible]

COMPILED BY M. M. CHECKED BY R. M. E. BORING DATE 2 MAY 1962

V-INSITU VANE SHEAR TEST	P-UNIT WEIGHT
M-MECHANICAL ANALYSIS	K-PERMEABILITY
U-UNCONFINED COMPRESSION	C-CONSOLIDATION
Q _u -TRIAXIAL CONSOLIDATED QUICK	CA-CASING
Q-TRIAXIAL QUICK	WL-WATER LEVEL IN CASING
S-TRIAXIAL SLOW	WT-WATER TABLE IN SOIL

SOIL PROFILE				SHEAR STRENGTH TONS/SG.FT. on $Q_{u/2}$		WATER CONTENT W%		SAMPLES						
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALES	PENETRATION TEST RESISTANCE BLOWS PER FOOT		O P.W. Δ LV		OTHER TESTS	CONDITION	TYPE	Nº	PENETRATION RESISTANCE	ELEV. RECOR.
2688 •		BOULDERS MIXED WITH CLAY		0								R.C.		2688 50%
				2										
				4										
				6										
				8										
				10										
				12										
				14										
				16										
				18										
2683 10%		BEDROCK		20								R.C.		2683 10%
				22										
				24										
				26										
				28										
				30										
				32										
				34										
				36										
				38										
2683 20%		END OF HOLE		40								R.C.		2683 20%

APPENDIX B

DIVING INSPECTION OF TRENT RIVER BRIDGE HIGHWAY 401, TRENTON, ONTARIO

GOLDER ASSOCIATES LTD.

**DIVING INSPECTION
OF
TRENT RIVER BRIDGE
(HWY 401)
TRENTON, ONTARIO**

**Inspection Date:
July 31 – August 1, 2007**

**Submitted
August 21, 2007**

ASI Group Project D20147

**ASI Group Ltd.
St. Catharines, Ontario, Canada**



TABLE OF CONTENTS

1.0	INTRODUCTION	1
2.0	METHODS	1
3.0	OBSERVATIONS.....	2
3.1	Pier A.....	2
3.2	Pier B.....	2
3.3	Pier C.....	3
3.4	Pier D.....	3
4.0	RECOMMENDATIONS.....	4

APPENDIX 1: Photographs

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE (HWY 401)
TRENTON, ONTARIO
ASI Group Project D20147
Inspection Date: July 31 – August 1, 2007**

1.0 INTRODUCTION

ASI Group Ltd. (ASI), of St. Catharines, Ontario was contracted by Golder Associates Ltd. to complete an underwater video inspection of the Trent River Bridge (Highway 401) in Trenton, Ontario.

The bridge itself was constructed in 1955. It is a four lane concrete structure with five spans. Inspected piers were B and C, which are surrounded by water on all sides, the east side of A and the west side of D.

The navigable channel of the Trent-Severn Waterway passes under the span bordered by Piers C and D. Current velocity at the time of inspection was negligible at less than 0.1 m/s.

2.0 METHODS

All diving operations were carried out in accordance with the Ontario Ministry of Labour Diving Regulation O. Reg. 629/94 amended to O. Reg. 115/04. A written Notice of Project (2007D-20887) was submitted to the Ontario Ministry of Labour before mobilizing to the work site.

A four-man commercial dive crew equipped with surface-supplied diving system and two-way voice communications performed the inspection from July 31 to August 1, 2007. The inspection video was recorded and edited into DVD format for presentation. All diving equipment was staged on board ASI's 24-foot diving support/work vessel, the *ASI Surveyor*.

3.0 OBSERVATIONS

For the purposes of this inspection, all comparisons are made to the previous inspection completed in 2000. All horizontal locations are documented with reference to the girder placement at the bridge seat at the top of each pier. For piers A through D, girders are numbered from 1 to 11 from north to south, with north being upstream. Benchmark information was gathered at the north nose of Pier B. The base of the lower notch at the top of pier was measured to the water surface. Water elevations remained constant throughout the entire inspection. The waterline at the time of the current inspection was identical to the waterline from the previous inspection (2000).

3.1 Pier A

Pier A was inspected on the east face only, the west face being dry.

Soundings showed that the mudline remains virtually unchanged since the last inspection. Bottom composition consisted of placed stone and rip-rap at either nose averaging 0.3 m in diameter. The mudline between girders 2 and 10 consisted of a mixture of silt/sand and zebra mussel shells. No exposed footing was found.

The concrete condition of the pier remains largely unchanged with the exception of the hairline crack at centre line of the pier that has extended down and terminates at the first cold joint beneath the waterline. An area of map cracking with light scaling of up to 10 mm exists at the second girder. There are three cold joints beneath the waterline that have minor spalling at intermittent areas.

A light layer of zebra mussels exist covering 15 to 20% of the entire concrete face.

3.2 Pier B

All faces of Pier B were inspected.

The mudline elevation has decreased with a loss of material of 10 to 30% on both faces with up to a 1 m loss on the east face at girders 7 through 10. As a result of the scour, the footing at the north nose is exposed with up to 0.3 m of vertical surface. The horizontal exposure is bordered by girder #1 on both faces (see photograph in Appendix 1). The top of footing to water surface was 4.42 m. No other exposed footing was found. Bottom composition at the north nose is a fine silt layer on top of natural river bottom and bedrock. On both faces, including the south nose, the bottom is a mixture of silt, sand and zebra mussel shells mixed into a mixture of cobbles that range in size between

0.1 m to 1 m in diameter. Probes taken at the south nose ranged between 0.3 m to 0.5 m. This would indicate that there is sufficient solid material to cover the footing.

The pier itself has some minor structural defects. Typically, scaling is on all surfaces to an average of 3 mm. Cold joints remain tight with some minor spalling along random areas along all joints. Random hairline cracks were found at girders 4 and 7 on the east face.

Zebra mussels covered up to 85% of the entire face up to one layer thick with a light layer of algae.

3.3 Pier C

All faces of Pier C were inspected.

There is a loss of material of up to 20% along the entire mudline with the most significant loss at the north nose of more than 1 m. However, there still remains significant material around the pier so that no footing was exposed. The bottom material consisted of a mixture of sand and silt on top of natural river bottom in the form of cobbles no more than 50 mm in diameter. The north nose had some larger placed stone at the base averaging 0.8 m in diameter.

Pier C is generally in good condition. The inspection found areas of map cracking below the water surface at girder 2 on the east face and girder 1 below the water surface on the west face. A 2 mm crack was found on the west face at girder 8 below the water surface (Appendix 1). There was a significant void around a cold joint at the north nose 0.3 m below the water surface. The dimensions of the void were 0.3 m long by 0.04 m high and up to 0.04 m deep (see photograph in Appendix 1). Concrete overpour was documented on the south nose just below the surface of the water (see photograph in Appendix 1).

A light layer of brown/green algae covered a single layer of zebra mussels. The zebra mussels covered 85% of all vertical surfaces.

3.4 Pier D

Only the west face of Pier D was inspected. The east face was dry.

The mudline along the north nose to girder 4 and girder 8 to the south nose was found to be a mixture of placed stone, coarse gravel and cobbles averaging 0.2 m in diameter. Between girders 4 and 8, the mudline was a mixture of sand and silt mixed in with cobbles. In general, the elevation of the mudline has decreased by 0.3 m. No footing was exposed.

Pier D itself is in fair condition. The entire underwater surface is scaled to no more than 10 mm. The majority of cold joints have spalling around them of various sizes at a maximum depth of 50 mm with aggregate exposed. In some cases rebar is exposed (see photograph in Appendix 1).

Zebra mussels cover an area of no more than 40% with a light algae layer on top.

4.0 RECOMMENDATIONS

Underwater inspections on a regular inspection schedule should be performed to monitor the exposed footing on the north nose of Pier B and possible exposure in the future of the north nose of Pier C. In addition, close monitoring of continued concrete deterioration of all pier faces with particular attention to Pier D.

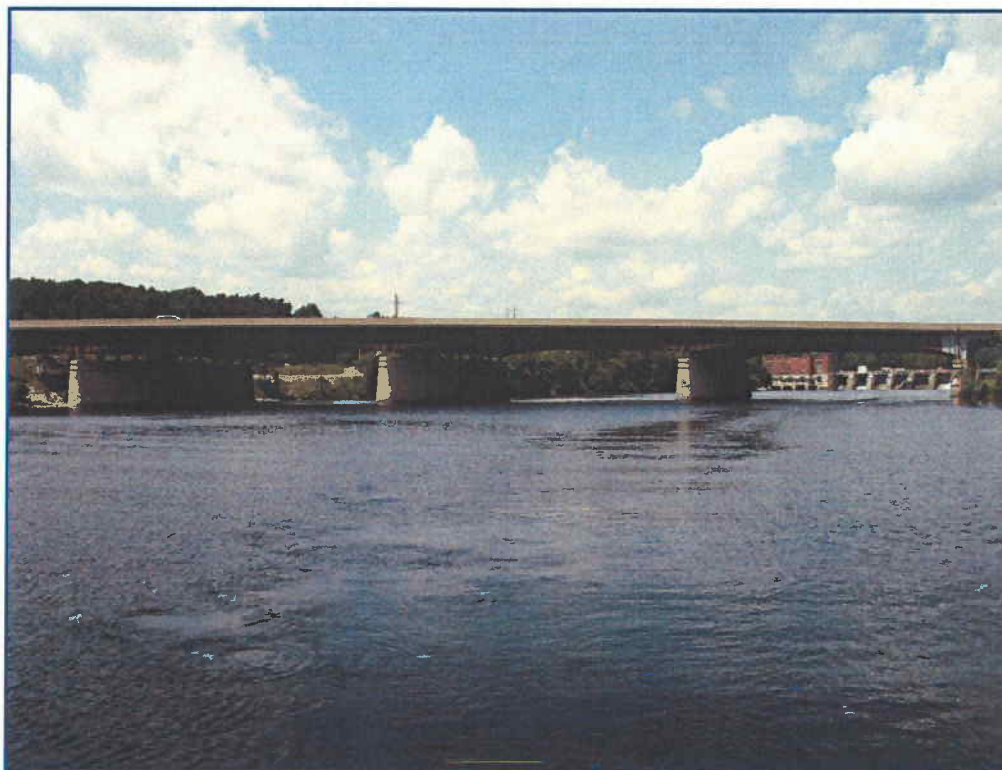
APPENDIX 1

Photographs





Photograph 1-1: North Elevation



Photograph 1-2: South Elevation

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

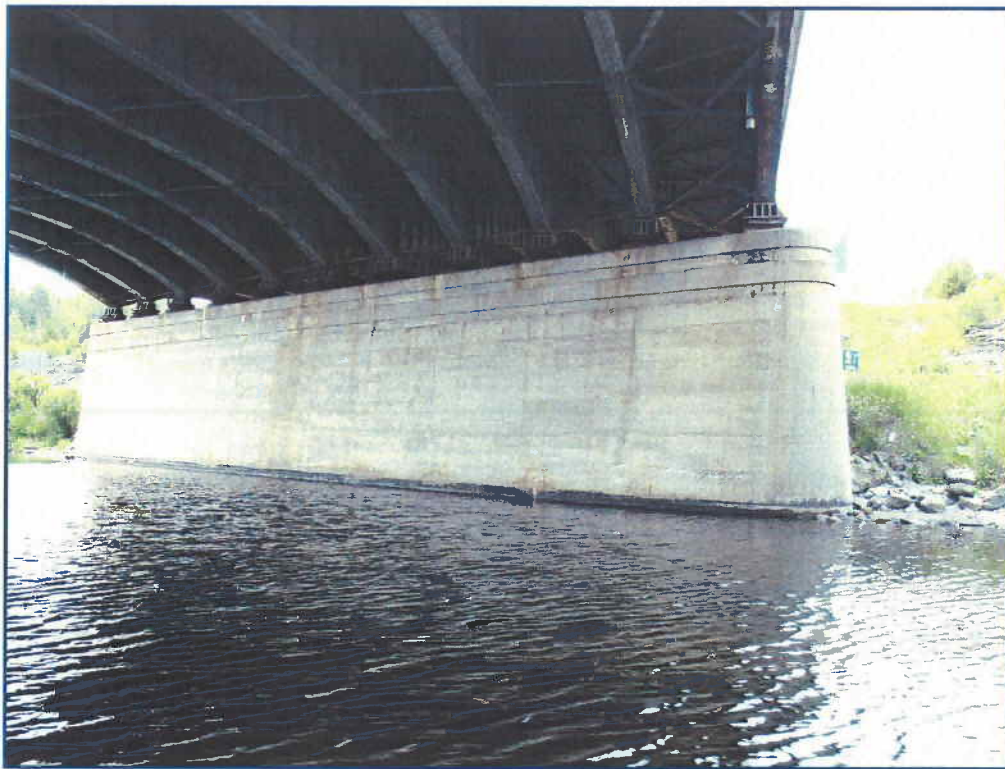
PHOTOGRAPHS

ASI PROJECT D20147

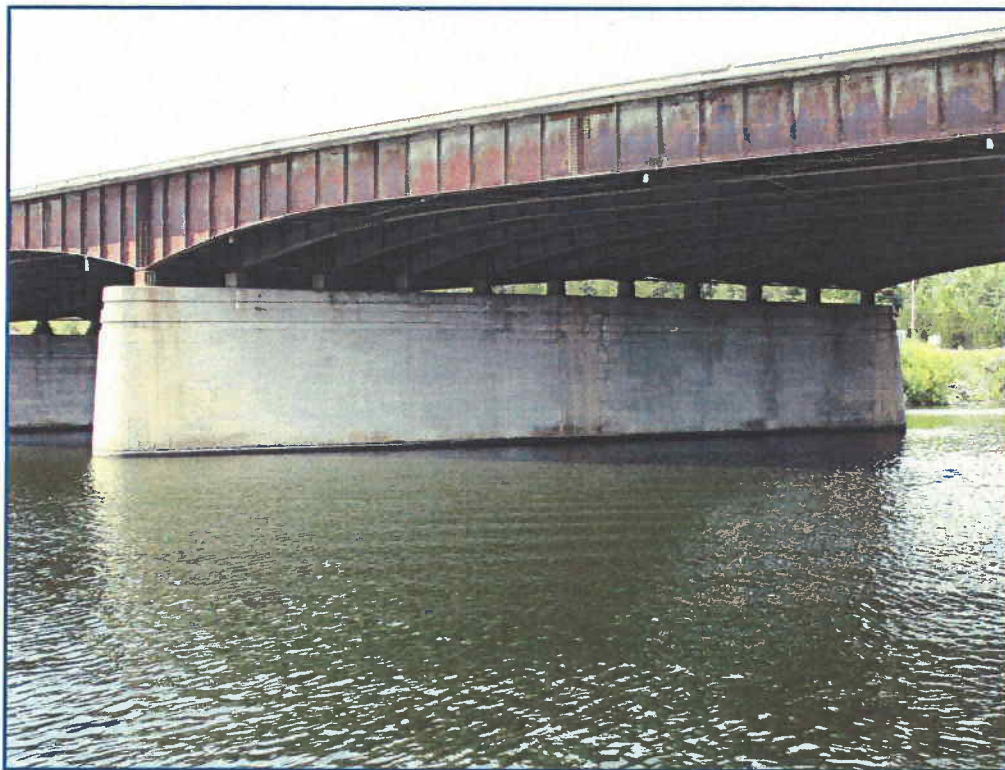
JULY 31, 2007

APPENDIX 1





Photograph 1-3: Pier A - East



Photograph 1-4: Pier B - East

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

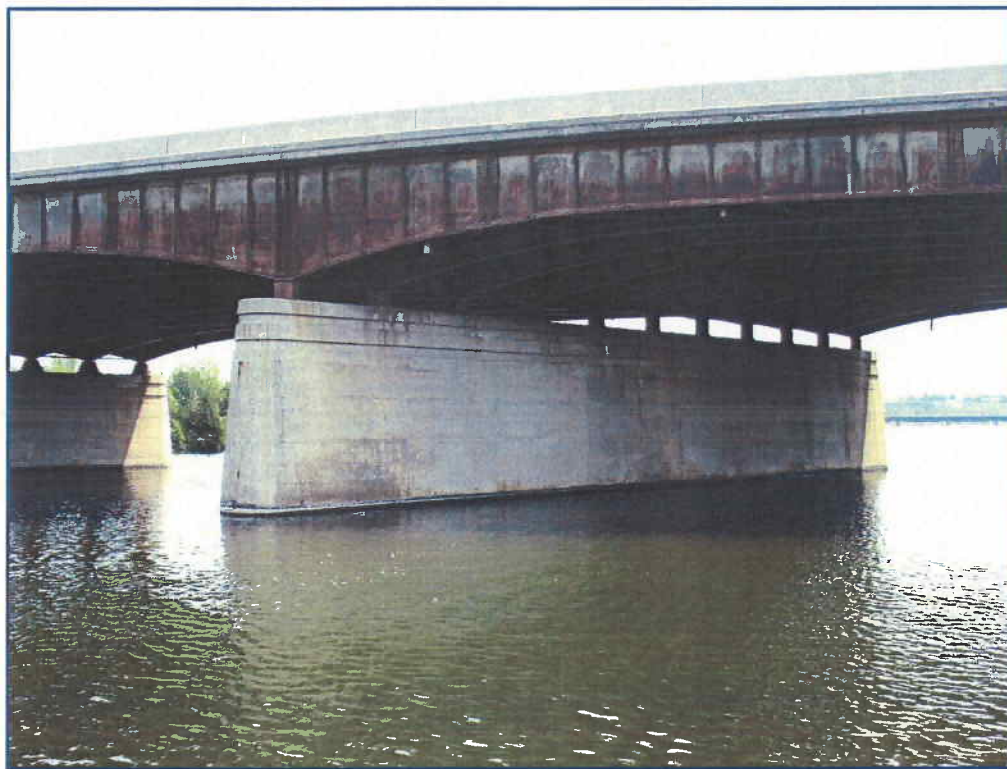
PHOTOGRAPHS

ASI PROJECT D20147

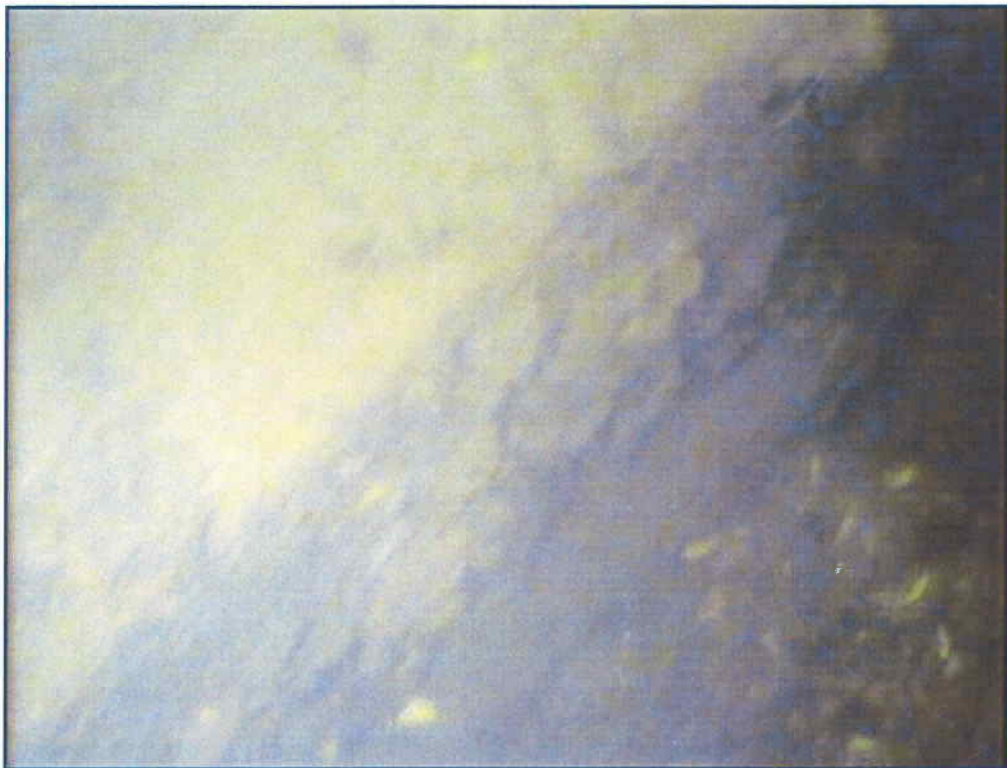
JULY 31, 2007

APPENDIX 1





Photograph 1-5: Pier B - West



Photograph 1-6: Pier B - Footing Exposed North Nose

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

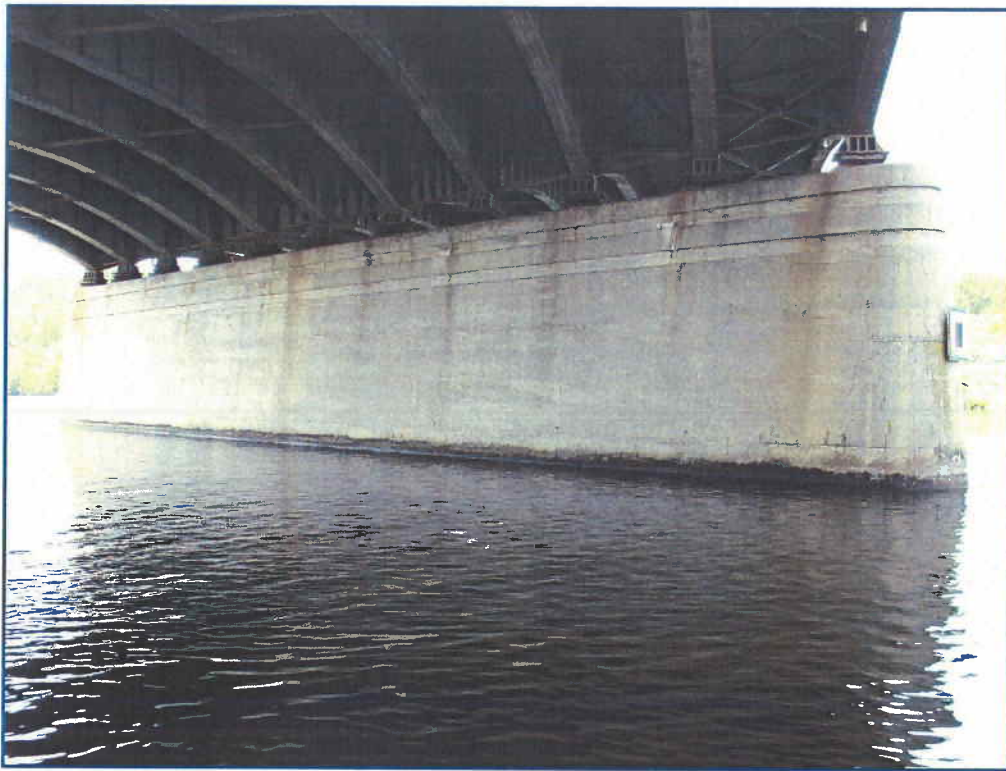
PHOTOGRAPHS

ASI PROJECT D20147

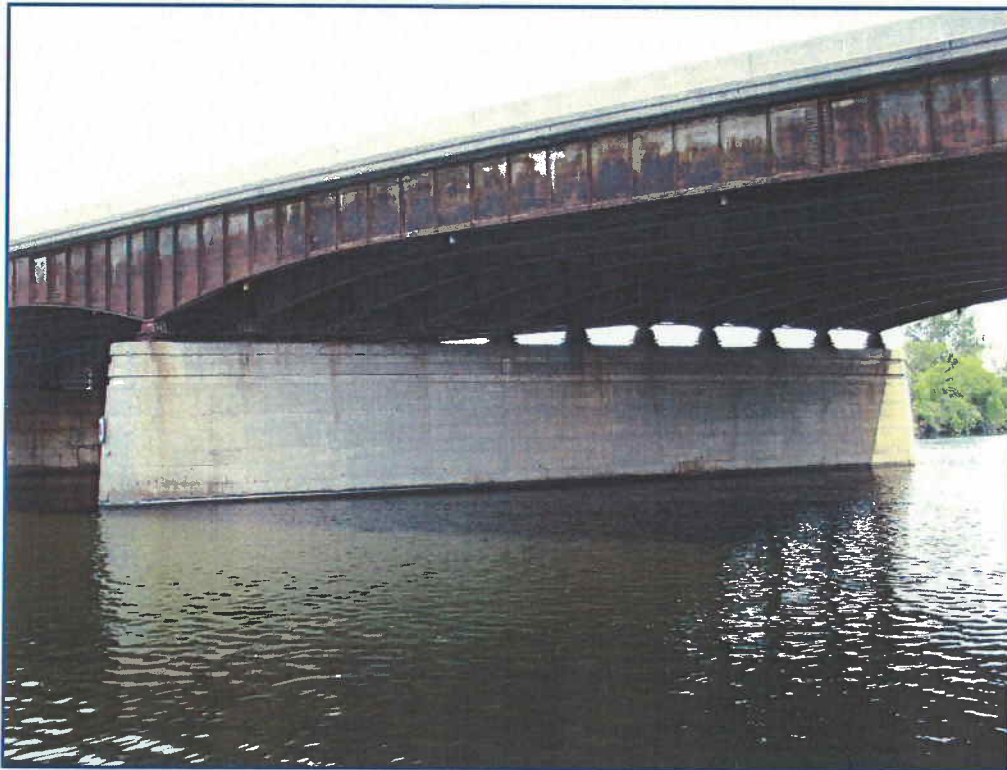
JULY 31, 2007

APPENDIX 1





Photograph 1-7: Pier C - East



Photograph 1-8: Pier C - West

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

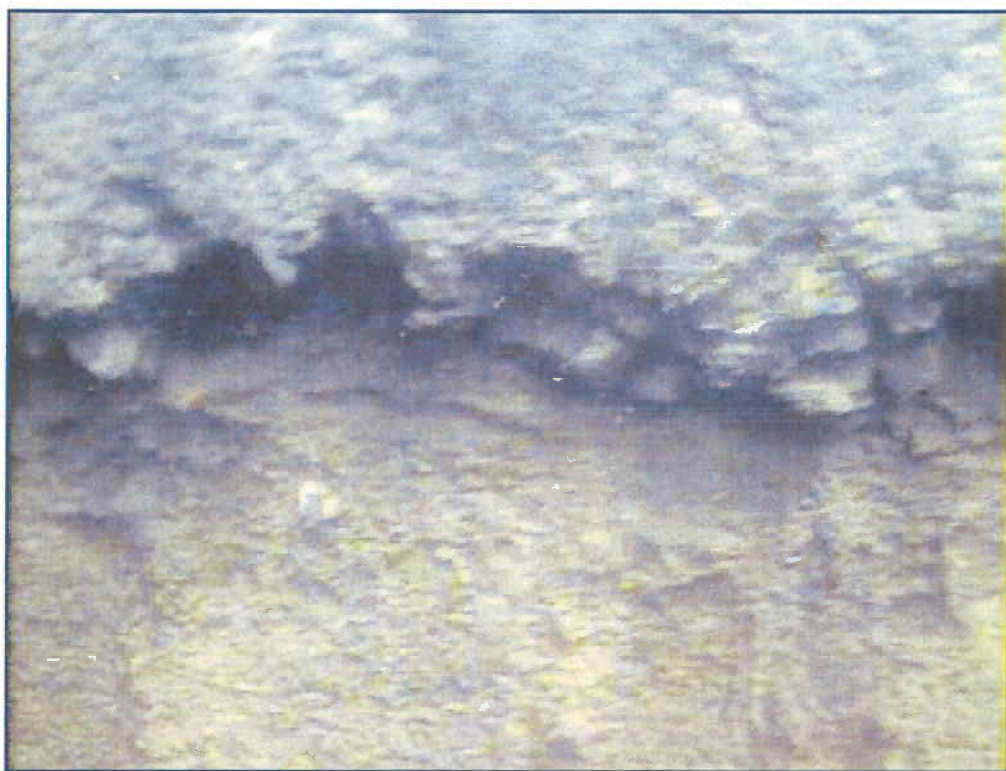
PHOTOGRAPHS

ASI PROJECT D20147

JULY 31, 2007

APPENDIX 1





Photograph 1-9: Pier C - Concrete Overpour at South Nose



Photograph 1-10: Pier C - Crack 2 mm on Centre Line of East Face

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

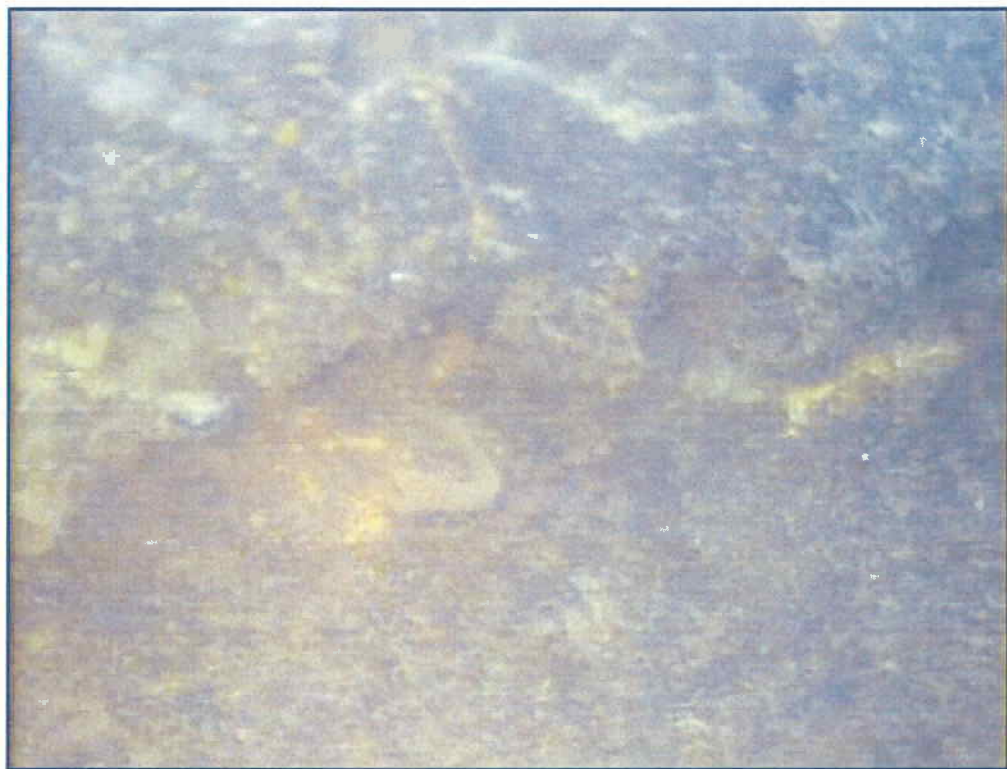
PHOTOGRAPHS

ASI PROJECT D20147

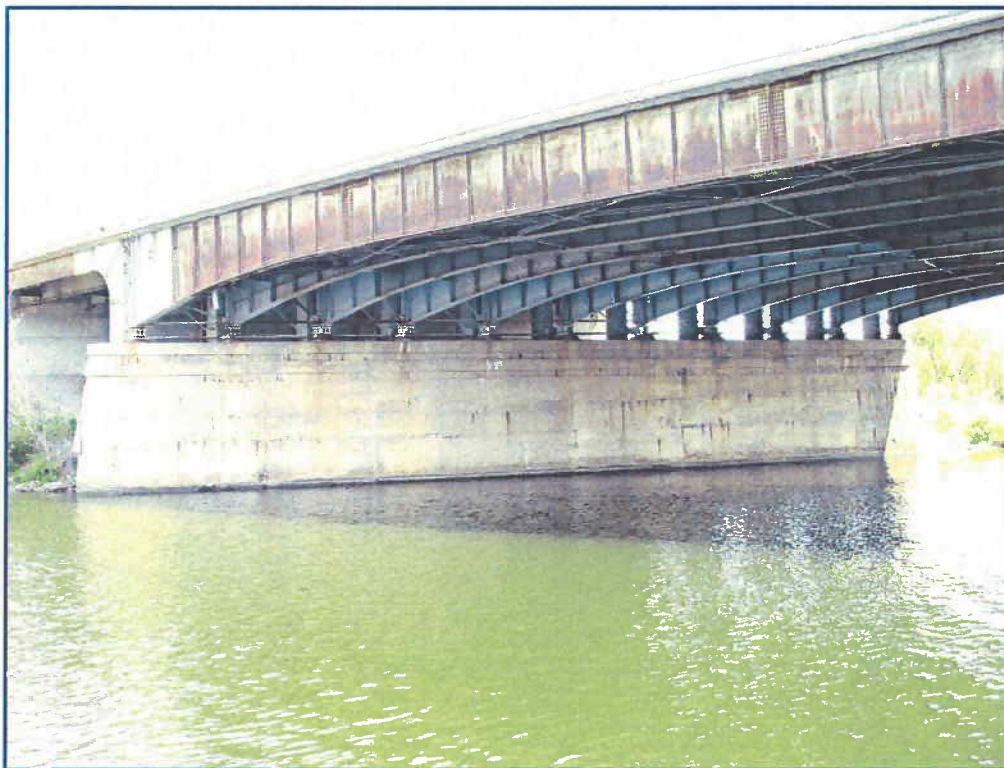
JULY 31, 2007

APPENDIX 1





Photograph 1-11: Pier C - Void on North Nose



Photograph 1-12: Pier D - West

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

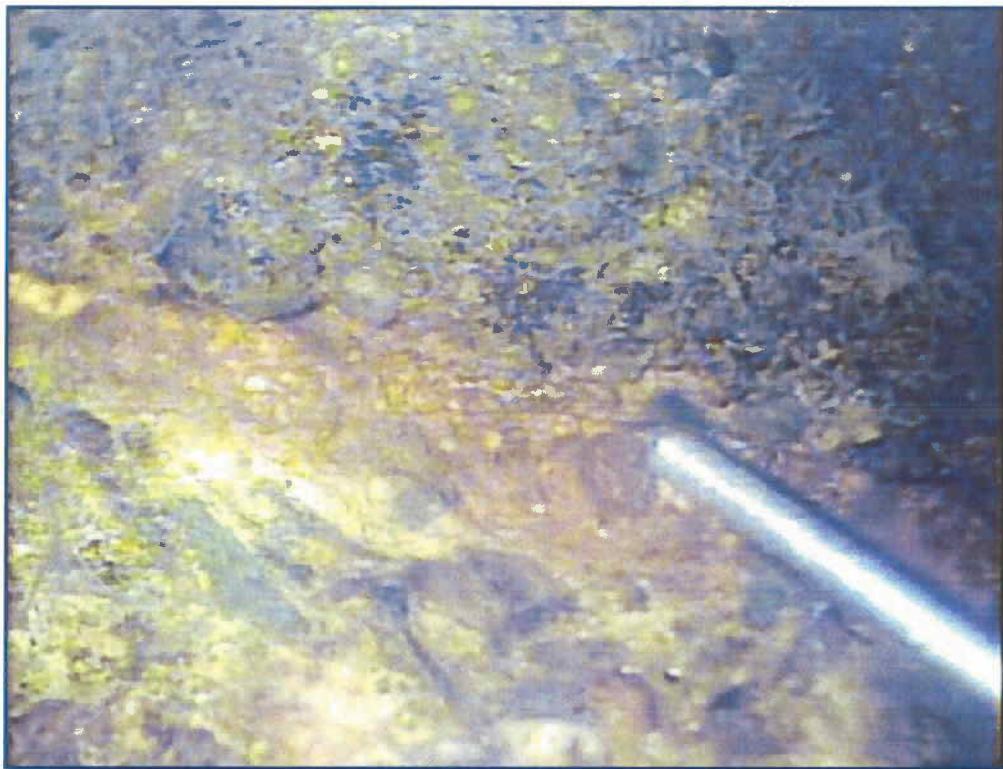
PHOTOGRAPHS

ASI PROJECT D20147

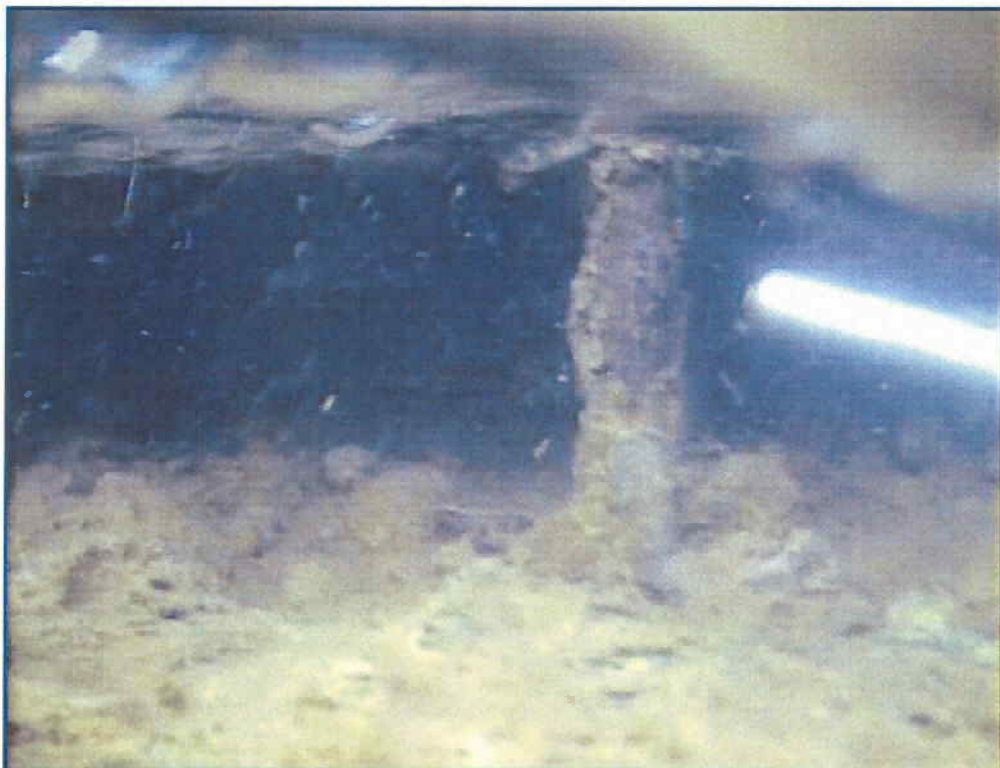
JULY 31, 2007

APPENDIX 1





Photograph 1-13: Pier D - Exposed Rebar in Spall. Girder #4



Photograph 1-14: Pier D - Exposed Rebar North Nose at Surface

**GOLDER ASSOCIATES LTD.
DIVING INSPECTION OF
TRENT RIVER BRIDGE
(HIGHWAY 401)**

PHOTOGRAPHS

ASI PROJECT D20147

JULY 31, 2007

APPENDIX 1



APPENDIX C

**NON-STANDARD SPECIAL PROVISIONS
AND OPERATIONAL CONSTRAINTS**

BEDROCK EXCAVATION - Item No.

Special Provision

The shaley limestone to calcareous shale bedrock at this site varies from weak to extremely strong. Appropriate construction equipment and procedures will be required for excavation into the bedrock. Bedrock excavation shall not disturb the existing Trent River bridge structure.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

COFFER DAMS – Item No.

Special Provision

1.0 SCOPE

As part of the work under this item, the Contractor shall:

- Design, supply, and install coffer dams to construct the footings for the east abutment widening;
- Cut off the top of the coffer dams below grade to the limits indicated in the Contract Drawings.

All work as shown on the Contract Drawings.

2.0 DEFINITION

Stamped:	Refers to drawings or details that have been reviewed and stamped "Conforms With Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer (QVE).
Quality Verification Engineer (QVE):	An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of cofferdams. The Contractor shall retain the QVE to ensure conformance with the contract document.
Coffer Dam Design Engineer:	An Engineer licensed to practice in the Province of Ontario who has a minimum of five (5) years of experience in the field of design and/or construction of bridges. In addition, the Coffer Dam Design Engineer shall have had responsible experience in the design of at least 5 other coffer dams. The Contractor shall retain the Coffer Dam Design Engineer to ensure conformance with the contract documents and issue certificate(s) of conformance for the design.

3.0 SUBMISSION AND DESIGN REQUIREMENTS

Design of concrete cofferdams shall be in accordance with CAN/CSA-S6-00.

Submission of Shop Drawings

All shop drawings submissions shall bear the seal and signature of the Coffer Dam Design Engineer.

The Contractor shall submit to the Quality Verification Engineer shop drawings for review and stamping.

At least two weeks prior to the commencement of coffer dam construction, the Contractor shall submit to the Contract Administrator, for information purposes only, four (4) sets of stamped drawings/calculations of the coffer dam system.

The Contractor shall, at least three (3) weeks prior to the commencement of the coffer dam installation, submit to the QVE for review, four (4) sets of drawings and calculations indicating:

- the coffer dam design;
- the location, type and dimensions of each coffer dam to be used;
- a schematic showing the configuration of all coffer dams;

- the thickness of the tremie plug to ensure stability of the design excavation and coffer dam and the pour sequence of the tremie concrete for which the coffer dam was designed to accommodate unbalanced loading from staged placement and variable heights of the tremie concrete.

The QVE shall review all calculations, construction details, shop drawings and procedures.

All submissions shall bear the seal and signature of the Cofferdam Design Engineer and QVE.

Certificates of Conformance

The Cofferdam Design Engineer shall inspect the installation of each coffer dam prior to the placing of the tremie concrete in that coffer dam. After the installation of the coffer dam has been completed, but before placing the tremie concrete, the Contractor shall submit a Certificate of Conformance to the Contract Administrator, sealed and signed by the Cofferdam Design Engineer. The Certificates of Conformance shall state that the coffer dam is in place, and has been installed in conformance with the stamped shop drawings and the Contract Drawings.

The Contractor will note that several Certificates of Conformance may be required, each to coincide with each coffer dam installation.

5.0 CONSTRUCTION

The soils at the site are glacially- and glaciofluvially-derived and should be expected to contain cobbles and boulders; in addition, obstructions may be present within the existing Highway 401 embankment fill. Appropriate equipment and procedures will be required to penetrate these obstructions during installation of the coffer dam.

Footing construction below the groundwater and/or river water levels must be carried out in dry conditions. The excavation shall be kept stable during the work.

The Contractor shall cut the coffer dam at the limits indicated on the Contract Drawings at the completion of the construction of the footings.

6.0 BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and materials to carry out the work.

TREMIE CONCRETE – Item No.

Special Provision

904.01 SCOPE

Section 904.01 of OPSS 904 is amended by the addition of the following:

As part of the work under this item, the Contractor shall:

- Construct tremie concrete at the base of the coffer dams as shown on the Contract Drawings;
- Core holes from the tremie concrete at each abutment and pier and deliver core samples to the Ministry designated testing laboratory.

All work as shown on the Contract Drawings.

904.07 CONSTRUCTION

Section 904.07 of OPSS 904 is amended by the addition of the following:

The base of the excavation for the east abutment widenings shall be inspected by a diver and cleaned of all overburden soils and loose fractured bedrock within 24 hours of placement of the tremie concrete. A second inspection and cleaning of overburden soils and loose fractured bedrock shall be completed under the direction of the Quality Verification Engineer within 4 hours of placement of the tremie concrete. A Certificate of Conformance shall be issued by the Contractor to the Contract Administrator prior to placement of the tremie concrete.

The Contractor's attention is directed to the requirement for synchronizing the tremie concrete placement with the unwatering operation to ensure that water within the cofferdams is not displaced into the Trent River, as specified under Unwatering Structure Excavation.

The Contractor shall provide underwater inspection via a certified dive team to monitor all phases of the tremie operation, and report status during concrete placement.

After unwatering the structure excavation, all poor, loose, delaminated, weak, or otherwise deficient tremie concrete shall be removed down to sound concrete. No additional payment will be made for additional concrete in footings required to fill the voids due to removal of the above noted tremie concrete. Any damage caused to dowels into rock shall be replaced by the Contractor at no cost to the owner.

904.08 QUALITY ASSURANCE

Section 904.08 of OPSS 904 is amended by the addition of the following:

- Prior to unwatering the structure excavation, the Contractor shall core the tremie concrete.

Cores shall be 100 mm nominal diameter, and shall penetrate the full depth of the tremie concrete, and a minimum 500 mm into bedrock. Cores shall be photographed and logged, and kept intact in a moist condition in accordance with CSA A23.3-14C. The cores will be inspected by the Contract Administrator.

The Contractor shall prepare three specimens from each core for compressive strength testing, in accordance with CSA A23.3-14C. Specimens shall be taken from core locations approved by the Contract Administrator.

Specimens shall be delivered to a testing laboratory approved by the Contract Administrator. The Ministry shall pay for the testing of initial specimens. Remaining core portions shall be relogged and retained until the Contract Administrator provides written permission for disposal.

Core holes shall be filled with 30 MPa concrete. Concrete in Footing shall not be placed until core hole concrete has reached a 10 MPa compressive strength.

Detection of the following shall be cause for rejection of the tremie concrete, as determined solely by the Contract Administrator:

- voids,
- contaminated areas,
- laitence seams,
- segregated areas,
- other defects in tremie concrete
- defects in the footing to rock interface
- all cores shall be tested and shall have a minimum 28-day compressive strength of 30 MPa.

The Contractor shall be responsible, at the Contractor's cost, for all remedial work and retesting required to restore defective work to the satisfaction of the Contract Administrator.

Non-shrink grout for Dowels into Rock as shown on the Contract Drawings or additional Dowels into Rock required by the Contractors operations, shall not be placed until the tremie concrete has been accepted by the Contract Administrator. The Contractor requires written permission from the Contract Administrator to proceed with placing non-shrink grout for Dowels into Rock at the abutment footings.

UNWATERING STRUCTURE EXCAVATION – Item No.

Special Provision

902.01 SCOPE

Section OPSS 902.01 of OPSS 902 is amended by the addition of the following:

As part of the work under this item, the Contractor shall unwater cofferdams. Foundations for the widening of the east abutment and wing walls will require excavation to between 1.5 m and 3.0 m below the groundwater level at the site, through cohesionless (sand and gravel to gravel) soils. These cohesionless soils, when subjected to unbalanced hydrostatic conditions, will slough and cave in without appropriate unwatering/temporary protection.

902.04 SUBMISSION AND DESIGN REQUIREMENTS

Section OPSS 902.04 of OPSS 902 is amended by the addition of the following:

At least two weeks prior to the commencement of cofferdam construction, the Contractor shall submit to the Contract Administrator, for information purposes only, four (4) sets of drawings of the unwatering system showing the unwatering methodology and measures in place to complete the work in dry conditions. The submission shall also provide details of the proposed methods of preventing unwatering or displaced water from directly entering the Trent River.

902.07 CONSTRUCTION

Section OPSS 902.07.06 of OPSS 902 is modified by the deletion of the second paragraph and replacement with the following:

Control of water shall be according to OPSS 518 and the following. Water from unwatering of the abutment cofferdams / excavations shall be contained in a settling pond or sediment basin located a minimum of 30 metres from the edge of water. The settling pond or sediment basin shall be of adequate size / design to allow settling out of sediments prior to any water flowing back to the Trent River. Discharge to the watercourse shall be over a vegetated area.

The Contractor shall also be required to take measures to ensure that water that is displaced during the placement of the tremie concrete does not enter the Trent River. The water within the cofferdam(s) shall be pumped to an on-shore settling pond or sediment basin as described above, in an operation synchronized with the placement of the tremie concrete. A second complete pumping system shall be provided on stand-by to ensure that neither the tremie concrete placement nor the removal of displaced water will be interrupted due to a failure of the primary unwatering system.

The Contractor is advised that water levels at the Trent River bridge site are dependent on operations at the upstream hydro-electric dam. The Contractor shall contact the dam operator to obtain information regarding their water control procedures, and shall conduct and coordinate the construction activities and dewatering efforts required under this Contract accordingly. The Ministry will not be held responsible for information provided by the dam operators.

DOWELS INTO ROCK – Item No.

Special Provision

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provision, except as extended herein. This document specifies additional requirements for the supply, installation and testing of dowels into rock for the structure footings.

1.2 Instructions to Contractor

- 1.2.1 These instructions are to be read in conjunction with the Contract Drawings.
- 1.2.2 A total of 1 test dowel into rock is required for the each structure footing.
- 1.2.3 Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

- 1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the installation of Dowels into Rock. Such experience shall have been obtained within the preceding five years on projects of similar nature and scope to the work required for this project.
- 1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.
- 1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

- 1.4.1 The Contractor shall prove the allowable bond stress by tests of the dowels into rock on non-production dowels into rock.

- 1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production dowels into rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.
- 1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.
- 1.4.4 The Contractor shall submit four copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

- 1.5.1 Dowels into rock: reinforcing steel bar and non-shrink grout.
- 1.5.2 Design Engineer: An Engineer who has a minimum of five years of experience in all aspects associated with the installation of dowels into rock, including drilling, grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the dowels into rock.
- 1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five years of experience in all aspects associated with the installation of dowels into rock, including drilling, grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

- 1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.
- 1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:
- All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
 - All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.
- 1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.

- 1.6.4 Working Drawings consisting of testing and installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.
- 1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:
- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of dowels into rock.
 - Test results verifying the 28-day strength of non-shrink grout.
 - The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
 - The procedures to verify hole length. Records of measurements that verify the hole length.
 - Records of all drilling procedures, rock conditions encountered, and installation times.
 - Test procedures for dowels into rock.
 - Drawings and design calculations for a suitable reaction system for the applied test loads.
 - Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
 - Drawings and details for reference system arrangement.
 - Current calibration curves shall be provided for all gauges.
 - Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
 - Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

- 1.7.1 Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout.

The Contractor shall provide the following information from the manufacturer for non-shrink grout:

- Data sheets for the non-shrink grout; and
- installation procedures.

3.0 EQUIPMENT

3.1 General

- 3.1.1 All equipment for the installation of the dowels into rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.
- 3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of dowels into rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

- 4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.
- 4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator.
- 4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

- 4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.
- 4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.
- 4.2.3 Dowels shall extend through the tremie concrete for the footing and into sound bedrock.
- 4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout

- 4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.
- 4.3.2 The placement of grout for the test dowels into rock shall be identical to the production dowels into rock.
- 4.3.3 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of dowels into rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

- 5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.
- 5.1.2 The Contractor shall install the number of dowels into rock specified in the contract documents for testing purposes. The purpose of the testing the dowels into rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.
- 5.1.3 The equipment, labour and materials for test dowels shall be identical to dowels into rock at the each structure foundation location.
- 5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.
- 5.1.5 The Quality Verification Engineer shall supervise the testing of the dowels into rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for dowels into rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.
- 5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the dowels into rock. The equipment and materials shall be capable of stressing the dowels into rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.
- 5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

- 5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.
- 5.2.2 The test dowels into rock shall be constructed at locations specified by the Contract Administrator.
- 5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

- 5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.
- 5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.
- 5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:
 - The beams shall be independently supported with the support firmly embedded in the ground.
 - The testing device shall not apply compression to the bedrock surrounding the test for the dowels into rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
 - Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.
- 5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

- 5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit

Working Drawings that include the above noted records to the Contract Administrator.

- 5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 153 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.
- 5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

- 5.5.1 The testing procedures shall safely load test the dowels into rock in tension at a rate of approximately 100kN per minute to the specified test load. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.
- 5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

- 5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the instalment of dowels into rock at each structure location.

Tests for dowels into rock shall have a capacity of at least [insert value] kN. The Quality Verification Engineer shall report on the acceptance of the tests for dowels into rock. The Quality Verification Engineer shall report on the testing of the dowels into rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for dowels into rock which are deemed as included as part of the work for the above noted item.

**CONTROL OF OVERBURDEN SOILS AND GROUNDWATER DURING CAISSON
INSTALLATION - Item No.**

Special Provision

Caisson excavations will be advanced through cohesionless fill and sand and gravel to gravel soils. Appropriate construction procedures and equipment will be required to minimize ground loss during drilling and concrete placement.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

CAISSON SOCKETS IN BEDROCK - Item No.

Special Provision

The shaley limestone to calcareous shale bedrock at this site varies from weak to extremely strong. Appropriate construction equipment and procedures will be required for construction of caisson foundation sockets within the bedrock.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION