GEOCRES No:
31D-595

FOUNDATION DESIGN CONFIRMATION REPORT
HIGHWAY 400 NBL & SBL STRUCTURES
PORT SEVERN, ONTARIO
GWP 2360-09-00 & GWP 2376-09-00
Contract No: DB-2013-2048

Report to

MMM Group Ltd.

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

Part 1 of the report presents the results of a review of the factual, subsurface information provided as part of the Design-Build (DB) contract package.

There are two structures included in the contract: structure over the Severn River and that over the Severn River Boat Channel. In each case, the existing SBL structure will be rehabilitated and will be widened to accommodate both southbound and northbound traffic. Following completion of this work, the existing northbound structures will be demolished.

A Foundation Investigation and Design Report was prepared for each site by Coffey. These reports are titled:

1. Foundation Investigation and Design Report, Proposed Widening of Southbound Highway 400 Bridge over the Severn River, W.P. 2360-06-00, Site 42-86/1&2, dated January 7, 2014
2. Foundation Investigation and Design Report, Proposed Widening of Southbound Highway 400 Bridge over the Severn River Boat Channel, Township of Baxter, MTO Central Region, W.P. 2376-09-00

These two reports are included as Appendix A and Appendix B, respectively, in this report.

In addition to reviewing these reports, Thurber carried out coring at the Severn River Bridge to investigate the condition of the existing mass concrete under the structure footings.

Thurber carried out this assignment as a sub-consultant to MMM Group Limited.

2 SITE DESCRIPTION

The sites lie on Highway 400 approximately 7.3 km north of the junction with Highway 12 and approximately 54 km north of Barrie. At this location the Severn River and Trent Severn Waterway empty into Georgian Bay.

The site geological setting is adequately described in the reports provided for the contract.

3 SITE INVESTIGATION AND FIELD TESTING

Site investigation and testing was carried out at each site by Coffey and the respective programs are described in the reports. Reference should be made to Coffey's reports for details of the investigations.

Thurber has reviewed the extent of these programs and is satisfied that sufficient data was gathered to support the preparation of geotechnical design recommendations for the structure foundations.

4 LABORATORY TESTING

Laboratory testing was carried out on selected samples and the results are shown in the reports. Sufficient information was made available to support the design recommendations.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference should be made to Coffey's reports in Appendices A and B for a detailed description of the subsurface conditions.

In summary, it is apparent the original site conditions consisted of exposed bedrock or bedrock overlain by a thin veneer of sand or silty sand and thin topsoil. Highway construction has resulted in the placement of fill and pavement structure. The fill can generally be described as silty sand, trace to some clay, trace gravel

6 ADDITIONAL INVESTIGATION

6.1 Concrete Coring

Thurber arranged and supervised the coring of the mass concrete supporting the footings of the Severn River bridge between June 12 and June 18, 2014.

Core was retrieved from two locations in the north abutment foundation and two locations in the south abutment foundation. Due to challenges in setting up equipment, 89 mm diameter core was retrieved to depths of less than 1 m and 35 mm core was retrieved to greater depths.

Core 1, in the south side, encountered bedrock at a depth of 4.8 m, approximate Elevation 170.2. Core 4, in the north side, encountered bedrock at a depth of 4.2 m, approximate Elevation 170.6.

Core photographs, including locations, and the results of compressive strength testing are included in Appendix C.

6.2 Test Pit Inspection

On May 28th, 2014, Thurber staff inspected the bedrock exposed in a test pit excavated immediately east of the east column of the existing north pier at the Severn River Boat Channel structure. The purpose of this excavation was to investigate the limits of rock excavation or disturbance from the construction of the existing SBL structure.

Sketch Sk. 1 in Appendix C shows the approximate surface of sound rock as observed in the test pit. The test pit was flooding to Elevation 176.2 and it was not possible to expose and observe the existing footing using the available equipment.

7 MISCELLANEOUS

The review and reporting have been carried out by Mr. Alastair Gorman, P.Eng. and the report has been reviewed by Dr. P.K. Chatterji, P.Eng., a Designated principal Contact for MTO Foundations projects.

Thurber Engineering Ltd.

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FOUNDATION DESIGN CONFIRMATION REPORT

HIGHWAY 400 NBL & SBL STRUCTURES

PORT SEVERN, ONTARIO

GWP 2360-09-00 & GWP 2376-09-00

Contract No: DB-2013-2048

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8 INTRODUCTION

This section of the report presents a review and, where necessary, updating of the geotechnical design recommendations based on the factual information as well as our understanding of the project. The plans and profiles and GA drawings used for preparation of this part of the report were provided by MMM Group Limited.

Both SBL bridges will be widened using construction that matches the existing, i.e. a rigid frame over the Severn River and a concrete deck on steel plate girders at the Boat Channel. Descriptions of the structures and embankments are to be found in Coffey's reports.

9 CONFIRMATION OF STRUCTURE FOUNDATIONS

The geotechnical recommendations for foundation design in Coffey's reports have been reviewed and have been confirmed to be generally appropriate for the design of the project. Comments of the more important points follow.

9.1 Severn River

In general the report presents adequate information on which to base geotechnical design recommendations for the foundations and approach fills.

This will be a rigid frame structure matching the existing SBL structure.

It is understood that the new footings will be placed on the existing mass concrete supporting the footings of the median retaining walls (which will be removed). The Preliminary GA shows the mass concrete founded at approximate Elevation 172 at the south abutment and Elevation 171 at the north abutment.

Borehole 4, drilled in the channel, shows bedrock at Elevation 172.3 near the south abutment and Borehole 6, also drilled in the channel, shows bedrock at Elevation 170.9.

As mentioned earlier, these boreholes were not drilled on the alignment of the foundation of the widened structure and some difference in bedrock elevation must be expected between the boreholes and the foundation. The mass concrete must be founded at least 300 mm into the bedrock and on an essentially level surface.

The results of coring the mass concrete are reported in Appendix C. At the core locations, bedrock was encountered as follows:

Location	Depth (m)	Elevation
South abutment	4.8	170.2
North abutment	4.2	170.6

Based on the results of compressive strength testing shown in Appendix C, the lowest strength value is 25.7 MPa at the top of the mass concrete at the south abutment.

Cofffey's report recommends a factored bearing resistance at ULS of 10,000 kPa on sound bedrock. This is an appropriate value for exposed, sound bedrock. However, if the existing mass concrete is used to support the new footings, it is recommended that a maximum value of 5,000 kPa be used for the existing mass concrete bearing on bedrock. The SLS condition will not govern for bearing on bedrock.

The maximum factored bearing pressure exerted by the new footing on the mass concrete must not exceed 5,000 kPa. A structural assessment of the mass concrete must also be carried out.

After the existing retaining wall footings have been removed, the top of the mass concrete must be visually inspected and any damaged or deteriorated concrete identified by this inspection must be removed.

There is a degree of constructability and cost risk associated with the exact bedrock topography.

9.2 Severn River Boat Channel

In general the report presents adequate information on which to base geotechnical design recommendations for the foundations and approach fills. A possible exception is the area of the north pier foundation where the investigation interprets sound bedrock whereas it is known that there is a disturbed zone from the construction of the existing bridge – as described below. The bedrock surface is variable across the site and since the boreholes do not delineate the footing locations exactly, there is some cost risk associated with rock elevations.

South Abutment

Underside of abutment – 182.0

Top of bedrock (from BH 11 and 12) – 176.0 to 176.3

Length of pile from the underside of abutment to bedrock – 5.7 to 6m.

This pile length meets the minimum 5 m given in the Integral Abutment Design Guidelines. The foundation recommendations provided by Coffey are acceptable. However, if piles socketed into bedrock have to be used, a 600 mm deep socket filled with 30 MPa concrete is considered to be adequate.

South Pier

Top of bedrock – 174.9 to 175.1 (sound bedrock interpreted at 174.3 to 174.7)

We recommend a spread footing bearing on bedrock for this pier. Coffey gave a factored ULS bearing resistance of 10,000 kPa, which is possible but very high and we would not recommend proportioning the footing on the basis of that pressure. A maximum of 5,000 kPa might be more reasonable and even at that, other consideration may govern the sizing.

Founding at Elevation 174.0 or lower should be satisfactory. However, factors to consider include:

1. The bedrock surface may vary across the width of the footing.
2. An essentially level founding surface must be prepared.
3. A coffer dam is most likely required.
4. When it comes to detail design, it may be worth considering a slightly higher elevation for the footing, with provision for structural concrete fill between the footing and the final bearing elevation.

North Pier

Top of bedrock 176.8 to 177.2 (sound bedrock 176.5 to 177.1)

Based on the FIDR, a spread footing founded at Elevation 176.5 would seem appropriate. However, it is known that the adjacent footing for the existing structure is founded at Elevation 172.5 and that bedrock excavation was carried out to reach this elevation.

As shown in the Sketch Sk 1, the face of the rock excavation essentially coincides with the east face of the west pier column. For practical reasons, the excavation could not be taken below Elevation 174±.

The investigation on site suggests that conditions at the east column are represented by Coffey's Borehole 15, drilled closest to the existing foundation, and showing 100% solid core recovery in 3 out of 4 core runs and 89 to 100% RQD below the top run. This suggests good founding conditions below Elevation 176 at a short distance away from the existing foundation.

The preliminary design recommendation for the north pier columns is caissons founded below Elevation 172.5 and in sound bedrock.

Based on information obtained through inspection of the test pit at the north pier, it is recommended that both columns be supported on spread footing bearing on bedrock at a bearing pressure of 5,000 kPa, factored ULS.

The east footing can be founded over the existing bedrock surface using mass concrete for levelling and dowels to resist sliding.

At the west column, it is recommended that the bedrock be excavated to the extent necessary to accommodate founding the footing at Elevation 172.5. However, bedrock excavation must not extend beyond what is necessary, in order not to jeopardize the bearing conditions at the east column.

North Abutment

Top of bedrock – 177.3 to 179.3 in the four boreholes that bracket the abutment area. Borehole 18 is closest to the abutment and in that borehole the bedrock was encountered at Elevation 178.5.

The Preliminary GA shows the underside of abutment at elevation 182.6. To meet the integral abutment guidelines, the free length of pile must extend to elevation 179.6 and fixity must be achieved below that elevation.

Provided the bottom of the rock socket is completely clean, the pile can be placed with a square cut end. However, based on the designer's preference, a steel pin may be attached to the pile tip to facilitate concrete flow around the tip. The hole above the concrete must be backfilled with loose sand conforming to the requirements of the integral abutment guidelines.

The piles must be at least 6 m long but an embedment of 600 mm in 30 MPa concrete is considered to be adequate. The following note is suggested for the installation of piles that will be concreted into a rock socket:

> "Auger to top of bedrock using a temporary liner. Diameter of augered hole to be at least 600mm and large enough to permit drilling or coring of a minimum 600mm dia hole into the bedrock. Drill or core a 600mm dia hole into bedrock to a depth of 6m below underside of abutment or 750mm into bedrock, whichever is deeper. Clean drill cuttings from base of hole. (Typ.).
>
> Place HP 310X110 pile resting on undisturbed bedrock at bottom of hole and fill bottom 600 to 750 mm of hole with 30MPa concrete. Fill remainder of hole with loose sand, removing the liner as the sand is placed. (Typ)."

10 CLOSURE

The engineering assessment and recommendations were prepared by Mr Alastair Gorman, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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Review Principal

Appendix A
Severn River Report

McCormick Rankin

Foundation Investigation and Design Report

Proposed Widening of Southbound Highway 400 Bridge
Over the Severn River, W.P. 2360-06-00,
Site 42-86/1&2, GEOCRES 31D-564

TRANETOB20462AA
07 January, 2014

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January 07, 2014

McCormick Rankin
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Attention: Mr. Ben Hui, P.Eng., M.Eng., Senior Project Manager

Dear Mr. Hui;

RE: Foundation Investigation and Design Report
Proposed Widening of Southbound Highway 400 Bridge over the Severn River
W.P. 2360-06-00, Site 42-86/1&2

Please find attached our foundation investigation and design reports relating to the above noted site.

If you have any comments or enquiries please contact the undersigned.

For and on behalf of Coffey.

Zuhtu Ozden, P.Eng.
Senior Principal

FOUNDATION INVESTIGATION REPORT
PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE OVER THE SEVERN RIVER
W.P. 2360-06-00, SITE 42-86/1&2, GEOCRES31D-564

McCormick Rankin

TRANETOB20462AA
January 07, 2014

REPORT

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CONTENTS

FOUNDATION INVESTIGATION REPORT
PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE
OVER THE SEVERN RIVER, W.P. 2360-06-00, SITE 42-86/1&2

1 INTRODUCTION

Coffey was retained by McCormick Rankin (MRC) to carry out a foundation investigation for the proposed Highway 400 southbound Bridge widening for realigned northbound lanes over the Severn River in the Township of Tay, Ontario.

The existing northbound Severn River Bridge is an approximately 31 m long single span, rigid frame concrete structure, supported on shallow foundations bearing on mass concrete inset 0.3 m into bedrock. This circa 1957 structure will be demolished. The existing southbound bridge which was built in 1991 will be widened to accommodate the proposed realigned northbound lanes. The widening will take place towards the median of the existing highway.

At the present time, the bridge widening is expected to be similar to the existing southbound bridge, which is a single span, rigid frame concrete structure with a clear span length of 27.5 m and a total length of 46.5 m.

The purpose of this investigation was to obtain information about the subsurface conditions at the proposed bridge widening site by means of boreholes, and to determine the engineering characteristics of the overburden soils and of the underlying bedrock, by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

The site is located on Highway 400 at the mouth of Severn River at Little Lake joining Georgian Bay, as shown on Drawing 1. The surrounding area is generally gently rolling and rock outcrops are visible in the vicinity.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located at the interface of Physiographic Regions ‘Algonquin Highland’ and ‘Carden Plain’.

The geology at the site is dominated by felsic igneous bedrock with shallow overburden. Bedrock at the site is known as granite and biotite gneiss of the Grenville Province.

According to Map 2418 of Ontario Geological Survey, the site is located immediately north of the confluence of Precambrian rocks with more recent Ordovician formations. The main body of geologic formations consist of late to middle Cambrian clastic metasediments which are comprised of conglomerate, greywacke, arkose, calcareous sandstone and siltstone, shale and derived metamorphic rocks, while in the vicinity of the site late Precambrian granitic to syenitic rocks are also found.

Previous site specific investigations show the presence of granite gneiss rocks.

Overburden, where present, consists of silty sands, either surficial loose deposits or as dense glacial till above the bedrock. Silty clay is also present in areas where bedrock is relatively deeper in occurrence. Organic mucks are also common in marshy areas.

3 INVESTIGATION PROCEDURES

The field work for this investigation was performed during the period of May 23 to June 14, 2013 and consisted of drilling and sampling eight boreholes. Boreholes 1, 2, 7 and 8, which were advanced from the top of the existing road embankment by augering, were terminated upon encountering refusal on the augers, on possible bedrock surface. The depth of these boreholes ranged from 5.8 to 10.7 m.

Boreholes 3 and 5 were also advanced from the top of the road embankment but in these boreholes rock coring was implemented upon encountering refusal at depths of 8.3 and 13.1 m, respectively. In these boreholes, the bedrock was proven by diamond drilling and obtaining NQ size rock cores to depth of 12.1 and 16.5 m, respectively, below the ground surface.

Boreholes 4, 4A and 6 were advanced on water from a barge, in the River. These boreholes were advanced in the overburden by washboring methods inside a steel casing. Upon encountering refusal to washboring at depths of between 3.7 m and 5.1 m below the water's surface in the River, the bedrock was proved in Boreholes 4 and 6 by rock coring and diamond drilling methods and obtaining BQ size rock cores to between 3.0 and 3.1 m below the bedrock surface or to depths of between 6.8 and 8.1 m below the water surface in the River. Overburden in Borehole 4 could not be sampled due to the presence of rock fill and therefore another borehole (BH 4A) was put down nearby Borehole 4, away from rock fill, in order to obtain samples of the overburden and to carry out standard penetration tests.

The drilling of boreholes put down from land was carried out by Davis Drilling of Milton, Ontario, while boreholes from the barge were effected by Walker Drilling of Utopia, Ontario.

The field work was carried out under the supervision and direction of an engineer from our office. The boreholes were advance using a track mounted or a barge mounted drilling rig, outfitted with tools and equipment for soil sampling and testing.

The boreholes were advanced using three different methods (i.e. continuous–flight, hollow-stem augers and washboring in the overburden and rock coring) depending on the subsurface conditions.

Samples in the overburden were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

Rock coring was implemented using NQ or BQ size cores.

Boreholes 1, 2, 3, 5, 7 and 8 were advanced by a track mounted CME 55 drill rig owned and operated by Davis Drilling Ltd. Of Milton, Ontario, while Boreholes 4, 4A and 6 were advanced from a barge using a D25 Diedrich type drill rig owned and operated by Walker Drilling Ltd. of Utopia, Ontario.

Groundwater conditions in the open boreholes were observed during drilling and upon completion. In addition, a piezometer was installed in each of Boreholes 2 and 8 to enable groundwater level monitoring in the boreholes over a prolonged period of time without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures. Boreholes 2 and 8, in which piezometers were installed, were not decommissioned, as piezometers may be useful during the construction. We recommend that a clause be included in the Contract Documents to decommission these two boreholes during the construction, as part of the Contract.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were determined by the client's surveyors. The survey information was provided to us.

The soil and rock samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory programme, consisting of natural moisture content, grain size analyses, and Atterberg Limit tests, was performed on selected representative soil samples and point load tests on selected rock cores. In addition selected rock cores were sent to Golder Associates Laboratory in Mississauga, Ontario to carry out unconfined compression tests.

4 SUBSURFACE CONDITIONS

The subsurface conditions were explored at eight boreholes plus a ninth borehole (BH 4A) adjacent to Borehole 4. The plan locations of the boreholes and profile are shown on Drawing No. 1, while stratigraphic sections at foundation locations are presented on Drawing Nos. 2 and 3.

Boreholes 1, 3, 5 and 7 were advanced from the top of the highway embankment, from the paved portion of the highway and contacted 120 to 190 mm of asphaltic concrete underlain by granular pavement fill, which is in turn underlain by embankment fill to depths of 7.3 to 13.1 m or to El. 176.6 to 172.1 m.

Borehole 2 was advanced from the unpaved portion of the highway embankment and contacted below a 0.1 m thick veneer of topsoil, embankment fill extending to a depth of 5.3 m below the ground surface or to El. 177.3 m.

In Boreholes 5 and 7, the embankment fill extends right down to the surface of the bedrock /inferred bedrock, while in Boreholes 1, 2 and 3, the embankment fill is underlain by a 0.5 m thick basal sand/silty sand layer, overlying the bedrock, at El. 176.8 to 175.9 m.

Borehole 8 was also advanced from the unpaved portion of the highway embankment and in this borehole, below 0.15 m topsoil, the embankment fill is underlain by 1.7 m of gravelly sand at a depth of 7.3 m or at El. 177.1 m, which is further underlain at a depth of 9.0 m below the ground surface or at El. 175.4 m, by a silty clay deposit. The silty clay deposit at this borehole location is 1.7 m thick and extends to 10.7 m (El. 173.7 m) where the surface of the bedrock was inferred from refusal to further augering.

Boreholes 4, 4A and 6 were advanced from a barge. Below 1.7 to 2.2 m water in the river/lake, the river/lake bottom was contacted at between El. 174.3 m and 173.8 m. The overburden encountered in Boreholes 4A and 8 consisted of basically sandy (granular) soils to the surface of the bedrock at El. 172.6 m and 170.9 m, respectively. In Borehole 4A, a 0.6 m thick silty clay layer was encountered, in between two layers of granular overburden soils. In Borehole 4, the overburden was mixed with rock fill.

In summary, below up to about 13 m of embankment fill and some native shallow overburden, the surface of the bedrock at the borehole locations were found/inferred at between El. 176.8 m (BH 2) and 170.9 m (BH 6).

At the locations of Boreholes 1, 2, and 3 on the east side of the River, the surface of the bedrock was contacted/inferred at El. 176.8 and 175.9 m (relatively level). However at the location of Borehole 4 it was contacted at El. 172.3 m (at an elevation of about 4 m lower). This is likely to be due to previous construction activities and possibly due to erosion by the River. On the west side of the River, the surface of the bedrock at Boreholes 5 and 8 were contacted/inferred at El. 172.1 and 173.7 m, respectively, while at Borehole 7, there appears to be a high point, as the surface of the bedrock at this location was inferred at El. 176.2 m. At Borehole 6, which was drilled in the River, the surface of the bedrock was contacted at El. 170.9 m (i.e. at a low elevation), probably due to river erosion or also possibly due to construction activities, similar to Borehole 4.

The bedrock was found to consist of greyish/pinkish granite gneiss of generally sound quality.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to amplify and complement these data.

4.1 Asphalt

Boreholes 1, 3, 5 and 7, which were advanced from the paved portion of the highway embankment, contacted 120 mm (BH 7) to 180-190 mm (BH 1, 3 and 5) of asphaltic concrete.

4.2 Topsoil

In Boreholes 2 and 8, which were drilled from the existing highway embankment, a 0.1 to 0.15 m thick topsoil layer was found at the ground surface level.

4.3 Pavement and Embankment Fill

Boreholes 1, 2, 3, 5, 7 and 8 were advanced from the existing highway embankment and contacted about 5.3 to 12.9 m thick pavement and/or embankment fill.

In Boreholes 5 and 7, the embankment fill was found to extend to the surface of the bedrock/inferred bedrock at depths/elevations of 13.1 m /172.1 m and 9.3 m/ 176.2 m, respectively.

In Boreholes 1, 2, 3 and 8, the embankment fill was found to be underlain by native overburden at depths of 5.3 to 7.8 m below the ground surface or at El. 177.3-176.4 m.

Granular pavement fill was contacted below the paved portion of the roadway, underlying the asphaltic concrete. The grain size distribution of four samples from the granular pavement fill is given in Appendix B in Figure B-1. These indicate the following grain size distribution:

Gravel:	22-40%
Sand:	46-63%
Silt & Clay:	12-16%

The embankment fill generally consists of a heterogeneous mixture of silty sand to sandy silt with traces to some clay and gravel size particles. From its grain size distribution and the general appearance of the samples from the fill, as retrieved by the split spoon sampler, it appears that the fill was derived from the indigenous glacial till deposits. The fill was found to be generally clean (i.e. devoid of deleterious soils/materials, such as organics). The presence of occasional clayey zones was also noted.

The grain size distribution of ten samples from the embankment fill is given in Figure B-2, in an envelope form, in Appendix B. The following grain size distribution is indicated:

Gravel:	2-10%

Sand:	49-66%
Silt:	18-27 %
Clay:	13-17%

Figure B-3 in Appendix B shows the grain size distribution of samples from the more siltier zones of the fill. The curves indicate the following grain size distribution:

Gravel:	2-9%
Sand:	34-41%
Silt:	33-46%
Clay:	17-18%

There are occasional gravelly zones which were encountered in the makeup of the embankment fill. Such a zone was contacted in Borehole 5 immediately beneath the pavement fill and was found to extend to a depth of 3.7 m or to El. 181.5 m. The grain size distribution curve of a sample is given in Figure B-4, indicating the following:

Gravel:	41%
Sand:	44%
Silt:	13%
Clay:	2%

The embankment fill is considered to be a typically granular (non-cohesive) soil. The presence of cobbles and boulders should always be anticipated in fill which are derived from glacial till (which the bulk of the embankment fill at this site appears to be), unless, of course, such coarser particle sizes were removed from the fill during its construction. As well, some of the coarser gravel, which is presented, may be misrepresented in the split-spoon-samples (i.e. the percentage of gravel may be higher than shown on the results presented). Standard Penetration Tests (SPT) performed in the embankment fill yielded N-values which generally ranged from 3 to 57 blows/0.3m. There are some higher recorded values, but there were attributed to the presence of oversize gravel particles in the fill. The recorded N-values indicate a very loose to very dense relative density. In most cases, the recorded average N-values lie in the range of 10 to 20 blows/0.3m, which indicate a generally compact material with some loose and occasional very loose and dense zones. From these results it appears that some systematic compaction was applied when the embankment was first constructed some twenty years ago, but the compactive effort was applied somewhat sporadically where some zones received little or no compaction.

4.4 Native Overburden

Natural (i.e. native) overburden was contacted in Boreholes 1, 2, 3, 4A, 6 and 8. The thickness of the native overburden at the borehole locations was found to range from 0.5 m at Boreholes 1, 2 and 3; 1.7 m at Boreholes 8 and 4A to 2.9 m at Borehole 6. The native overburden was found to typically consist of sandy (granular) soils but layers of a cohesive (silty clay) deposit were contacted in Boreholes 4A and 8, as described in the following paragraphs.

4.4.1 Silty Sand, Sand, Gravelly Sand and Sand & Gravel

Basically granular basal soils, consisting of silty sand to sand, were contacted in Boreholes 1, 2, 3, 4A and 6. At some borehole locations, these deposits were found to contain traces to some gravel.

These deposits were contacted in Boreholes 1, 2 and 3, immediately below the embankment fill at elevations ranging from 177.3 to 176.4 m and extended to the surface of the bedrock/inferred bedrock at a depth of 0.5 m below these elevations (i.e. 0.5 m thick deposit) at El. 176.8 to 175.9 m.

The grain size distribution of a sample from Borehole 3 is given in Figure B-5 in Appendix B, which indicates the following grain size range:

Gravel:	8%
Sand (mostly fine sand):	65%
Silt &Clay:	27%

These granular (non-cohesive) soils were found to be wet and water bearing and based on N-values of 38 to greater than 100 blows/0.3 m, their relative density is described as dense to very dense.

Boreholes 4A and 6 were advanced from a barge in the River. In Borehole 6, a 0.8 m thick sand layer was contacted immediately below the River bottom at El. 173.8 m. A Standard Penetration test performed in this deposit yielded an N-value of 7 blows/0.3 m, indicating a loose condition. In Borehole 4A, a 0.6 m thick sand layer was contacted at a depth of 1.1 m below the River bottom or at El. 173.2 m. This deposit extended to the surface of the bedrock and based on a recorded N-value of 22 blows/0.3 m, its relative density is described as compact.

In Borehole 8, a gravelly sand deposit was contacted below the embankment fill at depth/elevation of 7.3 m/177.1 m. The thickness of this deposit, which was identified as a possible fill, extended to depth/elevation of 9.0 m/175.4 m at the surface of underlying basal silty clay.

The grain size distribution of the sample recovered from this granular (non-cohesive) deposit is presented in Figure B-6 (Appendix B). The results are as follows;

Gravel:	26%
Sand:	65%
Silt &Clay:	9%

From a recorded N-value of 16 blows/0.3 m, the relative density of this layer can be described as compact.

Sand and gravel layers were contacted in Boreholes 4A and 6. In Borehole 4A, the deposit was contacted immediately below the River bottom at El. 174.3, and extended to the surface of underlying silty clay at El. 173.8 m (i.e. 0.5 m thick). From a recorded N-value of 6 blows/0.3 m this river bottom deposit is described as loose. In Borehole 6, another sand & gravel layer was contacted at a depth of 2.0 m below the River bottom. This deposit was found to be 0.9 m thick and extended to the surface of the bedrock at El. 170.9 m. From a recorded N-value of in excess of 100 blows/0.3 m, the relative density of this basal granular soil is considered very dense.

4.4.2 Silty Sand Till

Borehole 6 contacted at 0.6 m below the River bottom or at El. 173.0 m, a 1.2 m thick glacial till layer consisting of a heterogeneous mixture of silty sand with traces of gravel and clay size particles. The grain size distribution of a sample recovered from this granular (non-cohesive) deposit is given in Figure B-7 in Appendix B. The grain size distribution was found to be as follows;

Gravel:	12%
Sand:	62%

Silt &Clay: 26%

Standard Penetration tests performed in this deposit yielded N-values of 70 and in excess of 100 blows /0.3 m, which indicate a very dense relative density.

4.4.3 Silty Clay

A 0.6 m thick layer of silty clay was contacted in Borehole 4A at a depth of 0.5 m below the River bottom or at El. 173.8 m, sandwiched between two layers of granular soil. Silty clay was also encountered in Borehole 8, at a depth of 9.0 m (El. 175.4 m) and extended to the surface of the inferred bedrock at El. 173.7 m.

Atterberg Limits tests performed on two soil samples retrieved from this cohesive deposit yielded the following index values, as shown in the individual Record of Borehole Sheets and also on the Plasticity Chart in Figure B-8 (Appendix B):

Liquid Limit: 33-43%

Plastic Limit: 15-21%

Plastic Index: 18-22%

These results are characteristic of low to medium plasticity.

N values of 4 and 11 blows/0.3 m were recorded in Boreholes 4A and 8, respectively. Based on these results together with pocket penetrometer tests and visual & tactile examination of the recovered samples, the consistency of the silty clay encountered in Borehole 4A is described as very soft to soft, while in Borehole 8, its consistency is considered stiff.

This deposit is considered to be practically impervious. The deposit, as encountered in Borehole 4A, is considered weak and highly compressible.

4.5 Bedrock

In Boreholes 1, 2, 7 and 8, bedrock was inferred from refusal to augering while in Boreholes 3, 4, 5 and 6, upon encountering refusal on the augers, the presence of bedrock was proven by coring (i.e. diamond drilling) and obtaining rock cores to depths ranging from 3.0 to 3.8 m below the surface of the bedrock. In Boreholes 4 and 6 which were advanced by washboring methods from a barge, BQ size core samples were obtained, while in Boreholes 3 and 5, which were advanced from land, using a larger drilling rig, NQ size rock cores were obtained.

In boreholes where coring was effected, the bedrock was identified as granite gneiss, with a colour varying from light to medium (occasionally darkish) grey with a typically a pinkish tone and/or pink insets. Photographs of the rock cores are attached in Appendix D of this report.

The following table summarizes the bedrock elevations and condition in the boreholes.

Table 4.5.1

Borehole Number	Top of Bedrock Elevation (m)	Coring Size	Total Core Length (m)	T.C.R. (%)**	R.Q.D.(%)***
1	176.2*	N/A	N/A	N/A	N/A
2	176.8*	N/A	N/A	N/A	N/A
3	175.9	NQ	3.8	93-98	70-98
4	172.3	BQ	3.1	98-100	86-100
4A	172.6*	N/A	N/A	N/A	N/A

Borehole Number	Top of Bedrock Elevation (m)	Coring Size	Total Core Length (m)	T.C.R. (%)**	R.Q.D.(%)***
5	172.1	NQ	3.4	100	42-100
6	171.2-170.9	BQ	3.0	100	100
7	176.2*	N/A	N/A	N/A	N/A
8	173.7*	N/A	N/A	N/A	N/A

* inferred ** T.C.R. = total core recovery *** R.Q.D.= rock quality designation

N/A not applicable

From the above table, it can be seen that the surface of the bedrock was contacted or inferred between Elevations 176.8 m (BH 2) and 171.2/170.9 m (BH 6). It is noted that at the south abutment location at Boreholes 1, 2 and 3 locations, the surface of the bedrock is relatively higher and level (i.e. an elevation difference of only 0.9 m in the surface elevations of the bedrock at these three borehole locations) at between El. 176.8 and 175.9 m. But at the location of Borehole 4, the bedrock surface was contacted at El. 172.3 m (i.e. about 4 m lower). This is likely to be due to previous construction activities and possibly due to erosion by the River.

On the west side of the River (i.e. north abutment location), the surface of the bedrock at Boreholes 5 and 8 were contacted /inferred at El. 173.7 – 172.1 m, whereas at Borehole 7, it was inferred at El. 176.2 m (i.e. about 3 m higher). At Borehole 6, which was drilled in the River, the surface of the bedrock was contacted at El. 170.9 m (i.e. at a low elevation), probably, similar to Borehole 4, due to River erosion and/or previous construction activities.

In general, at most borehole locations the top 0.1 to 0.3 m of the bedrock was found to be highly fractured, but below this upper zone, the bedrock appeared to be rather sound.

The percentage of core recovery was 93-100 %, while the RQD values generally varied from 70 to 100 % (excluding the upper 0.3 m in Borehole 5 where the RQD value was only 42%). These values indicate a fair to excellent but generally good to excellent rock quality.

Based on these values and examination of the rock cores, the bedrock below about the top 0.3 m can be described as a sound and massive rock of good to excellent quality, at the cored locations.

To determine the compressive strength and hardness of the rock, a total of five samples were subjected to unconfined compressive testing. The unconfined compressive strength (UCS) of the tested samples ranged from 99.6 to 131.7 MPa with an average of 111.5 MPa. The results of these unconfined compressive tests are given in Appendix D.

Point Load Index tests were performed in our laboratory on 23 rock core samples. The test results are presented in Appendix B. $I_{s(50)}$ values ranging from 1.1 to 8.8 MPa and UCS values (using typical K=24) of 27.5 to 211.0 MPa were recorded.

Based on these results, the rock encountered at the site is classified as typically R4 to R5 (strong to very strong).

4.6 Groundwater Conditions

Groundwater conditions in the open boreholes were observed while drilling and upon completion of each borehole. However, Boreholes 4, 4A and 6 were put down from water's surface (from a barge) in the River, using washboring methods and as such no reliable water level observations could be made in these three boreholes.

In the remaining boreholes the groundwater table was inferred from the observations made in the boreholes to be at between El. 176 and 177 m.

In the piezometers installed in Boreholes 2 and 8, the groundwater table was measured twenty days after the installation at El. 177.1 m and 176.9 m, respectively.

It should be pointed out that the groundwater level at the site could be largely influenced by the regulated water level in the River depending on the requirements of the Trent-Severn waterway system. We took elevations of the water in the River once a day during the period of June 12-13 and 14, 2013, during which time it was measured to be between El. 175.9 and 176.0 m. It should however be pointed out that the water level may and probably did fluctuate during the course of each day. These values should therefore be considered approximate only.

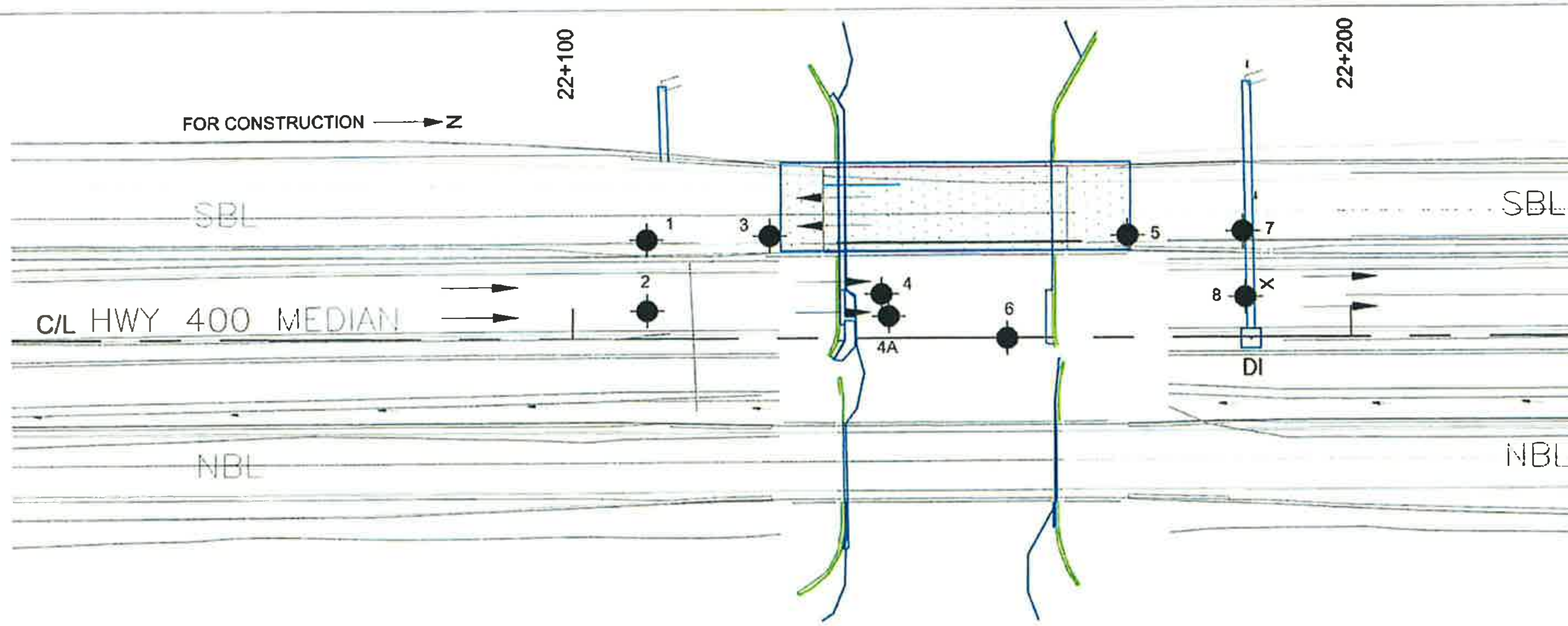
The groundwater table would also be subject to seasonal fluctuations and variations due to major weather events.

For and on behalf of Coffey.

Gwangha Roh, P.Eng., Ph. D.
Senior Geotechnical Engineer

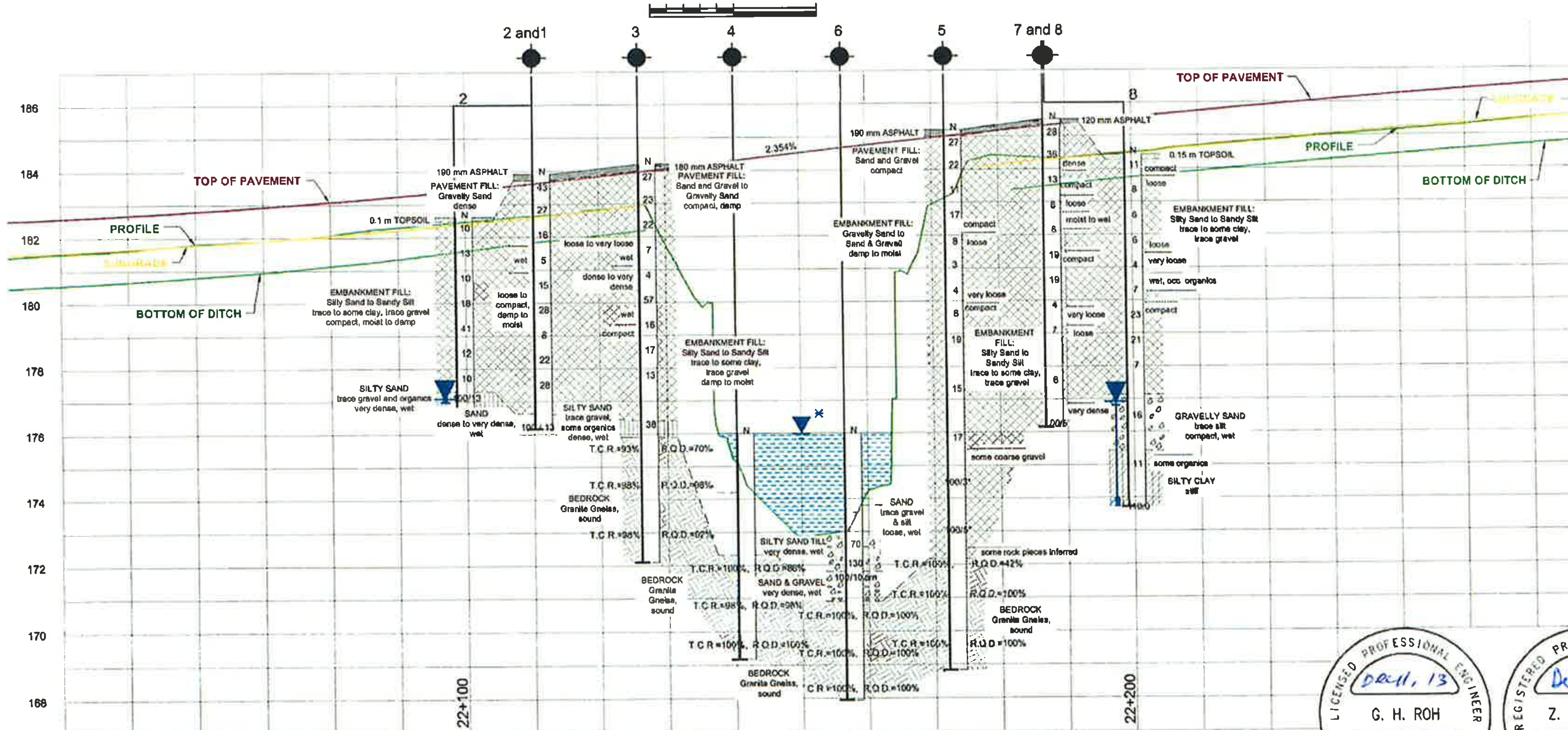
Zuhtu Ozden, P.Eng.
Senior Principal

Drawings



PLAN

SCALE 0 25



PROFILE

HORIZONTAL SCALE 0 25

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -

W.P.: 2360-06-00

SHEET

CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

KEY PLAN
N.T.S.

LEGEND

- Borehole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No	ELEVATION	STATION	OFFSET
BH1	183 9	22+110	12 8m Lt C/L
BH2	182 8	22+110	3 6m Lt C/L
BH3	184 2	22+125	13 3m Lt C/L
BH4	176 0	22+140	5 7m Lt C/L
BH5	185 2	22+172	13 2m Lt C/L
BH6	176 0	22+156	@ C/L
BH7	185 5	22+187	13.7m Lt C/L
BH8	184.4	22+187	5 3m Lt C/L
BH4A	176 0	22+141	2 9m Lt C/L

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

NOTE: This drawing is for subsurface information only.. Surface details and features are for conceptual illustration.

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REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-564

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE December , 2013	SITE 42-66/1&2
DRAWN SSH	CHECKED SS	APPROVED ZO	DWG 1

METRIC

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

CONT No. -
W.P.: 2360-06-00

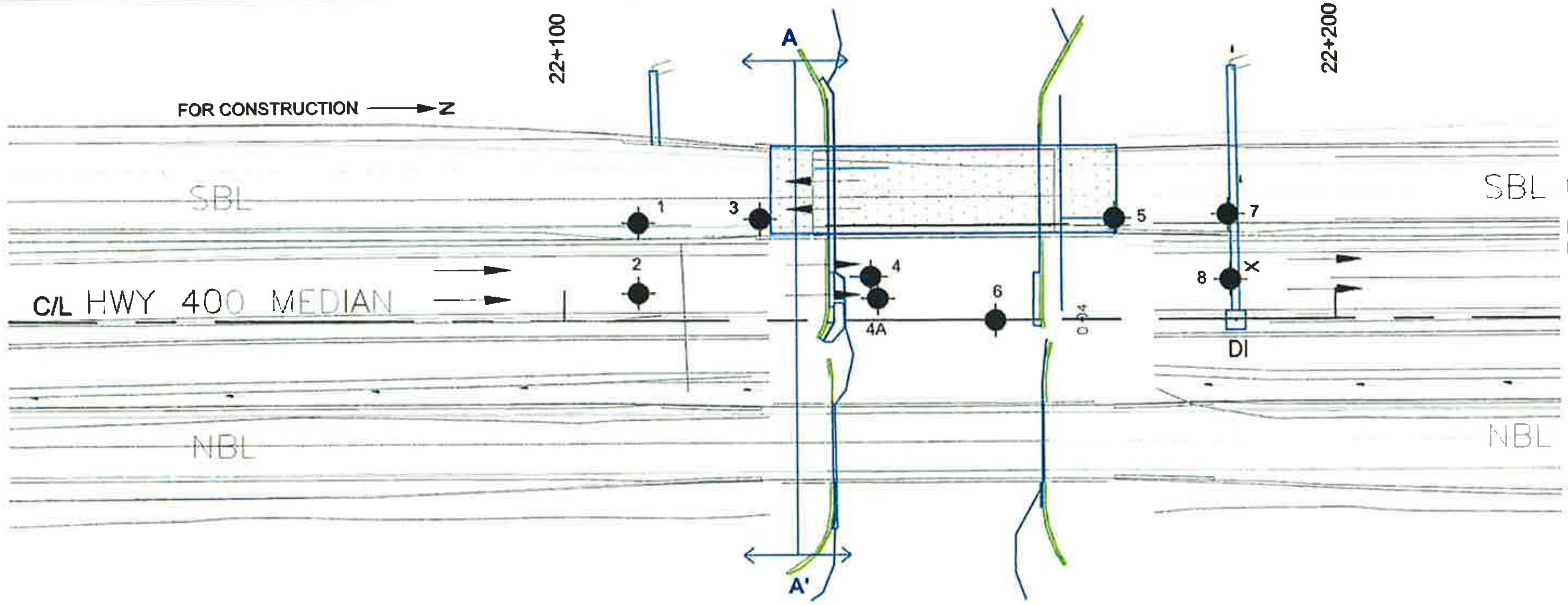
CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA-Section A-A'

SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

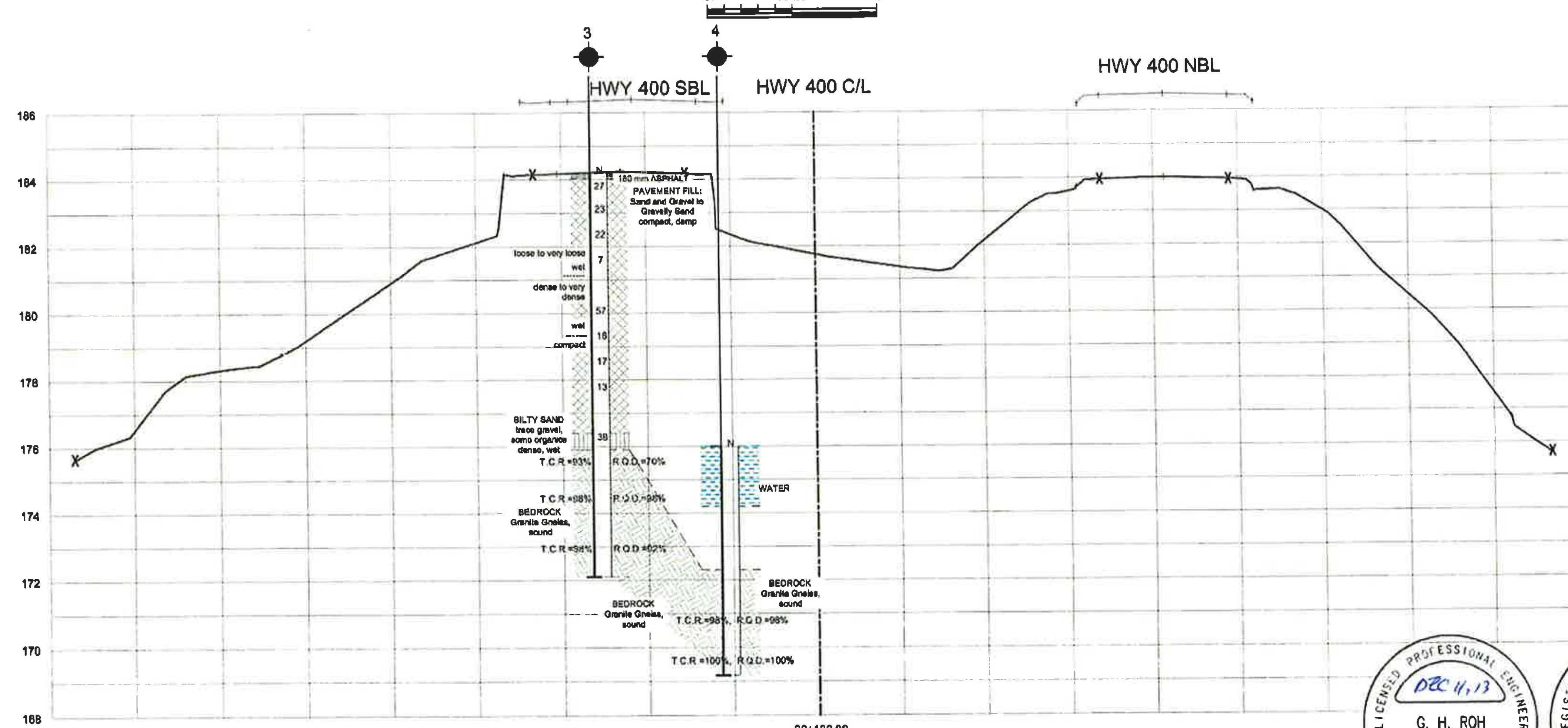
NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.



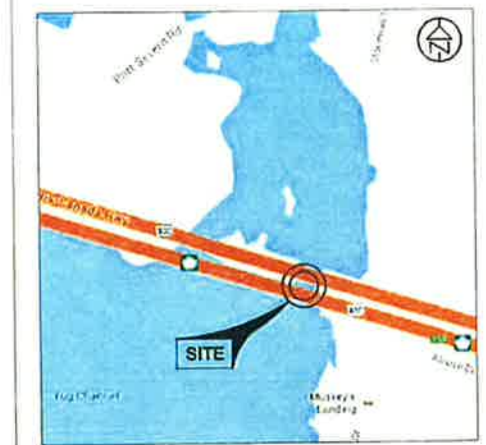
PLAN

SCALE 0 25



SECTION A-A'

HORIZONTAL SCALE 0 10



KEY PLAN
N.T.S.

LEGEND

Borehole

N Blows/0.3m (Std. Pen. Test, 475 J/blow)

A — A' Section

No	ELEVATION	STATION	OFFSET
BH3	184 2	22+125	13 3m Lt C/L
BH4	176 0	22+140	5 7m Lt C/L

-NOTE-

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LICENSED PROFESSIONAL ENGINEER
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REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-564

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE December, 2013	SITE 42-86/182
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 2

METRIC

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

NOTES:

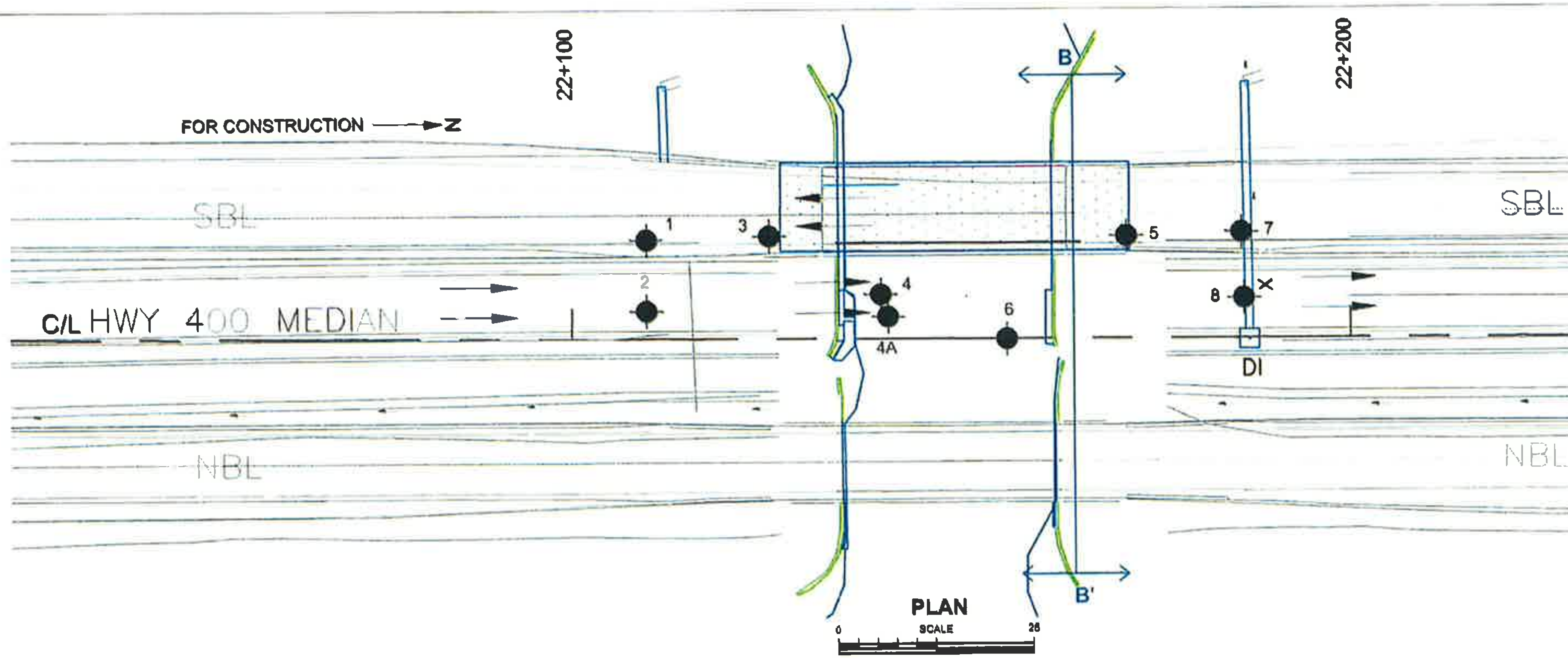
FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -
W.P.: 2360-06-00

CONTRACT A, HIGHWAY 400,
PORT SEVERN RIVER BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA - SECTION B-B'

SHEET

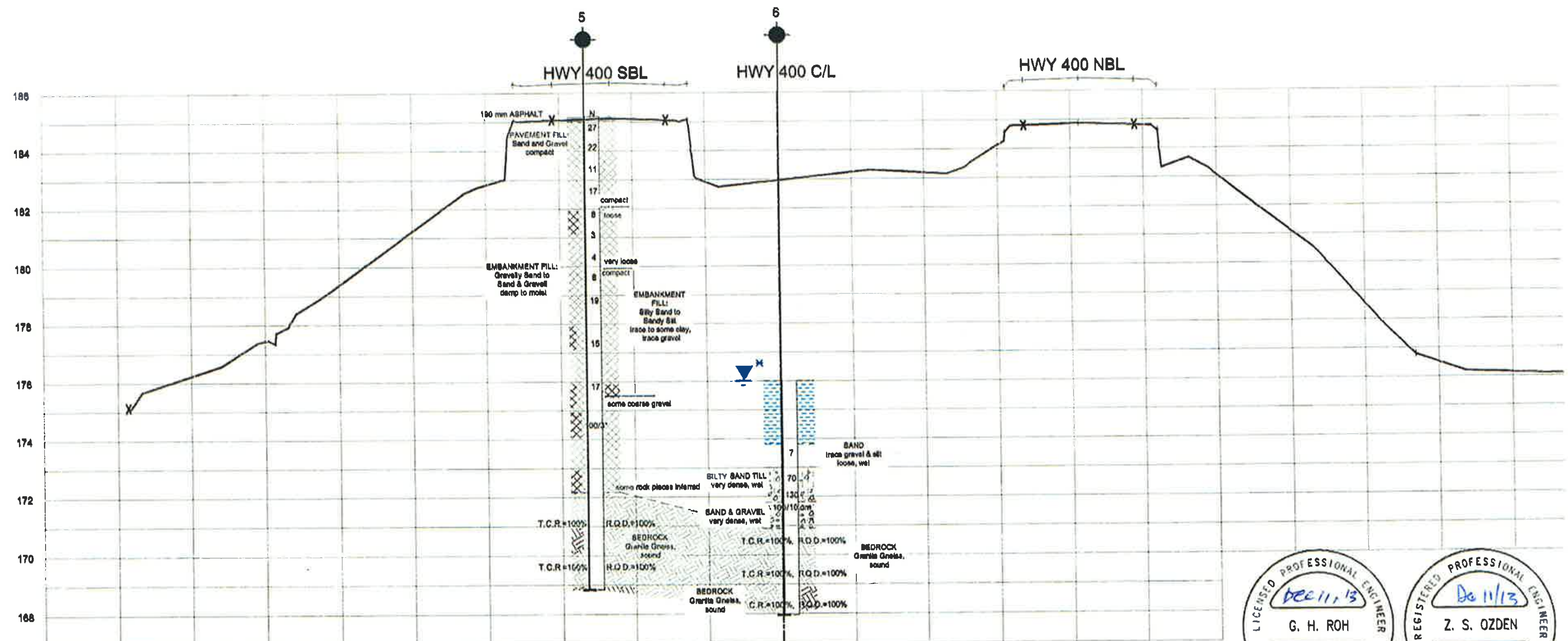
coffey geotechnics
SPECIALISTS MANAGING THE EARTH



PLAN

SCALE 0 25

KEY PLAN
N.T.S.



22+170.00

SECTION B-B'

HORIZONTAL SCALE 0 10

LEGEND

Borehole

N Blows/0.3m (Std Pen Test, 475 J/blow)

Water Level at Time of Investigation (W L NOT STABILIZED)

B-B' Section

No	ELEVATION	STATION	OFFSET
BH5	185.2	22+172	13.2m Lt C/L
BH6	176.0	22+166	@ C/L

-NOTE-

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

NOTE: This drawing is for subsurface information only.. Surface details and features are for conceptual illustration.

LICENSED PROFESSIONAL ENGINEER
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Z. S. OZDEN
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REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-564

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE December, 2013	SITE 42-86/182
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 3

Appendix A

Record of Borehole Sheets

Ministry of Transportation
Ontario

Foundation Design

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 1

METRIC

GWP WP 2360-09-00 | LOCATION 22+110, 12.8 m Lt C/L (N 4962143.863, E 287411.348) | ORIGINATED BY LG

DIST 5 HWY 400 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY SSH

DATUM Geodetic | DATE 30/05/2013 30/05/2013 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.9	GROUND SURFACE				
183.7 / 0.2	190 mm **ASPHALT**				
183.2 / 0.7	**PAVEMENT FILL:** Gravelly Sand brown, dense	1	SS	43	32 56 (12)
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, loose to compact, damp to moist	2	SS	27	3 58 22 17
		3	SS	16	
	wet	4	SS	5	
		5	SS	15	
		6	SS	28	2 34 46 18
		7	SS	8	
		8	SS	22	
		9	SS	28	
176.6 / 7.3	**SAND** grey, dense to very dense, wet	10	SS	100 / 13 cm	spoon wet and bouncing
176.2 / 7.8	End of Borehole Auger refusal @ 7.8 m Probable Bedrock Borehole open & dry on completion (not stabilized)				

ELEVATION SCALE: 183, 182, 181, 180, 179, 178, 177

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

GWP WP 2360-09-00 LOCATION 22+110, 3.6 m Lt C/L (N 4962152.644, E 287414.103) ORIGINATED BY LG

DIST 5 HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SSH

DATUM Geodetic DATE 30/05/2013 30/05/2013 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.6	GROUND SURFACE					
182.5 / 0.1	0.1 m **TOPSOIL**	1	SS	10	182	
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, compact, moist to damp	2	SS	13	181	
		3	SS	10		
		4	SS	18	180	2 56 26 16
		5	SS	41	179	
		6	SS	12	178	
		7	SS	10		2 66 19 13
177.3 / 5.3	**SILTY SAND** trace gravel and organics grey, very dense, wet	8	SS	100/13	177	spoon wet and bouncing
176.8 / 5.8	End of Borehole Auger refusal @ 5.8 m Probable Bedrock Borehole open & dry on completion (not stabilized) Piezometer installed to 5.6 m Water level in piezometer at 5.5 m (El. 177.1 m) on June 17, 2013					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

1 OF 1 **METRIC**

GWP	WP 2360-09-00	LOCATION	22+125, 13.3 m Lt C/L (N 4962148.179, E 287396.107)
DIST	5 HWY 400	BOREHOLE TYPE	Hollow Stem Auger + NQ Coring
DATUM	Geodetic	DATE	23/05/2013 23/05/2013
ORIGINATED BY	LG	COMPILED BY	SSH
CHECKED BY	ZO		

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.2	GROUND SURFACE					
184.0 0.2	180 mm **ASPHALT**				184	
	PAVEMENT FILL: Sand and Gravel to Gravelly Sand brown, compact, damp	1	SS	27		
		2	SS	23	183	36 48 (16)
		3	SS	22		40 46 (14)
181.9 2.3	**EMBANKMENT FILL:** Silty Sand to Sandy Silt trace to some clay, trace gravel damp to moist; loose to very loose wet brown	4	SS	7	182	
		5	SS	4	181	10 55 19 16
	-------- greyish brown dense to very dense wet	6	SS	57	180	
	-------- compact	7	SS	16		3 40 39 18
		8	SS	17	179	
	-------- grey	9	SS	13	178	
					177	
176.4 7.8	**SILTY SAND** trace gravel, some organics grey / black, dense, wet	10	SS	38	176	8 65 (27)
175.9 8.3	**BEDROCK** Granite Gneiss greyish / pink, sound; ---------fractured	11	RC	T.C.R.=93% R.Q.D.=70%		Auger refusal and Start of NQ coring at 8.3 m
		12	RC	T.C.R.=98% R.Q.D.=98%	175 174	
		13	RC	T.C.R.=98% R.Q.D.=92%	173	
172.1 12.1	End of Borehole Borehole open and dry upon completion (prior to coring), not stabilized					

+ ³, × ³ : Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 4

METRIC

GWP WP 2360-09-00	LOCATION 22+140, 5.7 m Lt C/L (N 4962159.701, E 287384.714)		ORIGINATED BY LG
DIST 5 HWY 400	BOREHOLE TYPE Washboring and BQ Rock Coring from barge		COMPILED BY SSH
DATUM Geodetic	DATE 13/06/2013 14/06/2013		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.0	WATER SURFACE						176	
0.0	**WATER**						175	
174.2 1.8	**River Bottom** wash bored through overburden without sampling due to the presence of rockfill (see Record of Borehole 4A for details of overburden)						174 173	
172.3 3.7	fractured ------------		1	RC	T.C.R.=100% R.Q.D=86%		172	
	BEDROCK Granite Gneiss sound mainly pink ------------ greyish / pink		2	RC	T.C.R.=98% R.Q.D=98%		171	
169.2			3	RC	T.C.R.=100% R.Q.D=100%		170	
6.8	End of Borehole							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

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

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 4A

METRIC

GWP	WP 2360-09-00	LOCATION	22+141, 2.9 m Lt C/L (N 4962162.711, E 287384.702)	ORIGINATED BY	LG
DIST 5	HWY 400	BOREHOLE TYPE	Washboring from barge	COMPILED BY	SSH
DATUM	Geodetic	DATE	14/06/2013 14/06/2013	CHECKED BY	ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.0	WATER SURFACE					176	
0.0	**WATER**					175	
174.3 / 1.7	**SAND AND GRAVEL** grey, loose, wet	1	SS	6		174	
173.8 / 2.2	**SILTY CLAY** reddish grey, soft	2	SS	4			
173.2 / 2.8	**SAND** some silt and gravel grey, compact, wet	3	SS	22		173	
172.6 / 3.4	End of Borehole See Record of Borehole 4 for continuation of stratigraphy Auger refusal @ 3.4 m Probable Bedrock						

Column headers: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 5

METRIC

GWP WP 2360-09-00 LOCATION 22+172, 13.2 m Lt C/L (N 4962162.063, E 287352.273) ORIGINATED BY LG

DIST 5 HWY 400 BOREHOLE TYPE Hollow Stem Augering and NQ Coring COMPILED BY SSH

DATUM Geodetic DATE 23/05/2013 23/05/2013 CHECKED BY ZO

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.2	GROUND SURFACE				
185.0 / 0.2	190 mm **ASPHALT**				
	PAVEMENT FILL: Sand and Gravel brown, compact	1	SS	27	
184.1 / 1.1		2	SS	22	
	EMBANKMENT FILL: Gravelly Sand to Sand and Gravel brown, damp to moist	3	SS	11	41 44 13 2
	compact	4	SS	17	
	loose	5	SS	8	
181.5 / 3.7	**EMBANKMENT FILL:** Silty Sand to Sandy Silt trace to some clay, trace gravel brown to 6 m, greyish brown to 10 m, grey below moist to wet to 6 m,moist 6 m to 10 m, wet below 10 m	6	SS	3	9 41 33 17
	very loose	7	SS	4	
	compact	8	SS	8	
	compact	9	SS	19	
		10	SS	15	6 58 20 16
		11	SS	17	
	some coarse gravel	12	SS	100/3*	* Nvalue influnced by coarse gravel
		13	SS	100/5*	
172.1 / 13.1	some rock pieces inferred				
	BEDROCK Granite Gneiss greyish / pink, sound; fractured	14	RC	T.C.R.=100% R.Q.D.=42%	Auger refusal and Start of NQ coring at 13.1 m
		15	RC	T.C.R.=100% R.Q.D.=100%	

Elevation scale: 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 5

METRIC

GWP WP 2360-09-00 LOCATION 22+172, 13.2 m Lt C/L (N 4962162.063, E 287352.273) ORIGINATED BY LG

DIST 5 HWY 400 BOREHOLE TYPE Hollow Stem Augering and NQ Coring COMPILED BY SSH

DATUM Geodetic DATE 23/05/2013 23/05/2013 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.2											
	BEDROCK Granite Gneiss greyish / pink, sound		16	RC	T.C.R.=100% R.Q.D=100%		170				
							169				
168.7 16.5	End of Borehole Borehole open and dry upon completion (prior to coring), not stabilized										

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

+ 3, × 3 : Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 6

METRIC

GWP WP 2360-09-00 LOCATION 22+156, @ C/L (N 4962170.075, E 287371.025) ORIGINATED BY LG

DIST 5 HWY 400 BOREHOLE TYPE HQ Casing + Washboring; BQ Coring from barge COMPILED BY SSH

DATUM Geodetic DATE 14/06/2013 14/06/2013 CHECKED BY ZO

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
176.0	WATER SURFACE					176	
0.0	**WATER**					175	
173.8 / 2.2	**River Bottom**				▼	174	
	SAND trace gravel and silt grey, loose, wet	1	SS	7			
173.0 / 3.0	**SILTY SAND TILL** grey, very dense, wet	2	SS	70		173	12 62 (26)
		3	SS	130		172	
171.8 / 4.2	**SAND AND GRAVEL** grey, very dense, wet broken rock pieces contacted below 4.8 m (possible shattered bedrock)	4	SS	100/10 cm			
170.9 / 5.1	**BEDROCK** Granite Gneiss greyish / pink, sound	5	RC	T.C.R.=100% R.Q.D.=100%		171	
		6	RC	T.C.R.=100% R.Q.D.=100%		170, 169	
		7	RC	T.C.R.=100% R.Q.D.=100%			
168.0 / 8.1	End of Borehole					168	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

METRIC

GWP WP 2360-09-00 | LOCATION 22+187, 13.7 m Lt C/L (N 4962166.060, E 287338.083) | ORIGINATED BY LG

DIST 5 HWY 400 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY SSH

DATUM Geodetic | DATE 29/05/2013 29/05/2013 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.5	GROUND SURFACE						
185.4 / 0.1	120 mm **ASPHALT**						
185.2 / 0.3	**PAVEMENT FILL:** Sand and Gravel, brown, compact	1	SS	28		185	22 63 (15)
184.9 / 0.6	**PAVEMENT FILL:** Gravelly Sand, brown, compact	2	SS	36			7 58 20 15
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, damp to moist; dense					184	
	compact	3	SS	13			
	loose	4	SS	8		183	
	moist to wet	5	SS	6		182	3 61 21 15
	compact	6	SS	19		181	
		7	SS	19			
	very loose	8	SS	4		180	
	loose	9	SS	7		179	
	moist to wet					178	
		10	SS	6			5 60 18 17
					▽	177	
176.2 / 9.3	very dense	11	SS	100/5			

End of Borehole
Auger refusal @ 9.4 m
Probable Bedrock
Borehole dry and open to 8.8 m upon completion (non stabilized)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m3

+ 3, × 3: Numbers refer to Sensitivity — 20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 8

METRIC

GWP: WP 2360-09-00 | LOCATION: 22+187, 5.3 m Lt C/L (N 4962174.284, E 287340.342) | ORIGINATED BY: LG

DIST: 5 HWY: 400 | BOREHOLE TYPE: Hollow Stem Auger | COMPILED BY: SSH

DATUM: Geodetic | DATE: 28/05/2013 28/05/2013 | CHECKED BY: ZO

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
184.4	GROUND SURFACE
184.3 / 0.2	0.15 m **TOPSOIL**
	EMBANKMENT FILL: Silty Sand to Sandy Silt, trace to some clay, trace gravel, greyish brown, moist to damp; compact / loose / loose / very loose / wet, occ. organics / loose / compact / loose / moist to wet
177.1 / 7.3	**GRAVELLY SAND** trace silt brown, compact, wet (possible fill)
175.4 / 9.0	**SILTY CLAY** brown, stiff; some organics
173.7 / 10.7	End of Borehole Auger refusal @ 10.7 m Probable Bedrock *Wet cave at 8.5 m upon completion Piezometer installed to 10.5 m Water level in piezometer 7.5 m (el 176.9 m) on June 17, 2013

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	11	
2	SS	8	3 61 21 15
3	SS	6	
4	SS	6	
5	SS	4	
6	SS	7	
7	SS	23	
8	SS	21	7 49 27 17
9	SS	7	
10	SS	16	sampler wet 26 65 (9)
11	SS	11	43
~~12~~	~~SS~~	~~110/0~~	spoon bouncing

ELEVATION SCALE: 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

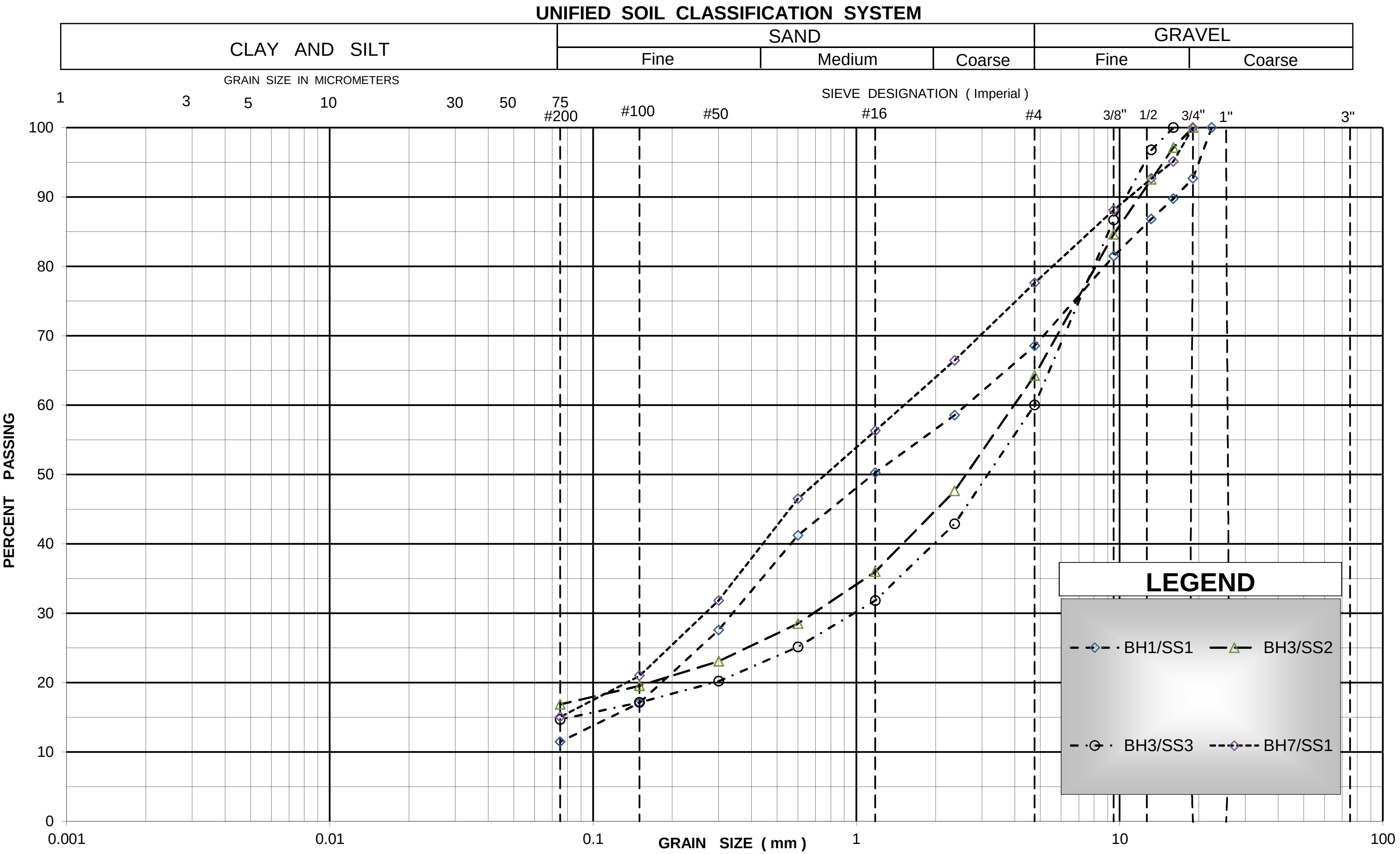
UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Test Results

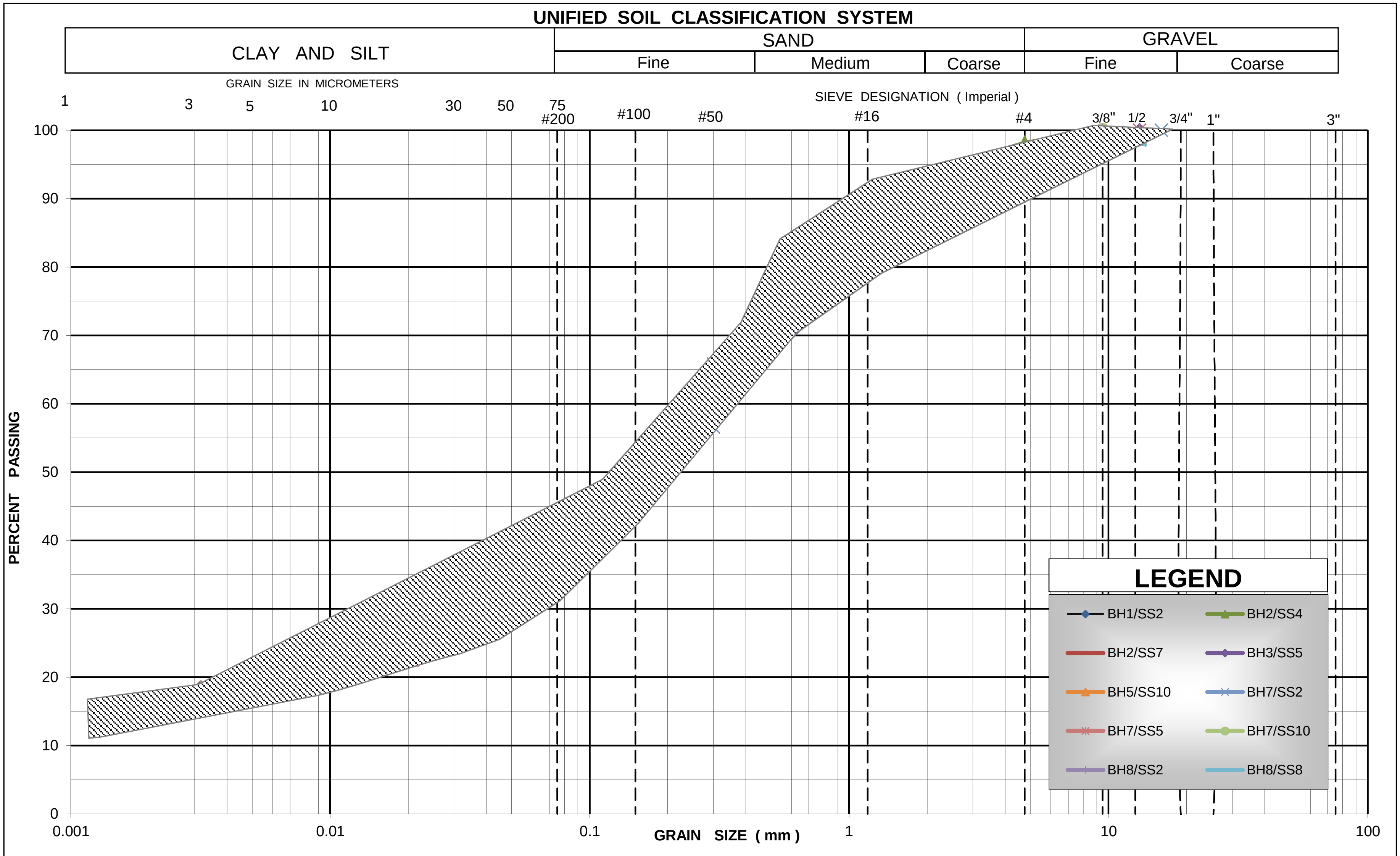


coffey geotechnics
SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

PAVEMENT FILL: Gravelly Sand, trace to some silt

Figure: B-1	
PROJECT # : TRANETOB20462AA	
DATE :	JULY, 2013



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

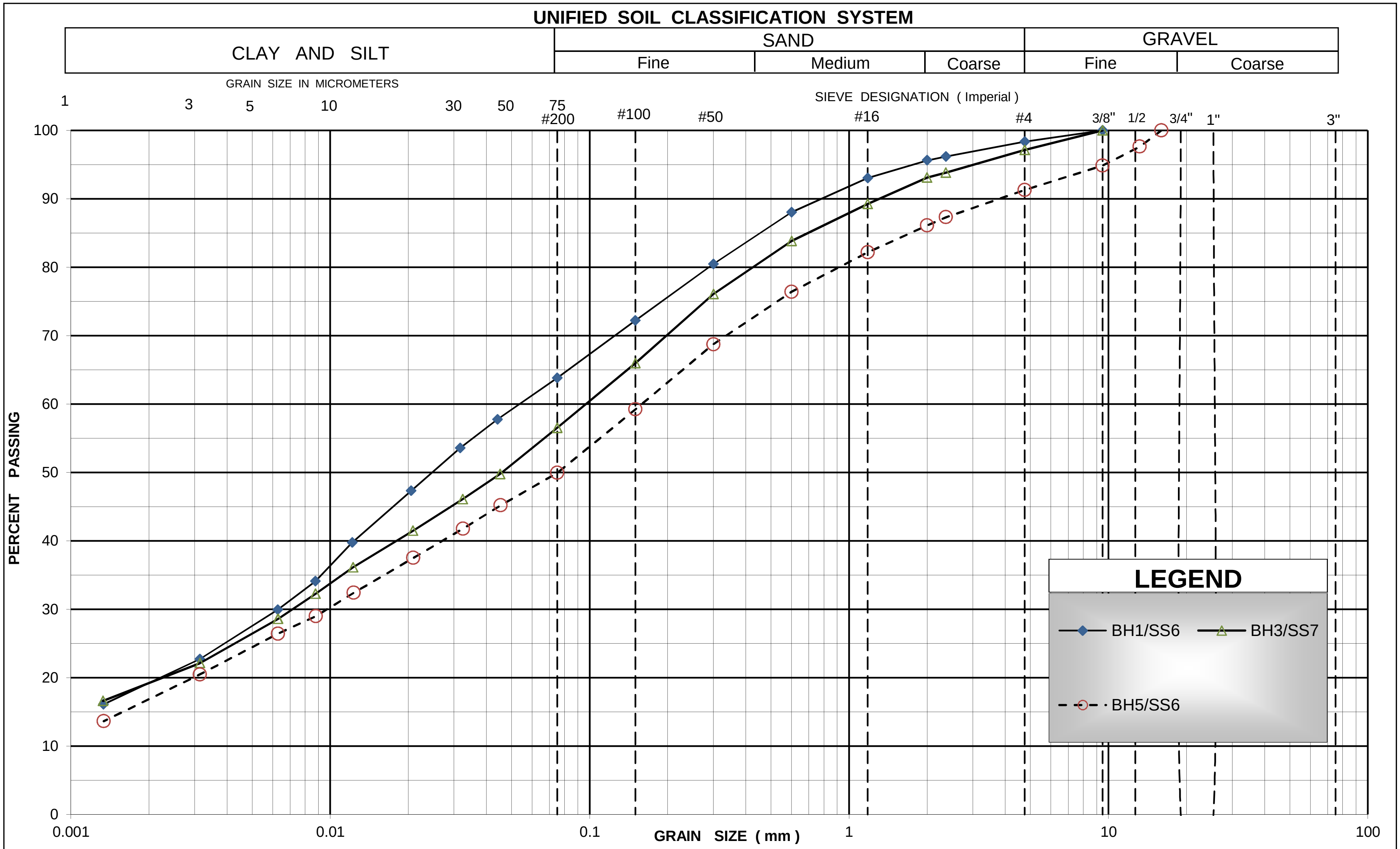
GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL: Silty Sand, trace to some clay, trace gravel

Figure: B-2

PROJECT # : TRANETOB20462AA

DATE : JULY, 2013



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

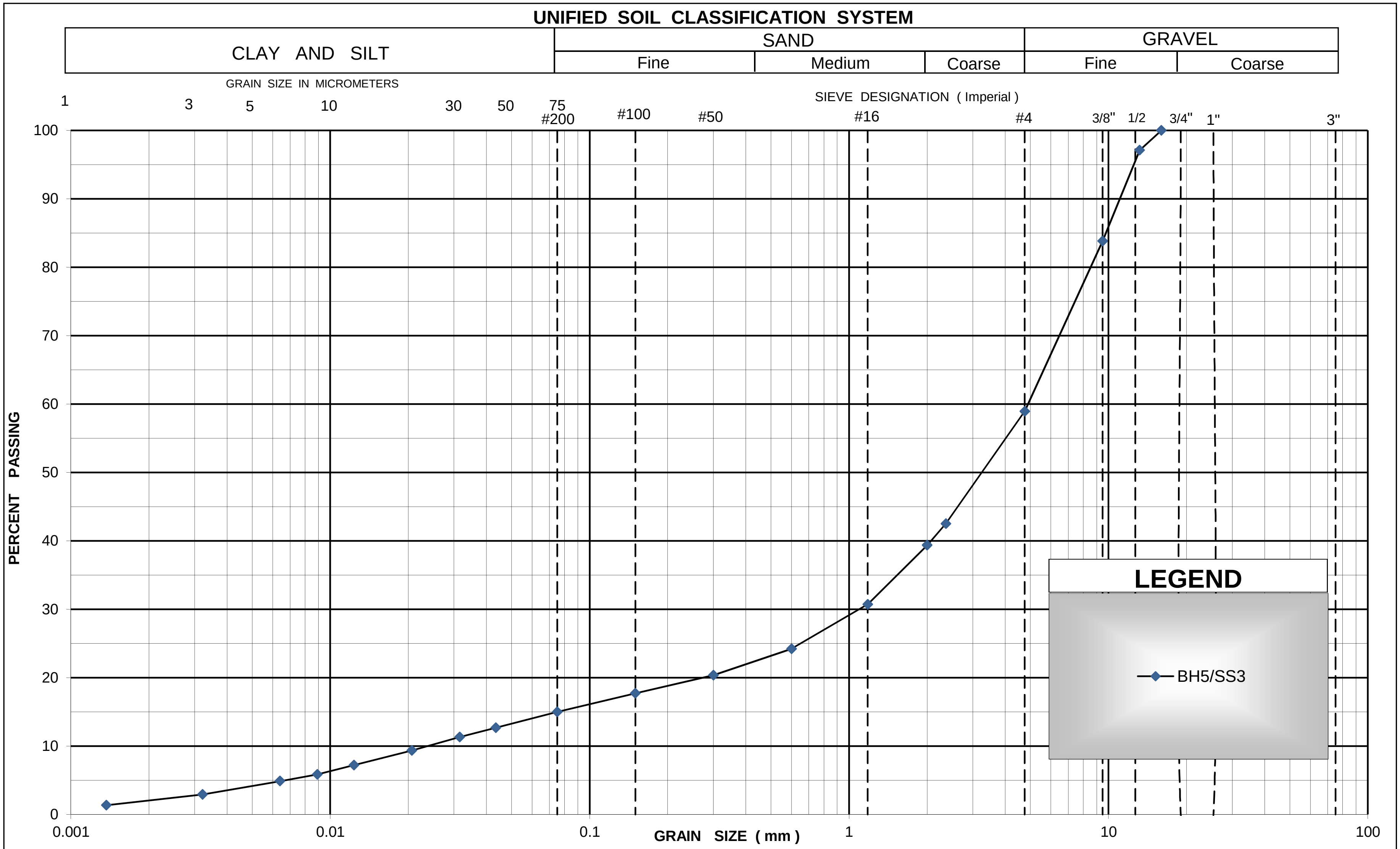
GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL: Sandy Silt, trace to some clay, trace gravel

Figure: B-3

PROJECT # : TRANETOB20462AA

DATE : JULY, 2013



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

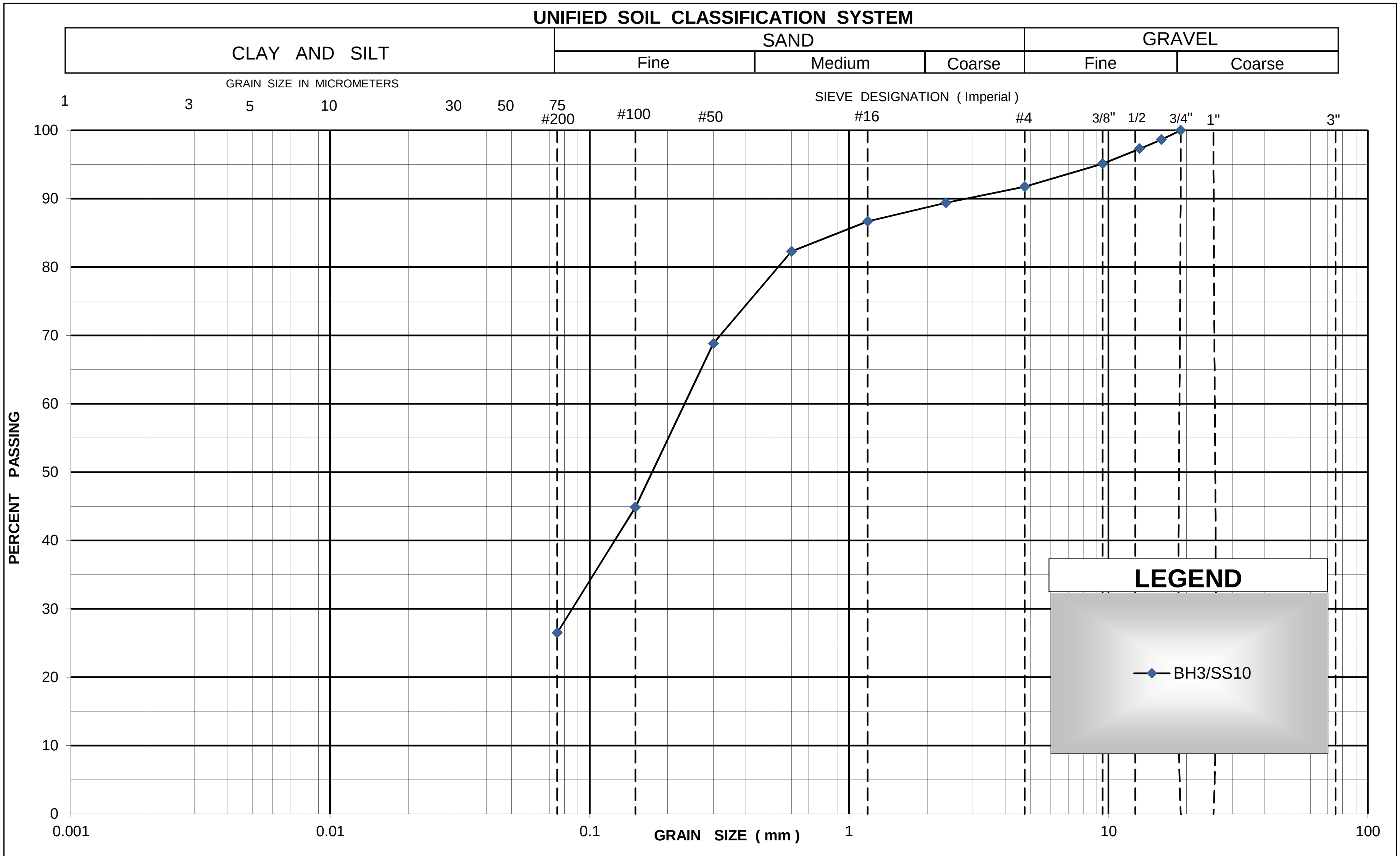
GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL: Sand and Gravel, trace silt

Figure B-4

PROJECT # : TRANETOB20462AA

DATE : JULY, 2013

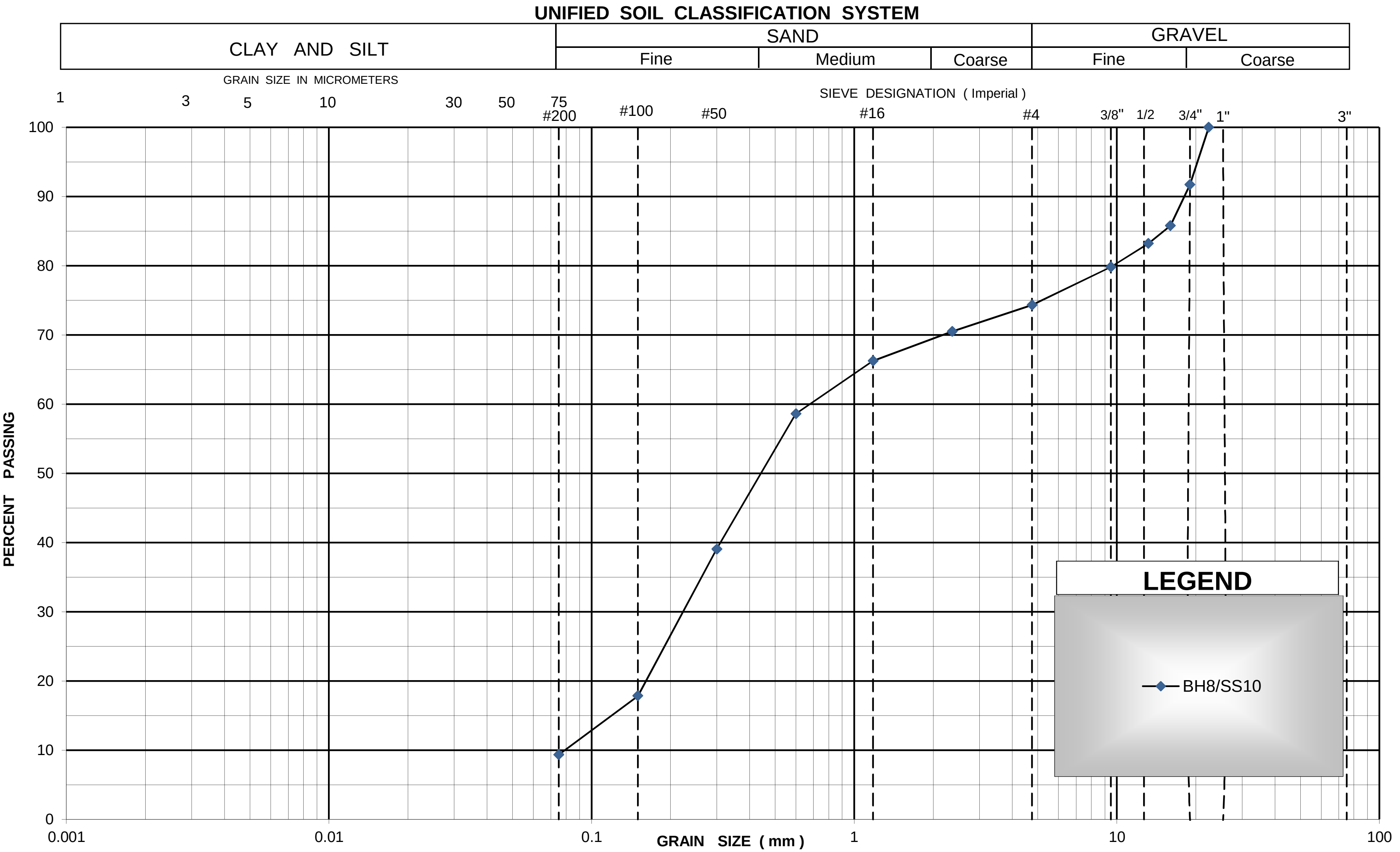


coffey geotechnics SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

SILTY SAND

Figure B-5	
PROJECT # : TRANETOB20462AA	
DATE :	JULY, 2013



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

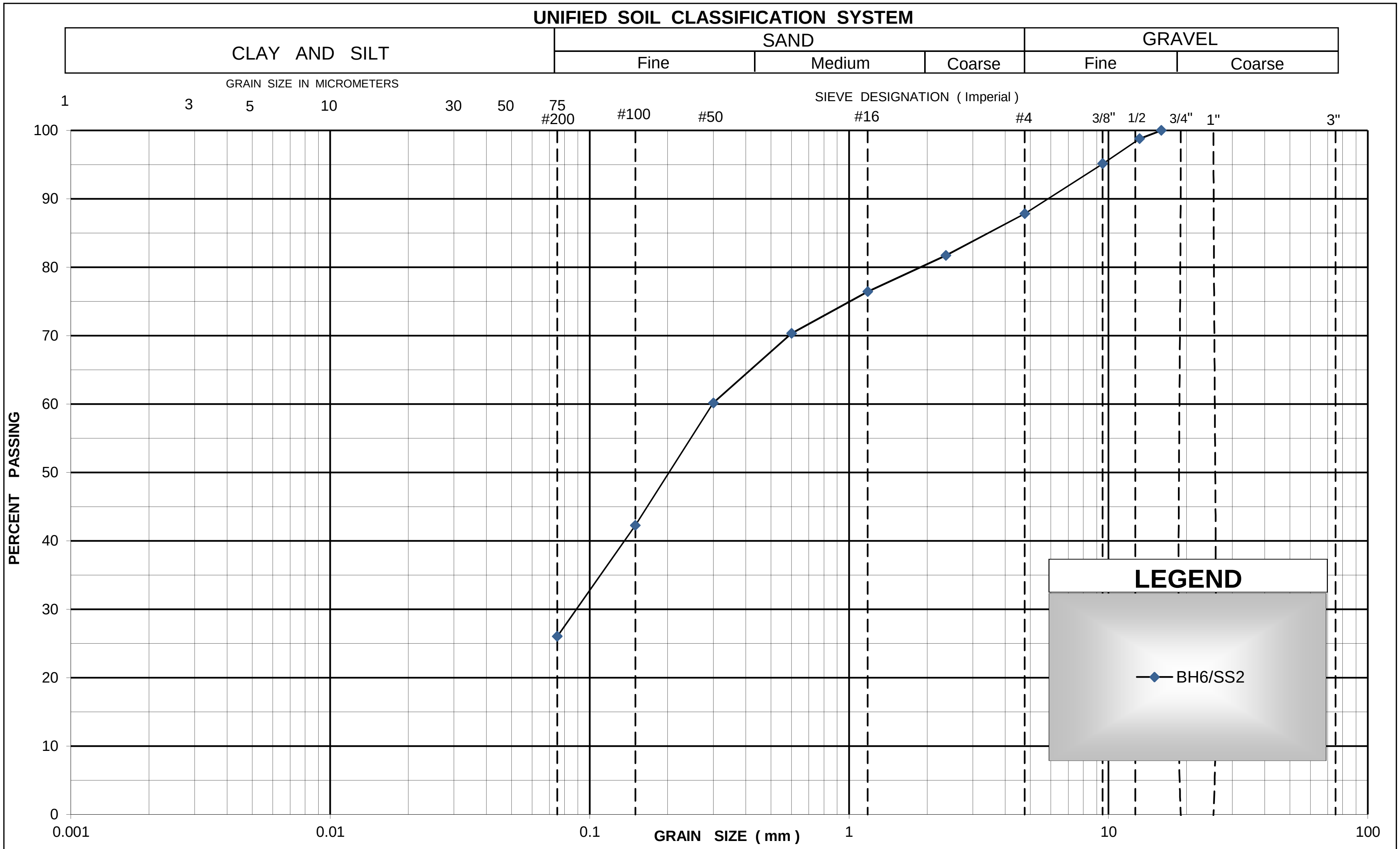
GRAIN SIZE DISTRIBUTION

GRAVELLY SAND

Figure B-6

PROJECT # : TRANETOB20462AA

DATE : JULY, 2013

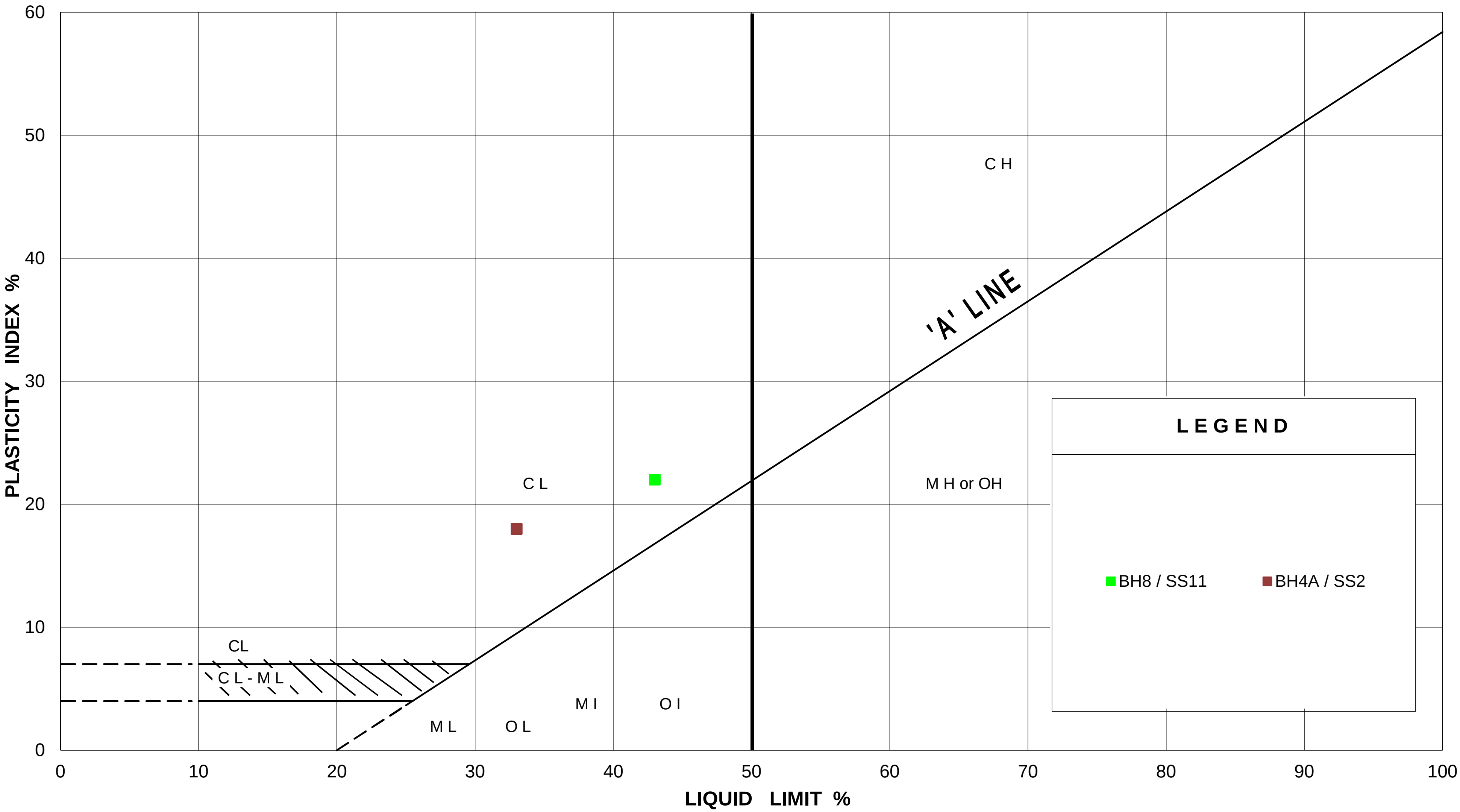


coffey geotechnics SPECIALISTS MANAGING THE EARTH

GRAIN SIZE DISTRIBUTION

SILTY SAND TILL

Figure B-7	
PROJECT # : TRANETOB20462AA	
DATE :	JULY, 2013



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PLASTICITY CHART

Silty Clay

PROJECT NO. TRANETOB20462AA

Figure B-8

DATE JULY, 2013

Appendix C

Site Photographs

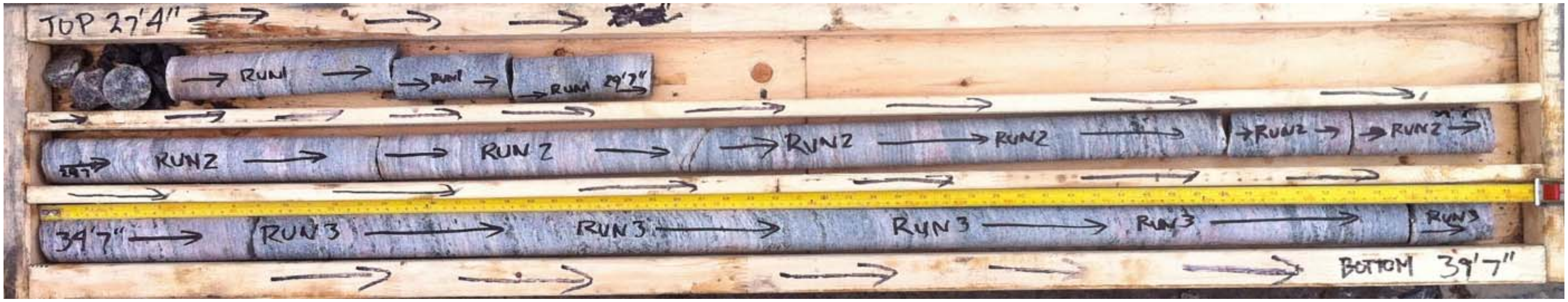
Photograph 1. Borehole 3 looking east (south)

Photograph 2. Borehole 2 looking west (north)

Photograph 3. Boreholes 4 and 4A looking east (south)

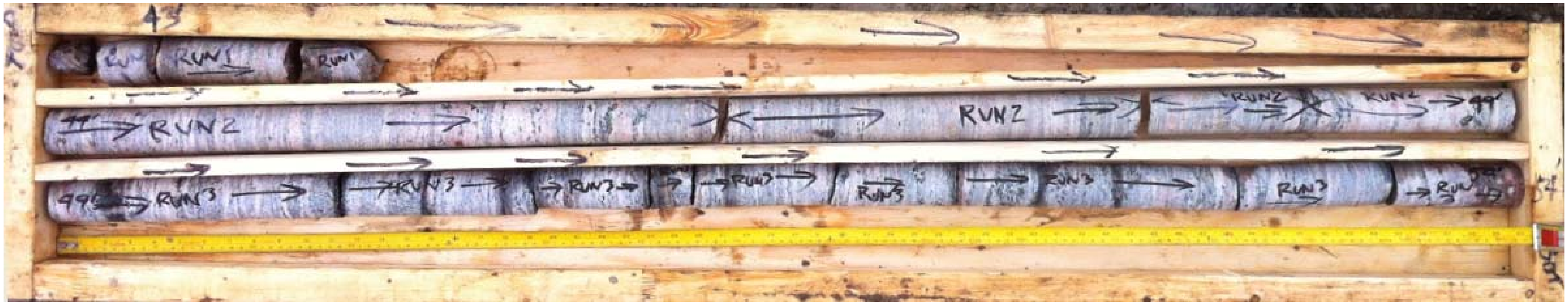
Appendix D

Rock Core Photographs and Test Results



BH 3 (wooden box is 5 feet long)

BH 4 (wooden box is 5 feet long)



BH 5 (wooden box is 5 feet long)

BH 6 (wooden box is 5 feet long)

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	3	SAMPLE DEPTH, m	8.7-9.0

TEST CONDITIONS

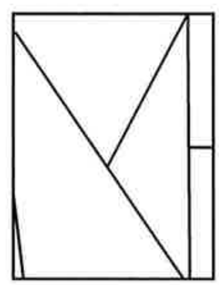
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.55	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.72	UNIT WEIGHT, kN/m^3	26.52
SAMPLE AREA, cm^2	17.47	DRY UNIT WT., kN/m^3	26.50
SAMPLE VOLUME, cm^3	184.29	SPECIFIC GRAVITY	-
WET WEIGHT, g	498.56	VOID RATIO	-
DRY WEIGHT, g	498.16		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	104.4

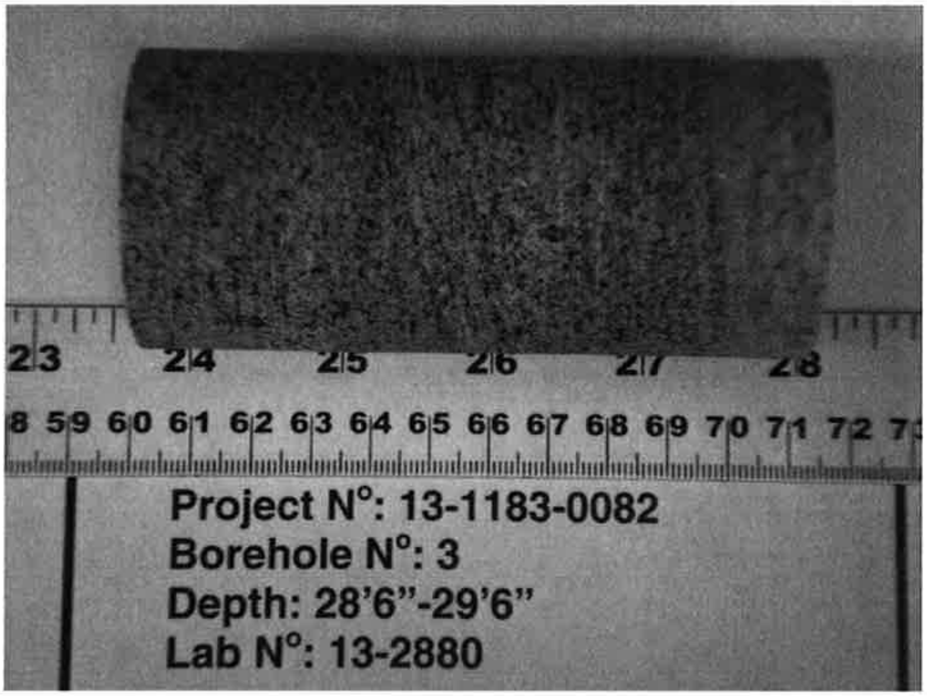
REMARKS:		DATE:	7/24/2013

Checked By: Ro

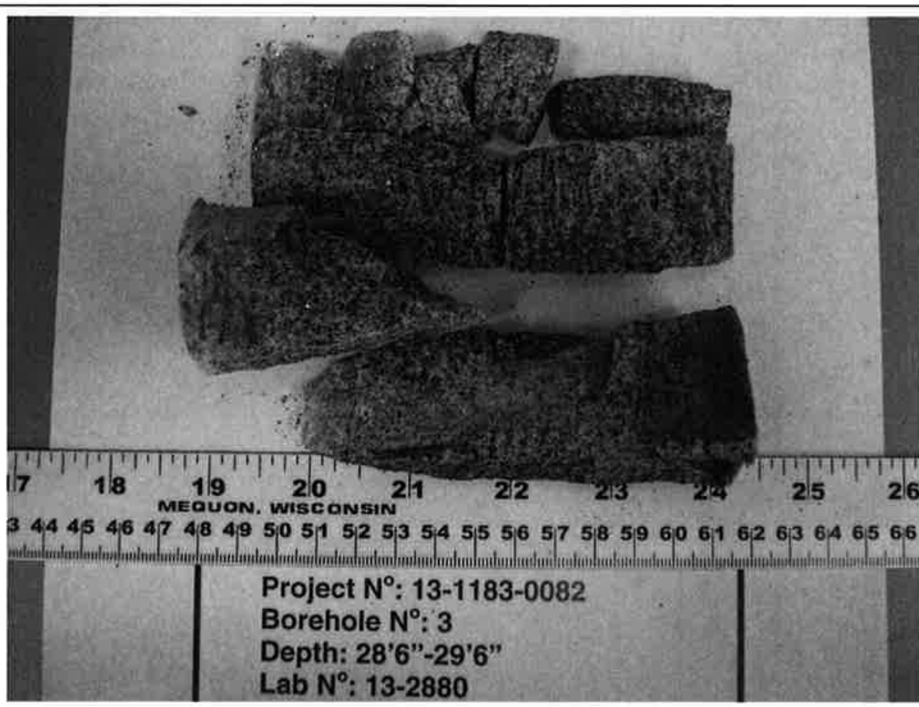
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. ro

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	4	SAMPLE DEPTH, m	2.1-2.4

TEST CONDITIONS

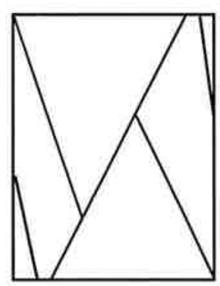
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.06	WATER CONTENT, (specimen) %	0.13
SAMPLE DIAMETER, cm	3.62	UNIT WEIGHT, kN/m^3	25.54
SAMPLE AREA, cm^2	10.30	DRY UNIT WT., kN/m^3	25.50
SAMPLE VOLUME, cm^3	82.98	SPECIFIC GRAVITY	-
WET WEIGHT, g	216.17	VOID RATIO	-
DRY WEIGHT, g	215.89		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	131.7

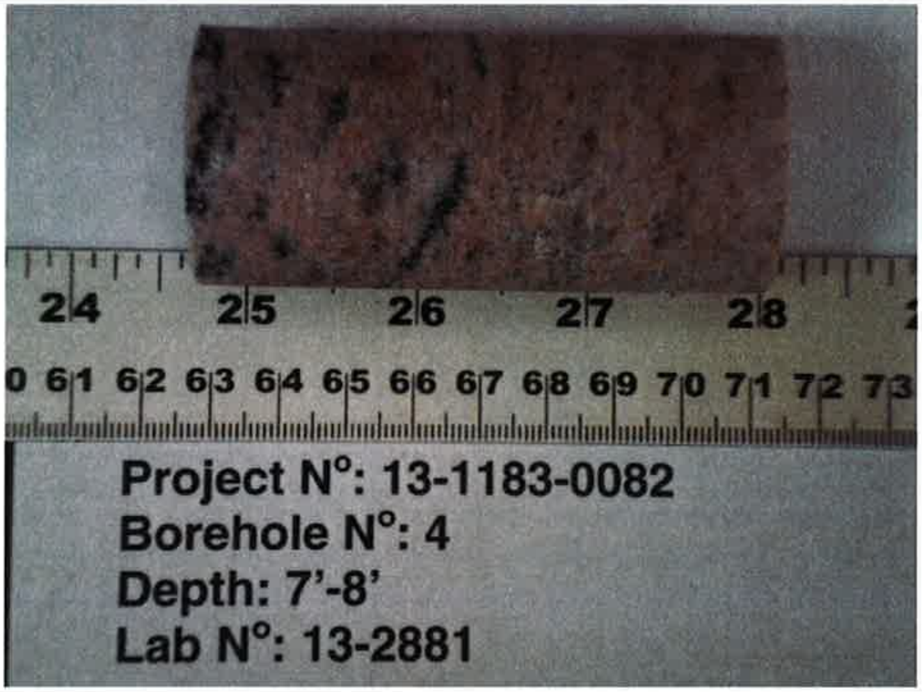
REMARKS:		DATE:	7/24/2013

Checked By: RO

Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION

AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RO

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	5	SAMPLE DEPTH, m	13.2-13.4

TEST CONDITIONS

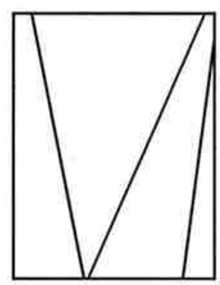
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.20

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.41	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	26.09
SAMPLE AREA, cm^2	17.56	DRY UNIT WT., kN/m^3	26.07
SAMPLE VOLUME, cm^3	182.84	SPECIFIC GRAVITY	-
WET WEIGHT, g	486.64	VOID RATIO	-
DRY WEIGHT, g	486.25		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	105.1

REMARKS:

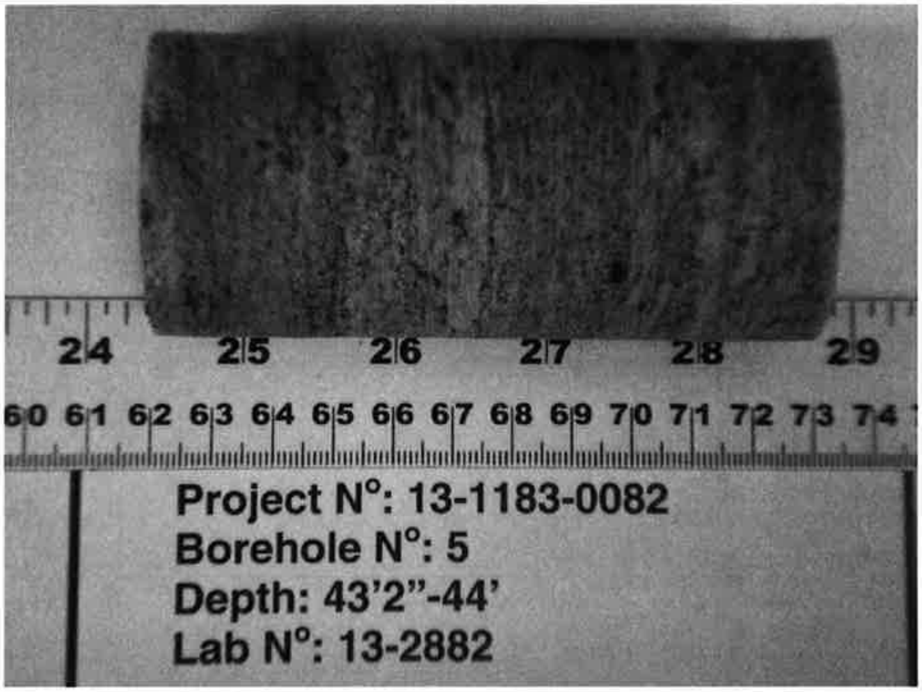
DATE: 7/24/2013

Checked By: *Ro*

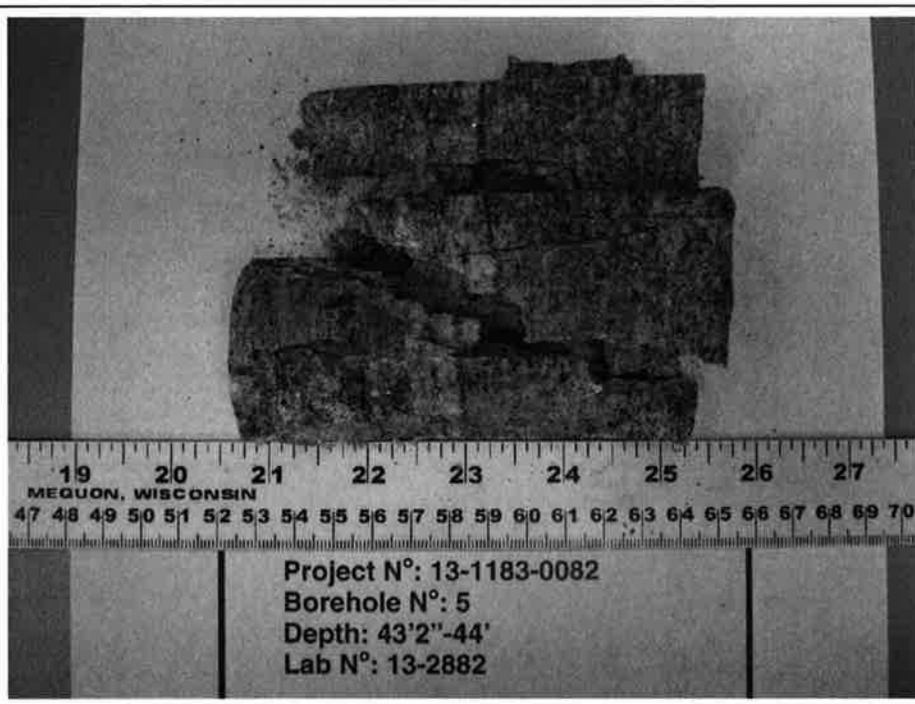
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RA

UNCONFINED COMPRESSION TEST (UC)
ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	5	SAMPLE DEPTH, m	13.7-14.3

TEST CONDITIONS

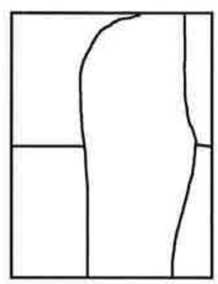
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.59	WATER CONTENT, (specimen) %	0.07
SAMPLE DIAMETER, cm	4.74	UNIT WEIGHT, kN/m^3	26.25
SAMPLE AREA, cm^2	17.64	DRY UNIT WT., kN/m^3	26.24
SAMPLE VOLUME, cm^3	186.70	SPECIFIC GRAVITY	-
WET WEIGHT, g	500.04	VOID RATIO	-
DRY WEIGHT, g	499.69		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	99.6

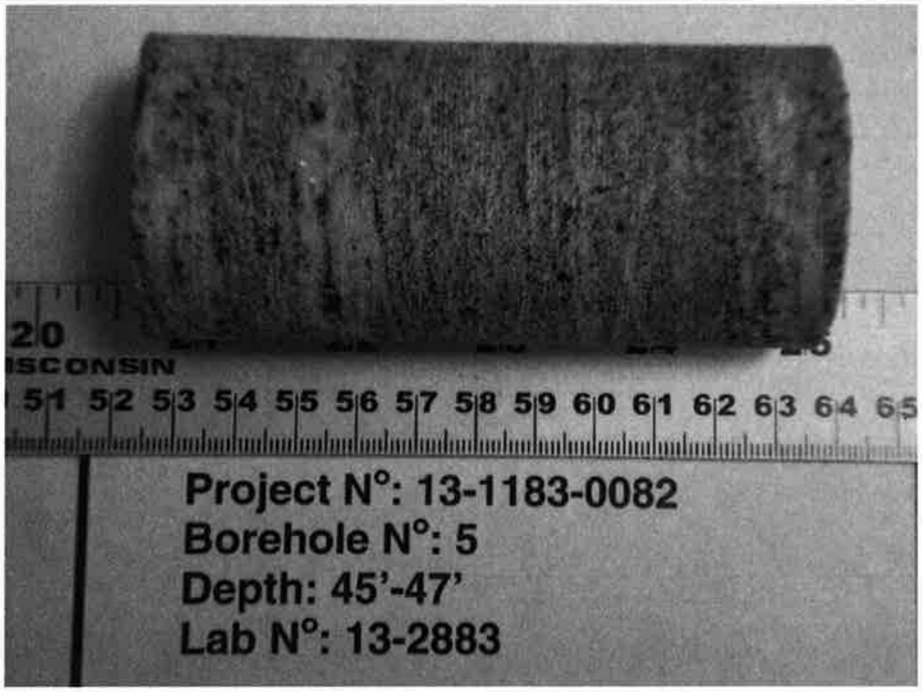
REMARKS:		DATE:	7/24/2013

Checked By: Ro

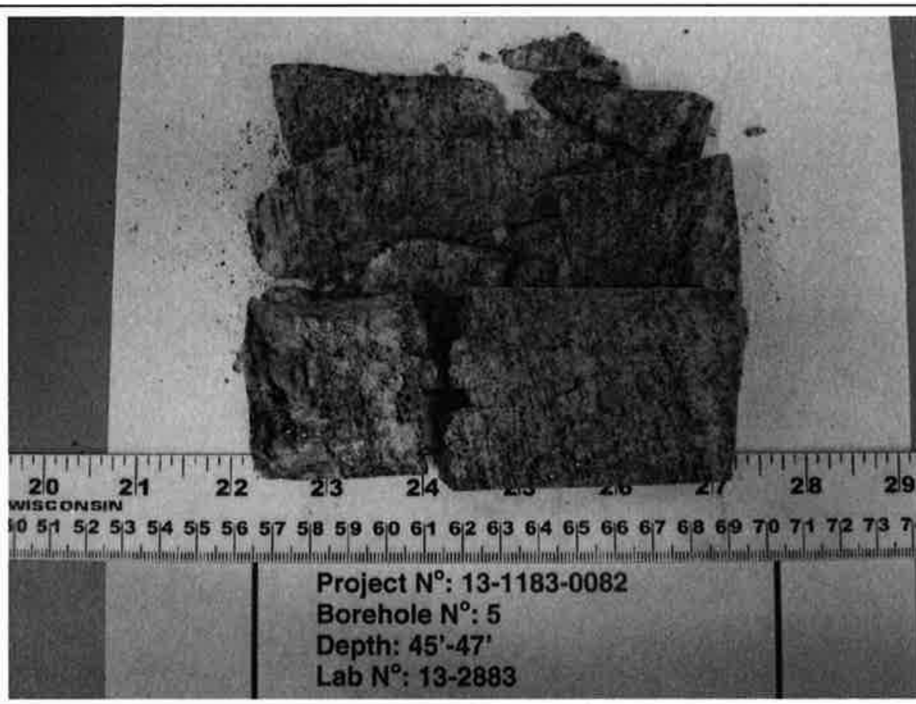
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. Ro

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	6	SAMPLE DEPTH, m	2.9-3.4

TEST CONDITIONS

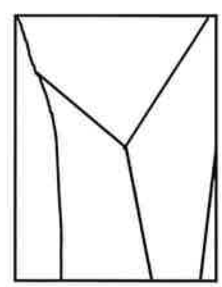
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	8.00	WATER CONTENT, (specimen) %	0.09
SAMPLE DIAMETER, cm	3.62	UNIT WEIGHT, kN/m^3	26.15
SAMPLE AREA, cm^2	10.28	DRY UNIT WT., kN/m^3	26.13
SAMPLE VOLUME, cm^3	82.21	SPECIFIC GRAVITY	-
WET WEIGHT, g	219.28	VOID RATIO	-
DRY WEIGHT, g	219.08		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	116.8

REMARKS:

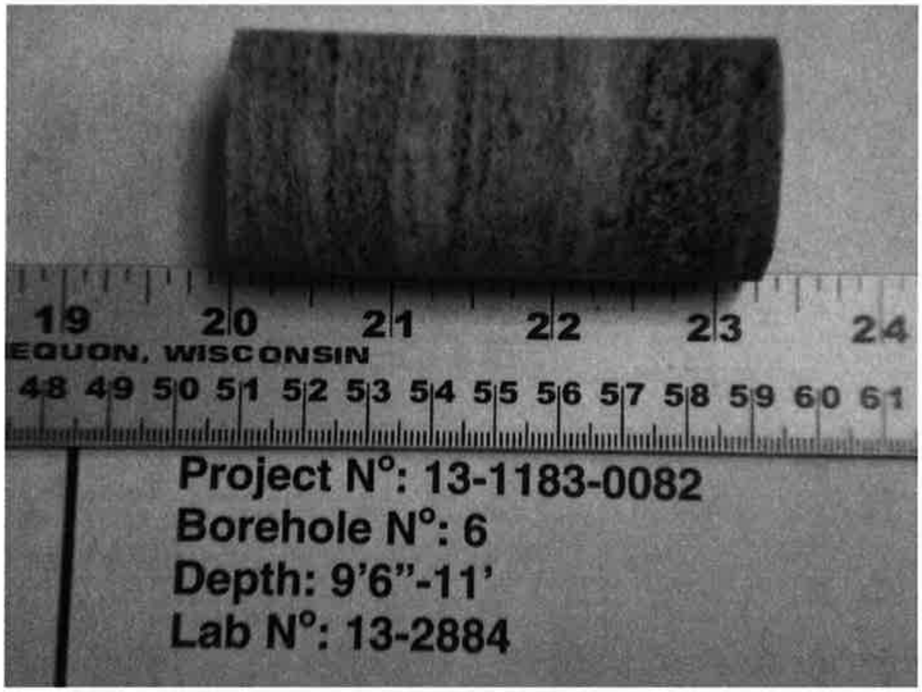
DATE: 7/24/2013

Checked By: RO

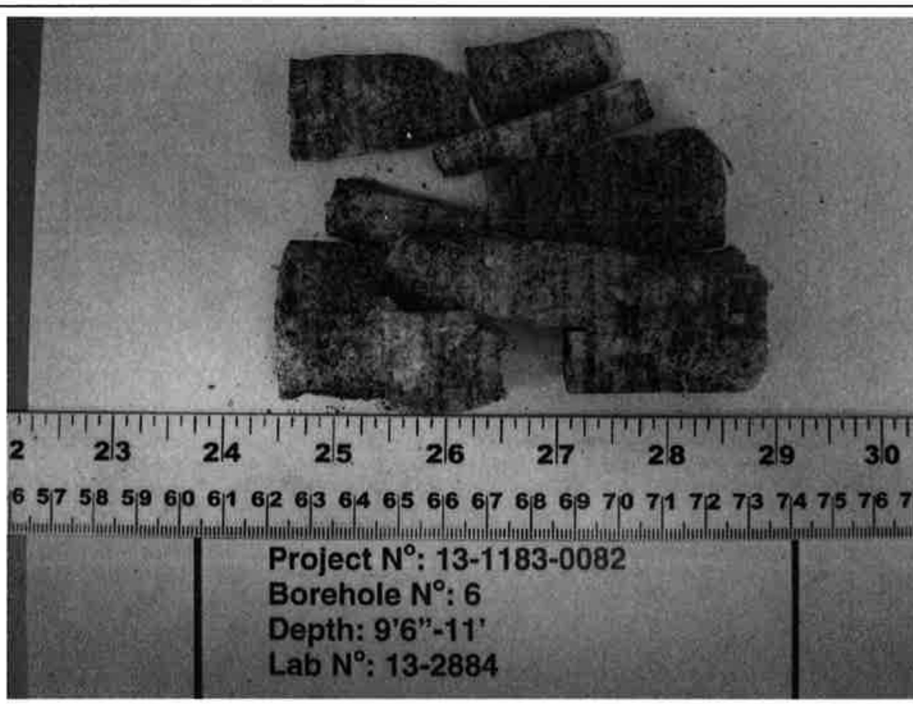
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RO

Borehole No.	Run No.	Depth (ft)	Depth (m)	Test Type	Length (mm)	Core Diameter (mm)	Force (kN)	Rock Type	Is (MPa)	Is(50) (MPa)	Equivalent UCS (MPa)
BH3	2	29.67	9.04	A	42	48	17.348	GNEISS	6.758	6.799	163.2
	2	29.83	9.09	D		48	12.238	GNEISS	5.215	5.2	125.2
	3	34.67	10.57	A	47	48	20.28	GNEISS	7.060	7.3	174.8
	3	34.83	10.62	D		48	9.72	GNEISS	4.142	4.1	99.4
BH4	1	6.5	1.98	A	37	37	9.803	GNEISS	5.624	5.2	124.5
	1	6.58	2.01	D		37	5.761	GNEISS	3.675	3.7	88.2
	1	6.75	2.06	A	37	37	15.379	GNEISS	8.823	8.1	195.2
	2	12.67	3.86	D		37	8.981	GNEISS	5.729	5.7	137.5
	2	12.83	3.91	A	36	37	10.975	GNEISS	6.471	5.9	142.3
	3	15.33	4.67	D		37	5.88	GNEISS	3.751	3.8	90.0
	3	15.5	4.72	A	40	37	14.578	GNEISS	7.736	7.3	174.2
BH5	1	43.25	13.18	A	40	48	20.153	GNEISS	8.244	8.2	196.9
	1	43.75	13.34	D		48	6.206	GNEISS	2.645	2.6	63.5
	2	44.08	13.44	A	53	48	26.871	GNEISS	8.296	8.8	211.0
	2	44.25	13.49	D		48	14.044	GNEISS	5.985	6.0	143.6
	3	51.25	15.62	D		48	12.029	GNEISS	5.126	5.1	123.0
	3	51.67	15.75	A	46	48	15.025	GNEISS	5.344	5.5	131.7
BH6	1	9.75	2.97	A	29	37	11.876	GNEISS	8.693	7.6	182.1
	1	9.92	3.02	D	37	37	1.798	GNEISS	1.147	1.1	27.5
	2	14.42	4.40	A	31	37	6.806	GNEISS	4.660	4.1	99.1
	2	14.58	4.44	D	37	37	7.356	GNEISS	4.692	4.7	112.6
	3	17.58	5.36	A	28	37	11.402	GNEISS	8.644	7.5	179.7
	3	17.75	5.41	D	37	37	8.625	GNEISS	5.502	5.5	132.0

Appendix E

Explanation of Terms Used in the Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	W_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	W_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	W_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_P)/ I_P$	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L - W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION DESIGN REPORT PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE OVER THE SEVERN RIVER W.P. 2360-06-00, SITE 42-86/1&2, GEOCRES31D-564

McCormick Rankin

TRANETOB20462AA
January 07, 2014

REPORT

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

CONTENTS

FOUNDATION DESIGN REPORT
PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE
OVER THE SEVERN RIVER, W.P. 2360-06-00, SITE 42-86/1&2

5 DISCUSSION AND RECOMMENDATIONS

McCormick Rankin (MRC) has been studying the feasibility of replacing/rehabilitation of the existing Highway 400 Bridges over the Severn River in the Township of Tay.

Existing bridge information based on available bridge drawings is summarized in the table presented below.

Table 5.1 Bridge Information

Title	Site Number	Year Built	Length (m)	Width (m)	Existing Structure Type	Proposed Structure Strategy
Severn River Bridge, NBL	42-86/1	1957	27.4	10.4	Single Span Rigid Frame	Replacement
Severn River Bridge, SBL	42-86/2	1991	27.5	12.0	Single Span Rigid Frame	Rehabilitation

In 2012, Coffey prepared preliminary geotechnical investigation reports based on existing information (i.e. desk top study – no boreholes drilled) to aid MRC in their study. More recently, MTO and MRC decided to demolish the existing circa 1957 northbound lanes bridge and to replace it by widening the existing concrete southbound lanes structure, to accommodate the northbound traffic.

The investigation deals with the proposed widening of the Highway 400 southbound Severn River Bridge. The investigation report for the proposed widening of the Highway 400 southbound Severn River Boat Channel Bridge is presented under separate cover.

As mentioned before, it is our understanding that existing SBL bridge will be rehabilitated and widened towards the median to carry the realigned NBL. After the rehabilitation and widening of the southbound structure, the existing northbound lanes bridge will be demolished.

The subsurface conditions were explored during this investigation at eight borehole locations. The boreholes that were drilled from the top of the existing highway embankment show that the embankment fill consists of typically silty sand to sandy silt with traces to some clay and gravel. In Boreholes 5 and 7, the fill extends to the surface of the bedrock/inferred bedrock, while in Boreholes 1, 2, 3 and 8, the embankment fill is underlain by shallow, basal native overburden soils, underlain by bedrock. Boreholes 4, 4A and 6 were advanced from a barge (in the River) and contacted some shallow overburden to the surface of the bedrock.

The natural (native) overburden over the bedrock at the borehole locations was found to be 0.5 m thick at Boreholes 1, 2 and 3; 1.7 m at BH 4A and 2.9 m to 3.4 m at Boreholes 6 and 8. In BH4, the overburden was found to be mixed with rock fill. The natural (native) overburden was found to consist of generally granular type materials (i.e. silty sand to sand and gravel, but generally sand); however, in Boreholes 4A and 8, a cohesive deposit (silty clay) was encountered. The thickness of this cohesive material at the locations of Boreholes 4A and 8 was found to be 0.6 m and 1.7 m, respectively, with a consistency described as very soft to soft at Borehole 4A and stiff at Borehole 8.

At the borehole locations the presence of bedrock was inferred/proven at El 176.8-175.9 m at Boreholes 1, 2, 3 and 7; at El. 173.7 m at BH 8 and at El. 172.3 to 171.2/170.9 m at Boreholes 4, 5 and 6.

5.1 Foundations

We understand that the proposed bridge, which will carry the northbound traffic of the highway, will be constructed adjacent to the existing southbound lanes bridge by widening. The existing southbound bridge is single span, rigid frame structure, with a clear span length of 27.5 m (i.e. from inside to inside of the abutment wall), as shown on the General Arrangement Drawing in Appendix F. The available information also shows that the abutments are supported on shallow spread footing foundations, but mass concrete was used to raise the grade by about 1.5 to 3.0 m to El. 174.5 and 175 m, on which spread footing foundations were constructed. The mass concrete was set 0.3 m into the bedrock.

5.1.1 Spread Footing Foundations

The structure widening can be supported on similar type foundations as the existing bridge foundations (i.e. spread footings on mass concrete set about 0.3m into the bedrock). In this instance (if replacement of the existing mass concrete under the foundations is required), the existing mass concrete will need to be removed to the surface of the sufficiently sound bedrock. However, to reduce the need, cost and constructability of a cofferdam at this location, as well as possible extensive rock excavation due to the previously reported overbreak potential of bedrock at the site, consideration can be given to leaving the existing mass concrete in place if the existing mass concrete is found sufficiently in a good condition to accomodate the anticipated loading conditions induced by the proposed bridge widening.

These two options are discussed in the following paragraphs.

Of the boreholes drilled, Boreholes 4 and 6 are located closer to the abutment locations. In these boreholes, which were advanced from the surface of the water in the River, from a barge, the River bottom was found at El. 174.2 and 173.8 m, respectively. The thickness of the overburden was measured to be 1.9 and 2.9 m and the surface of the bedrock was encountered at El. 172.3 and 170.9 m, respectively. Assuming that the bottom of the footing at the south abutment location will be at El. 174.0 m, and if the surface of the bedrock is similar to that encountered in Borehole 4 (i.e. El. 172.3 m), it can be expected that an approximately 2 m of mass concrete will be required, after setting the mass concrete to about 0.3 m below the bedrock surface. At the north abutment location, the underside of the existing footing, based on the information provided to us, appears to be at El. 174.5 m. In Borehole 6, the surface of the relatively sound bedrock was contacted at El. 170.9 m. While some variations can be expected regarding the surface of the bedrock, assuming that this elevation is representative of the sound bedrock surface in the general area, a grade raise (i.e. mass concrete) of 3.6 m can be used for preliminary estimating purposes (i.e. from El. 170.9 to 174.5 m). If this amount of grade raise is considered objectionable/uneconomical, then consideration can be given to the use of drilled and cast-in-place concrete (caisson) foundations, as will be discussed later in this report.

In general foundations bearing on the surface of the bedrock should be set 0.2 to 0.3 m into the sufficiently sound bedrock.

The following geotechnical resistances are available for footings bearing on level, sound bedrock:

- Factored Bearing Resistance at U.L.S. = 10,000 kPa
- Bearing Resistance at S.L.S. will not govern

If the foundations are to be constructed adjacent to sloping ground, stability must be assured by socketing/keying-in the foundations sufficiently into the bedrock and/or doweling/anchoring into the bedrock. In addition, the footing must be placed on sufficiently level rock surface. If necessary, the bedrock surface can be flattened by levelling or making benches or the problem may be alleviated by providing dowels. As well, it should be ensured that the rock beneath the footing level will not be subject to detrimental scour or frost effects which might jeopardize the footings.

As mentioned before, as a second option, consideration can be given to utilizing the existing mass concrete which supports the existing foundations (e.g. existing retaining walls which will be demolished). This may involve the improvement or the extension of the mass concrete. In this case, the existing mass concrete which will be re-used, including surrounding bedrock, should be inspected to verify their condition and suitability, by qualified personnel and approved. This may involve underwater inspection, depending on the water level in the River. The strength of the existing concrete and its condition need also be verified to ensure the capability of the existing concrete to carry the required loads and to resist further scour/erosion and deterioration. These may require destructive (i.e. obtaining and testing core samples) and or non-destructive testing for verification. We recommend that an NSSP be provided in the Contract Documents for this purpose, if the existing mass concrete is to be utilized for the proposed widening.

For inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with the Canadian Highway Bridge Design Code (CHBDC CAN/CSA, S6-06).

For the evaluation of the sliding resistance of the foundations, the friction factor (ultimate) between the underside of the concrete footing and the clean and sufficiently roughened bedrock surface can be taken as 0.6. Horizontal shear resistance can be supplemented by keying-in to the bedrock and utilizing the passive rock resistance and/or shear in grouted dowels and/or rock anchors. We recommended a minimum dowel length of 1.2 m, but not less than 0.6 m into sound bedrock. Provided that the surface of the mass concrete is sufficiently clean, a friction factor (ultimate) between the underside of the new concrete footing and the existing mass concrete can also be taken as 0.6.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 1000 kPa and resistance at S.L.S. need not be considered. The upper 0.5 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.2 m into the sound rock (embedded length in the sufficiently sound rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor ground resistances should also be checked.

For spread footing foundations, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer appointed by QVE and who is familiar with the findings of this investigation. This is important for this site, since the surface of the bedrock appears to be sloping/variable and that the upper 0.2 to 0.3 m appears to be generally shattered.

Normally for frost protection in this geographic area, the footings should have a permanent earth cover of not less than 1.6 m. If the footings are placed on sufficiently massive rock (i.e. no jointing, cracks, fissures, etc.,) it may be possible to reduce the thickness of frost protection or even eliminate it. For this purpose the following approach can be taken. The surface of the bedrock on which the footing is to be supported should be made level and carefully inspected by a Geologist or a Geotechnical Engineer. The surface of the rock to receive the footing must be free of open fractures, jointing, cracks, fissures or bedding planes, or any other defects which water can get into and cause problems due to frost. This is also applicable to rock surrounding the footing footprint. These areas must also be defect free or made so, such that water could not enter to cause problems with the rock supporting the footing (i.e. further opening the existing defects or causing heave due to frost action). This would not be applicable to footings in water, if it can be ensured that freezing will not occur at the surface of rock level. From the borehole data and the anticipated founding depths, it is unlikely that frost will present a problem for footings placed on bedrock, but the above statements regarding frost protection are included herein for the sake of completeness and in case the rock surface at the footing locations is found within frost depth.

The rock must also be checked for any planes or other defects which may cause the footings to slide towards the River. These are standard field features which are normally evaluated by a Geologist or Geotechnical Engineer, provided they are experienced enough.

If rock blasting is required/permitted for excavation, it should be controlled in order to avoid over-breaking of bedrock and also to prevent any damage to the existing bridge and its support elements. In our opinion, however, rock blasting should not be permitted. Wherever rock is over-excavated, it should be inspected and approved by a Geotechnical Engineer and filled up with same class concrete as the foundation concrete.

In addition, the bearing surface should be cleaned and made free from any loose debris prior to concreting of foundations.

Any mass concrete used to raise the grade to the underside of the footings should be of sufficiently good quality to resist possible erosional forces that may exist in the River.

5.1.2 Deep Foundations

Because of the presence of variable and rather shallow depths to the surface of the bedrock encountered at the site, the use of driven piles is considered unsuitable to support the proposed bridge widening.

The use of spread footings by duplicating the existing structure foundation, while presenting the most logical solution, will likely involve overburden excavation below the River bottom to the surface of the bedrock. In Boreholes 4A and 6, located closest to the proposed footing locations, the bulk of the overburden soils which will cave-in immediately upon excavation and thus the sides of the excavation will need proper support. As well the use of mass concrete to raise the grade may be uneconomical. For these reasons, the use of drilled and cast-in-place concrete piles (caisson) may possibly represent an attractive solution. This approach can also be expected to reduce some of the shoring effort.

Existing mass concrete may cause problems for deep foundation construction at the site.

5.1.2.1 Cast-In-Place Concrete Pile (Caisson) Foundations

Cast-in-place concrete piles (drilled caissons) can be considered and caissons socketed into the bedrock would be required to resist the axial and lateral loads. Vibrations should not present major problems, except possibly when extending the caissons into the bedrock (i.e. while socketing into the bedrock) or if rock fill is encountered in the overburden (e.g. BH 4 location). While excavating, rock adjacent to caisson should not be shattered (damage to the bedrock should be minimized).

Geotechnical resistances of cast-in-place concrete piles increase with socket depth into the bedrock. For caissons which extend not less than 0.3 m into the relatively sound bedrock, 10,000 kPa can be used (end bearing resistance at ULS). The minimum caisson penetration depth below the sufficiently sound bedrock surface may need to be increased depending on the degree of sloping of the bedrock surface to avoid sliding of the caisson due to unbalanced horizontal forces.

The minimum spacing of the caissons centre to centre should normally not be less than three diameters as per CHBDC S6-06. As well, a minimum caisson diameter of 0.76 m is recommended to enable the base inspection and cleaning, if required. However, if there is a compelling reason for the use smaller diameter caissons, this requirement can be looked into.

As was mentioned before, if the rock surface in front of the caisson is sloping and the caisson is located close to the sloping surface, this geometry may adversely affect the resistance, in particular the horizontal resistance. As well, if the rock around the caisson is shattered during the construction, this too will adversely affect the resistances and as such excessive shattering of the rock in the vicinity of the caissons must be avoided. As per OPSS 903, the caisson bottom may if necessary be stepped on sloping bedrock condition, with each step not greater than ¼ the diameter of the bearing area.

Excavation methods shall be such that the sides and bottom of the hole are straight and free of loose material that might prevent intimate contact of the concrete with undisturbed soil or bedrock.

The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent ‘necking’.

It should be pointed out that the presence of rock fill was inferred below the River bottom while advancing Borehole 4 and if this happens during caisson installation it can present problems.

5.1.2.2 Micropiles

Another alternative would be to use micropiles. Similar to the use of caissons, this method can be expected to reduce the extent of excavations, concreting and shoring.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can be installed in most soil and rock types, ground conditions as well as through existing mass or reinforced concrete (i.e. reinforcing steel bars should not present problems). A permanent steel casing is typically used to avoid the grout loss into the voids in the rock fill and to protect the micropile from being exposed to environments. Micropiles can withstand axial and/or lateral loads. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, ground, and the environment. They can be installed in access-restrictive environments as well. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing structures. Micropile structural capacities, by comparison, rely on high capacity steel elements to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout & ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors.

Geotechnical resistances for design purposes will depend on the type and installation methods used. For preliminary estimating purposes a value of 1000 kPa between the sound granite gneiss and grout can be used but the upper 0.5 m of the bedrock should be ignored. If the use of micropiles is to be considered, this should be further discussed with us.

The use of micropiles can be expected to be more costly than spread footing and caisson options. However, this and other details can be discussed with a specialized contractor; we will be pleased to facilitate this, if requested.

5.1.3 Summary of Foundation Options

Supporting the bridge widening on spread footings, duplicating the existing supports or re-use of the existing mass concrete (upon approval), is, in our opinion, the most obvious choice.

Consideration can be given to the use caisson for expediency, including reducing the amount of overburden excavation and possible shoring in comparison with the spread footing foundations option.

The use of micropiles is considered a third option, although it is likely to be the least economical.

5.2 Approach Embankments

It is anticipated that the existing Highway 400 southbound lanes embankment will be widened to accommodate the realigned northbound traffic. The embankment (top) grade will be at about 185 m on the north (i.e. towards Parry Sound) side and about 184 m on the south (i.e. towards Barrie) side of the bridge widening, which we understand will duplicate the existing bridge and embankment structures.

Below embankment fill or below the River bottom, the natural (native) soil strata at the borehole locations were contacted at Elevations ranging from 177.3 to 173.1 m and, thus, the anticipated grade raise over and above the existing natural grades (i.e. original grade or o.g. levels) can be expected to range from about 7 m at the location of Boreholes 1 and 2 to about 13 m at the location of Borehole 5.

Along the proposed south approach, at the location of Boreholes 1, 2 and 3, below the embankment fill, an approximately 0.5 m thick non-cohesive basal soil deposit was contacted overlying the bedrock. Based on these findings and assuming that all stripping is properly executed as per MTO convention, there should be no concerns with foundation instability at these borehole locations and the embankments can be constructed with normal 2H:1V side slopes and, needless to say, flatter side slopes can be used, if desired.

At Borehole 4A location, however, a 0.6 m thick silty clay layer was contacted at a depth of 0.5 m below the River bottom. This deposit must be removed from beneath the footprint of the proposed embankment for foundation stability of the proposed embankment widening.

If this layer is contacted at other locations, it must be removed. This is a possibility as Boreholes 1, 2 and 3 were drilled within the footprint of the existing embankment and the weak clay may have been removed when the embankment was first constructed, but it may exist elsewhere.

At the north approach, Boreholes 5 through 8 were drilled. Boreholes 5 and 7 were put down from the top of the existing embankment fill and contacted fill to the surface of the bedrock. It is possible that at these locations, the bedrock was exposed or any natural soil may have been stripped, during the construction of the existing embankment, including any weak clays. Assuming that all the unsuitable soils, including weak clays, will be removed from the construction of the new embankment, foundation stability of the embankments should not present any problems. When making this statement, it is also assumed that proper stripping was carried out including the removal of all weak clays from underneath the existing embankment side slope which will be widened. The removal is for stability issues and also to prevent possible excessive settlements.

BH 8 was also drilled from the top of the approach embankment. This borehole contacted below the embankment fill, a 1.7 m thick gravelly sand layer (which was identified as possible fill) at a depth of 7.3 m (El. 177.1 m), underlain by a 1.7 m thick silty clay layer to the surface of the inferred bedrock at El. 173.7 m. Unlike the silty clay deposit at BH 4A, this material was found to be of stiff consistency and should not pose a foundation instability problem; thus, it need not be removed. If, however, when stripping, if weaker clayey soils (similar to BH 4A), these must be stripped for foundation stability. This can be done by digging test pits, where necessary, to the surface of the bedrock to verify this condition. It should also be kept in mind that BH8 was advanced from the top of the existing embankment and the silty clay encountered in this borehole would be compressed (i.e. consolidated) under the weight of the embankment and thus would have gained strength. This would not be the case where the silty clay would be present near the toe or beyond the toe of the embankment. In short, all weak silty clay must be removed for embankment stability.

BH 6 was put down from the River and contacted, in sequence, sand, silty sand till and sand & gravel, to the surface of bedrock, at a depth of 2.9 m below the River bottom. These deposits are not considered to pose an embankment foundation instability problem.

In summary, the soils encountered in the boreholes do not entail a slope stability concern, except for the weak silty clay layer contacted in BH 4A and possibly BH 8. Based on the previous desktop study and on the present borehole data, there is evidence that weak silty clay layers exist in the general area. These must be removed from beneath the footprint of the embankment. For this purpose test pits can be dug under the guidance of an experienced Geotechnical Engineer. If weak clay is encountered, it must be removed. Both test pitting and removal any unsuitable soils must be carried out in a manner so as not to induce a failure of the existing embankment. This can be accomplished by removing the unsuitable soils in short (say maximum 4 m wide) sections, perpendicular to the existing embankment and backfilling without undue delay.

We recommend that an NSSP be issued to ensure that these procedures are followed.

After stripping, the exposed subgrade should be inspected and approved. After approval, any overburden subgrade should be properly compacted from the surface, where feasible, using a suitably heavy compactor. If necessary, the groundwater level should be lowered to at least 0.7 m below the subgrade level before any proof rolling and the application of any significant compaction effort. The dewatering can be achieved by gravity drainage and pumping from strategically placed sumps and if necessary, ditches.

If filling is required to be conducted below the water level in the River, the fill material to be placed below the water level will need to consist of suitable granular soils to about 0.5 m above the water level in order effect proper compaction. Erosion and scour protection will need to be provided.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized 2H:1V side slope can be used for the construction of the approach fills, provided that the founding subgrade is prepared as discussed earlier in this section. Proper erosion control measures should be implemented by prompt seed and cover (OPSS 803) and sodding (OPSS804).

The existing embankment side slopes should be properly benched as per MTO standard (OPSD 208.010) where the embankment widening is proposed.

The material used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Selected Subgrade Materials – OPSS 1010). Fill used for construction of the embankment should be in accordance with OPSS 212 and fill placement should meet or exceed the requirement of OPSS 501 and OPSS 206. Construction should be in accordance with SP 206S03. Quality assurance should be provided as per MTO standard 501.08 (OPSS 501).

Based on the findings of the boreholes, the anticipated embankment foundation settlements under the stress generated by grade raise (to El. 184 m on the south abutment side and 185 m on the north abutment side) are expected to be within tolerable limits for a flexible pavement, provided that proper stripping is carried out, as discussed above, including the removal of weak and compressible silty clay layers. This is because at the location of Boreholes 1, 2, 3, 5 and 7, little or no overburden was contacted. In the remaining boreholes (i.e. Boreholes 4A, 6 and 8), the anticipated total settlement, after the embankment is raised to its final level, are 30 mm or less, which, in our opinion, will not necessitate surcharging or preloading, especially since some of these settlements can be expected to take place within several weeks after the grade raise.

In addition to foundation settlements, the newly built embankments can be expected to undergo settlements under their own weight. The magnitude of these settlements will depend on the materials used and compaction effort applied (i.e. construction procedures), while the rate of settlement will depend on the materials used to build the embankments (e.g. granular soils will settle much more rapidly compared with clayey fills). Assuming that an average SSM type soil embankment fill will be used, the settlement of the new embankment under its own weight should be substantially completed within about three months. Assuming that proper compaction procedures are followed, the magnitude of settlement of a typical 10 m high embankment fill under its own weight would be about 50 mm, bringing the maximum total settlement including the foundation settlement to about 80 mm. We recommend that, in order to reduce the detrimental effects of such settlements, the paving of the road be delayed by about four weeks after the placement of granular pavement fill.

As well, it is recommended that any excessive settlements and lateral movements should be observed during the construction with the view to rectify such problems, should they occur.

It should also be pointed out that some settlement of the existing embankments can be expected due to widening, because of stress superposition from the widened section. Assuming that all the unsuitable soils under the sloping portion of the embankment (on which additional soils are to be placed) were removed when the embankment was first constructed, these settlements should not be excessive and should not cause major problems (i.e. extensive cracking of the existing pavement).

It should however be pointed out that settlements of this magnitude (i.e. 80 mm) are only applicable to high embankments (i.e. of the order of 10 m). It is our understanding however that with the present design, the filling (i.e. embankment widening) will only occur towards the median side. From the cross sectional drawings provided by MRC (see Appendix F) the grade raise is a maximum of 1.5 to 2.0 m at the median ditch gradually decreasing towards the existing roadway. This is because when the existing southbound bridge was built in 1990's, the space between the existing northbound embankment and the newly built southbound embankment was filled, leaving only a 1.5 to 2.0 m deep median ditch. In this case, the anticipated settlement under this amount of fill (i.e. 1.5 to 2.0 m) is 25 mm at the south abutment side and 30 mm at the north abutment side. These settlements are not considered excessive, but they will translate into differential settlements between the edge of the existing embankment and the existing ditch location. However, as the transition from the existing edge of embankment (i.e. zero grade raise) and the ditch location is very gradual, these differential settlements are expected to be within tolerable limits for a flexible pavement.

In summary if the widening of the existing embankment is only towards the median side, as presently planned, the grade raise will gradually increase from zero from the edge of the existing pavement towards the median ditch where it entail a gradual 1.5 to 2.0 m grade raise. Based on the available borehole data this should cause no major cracking neither of the existing nor the new embankment, provided the subgrade is properly prepared after stripping and the new fill is properly compacted as per MTO convention.

5.2.1 Retaining Wall

We understand that the project includes the construction of a retaining wall on the north and south sides of the widened highway. Based on the GA drawing, near vertical facing retained soil system (RSS) is the presently preferred option for the proposed retaining wall construction. The height of the wall can be expected to be of the order of 3 to 9 m depending on the location (about 9 m high near the abutment and 3 m at the end of the retaining wall). Typical retaining wall options are as follows;

- Conventional Cast-in-place Reinforced Concrete Cantilever Retaining Wall
- Contiguous Caisson Retaining Wall
- Mechanically Stabilized Earth /Retained Soil System (MSE/RSS) Wall

These options based on the available subsurface data, are discussed in the following paragraphs to cover the geotechnical issues of the proposed retaining walls at the Highway 400 Port Severn River Bridge site.

The available borehole data show that the possible retaining wall locations are probably underlain by fill which generally range in thickness from 7 to 13 m. The fill in Borehole 8 is underlain by about 1.7 m thick gravely sand, which is further underlain by 1.7 m thick silty clay. Below the silty clay in Borehole 8 and fill in Boreholes 5 and7, bedrock was contacted/inferred.

If the proposed retaining wall will be placed on a sloping ground (i.e. embankment or berm side slope), stability of the existing slope should be maintained during the construction.

5.2.1.1 Conventional Cast-in-place Reinforced Concrete Retaining Wall

The use of conventional cast-in-place reinforced concrete retaining wall may be a feasible option for the proposed retaining wall construction. In this instance the foundations of the wall will need to be extended to the surface of the sufficiently sound bedrock (i.e. typically 0.2 to 0.3 m below the surface of bedrock). This can be achieved by using drilled and case-in-place concrete piles (i.e. caissons) or using spread footing foundations. The depths of such foundation were discussed in section 5.1.1 and 5.1.2 of this report and will not be elaborated here, especially since it will probably present a less cost effective option in comparison with the presently chosen RSS wall option.

5.2.1.2 Contiguous Caisson Retaining Wall

A contiguous caisson type retaining wall would be suitable for the prevailing subsurface conditions. This consists of vertically drilled holes which are interlocked and filled with a suitable concrete mix. A steel I-beam is typically placed in the holes at every 2 to 3 m before concreting, if tie backs are required. At this site the caissons will need to be extended into the bedrock for fixity and this will likely render this system less economical than an RSS type wall. In addition, the visible surface of the wall will probably need to be treated for aesthetic reasons, which will render caisson wall even less economical.

5.2.1.3 Retained Soil System (RSS)

Consideration can also be given to the use of a retained soil system (RSS) wall, provided there is sufficient horizontal space to implement this option. Vertical wall facing segmental concrete panel RSS with reinforcement installed within backfill (i.e. Tensar/Nilex Acres, Terrafix Terrafort) may be a feasible option based on the GA drawing. Typically, this type of RSS wall is supported on a granular bearing pad. In this instance, the minimum thickness of this granular pad supporting the RSS fill is 0.4 m. But RSS supplier/Contractor may increase this recommended minimum thickness. From the GA drawing, the based of the deeper portion of the RSS wall (i.e. near the proposed abutment) appears to be at about El. 176 m gradually rising with increased distance from the abutment. The available borehole data indicate that at about El. 176 m, either native overburden or embankment fill may be encountered. As the grade for the base of the proposed wall rises, the base can be expected to site on the existing embankment fill. There are some weak zones in the embankment fill which appear not to have been systematically compacted when the embankment was first built, as evidenced by sporadic low N-values. As well there may be some weak or organic soils, such as weak clays in the native (natural) overburden soils overlying the bedrock. For this reason, after stripping to the bottom elevation of the proposed granular fill pad, the exposed subgrade should be inspected, evaluated and approved by qualified personnel. If unsuitable and/or uncompacted soils are found or probed, they should be replaced with compacted suitable material. If the excavated soils are found to be of reasonable quality they can be re-used. The fill should be compacted to at least 97 % of its Standard Proctor Maximum Dry Density (SPMDD). The granular pad should be compacted to at least 98 % the SPMDD. This procedure should reduce the magnitude of any differential settlements to tolerable limits. As well, the global stability is expected to be acceptable, if unsuitable founding soils are removed, if and where necessary to the surface of the bedrock. Internal stability is the responsibility of the RSS supplier/Contractor.

Typically, the facing panels of the RSS wall are supported on a strip footing which is placed on a granular bearing pad. The thickness of this granular pad varies but is generally between 0.6 and 1.0 m. As mentioned before, because of the presence of weak zones in the embankment fill a minimum 1.0 m thick pad is recommended, but it should be extended deeper if during excavation and inspection, weak soils are found. These should be removed and replaced, if necessary, beyond the 1.0 m depth.

The granular pad supporting the facing panels should be extend at least 1.0 m beyond the perimeter of the footing and compacted to not less than 97 % of the SPMDD of the granular fill material. In that event, a factored geotechnical resistance of up to 220 kPa at ULS and resistance of 140 kPa at SLS would be available. For a subgrade prepared in accordance with our recommendations, for the quoted SLS value, the estimated maximum settlement is 30 mm.

5.2.1.4 Retaining Wall Backfill

Approved free draining & frost free granular materials in accordance with MTO standards (OPSS 1010, OPSD 3101.150 and OPSD 3101.200) should be used to backfill the retaining wall. Proper drainage system should be provided to prevent unexpected hydrostatic water pressure build up behind the retaining wall.

5.3 Lateral Earth Pressures

Backfill behind the abutments and associated retaining structures should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150 and OPSD 3101.200.

Free-draining backfill materials (i.e. Granular ‘A’ or Granular ‘B’ Type I or Type II, with minus 0.075 mm sieve size material not exceeding 5%) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C. For design purposes, the following static parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, ϕ = 35° (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

K_a = 0.27	K_b = 0.35
K_o = 0.43	K* = 0.45

Compacted Granular 'B' Type I

Angle of Internal Friction, ϕ = 32° (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

K_a = 0.31	K_b = 0.41
K_o = 0.47	K* = 0.57

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure.

K* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding (e.g. when supported on bedrock as is the case for this project), then at rest pressures should be used in accordance with Canadian Highway Bridge Design Code (CHBDC S6-06). The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the CHBDC Commentary can be consulted. K* is typically used when the retaining structure is supported on unyielding foundations, such as spread footings on bedrock. We recommend that where the lateral yield of the retaining structure may render the use of active soil pressure (i.e. the use of K_a may be possible), the intermediate pressure coefficient K_b be adopted to allow for future changes in the pressure distribution due to vibrations induced by the highway traffic.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

5.4 Seismic Design

Seismic analysis is not required for single span bridges regardless of seismic performance zone except for single span truss bridges as per Clause 4.4.5.2 of CHBDC CAN/CSA-S6-06. For this reason seismic analysis is not required for this project, as the proposed bridge is a single span structure.

As the proposed structure will be supported on sound bedrock, the foundation materials are considered not liquefiable.

5.5 Construction Comments

All excavations, shoring and backfilling should be carried out in conformance with the Occupational Health and Safety Act (OHSA) 213/91, as well as the following specifications.

- OPSS 539 Construction Specification for Temporary Protection System
- OPSS 902 Construction Specification for Excavation and Backfilling – Structures

The boreholes show that the excavation can be expected to extend through fill material at some locations, to the surface of the bedrock, while at other locations, the fill is underlain by some basal overburden immediately above the bedrock. Overburden was also contacted below the River bottom. The composition of overburden at the borehole locations was found to range from silty sand till, silty fine sand to sand and sand & gravel. Silty clay was also contacted at two borehole locations. These soils can be classified as

Granular Pavement Fill	Type 3 soil
Embankment Fill	Type 3 soil above groundwater table
(typically silty sand to sandy silt	Type 4 soil below groundwater table
with traces to some clay and gravel)	
Silty Sand to Sand & Gravel	Type 3 soil above groundwater table
	Type 4 soil below groundwater table
Glacial Till (dense to very dense)	Type 2 soil above groundwater table
	Type 4 soil below groundwater table
Silty Clay (stiff to hard)	Type 3 soil above groundwater table
	Type 4 soil below groundwater table
Silty Clay (very soft to firm)	Type 4 soil

The bridge foundations are expected to be supported on the bedrock. Therefore, dewatering will only be required to facilitate the excavations through the overburden and to enable inspections to verify the condition of the bedrock, as well as to facilitate mass concrete pour to raise the grade to the underside of the proposed footings and the construction of footings.

It is expected that at least some of the foundation construction work will be carried out below the water level in the River. The severity of the unwatering can possibly be reduced by regulating the level of the water (i.e. lowering) in the River by means of the existing upstream control structure. Regardless, however, some sort of cofferdam will be required to prepare the foundations on the bedrock, for concrete pour, etc. Tight interlocking steel sheet piling extending to the surface of the bedrock can be considered. This may however be costly and it may not provide a sufficiently tight enclosure, if the rock surface is not level. Sand bagging and pumping from within the cofferdam enclosure can also be considered. There are also other methods used by some contractors such as plastic bladder enclosure, etc. to provide easier working environment within the River. These decisions are however generally left to the discretion of the Contractor.

With respect to unwatering there is an advantage in leaving the existing mass concrete in rather than removing it. This is because it is generally difficult and costly to extend tight interlocking sheeting into bedrock (for dewatering/unwatering purposes). However if the existing mass concrete is left in place (after ensuring that it is sufficiently sound) the cofferdam sheeting can be braced/supported against/on the existing mass concrete (when extending the existing mass concrete and the footing). Thus utilizing the existing mass concrete presents an advantage in this respect.

Some dewatering will also be required to facilitate stripping and the construction of the new embankment fills, which on land, can normally consist of gravity drainage and pumping form strategically placed sumps, as discussed before.

Shoring will likely be required to construct the new abutments (abutting into the existing abutments) and the approach fills.

In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing/rakers). In this instance, the use of tiebacks will also likely be required. The soldier piles can be expected to extend into the bedrock. Tiebacks would extend, through the fill and some shallow overburden, into the bedrock. Tiebacks should be assumed to derive their resistance from the bedrock only (i.e. resistance from the overburden should be ignored). For preliminary design purposes, the factored rock/grout bond resistance at U.L.S. can be taken as 800 kPa and resistance at S.L.S. need not be considered.

The shoring system should be designed so that the lateral movement of any portion of the shoring system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work. As mentioned before all shoring should be in accordance with OPSS 539.

Table 5.5.1

Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m^3)
Granular Embankment Fill	0.32	0.49	3.1	21.0
Embankment Fill (typical)	0.36	0.53	2.8	20.0
Silty Sand/Sand	0.33	0.50	3.0	19.0
Gravelly Sand, Sand & Gravel	0.32	0.49	3.1	20.5
Silty Sand Till (compact to dense)	0.31	0.47	3.2	21.5
Bedrock	0.20	0.40	5.0	24.0

It should be pointed out that the presence of cobbles and boulders can be expected within the fill and the overburden, as well as the presence of rock fragments within the lower portion of the natural overburden, immediately above the bedrock. As was mentioned before, rock fill was found at Borehole 4 location in the River (Borehole 4 was located 2.7 m away from Borehole 4A in which no rock fill or boulders were found). We recommend possible presence of cobbles and boulders in the fill or the natural overburden, as well as the presence of rock fill be ‘red-flagged’ in the Contract Documents.

Due to the fact that existing and proposed structures will be attached, vibrations should be monitored during the proposed bridge construction (if rock and/or mass concrete excavation may include percussion type penetration or other methods causing vibration). Special provision for vibration monitoring is given in Appendix H. An NSSP should be issued in this respect.

5.6 Scour and Erosion Protection

If required, scour protection and erosion control should be designed (if required) by an experienced Hydraulic Engineer.

5.7 Frost Protection

Design frost protection depth for the general area is 1.6 m. Therefore, a permanent soil cover of 1.6 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, placed on overburden or shattered/fractured rock. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

The Limitations of Report, as quoted in Appendix I, are an integral part of this report.

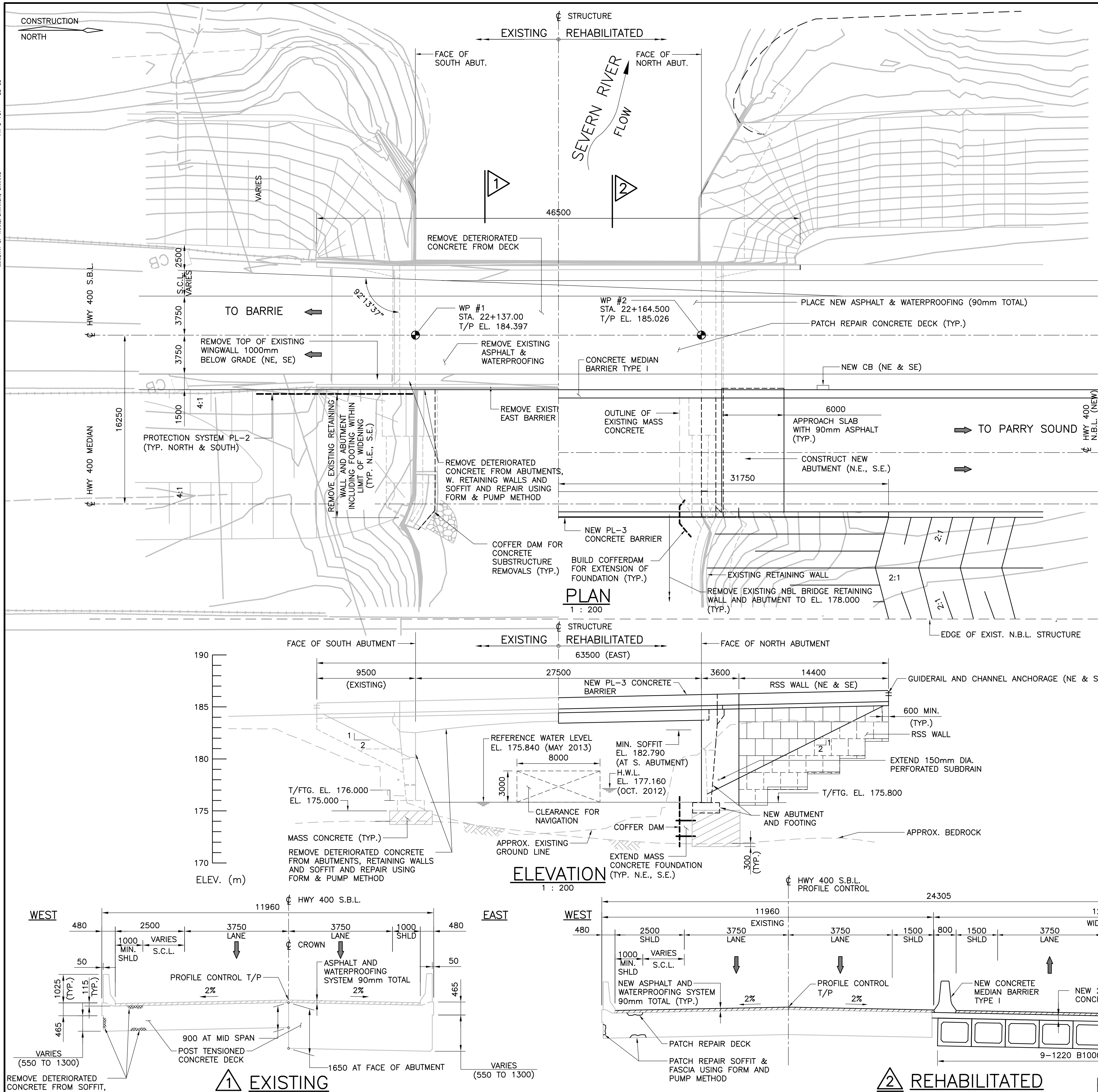
For and on behalf of Coffey.

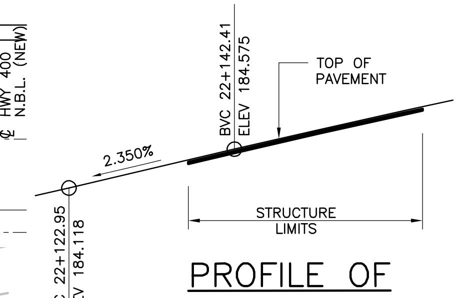
Gwangha Roh, P.Eng., Ph.D.
Senior Geotechnical Engineer

Zuhtu Ozden, P.Eng.
Senior Principal

Appendix F

GA Drawings and Cross Sectional Drawings





PROFILE OF HWY 400 S.B.L.

N.T.S.

LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGING

APPLICABLE STANDARD DRAWINGS:

OPSD 912.4800	GUIDE RAIL SYSTEM, CONCRETE BARRIER, PERMANENT CONNECTION INSTALLATION, SINGLE STEEL BEAM TO CONCRETE BARRIER
OPSD 3419.100	BARRIERS AND RAILINGS – STEEL GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD 3941.200	FIGURES IN CONCRETE – SITE NUMBER AND DATE LAYOUT

GENERAL NOTES

CLASS OF CONCRETE:

PRECAST GIRDERS	50MPa
REMAINDER UNLESS OTHERWISE NOTED	30MPa

CLEAR COVER TO REINFORCING STEEL:

DECK	TOP	70 ± 20
	BOTTOM	40 ± 10
REMAINDER		70 ± 20 UNLESS OTHERWISE NOTED

REINFORCING STEEL:

REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12–1 AND SS12–2, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.

SAWCUTS WHERE INDICATED SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL WHICHEVER IS LESS.

FOR TRAFFIC STAGING AND MAINTENANCE OF TRAFFIC SEE CONSTRUCTION STAGING DRAWINGS.

PROTECTION SYSTEMS REQUIRED TO COMPLETE THE WORK SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA BY CONTRACTOR. LIMITS OF PROTECTION SYSTEM TO BE DETERMINED BY CONTRACTOR. PROTECTION SYSTEMS SHALL BE SUFFICIENT FOR ALL ACCESS AND WORKING PLATFORMS.

COFFER DAMS REQUIRED TO COMPLETE THE FOUNDATION EXTENSION AND SUBSTRUCTURE REMOVALS IN THE DRY SHALL BE DESIGNED BY THE CONTRACTOR.

BACKFILL SHOULD NOT BE PLACED UNTIL THE DECK HAS REACHED 75% OF ITS SPECIFIED STRENGTH. BACKFILL SHOULD BE PLACED SIMULTANEOUSLY AT BOTH ENDS OF THE STRUCTURE KEEPING THE HEIGHT OF BACKFILL THE SAME. AT NO TIME SHALL THE DIFFERENCE IN HEIGHT OF BACKFILL BE GREATER THAN 500mm.

LIST OF ABBREVIATIONS:

T/P	– DENOTES TOP OF PAVEMENT
T/FTG.	– DENOTES TOP OF FOOTING
RSS	– DENOTES RETAINED SOIL SYSTEM

LICENSED PROFESSIONAL ENGINEER J.S. YUN 100079999 PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER B. R. BRIDGES PROVINCE OF ONTARIO

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

DISTRICT
CONT. No.
WP No. 2360–09–00

HIGHWAY 400 NBL & SBL
SEVERN RIVER BRIDGE
BRIDGE REHABILITATION

SHEET

PRELIMINARY GENERAL ARRANGEMENT

McCORMICK RANKIN
A member of MMM GROUP

METRIC

REVISIONS | DESCRIPTION

DESIGN AY	CHK BB	CODE CHBDC–06	LOAD CL–625–ONT	DATE MAR/13	
DRAWN CA	CHK AY	SITE 42–86/1&2	STRUCT	SCHEME	DWG P1

MINISTRY OF TRANSPORTATION, ONTARIO PR–D–707 88–05
CAD FILE LOCATION AND NAME: S:\2012\32\3212093\310–4 PORT SEVERN RIVER SBL\S3212093–300––001GA.dwg
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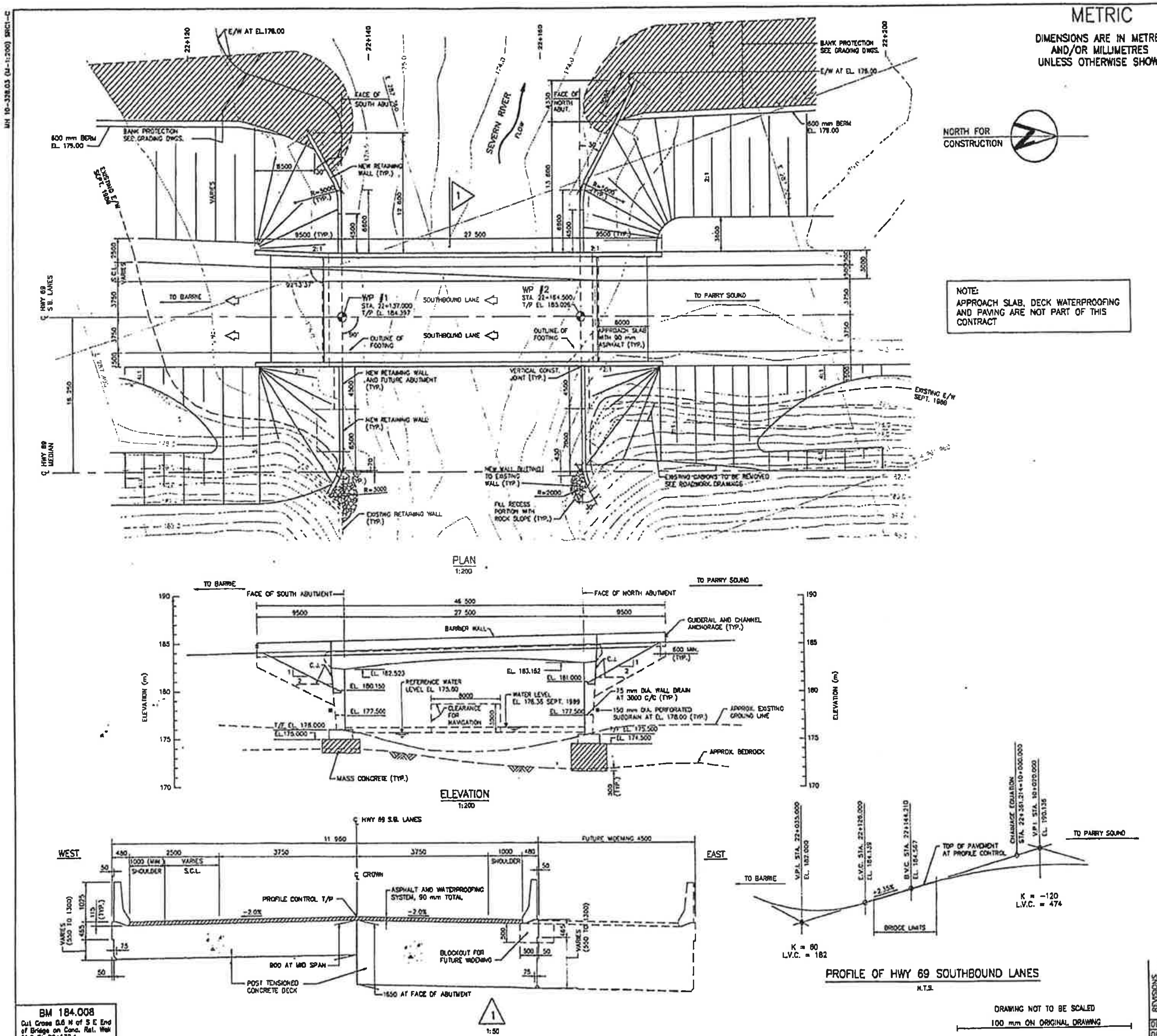
DATE PLOTTED: 8/14/2013 4:17:24 PM BY: RICHIE RAPISTA

Drawing E-6



GENERAL NOTES

1. CLASS OF CONCRETE
   - DECK — 35 MPa
   - REMAINDER — 30 MPa
2. REINFORCING STEEL
   REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE NOTED. BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS.
3. CLEAR COVER TO REINFORCING STEEL
   - FOOTINGS — 100±25
   - ABUTMENTS, WINGWALLS AND RETAINING WALLS
     - FRONT FACE — 80±20
     - BACK FACE — 70±20
   - DECK SLAB
     - TOP — 70±20
     - BOTTOM AND SIDES — 50±10
   - REMAINDER (UNLESS NOTED) — 70±20
4. CONSTRUCTION NOTES.
   BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF THE BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 500 mm.
   FOOTINGS SHALL BE SET 300 mm INTO SOUND BEDROCK.
   ROCK SURFACES IN OVER-EXCAVATED AREAS SHALL BE SUBJECT TO APPROVAL BY THE ENGINEER.
   OVER-EXCAVATION SHALL BE REPLACED WITH CONCRETE OF SAME CLASS AS FOOTING CONCRETE.
   SEVERN RIVER COULD BE TEMPORARY CLOSED TO BOAT TRAFFIC DURING CONSTRUCTION WITH THE MINISTRY APPROVAL.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATA
3. FOOTING AND RETAINING WALL LAYOUT
4. FOOTING REINFORCING
5. RETAINING WALLS REINFORCING
6. WINGWALLS AND DETAILS
7. DECK LAYOUT AND SCREED ELEVATIONS
8. DECK AND ABUTMENT REINFORCING
9. PRESTRESSING LAYOUT AND DETAILS
10. 6000 mm APPROACH SLAB
11. BARRIER WALL
12. AS CONSTRUCTED ELEVATIONS AND DIMENSIONS
13. STANDARD DETAILS
14. QUANTITIES – STRUCTURE 1

LEGEND

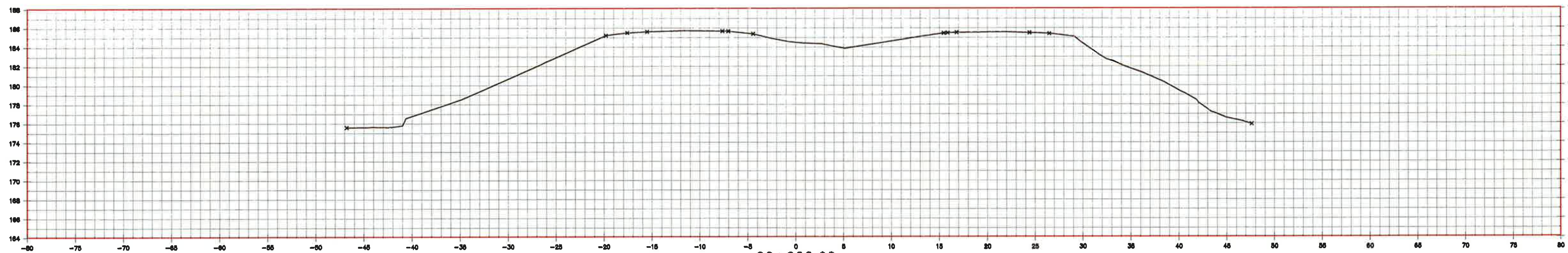
- T/F – TOP OF FOOTING
- T/C – TOP OF CONCRETE
- T/P – TOP OF PAVEMENT
- WP – WORKING POINT
- E/W – EDGE OF WATER
- S.C.L. – SPEED CHANGE LANE
- HWL – HIGH WATER LEVEL
- C.J. – CONSTRUCTION JOINT

APPLICABLE STANDARD DRAWINGS

- DD-3501 MINIMUM GRANULAR BACKFILL REQUIREMENTS
- DD-3504 RETAINING WALL BACKFILL REQUIREMENTS

REVISIONS	DATE	BY	DESCRIPTION

DESIGN CL	CHK. ATC	CODE OHBDC-83	LOAD CLASS A	DATE DEC./90	
DRAWN HT	CHK. ATC	SITE 42-865	STRUCT.	SCHEME	DWG. 1



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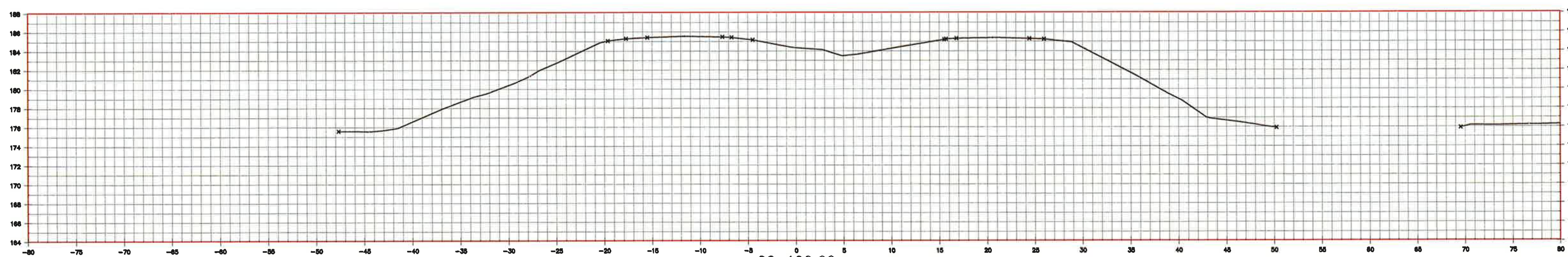

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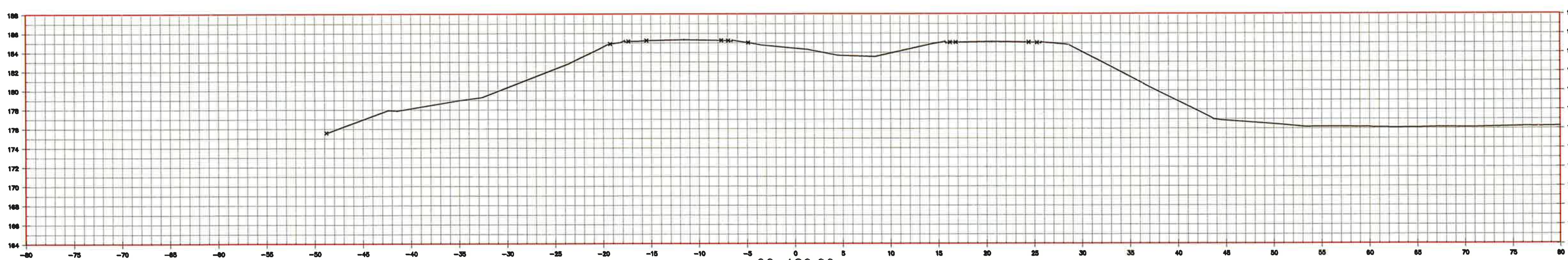

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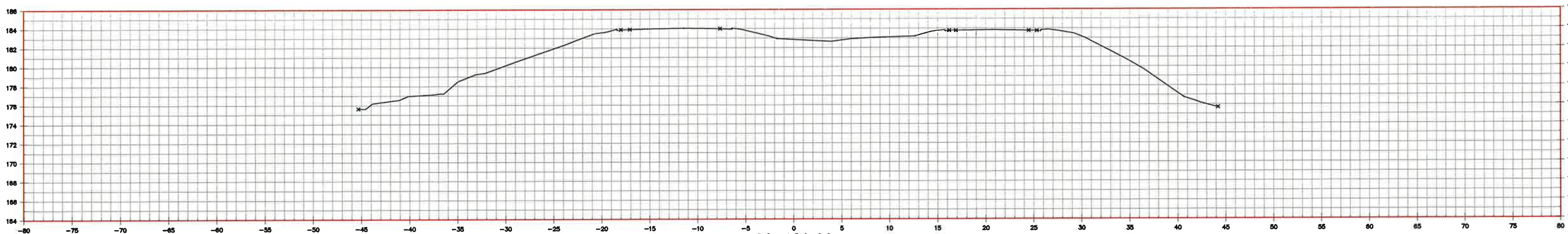

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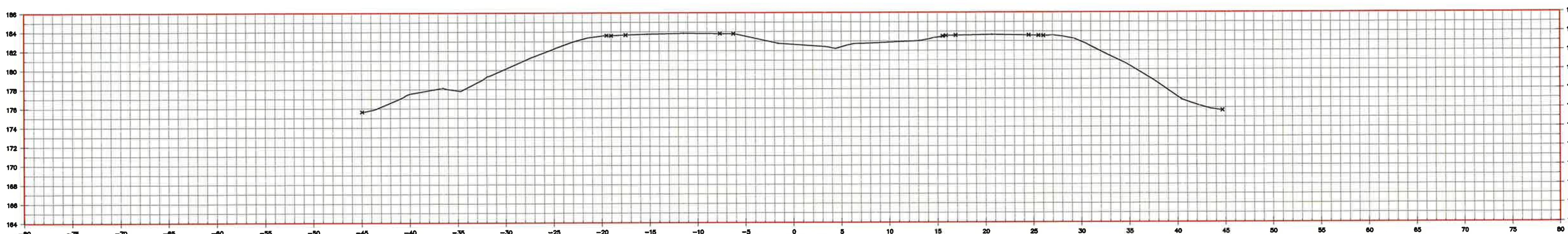

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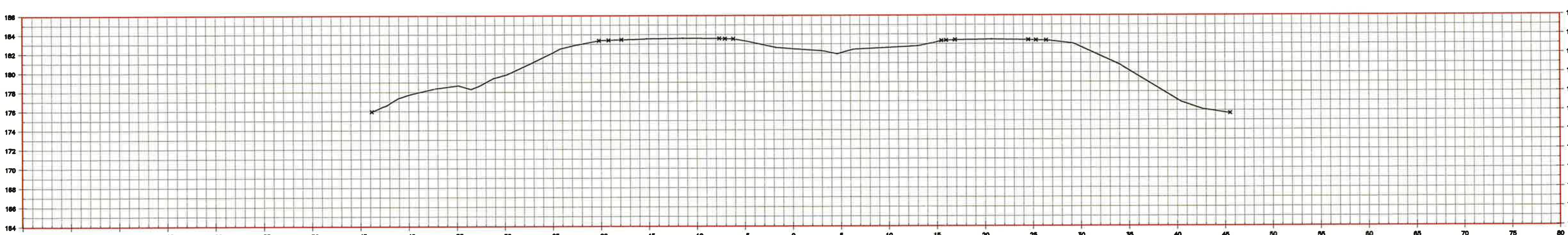

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Appendix G

Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Table G-1

Foundation Options for Severn River Bridge Widening

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	-Lower cost than deep foundation options -Dewatering and unwatering required -May require extensive shoring	-Greater shoring effort will likely be needed in comparison with caisson and micropile options	Low to Medium	-Feasible -Temporary support system is required
Shallow Foundations with re-use of the existing mass concrete	-Lower cost than other options including shallow foundations directly on bedrock after removing the existing mass concrete -Re-use of the existing mass concrete is subject to its condition	-Re-use of the existing mass concrete will reduce shoring and dewatering/unwatering efforts -If extension of existing mass concrete is required, dewatering and unwatering will be required	Low to Medium	-Feasible subject to the existing mass concrete condition -Partial replacement of existing mass concrete and/or extension of existing mass concrete will be required.
Driven H-pile foundations	-May reduce shoring effort	-Existing mass concrete may create problems	Medium	-Not feasible for the prevailing subsurface conditions
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-May reduce shoring effort	- Existing mass concrete may create problems, but to a lesser degree than driven piles	Medium	-Can be considered if shoring is expected to be extensive
Micropiles	-May reduce shoring effort -Equipment easier to operate under low overhead and restricted access conditions -Can be installed through mass concrete if encountered	-Rock fill, if encountered, may create problems during installation but to a lesser extent than caisson option	Higher in comparison with other options	-Would merit consideration if it reduces shoring effort and there is problems with access and equipment overhead, as well as existing mass concrete

Appendix H

List of OPSS, OPSD and Non-standard Specifications

List of OPSDs, OPSSs and Non-standard Specifications

OPSDs

OPSD 208.01 Benching of Earth Slopes

OPSSs

OPSS206 - Construction Specification for Grading

OPSS212 - Construction Specification for Borrowing

OPSS 501 - Construction Specification for Compacting

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 803 - Construction Specification for Sodding

OPSS804 - Construction Specification for Seed and Cover

OPSS 903 – Construction Specification for Deep Foundations

OPSS.PROV 1010 – Material Specification for Aggregates-Base, Sub base, Select Subgrade, and Backfill Material

NSSP Wording

Special Provision

Removal of Unsuitable Soils - Item No.

A weak silty clay layer contacted in Borehole 4A. Based on the previous desktop study and on the present borehole data, there is evidence that weak silty clay exist in the general area. These must be removed from beneath the footprint of the embankment. For this purpose test pits can be dug under the guidance of an experienced Geotechnical Engineer. If weak clay is encountered, it must be removed. Both test pitting and removal any unsuitable soils must be carried out in a manner so as not to induce a failure of the existing embankment. This can be accomplished by removing the unsuitable soils in short (say maximum 4 m wide) section, perpendicular to the embankment and backfilling without undue delay.

Special Provision

Vibration Monitoring

The vibration monitoring equipment shall be placed on the existing and newly widened structure such that it will not be disturbed. The location should be as close as possible to the construction works.

The vibrations at the existing structure shall not exceed 100 mm/s (peak particle velocity).

The Contractor shall take readings during the construction. The results shall be submitted to the Contract Administrator at the end of each day.

If the readings are not within the limits stated above, the Contractor must alter his/her construction procedures until the vibrations on the existing and newly built structure are within acceptable levels.

Appendix I

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

Appendix B
Severn River Boat Channel Report

coffey

McCormick Rankin

Foundation Investigation and Design Report

Proposed Widening of Southbound Highway 400 Bridge Over the Severn River Boat Channel, Township of Baxter,
WP 2376-09-00, Site 42-87/1&2, GEOCRES 31D-566

TRANETOB20462AA
07 January 2014

When you think with a global mind problems get smaller coffey

January 07, 2014

McCormick Rankin
2655 North Sheridan Way, Suite 300
Mississauga, Ontario
L5K 2P8

Attention: Mr. Ben Hui, P.Eng., Senior Project Manager

Dear Mr. Hui,

RE: Foundation Investigation Report, Proposed Widening of Southbound Highway 400 Bridge over the Severn River Boat Channel, Township of Baxter, MTO Central Region, W.P. 2376-09-00, Site 42-87/1&2, GEOCRES 31D-566

Please find attached our foundation investigation and design reports relating to the above noted site.

If you have any comments or enquiries, please contact the undersigned.

For and on behalf of Coffey.

Zuhtu Ozden, P.Eng.
Senior Principal

FOUNDATION INVESTIGATION REPORT, PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE OVER THE SEVERN RIVER BOAT CHANNEL, TOWNSHIP OF BAXTER, MTO CENTRAL REGION, W.P. 2376-09-00, SITE 42-87/1&2, GEOCRES 31D-566

McCormick Rankin

Project: TRANETOB20462AA
January 07, 2014

REPORT

Coffey
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

CONTENTS

FOUNDATION INVESTIGATION REPORT PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE OVER THE SEVERN RIVER BOAT CHANNEL, TOWNSHIP OF BAXTER, MTO CENTRAL REGION, W.P. 2376-09-00, SITE 42-87/1&2

1 INTRODUCTION

Coffey was retained by McCormick Rankin (MRC) to carry out a foundation investigation for the proposed Highway 400 Southbound Bridge widening for realigned northbound lanes over the Severn River Boat Channel in the Township of Baxter, Ontario.

The existing northbound Severn River Boat Channel Bridge is an approximately 93.7 m long, 11.3 m wide, open spandrel deck arch bridge. This structure, which was built in 1957, will be demolished. The existing southbound bridge, built in 1992, will be widened to accommodate the proposed realigned northbound lanes. The widening will take place towards the median of the existing highway.

The proposed bridge widening is planned to be identical (width in GA is different) to the existing southbound bridge, which is a three span slab on steel I-girder structure. The existing bridge has a length of 118 m and a width of 12 m. It was built in 1992 and is supported on integral abutments with piers supported on bedrock.

The purpose of this investigation was to obtain information about the subsurface conditions at the proposed bridge widening site by means of boreholes, and to determine the engineering characteristics of the overburden soils and of the underlying bedrock, by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

The site is located on Highway 400 at the mouth of Severn River Boat Channel at Little Lake joining Georgian Bay, as shown on Drawing 1. The surrounding area is generally gently rolling and rock outcrops are visible in the vicinity.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located at the interface of Physiographic Regions ‘Algonquin Highland’ and ‘Carden Plain’.

The geology at the site is dominated by felsic igneous bedrock with shallow overburden. Bedrock at the site is known as granite and biotite gneiss of the Grenville Province.

According to Map 2418 of Ontario Geologic Survey, the site is located immediately north of the confluence of Precambrian rocks with more recent Ordovician formations. The main body of geologic formations consists of late to middle Cambrian clastic metasediments which are comprised of conglomerate, greywacke, arkose, calcareous sandstone and siltstone, shale and derived metamorphic rocks, while in the vicinity of the site late Precambrian granitic to syenitic rocks are also found.

Previous site specific investigations show the presence of granite gneiss rocks.

Overburden, where present, consists of silty sands, either surficial loose deposits or as dense glacial till above the bedrock. Silty clay is also present in areas where bedrock is relatively deeper in occurrence. Organic mucks are also common in marshy areas.

3 METHOD OF INVESTIGATION

The field work for this investigation was performed between the period of May 15 and June 12, 2013, and consisted of drilling and sampling of twelve boreholes. The boreholes were numbered from 9 to 20 to continue the numbering sequence of the boreholes drilled (i.e. Boreholes 1 to 8) at the nearby Severn River bridge site (report prepared under separate cover).

Boreholes 9, 10, 19 and 20, which were put down from the top of the existing road embankment by hollow-stem augering, were terminated upon encountering refusal on the augers, probably on the surface of the bedrock. The depths of these boreholes ranged from 9.2 to 11.0 m below the existing grades.

Boreholes 11, 12, 17 and 18 were also advanced from the top of the existing road embankment by hollow-stem augering; however, in these boreholes, upon encountering refusal on the augers at depths of 8.0 to 11.1 m below the ground surface, the boreholes were further advanced by diamond drilling methods and rock cores were obtained. The length of rock coring ranged from 2.3 to 4.1 m and the depths of the boreholes ranged from 10.5 to 14.3 m below the ground surface (i.e. top of embankment).

Boreholes 13 and 14 were drilled using a drill rig mounted on a barge, from the surface of water in the Channel. At the time of our investigation, the depth of the water in the Channel was 0.8 to 1.0 m at the borehole locations, and no overburden was found (i.e. rock was exposed at the Channel bottom). Consequently, these two boreholes were advanced 3.4 to 4.0 m by NQ and BQ size rock coring, below the bottom of the Channel.

Boreholes 15 and 16 were advanced using a portable drill rig by manual wash boring methods in the overburden. Upon encountering refusal to further advancing the boreholes by wash boring methods, the bedrock was cored by 3.7 – 4.1 m and BQ size rock cores were obtained. The depth of the boreholes were 6.5 and 6.3 m, respectively.

The drilling of Boreholes 9 through 12 and 17 through 20 was carried out using a track-mounted CME 55 drill rig owned and operated by Davis Drilling of Milton, Ontario. Boreholes 13 and 14, which were advanced from a barge, were drilled using a D25 Diedrich type drill rig, owned and operated by Walker Drilling of Utopia, Ontario. Boreholes 15 and 16 were put down using a portable Pionjar 120 drilling system (due to limited access), owned and operated by Sonic Soil Sampling of Concord, Ontario.

Samples in the overburden were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS-split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.3 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

In Boreholes 15 and 16, where manual drilling was effected in the overburden, a 31.8 kg hammer was used, instead of the standard 63.5 kg hammer. The recorded resistance values in these two boreholes were divided by two, to obtain approximate equivalent N-values in the overburden.

Groundwater conditions were observed during drilling and upon completion free-standing water levels were measured. In addition, a piezometer was installed in each of Boreholes 12 and 18 to enable us to monitor the groundwater table over a prolonged period of time, without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures. Boreholes 12 and 18 were not grouted, as measuring the groundwater levels at the time of construction may be useful. We recommend, however, a clause be included in the contract to decommission these piezometers at the time of construction.

The field work was carried out under the supervision and direction of technical personnel from our office. The borehole locations were established in the field by Coffey engineering staff, in relation to the existing site features. The borehole locations and the geodetic ground surface elevations at the borehole locations were subsequently determined by MRC's surveyors, who provided this information to us.

The soil and rock samples obtained from the boreholes were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory programme, consisting of natural moisture content and grain size analyses was performed on selected representative soil samples and point load tests on selected rock cores. Subsequently, some selected rock cores were shipped to Golder Associates' Laboratory, in Mississauga, Ontario, for unconfined compressive strength testing.

4 SUBSURFACE CONDITIONS

The subsurface conditions were explored at twelve borehole locations. The locations of the boreholes are shown on Drawing No. 1. Stratigraphic sections and profiles are presented in Drawing Nos. 2 to 5.

Boreholes 9, 11, 17 and 19 were advanced from the paved road surface and contacted a 160 to 230 mm thick asphalt layer underlain by pavement and embankment fill to depths ranging between 8.2 and 11.1 m. In Boreholes 9 and 19, a veneer of overburden was contacted underlain by bedrock, while in Boreholes 11 and 17 the embankment fill is underlain directly by bedrock.

Boreholes 10, 12, 18 and 20 were also put down from the top of the highway embankment, but from an unpaved portion. Below some topsoil, these boreholes encountered embankment fill to 7.6 to 9.4 m below the ground surface. In BH 10, the embankment fill was found to extend to the surface of the bedrock, while in the remaining three boreholes the bedrock is overlain by a 0.4 to 0.7 m thick native overburden.

In summary, the embankment fill was found to extend to depths of 7.6 to 11.1 m below the ground surface or to El. 179.6 to 176.0 m. In Boreholes 10, 11 and 17, it extends to the surface of the bedrock while in the remaining seven boreholes which were drilled from land, it is underlain by 0.1 to 0.7 m thick native overburden (i.e. excluding Boreholes 13 and 14 which were advanced from water's surface).

Boreholes 15 and 16 were put down from a lower elevation and these boreholes contacted below a veneer of topsoil an approximately 2 m thick fill layer (to elevations 177.6 m and 177.3 m, respectively), underlain by a 0.1 m to 0.8 m thick native sand deposit, overlaying the bedrock.

No fill or overburden was contacted in Boreholes 13 and 14, put down from the Channel, from a barge. Here, the bedrock was exposed at the Channel base.

Bedrock was contacted or inferred in all the boreholes. Bedrock was contacted in Boreholes 10, 11 and 17 directly below the embankment fill. In Boreholes 9, 12, 15, 16, 18, 19 and 20, it was encountered below a relatively thin layer of basal overburden, while in Boreholes 13 and 14 it was contacted immediately below the Channel bottom. The surface of the bedrock at the borehole locations was encountered between El. 179.6 m (BH 17) to 174.9 m (BH 13).

The bedrock was found to consist of greyish/pinkish granite gneiss of generally sound quality.

Details of the subsurface conditions encountered in the boreholes are given on the individual Record of Borehole Sheets in Appendix A. Detailed laboratory test results (soil and rock) are enclosed in Appendices B and D.

The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the soil and groundwater conditions may vary in between and beyond borehole locations.

4.1 Asphalt

Boreholes 9, 11, 17 and 19 were put down from the surface of the paved highway and consequently contacted asphaltic concrete, ranging in thickness from 160 mm (BH 9) to 230 mm (BH 11). In Boreholes 17 and 19, the thickness of the asphalt was found to be 180 and 190 mm, respectively.

4.2 Topsoil

Boreholes 10, 12, 15, 16, 18 and 20, which were drilled from the existing highway embankment, off the roadway, contacted a 0.05 to 0.15 m thick veneer of topsoil at the ground surface. In addition, in BH 19 an approximately 0.1 m thick topsoil layer was contacted immediately below the embankment fill at a depth of 9.1 m immediately overlying the bedrock.

4.3 Pavement and Embankment Fill

4.3.1 Pavement Fill

As mentioned before, Boreholes 9, 11, 17 and 19 were advanced from the top of the paved road surface and these boreholes contacted, below the asphaltic concrete, granular pavement fill which extended to 0.9 m (Boreholes 9 and 19) and 1.3 m (BH 11).

The grain size distribution of three samples from the granular pavement fill is given in Figure B-1 (Appendix B). The following grain size distribution is indicated:

Gravel:	24-40%
Sand:	46-62%
Silt:	14-17%

N-values recorded in the pavement fill range from 25 to 48 blows/0.3m, which indicate a compact to dense relative density of the compacted granular pavement fill.

4.3.2 Embankment Fill

Underlying the pavement fill in Boreholes 9, 11, 17 and 19 and the topsoil in Boreholes 10, 12, 18 and 20, embankment fill was found to extend to depths ranging from 8.2 to 11.1 m below the ground surface.

In Boreholes 10, 11 and 17, the embankment fill was found to extend to the surface of bedrock/inferred bedrock at depths of 8.2 to 11.1 m below the ground surface or at El. 179.6 to 176.0 m, while in the remaining boreholes the embankment fill was found to extend to the surface of the overburden at depths of 7.6 to 10.6 m (El. 178.9 – 176.2 m).

Boreholes 15 and 16 were put down near the toe of the embankment, using portable equipment. These boreholes encountered fill extending to 2.0 m (El. 177.6 m) and 2.1 m (El. 177.3 m), respectively.

The embankment fill generally consists of a heterogeneous mixture of silty sand to sandy silt with trace to some clay and gravel size particles. From a visual examination of the split-spoon samples obtained from the material and the results of the grain-size analyses on the samples, it can be surmised that the source of the embankment fill is the local glacial till soils. In general, the fill appeared to be relatively clean (i.e. devoid of organic and other deleterious material); the presence of some clay in the fill was noted.

The grain-size distribution of fourteen samples from the embankment fill is given in an envelope form in Figure. B-2 in Appendix B, showing the following grain-size distribution:

Gravel:	2-8%
Sand:	45-58%
Silt:	21-36%
Clay:	15-18%

There are in the embankment fill somewhat sandier zones. Figure B-3 in Appendix B presents the grain size distribution of two such samples from Boreholes 9 and 11. The following is the grain-size distribution indicated:

Gravel:	2-4%
Sand:	61-66%
Silt:	16-24%
Clay:	11-16%

In BH 10, the lower portion of the fill below 6.0 m (El. 179.7 m) was found to be even more sandy. The grain-size distribution of a sample from between 6 and 7 m depth from this borehole is presented in Figure B-4 (Appendix B), which indicates 4% gravel, 83% sand, 9% silt and 4% clay size particles. As well in BH 20, the lower portion of the fill below about 7.6 m (El. 178.9 m) was found to be a basically gravelly fine sand with some silt lenses.

As was also mentioned before, the presence of occasional siltier and more clayey zones was also noted. Figure B-5 in Appendix B presents the grain-size distribution of a sample from BH 20 from such a zone, which shows 3% gravel, 36% sand, 41% silt and 20% clay size particles.

The fill can be classified as a basically granular (i.e. non-cohesive) soil type, with occasional cohesive zones.

Standard Penetration Tests performed in the embankment fill yielded N-values which range from 3 to in excess of 60 blows/0.3 m, but typically between 12 and 24 blows/0.3 m, indicating a very loose to very dense relative density but generally compact. There are occasional weak (i.e. loose to very loose) zones as evidenced by N-values of between 3 and 8 blows/0.3 m in Boreholes 10, 17, 18, 19 and 20 and particularly in Boreholes 11 and 12 as rather thick zones.

From these results, it can be concluded that the embankment fill generally received adequate compaction when it was first constructed in the early 1990's but there are sporadic zones which did not receive compaction, especially in BH 11 in the upper 4 m and in BH 12 below the top 2 m.

Boreholes 15 and 16 were put down from a lower level near the toe of the highway embankment. In these boreholes the adjusted and approximately equivalent resistance values in the fill were between 2 and 9 blows/0.3 m, indicating a very loose to loose relative density. The grain-size distribution of two samples encountered in these two boreholes is given in Figure B-6 (Appendix B). The results are as follows:

Gravel:	2-4%
Sand:	49%
Silt:	29-32%
Clay:	17-18%

These are considered similar to the embankment fill material grain-size distributions encountered in the other boreholes.

4.4 Native Overburden

Thin basal native overburden deposits were encountered in Boreholes 9, 12, 15, 16, 18, 19 and 20, underlying the embankment fill.

The thickness of the native overburden at the borehole locations was found to range from 0.1 m (Boreholes 16 and 19) to 0.8 m (Borehole 15).

In BH 19, a 0.1 m thick veneer of topsoil was contacted, underlying the embankment fill. In the remaining boreholes, the natural overburden was found to consist of granular (non-cohesive) soils, ranging from silty sand to sand.

A modified Standard Penetration Test in BH 15, using portable equipment, yielded an equivalent N-value of 19 blows/0.3 m, which indicates a compact condition.

In Boreholes 9 and 18, N-values in excess of 100 blows/0.3 m were recorded and based on this, the relative density of the soil is described as very dense. In the remaining boreholes, the recorded values may not be reliable (i.e. presence of rock pieces immediately above the bedrock surface). But there is some evidence that the overburden soils are generally dense to very dense. A word of caution is however in order in this respect. These resistances reflect the values beneath considerable embankment fill. As such, the soil is likely to have densified under the weight of the fill and may not reflect the denseness condition of the overburden soils beyond the embankment fill influence zone.

4.5 Bedrock

Bedrock was encountered/inferred at all borehole locations.

In Boreholes 9, 10, 19 and 20, the presence of bedrock was inferred from refusal to augering, while in the remaining eight boreholes upon encountering refusal, the presence of bedrock was proven by diamond drilling and obtaining rock cores to depths ranging from 2.3 and 3.2 m in Boreholes 17 and 11, respectively, to between 3.4 and 4.1 m in Boreholes 12, 13, 14, 15, 16 and 18.

Generally, NQ size cores were obtained. However, Boreholes 13 and 14 were advanced from a barge using smaller equipment and thus both NQ and BQ size coring was effected. Similarly, Boreholes 15 and 16 were advanced using portable equipment and as a result, in these boreholes BQ size cores were retrieved.

From the cores, the bedrock was identified as granite gneiss. Its colour was found to range from light to medium and occasionally dark grey with a pinkish tone and/or pink insets.

The following table summarizes the bedrock surface elevations and the condition of the bedrock, as revealed by the rock cores obtained from the boreholes.

Photographs of the rock cores are included in Appendix D.

Table 4.5.1: Bedrock Surface Elevations and Rock Details

Borehole Number	Top of Bedrock Elevation (m)	Coring Size	Total Core Length (m)	T.C.R. (%)**	R.Q.D. (%)***
9	175.8*	N/A	N/A	N/A	N/A
10	176.2*	N/A	N/A	N/A	N/A
11	176.0	NQ	3.2	97-100	90-100
12	176.3	NQ	4.1	95-100	90-98
13	174.9	BQ & NQ	4.0	100	35-100
14	175.1	BQ	3.4	100	82-100
15	176.8	BQ	3.7	96-100	23-100
16	177.2	BQ	4.1	91-100	91-100
17	179.6	NQ	2.3	83-100	83-100
18	178.5	NQ	4.1	98-100	75-100
19	178.6*	N/A	N/A	N/A	N/A
20	177.2*	N/A	N/A	N/A	N/A

* inferred

** T.C.R. = Total Core Recovery

*** R.Q.D. = Rock Quality Designation

From the table presented, it can be seen that the surface of the bedrock was contacted or inferred between elevations 179.6 m (at BH 17) and 174.9 m (at BH 13). This represents an elevation difference of 4.7 m over a horizontal distance of about 100 m. It is possible that the surface of the rock underlying the Channel may have been lowered by blasting when the Channel itself was first built. If this hypothesis is true, this operation would likely to have modified the rock surface elevation at Boreholes 13 and 14 and indeed here the recorded surface of the bedrock is the lower (i.e. 174.9 m and 175.1 m, respectively). It may also possibly have affect the rock surface elevation at Boreholes 15 and 16 and again the rock surface elevation at these locations is somewhat lower than the elevations at Boreholes 17 and 18 which are on the same side of the Channel (i.e. El. 177.2 – 176.8 m vs 179.6 – 178.5 m).

On the south side of the Channel (i.e. towards Barrie) in Boreholes 9 through 12, the surface of the rock appears to be relatively level ranging from El. 176.3 m to 175.8 m, while to the north (i.e. towards Parry Sound) it appears to be more undulating at Boreholes 17 through 20, ranging from El. 179.6 m to 177.3 m; in particular BH 17 seems to represent a peak at El. 179.6 m.

In general, the top 0.1 to 0.4 m of the bedrock, as determined by core results, was found to be highly fractured. The depth of fracturing in BH 13 was found to be 0.6 m but this is likely to be the result of blasting operations when the Channel was built.

The percentage of core recovery in the boreholes ranged from 83 – 100%, but generally ranged from 93 to 100%. The RQD values (i.e. Rock Quality Designation) were recorded between 23 and 100% but generally were between 75 and 100%. The lowest RQD values of 23 and 35% were obtained within the upper zones of the core samples immediately below or near the Channel bottom at Boreholes 15 and 13, respectively, probably reflecting the effects of previous blasting operations. If these latter two values are discarded, the recorded RQD values are indicative of a good to excellent rock quality (see Appendix A), at the cored locations.

To determine the compressive strength and hardness of the rock, a total of five samples were subjected to unconfined compressive testing. The unconfined compressive strength (UCS) of the tested samples ranged from 49.7 to 109.3 MPa with an average of 77.9 MPa. The results of these unconfined compressive tests are given in Appendix D.

Point Load Index tests were performed in our laboratory on 20 rock core samples. The test results are presented in Appendix D. Is(50) values ranging from 2.4 to 8.6 MPa and UCS values (using typical K=24) of 57.7 to 206.3 MPa were recorded.

Based on these results, the rock encountered at the site is classified as typically strong to very strong.

4.6 Groundwater Conditions

Groundwater conditions in the open boreholes were observed while drilling and upon completion of each borehole. In addition, piezometers were installed in Boreholes 12 and 18 to enable us to monitor groundwater levels over a prolonged period of time, without interference from surface water. As Boreholes 13 and 14 were advanced from a barge in the Channel, no groundwater observations could be made in these two boreholes. At these borehole locations at the time of our investigation the depth of water in the Channel was 1.0 m and 0.8 m, respectively and the water surface elevation in Channel was at 175.9 m.

On the south side of the Channel (i.e. towards Barrie) in Boreholes 9, 10 and 11, the groundwater was measured, upon completion of each borehole (i.e. not necessarily stabilized) and from soil moisture and the wetness condition of the sampler, to be at between El. 178 and 176 m. In the piezometer installed in BH 12, the groundwater level was measured twenty days after the installation at El. 178.8 m. From these it is concluded that at the time of our investigation the groundwater table at the south side of the site was between El. 179 – 176 m.

On the north side of the Channel, in Boreholes 15, 16, 17, 18, 19 and 20, the water level during and upon completion (i.e. not necessarily stabilized) was at El. 179 – 178 m. In BH 18, the sampler was found to be wet at El. 179 m but subsequently in the piezometer installed, it was recorded eighteen days after drilling at El. 181.7 m. All these observations indicate that at the time of our investigation the groundwater level on the north side was between El. 182 – 178 m.

It should be pointed out that the groundwater table at the site can be expected to fluctuate seasonally and in response to major weather events.

It should also be pointed out that the groundwater level at the site would also be influenced by the water level in the Channel, which is regulated. We took elevations of the water in the Channel once a day during the period of June 12 – 13 and 14, 2013, during which time it was measured to be between 176.0 and 175.9 m. However, the water level in the Channel would fluctuate as it is controlled (regulated) by the Trent Severn waterway system authority.

For and on behalf of Coffey Geotechnics Inc.

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Zuhtu Ozden, P.Eng.
Senior Principal

Drawings

KEY MAP

Severn River Boat Channel Bridges on Highway 400

METRIC

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

NOTES:

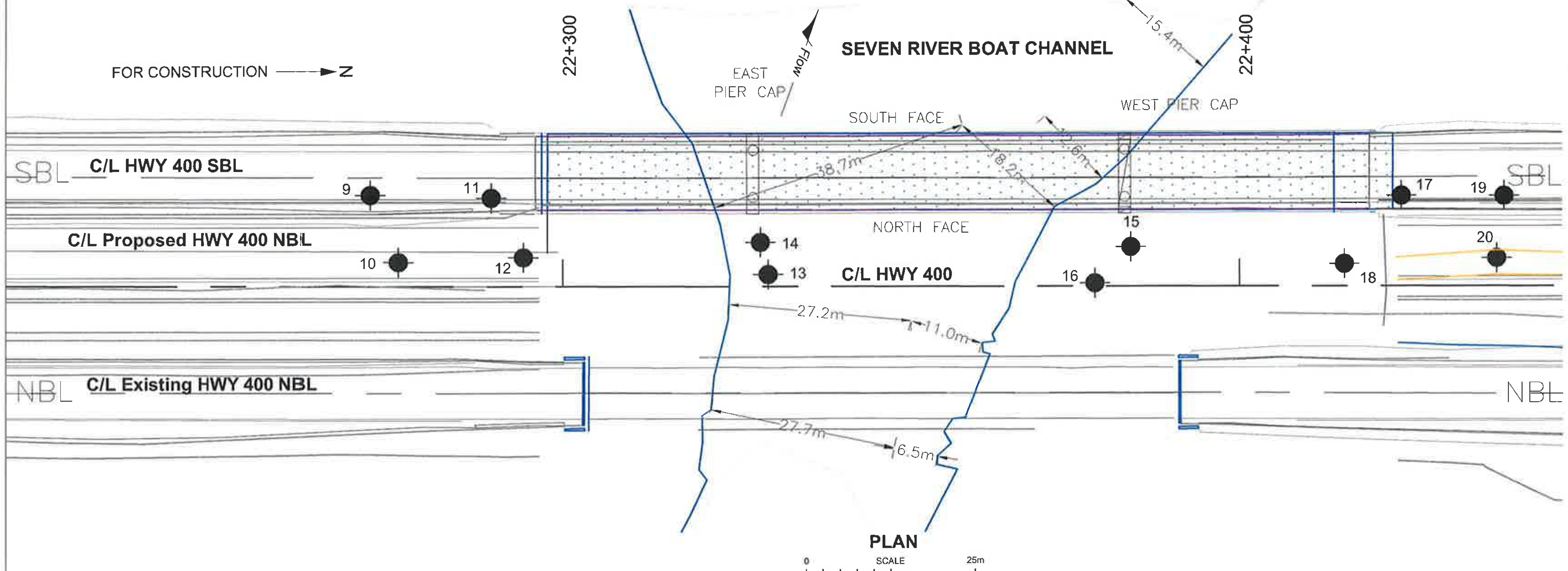
FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -

W.P.: 2376-09-00

CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BOAT CHANNEL BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA

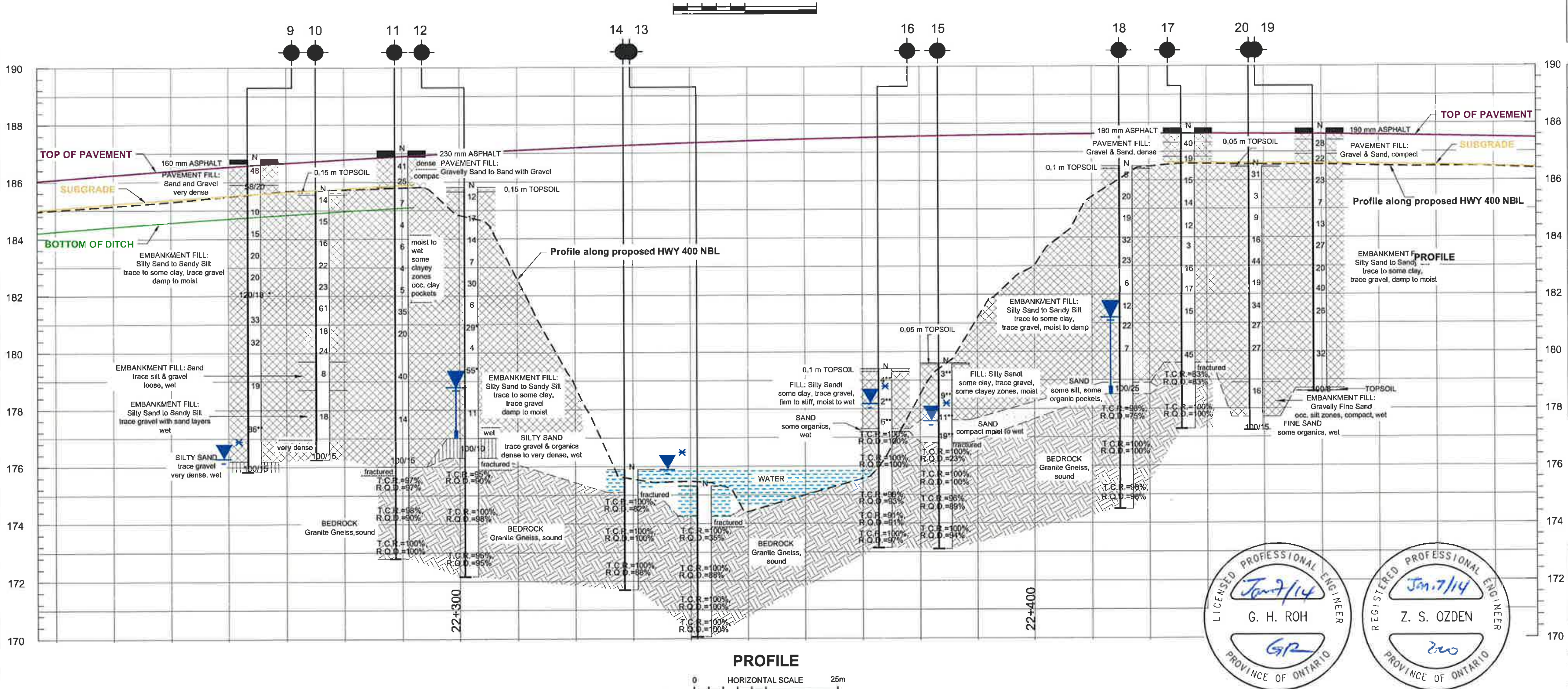
SHEET



PLAN

0 SCALE 25m

KEY PLAN
N.T.S.



PROFILE

0 HORIZONTAL SCALE 25m

LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEVATION	STATION	OFFSET
BH9	186.8	22+272	13.8m Lt C/L
BH10	185.7	22+276	3.6m Lt C/L
BH11	187.1	22+289	13.2m Lt C/L
BH12	185.8	22+294	4.3m Lt C/L
BH13	175.9	22+330	1.8m Lt C/L
BH14	175.9	22+329	6.6m Lt C/L
BH15	179.6	22+384	5.9m Lt C/L
BH16	179.4	22+379	0.5m Lt C/L
BH17	187.8	22+424	13.4m Lt C/L
BH18	186.5	22+415	3.3m Lt C/L
BH19	187.8	22+439	13.4m Lt C/L
BH20	186.5	22+438	4.2 m Lt C/L

-NOTE-

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

NOTE: This drawing is for subsurface information only.. Surface details and features are for conceptual illustration.

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Jan. 7/14
Z. S. OZDEN
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REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-566

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE January , 2014	SITE 42-87/1&2
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 1

METRIC

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

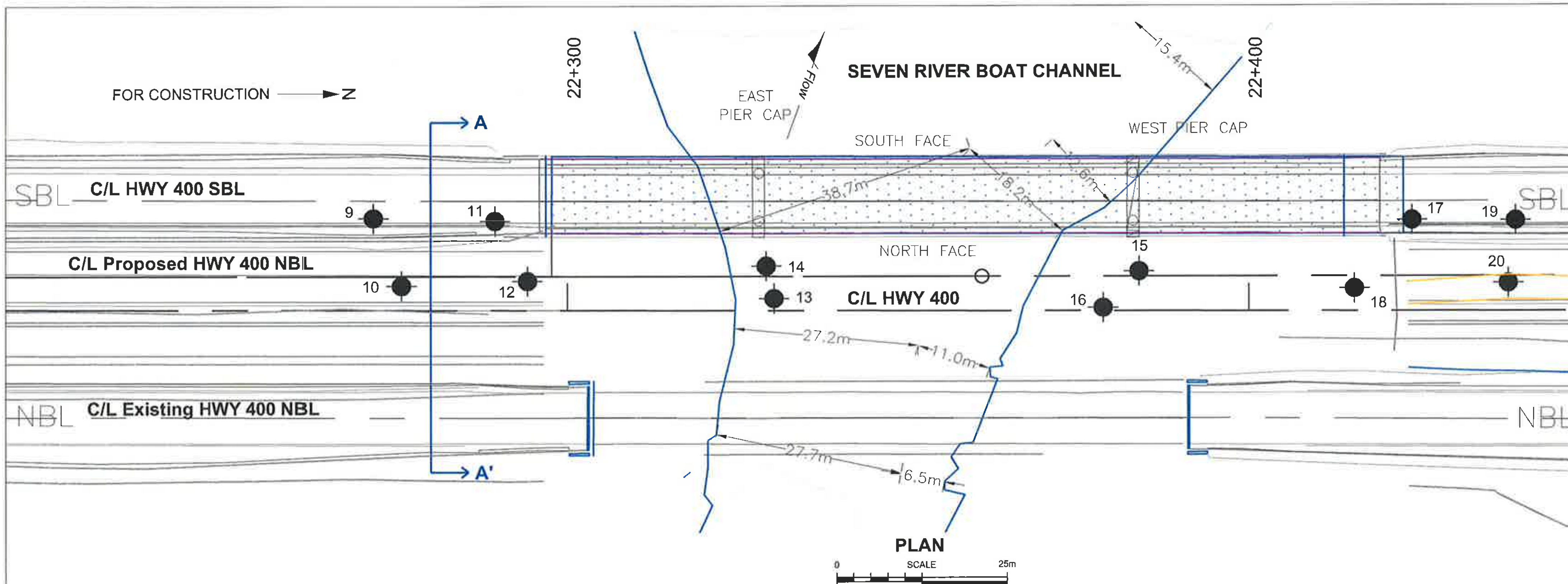
CONT No. -
W.P.: 2376-09-00

CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BOAT CHANNEL BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA

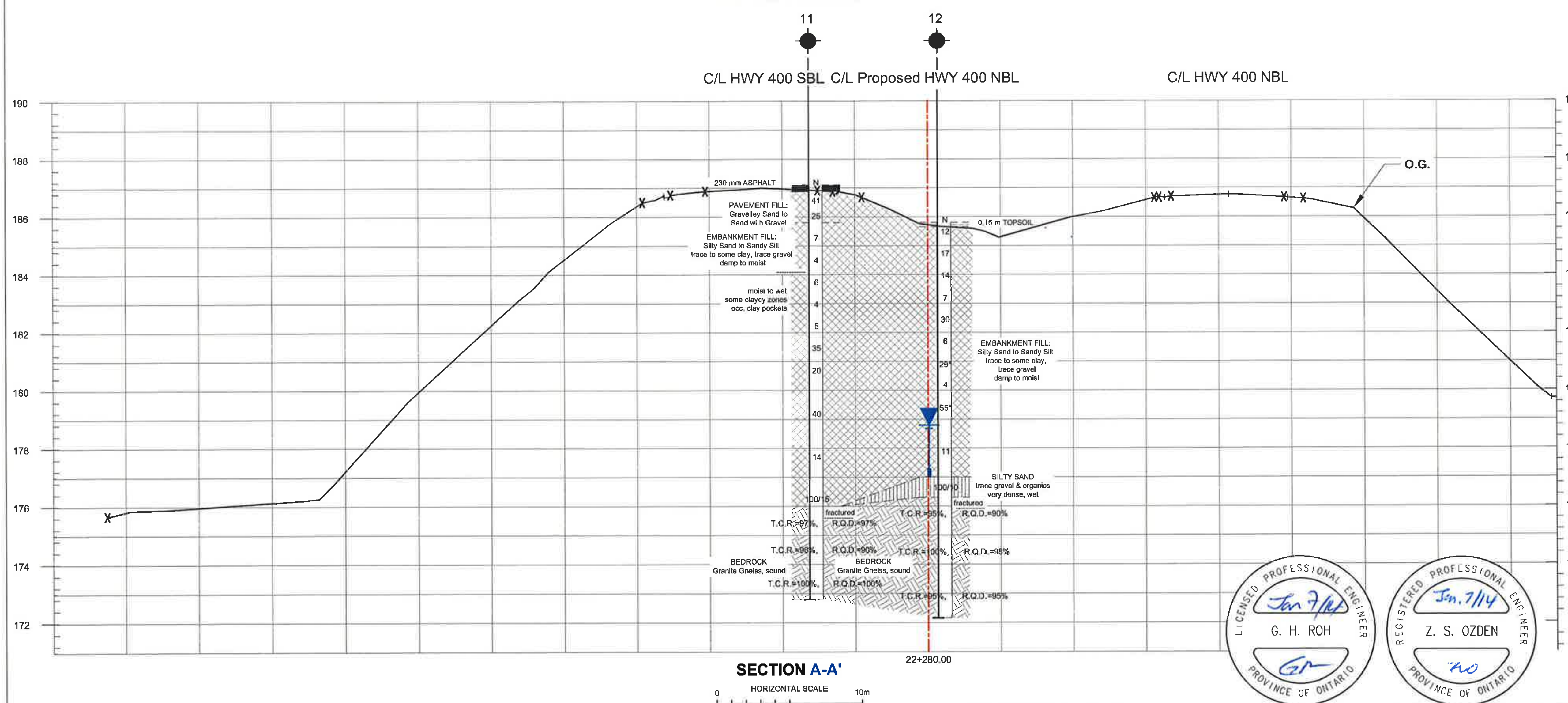
SHEET

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.



PLAN



SECTION A-A'

KEY PLAN
N.T.S.

LEGEND

- Borehole
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer
- A A' — Section

No.	ELEVATION	STATION	OFFSET
BH11	187.1	22+289	13.2m Lt C/L
BH12	185.8	22+294	4.3m Lt C/L

-NOTE-
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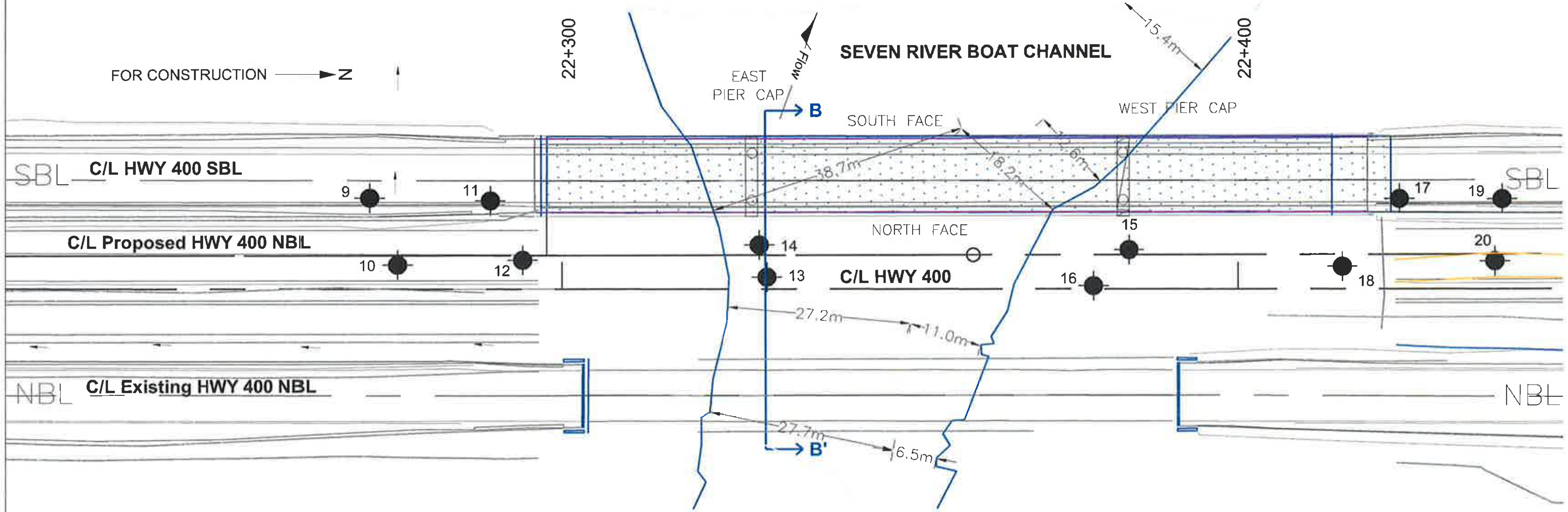
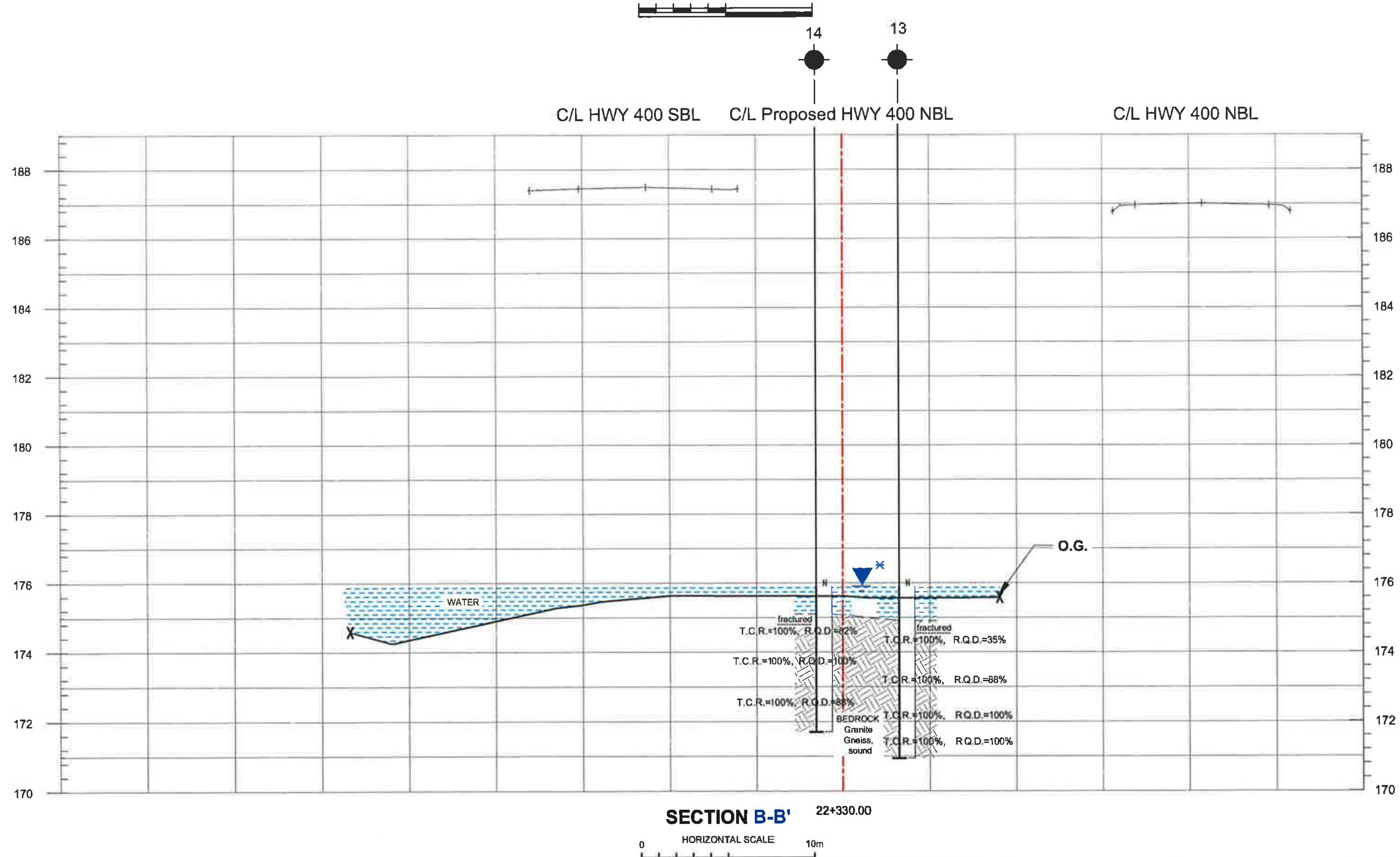
REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-566

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE January , 2014	SITE 42-87/1&2
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 2



PLAN



SECTION B-B' 22+330.00

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -
W.P.: 2376-09-00

CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BOAT CHANNEL BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

KEY PLAN
N.T.S.

LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer
- Section

No.	ELEVATION	STATION	OFFSET
BH13	175.9	22+330	1.8m Lt C/L
BH14	175.9	22+329	6.6m Lt C/L

-NOTE-
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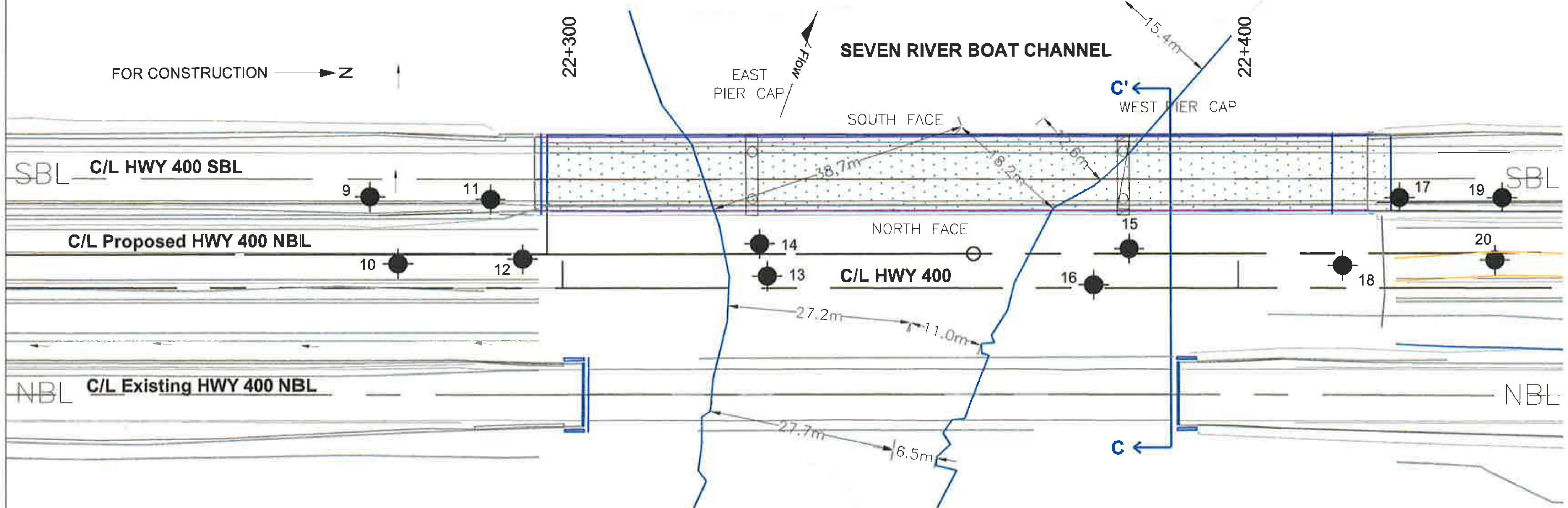
REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-566

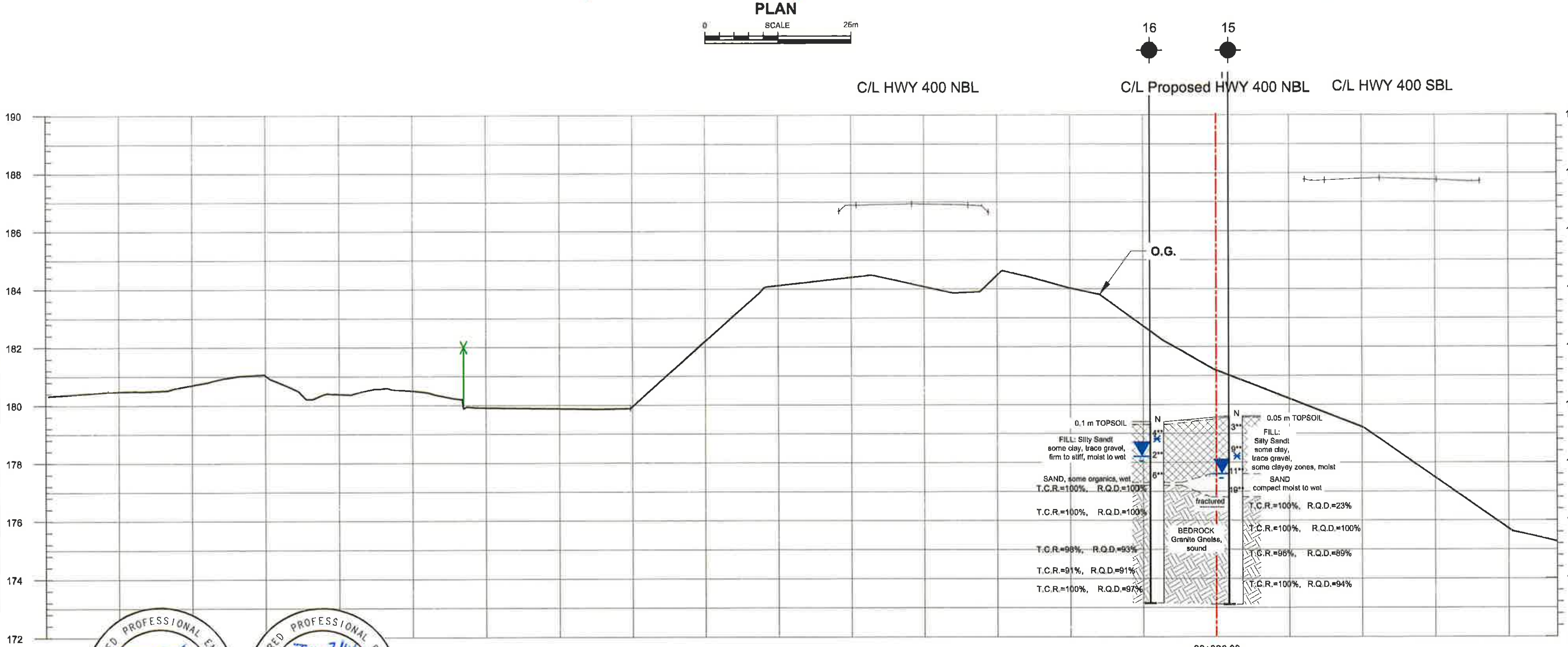
TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE January, 2014	SITE 42-87/1&2
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 3

FOR CONSTRUCTION → N



PLAN

SCALE 0 25m



PROFESSIONAL ENGINEER
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SECTION C-C'

HORIZONTAL SCALE 0 10m

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -

W.P.: 2376-09-00

CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BOAT CHANNEL BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA

SHEET

KEY PLAN
N.T.S.

LEGEND

- Borehole
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer
- C C' Section

No.	ELEVATION	STATION	OFFSET
BH15	179.6	22+384	5.9m Lt C/L
BH16	179.4	22+379	0.5m Lt C/L

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

NOTE: This drawing is for subsurface information only.. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-566

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE January , 2014	SITE 42-87/1&2
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 4

METRIC

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

NOTES:

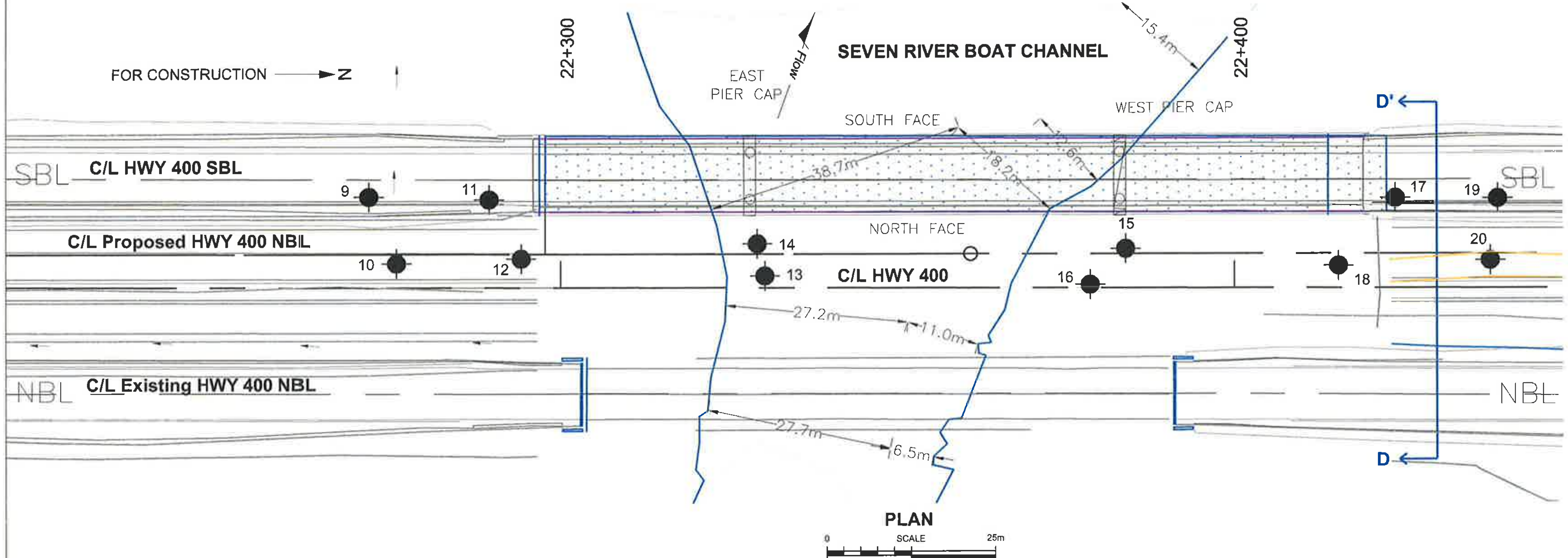
FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No. -

W.P.: 2376-09-00

CONTRACT A, HIGHWAY 400, PORT SEVERN RIVER BOAT CHANNEL BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA

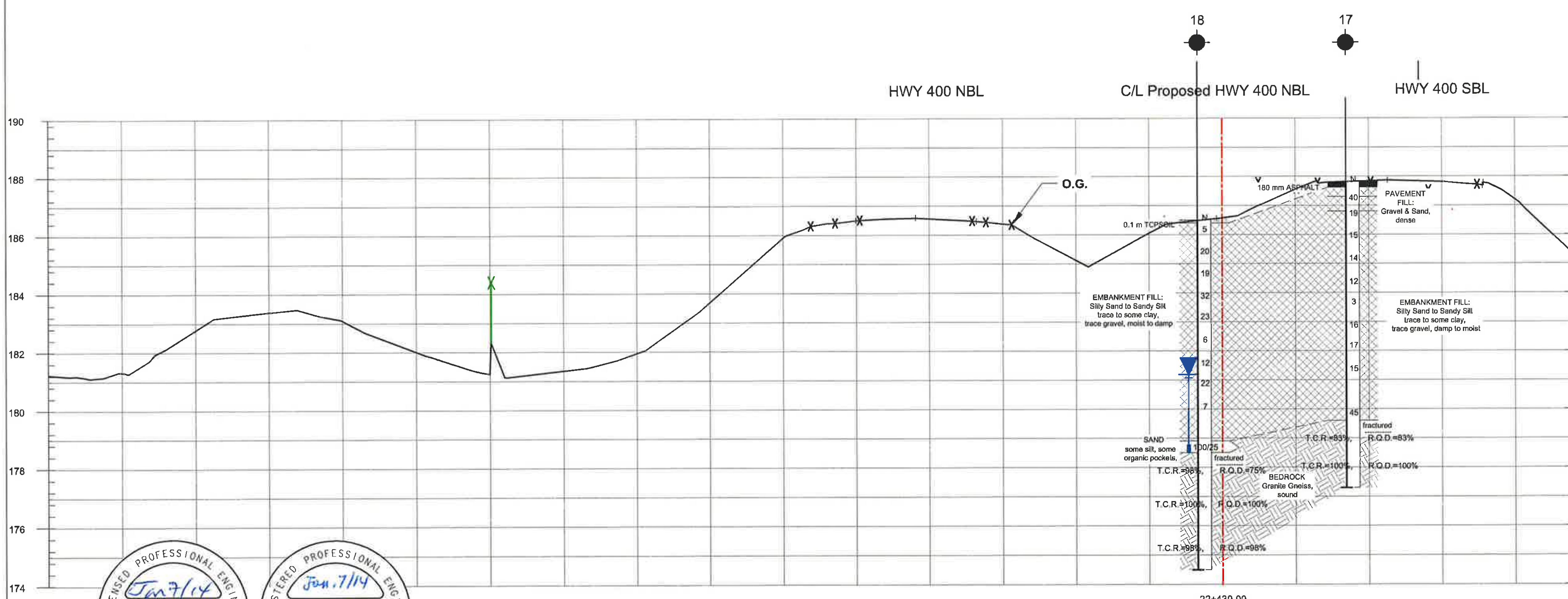
SHEET



PLAN

SCALE 0 25m

KEY PLAN
N.T.S.



SECTION D-D'

HORIZONTAL SCALE 0 10m

LICENSED PROFESSIONAL ENGINEER Jan 7/14 G. H. ROH GR PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER Jan. 7/14 Z. S. OZDEN ZO PROVINCE OF ONTARIO

LEGEND

- Borehole
- N — Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer
- D D' Section

No.	ELEVATION	STATION	OFFSET
BH17	187.8	22+424	13.4m Lt C/L
BH18	186.5	22+415	3.3m Lt C/L

-NOTE-

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

NOTE: This drawing is for subsurface information only.. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No -31D-566

TRANETOB20462AA			DIST
SUBM'D	CHECKED	DATE January , 2014	SITE 42-87/1&2
DRAWN SSH	CHECKED GR	APPROVED ZO	DWG 5

Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE No 9

METRIC

GWP	W.P. 2376-09-00	LOCATION	22+272, 13.6 m Lt C/L (N 4962191.991, E 287256.512)
DIST	5 HWY 400	BOREHOLE TYPE	Hollow Stem Auger
DATUM	Geodetic	DATE	22/05/2013 22/05/2013

ORIGINATED BY LG
COMPILED BY SSH
CHECKED BY ZO

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.8	GROUND SURFACE						
186.6 / 0.2	160 mm **ASPHALT**						
	PAVEMENT FILL: Sand & Gravel brown, very dense	1	SS	48			40 46 (14)
185.9 / 0.9		2	SS	58/20		186	
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, compact, damp to moist	3	SS	10		185	4 57 21 18
		4	SS	15		184	
		5	SS	20		183	7 58 29 16
		6	SS	20			
	dense	7	SS	120/18 *		182	*spon bouncing probable boulder N-value not reliable
		8	SS	33		181	
	compact	9	SS	32		180	4 61 24 11
		10	SS	19		179	
						178	
		11	SS	86**		177	**oversized gravel N-value not reliable
176.2 / 10.6	**SILTY SAND** trace gravel grey, very dense, wet				▼***		
175.8 / 11.0		12	SS	100/15		176	wet spoon

End of Borehole
Auger refusal @ 11.0 m
Probable Bedrock
***Water level @ 10.5 m (El. 176.3 m) upon completion (not stabilized)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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

Ministry of Transportation
Ontario

Foundation Design

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 10

METRIC

GWP	W.P. 2376-09-00	LOCATION	22+276, 13.6 m Lt C/L (N 4962202.741, E 287255.590)	ORIGINATED BY	LG
DIST	5 HWY 400	BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	SSH
DATUM	Geodetic	DATE	27/05/2013 27/05/2013	CHECKED BY	ZO

SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
185.7	GROUND SURFACE							
185.6 0.2	0.15 m **TOPSOIL**	1	SS	14				
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, compact	2	SS	15	3	58	22	17
		3	SS	16				
		4	SS	22				
		5	SS	23	5	49	31	15
	very dense	6	SS	61				
		7	SS	18				
		8	SS	24				
179.7 6.0	**EMBANKMENT FILL:** Sand trace silt & gravel brown, loose, wet	9	SS	8	4	83	9	4
178.7 7.0	**EMBANKMENT FILL:** Silty Sand to Sandy Silt trace gravel with sand layers grey, compact, wet	10	SS	18				
	very dense	11	SS	100/15				
176.3 9.4	End of Borehole Auger refusal @ 9.5 m Probable Bedrock *wet cave at 7.9 m (El. 177.8 m) upon completion (not stabilized)							

Elevation scale: 185, 184, 183, 182, 181, 180, 179, 178, 177

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

15-⊕-5 (20, 10) (%) STRAIN AT FAILURE

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 11

METRIC

GWP W.P. 2376-09-00	LOCATION 22+289, 13.2 m Lt C/L (N 4962197.730, E 287239.700)		ORIGINATED BY LG
DIST 5 HWY 400	BOREHOLE TYPE Hollow Stem Auger & NQ Coring		COMPILED BY SSH
DATUM Geodetic	DATE 16/05/2013	16/05/2013	CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.1	GROUND SURFACE					187	
186.9 / 0.2	230 mm **ASPHALT**						
	PAVEMENT FILL: Gravelly Sand to Sand with Gravel, brown; dense	1	SS	41			32 51 (17)
185.8 / 1.3	compact	2	SS	25		186	
	EMBANKMENT FILL: Silty Sand to Sandy Silt, trace to some clay, trace gravel, greyish brown, moist to damp	3	SS	7		185	
		4	SS	4		184	
	moist to wet, some clayey zones, occ. clay pockets	5	SS	6			
		6	SS	4		183	
	loose to very loose	7	SS	5		182	4 45 33 18
	compact to very dense	8	SS	35		181	Augers grinding @ 5.5 m, possible cobble, N-value may not be reliable
		9	SS	20		180	
		10	SS	40		179	2 66 16 16
		11	SS	14		178, 177	
176.0 / 11.1	grey	12	SS	100/15		176	Auger refusal and start of NQ coring at 11.1 m
	BEDROCK Granite Gneiss, greyish / pink, sound; fractured	13	RC	T.C.R.=97% R.Q.D.=97%		175	
	greyish	14	RC	T.C.R.=98% R.Q.D.=90%		174	
172.8 / 14.3		15	RC	T.C.R.=100% R.Q.D.=100%		173	
	End of Borehole Borehole open and dry upon completion of overburden drilling (not stabilized)						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity   15-⊕-5 (20 / 10) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 12

METRIC

GWP W.P. 2376-09-00	LOCATION 22+294, 4.4 m Lt C/L (N 4962207.608, E 287237.829)		ORIGINATED BY LG
DIST 5	HWY 400	BOREHOLE TYPE Hollow Stem Auger & NQ Coring	COMPILED BY SSH
DATUM Geodetic	DATE 27/05/2013 27/05/2013		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.8	GROUND SURFACE						
185.7 0.2	0.15 m **TOPSOIL**	1	SS	12			
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, damp to moist	2	SS	17	185		
	compact	3	SS	14	184		
	loose	4	SS	7	183		5 51 27 17
	compact	5	SS	30	182		
	loose	6	SS	6			
	compact	7	SS	29*	181		
	very loose to compact	8	SS	4	180		
		9	SS	55*	179		* oversize gravel in sampler, N-value may not be reliable
		10	SS	11	178		
177.0 8.8	wet				177		
	SILTY SAND trace gravel & organics grey, dense to very dense, wet	11	SS	100/10			
176.3 9.5	fractured				176		Auger refusal and start of NQ coring at 9.5 m
	BEDROCK Granite Gneiss greyish / pink, sound	12	RC	T.C.R.=95% R.Q.D.=90%	175		
		13	RC	T.C.R.=100% R.Q.D.=98%	174		
		14	RC	T.C.R.=95% R.Q.D.=95%	173		
172.2 13.6	End of Borehole Piezometer installed to 8.6 m. Water level in piezometer @ 2.5 m upon installation (not reliable due to water used for coring). Water level in piezometer @ 7.0 m (El. 178.8 m) on June 17, 2013						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L — WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 13

METRIC

GWP: W.P. 2376-09-00
LOCATION: 22+330, 1.8 m Lt C/L (N 4962220.901, E 287204.168)
ORIGINATED BY: LG

DIST: 5 HWY: 400
BOREHOLE TYPE: NQ and BQ Coring from barge
COMPILED BY: SSH

DATUM: Geodetic
DATE: 11/06/2013 11/06/2013
CHECKED BY: ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
175.9	WATER SURFACE					
0.0	**WATER**					
174.9 / 1.0	Channel Bottom				175	
	fractured; **BEDROCK** Granite Gneiss greyish / pink sound below 0.6 m	1	RC	T.C.R.=100% R.Q.D.=35%	174	
		2	RC	T.C.R.=100% R.Q.D.=88%	173	
		3	RC	T.C.R.=100% R.Q.D.=100%	172	
		4	RC	T.C.R.=100% R.Q.D.=100%		change to BQ coring @ 3.0 m below channel bottom
170.9 / 5.0	End of Borehole NQ coring to 3.0 m below channel bottom then BQ coring to the end of the borehole				171	

Column headings (no data plotted): STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; GR SA SI CL

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 14

METRIC

GWP	W.P. 2376-09-00	LOCATION	22+329, 6.6 m Lt C/L (N 4962215.984, E 287203.809)	ORIGINATED BY	LG
DIST 5	HWY 400	BOREHOLE TYPE	BQ Coring from barge	COMPILED BY	SSH
DATUM	Geodetic	DATE	12/06/2013 12/06/2013	CHECKED BY	ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
175.9	WATER SURFACE										
0.0	**WATER** Channel Bottom										
175.1 0.8	fractured		1	RC	T.C.R.=100% R.Q.D.=82%		175				
	BEDROCK Granite Gneiss greyish / pink sound		2	RC	T.C.R.=100% R.Q.D.=100%		174				
			3	RC	T.C.R.=100% R.Q.D.=88%		173				
171.7 4.2							172				
	End of Borehole Borehole was open										

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

20 / 15 –O– 5 / 10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 15

METRIC

GWP W.P. 2376-09-00	LOCATION 22+384, 5.9 m Lt C/L (N 4962233.161, E 287151.839)		ORIGINATED BY LG
DIST 5 HWY 400	BOREHOLE TYPE Manual Drilling by Washboring & BQ Coring		COMPILED BY SSH
DATUM Geodetic	DATE 03/06/2013	04/06/2013	CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.6	GROUND SURFACE						
179.6 / 0.1	0.05 m **TOPSOIL**						
	FILL: Silty Sand some clay, trace gravel, some clayey zones brown, very loose to loose, moist	1	SS	3**		179	
		2	SS	9**			4 49 29 18
177.6 / 2.0		3	SS	11**	▼*	178	
176.8 / 2.8	**SAND** with silty zones, trace gravel brown to 2.4 m, grey below, compact, moist to wet	4	SS	19**		177	
	fractured	5	RC	T.C.R.=100% R.Q.D.=23%			Auger refusal and start of BQ coring at 2.8 m
	BEDROCK Granite Gneiss greyish / pink sound	6	RC	T.C.R.=100% R.Q.D.=100%		176	
		7	RC	T.C.R.=96% R.Q.D.=89%		175	
173.1 / 6.5		8	RC	T.C.R.=100% R.Q.D.=94%		174	

End of Borehole
*water level @ 2.0 m (El. 177.6 m) upon completion (not stabilized)
**31.8 kg hammer used instead of standard 63.6 kg hammer, due to manual drilling; recorded resistance values were divided by 2 to obtain approximate equivalent N-values

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+ 3, × 3: Numbers refer to Sensitivity   15 ⊕ 5 (20, 10) (%) STRAIN AT FAILURE

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 16

METRIC

GWP W.P. 2376-09-00	LOCATION 22+379, 0.5 m Lt C/L (N 4962236.712, E 287158.483)		ORIGINATED BY LG
DIST 5 HWY 400	BOREHOLE TYPE Manual Drilling by Washboring & BQ Coring		COMPILED BY SSH
DATUM Geodetic	DATE 03/06/2013	05/06/2013	CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
179.4	GROUND SURFACE						
179.3 / 0.1	0.1 m **TOPSOIL**						
	FILL: Silty Sand some clay, trace gravel brown, very loose to compact firm to stiff, moist to wet	1	SS	4**		179	
		2	SS	2**	▼*	178	2 49 32 17
		3	SS	6**			
177.3 / 2.1	**SAND** some organics, brown, wet	4	RC	T.C.R.=100% R.Q.D.=100%		177	Auger refusal and start of BQ coring at 2.2 m
177.2 / 2.2	**BEDROCK** Granite Gneiss greyish / pink sound	5	RC	T.C.R.=100% R.Q.D.=100%		176	
		6	RC	T.C.R.=98% R.Q.D.=93%		175	
		7	RC	T.C.R.=91% R.Q.D.=91%		174	
		8	RC	T.C.R.=100% R.Q.D.=97%			
173.2 / 6.3	End of Borehole						

End of Borehole
* Water level before coring @ 1.2 m (El. 178.2 m) (not stabilized)
**31.8 kg hammer used instead of standard 63.6 kg hammer, due to manual drilling recorded resistance values were divided by 2 to obtain approximate equivalent N-values

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³

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

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 17

METRIC

GWP	W.P. 2376-09-00	LOCATION	22+424, 13.4 m Lt C/L (N 4962237.999, E 287111.392)
DIST	5 HWY 400	BOREHOLE TYPE	Hollow Stem Auger & NQ Coring
DATUM	Geodetic	DATE	15/05/2013 15/05/2013

ORIGINATED BY LG
COMPILED BY SSH
CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.8	GROUND SURFACE					
187.6 / 0.2	180 mm **ASPHALT**					
187.3 / 0.5	**PAVEMENT FILL:** Gravel & Sand, brown, dense	1	SS	40	187	
186.8 / 1.0	**PAVEMENT FILL:** Gravelly Sand, brown, dense	2	SS	19		
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, compact, damp to moist	3	SS	15	186	
		4	SS	14	185	8 54 23 15
		5	SS	12	184	
	very loose, wet	6	SS	3		4 52 36 18
	brown from 4.5 to 7.5 m	7	SS	16	183	
		8	SS	17	182	
		9	SS	15	181	
	dense	10	SS	45	180	
179.6 / 8.2	fractured	11	RC	T.C.R.=83% R.Q.D.=83%	179	Auger refusal and start of NQ coring at 8.2 m
	BEDROCK Granite Gneiss greyish / pink sound	12	RC	T.C.R.=100% R.Q.D.=100%	178	
177.3 / 10.5	End of Borehole. Borehole open and dry upon completion of overburden drilling (not stabilized)					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L
WATER CONTENT (%): 10 20 30
UNIT WEIGHT γ kN/m^3
GROUND WATER CONDITIONS

+3, ×3: Numbers refer to Sensitivity   20 / 15 5 / 10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 18

1 OF 1 **METRIC**

GWP W.P. 2376-09-00	LOCATION 22+416, 3.3 m Lt C/L (N 4962245.103, E 287122.481)		ORIGINATED BY LG
DIST 5 HWY 400	BOREHOLE TYPE Hollow Stem Auger & NQ Coring		COMPILED BY SSH
DATUM Geodetic	DATE 29/05/2013	29/05/2013	CHECKED BY ZO

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) (10 20 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS (GR SA SI CL)
186.5	GROUND SURFACE				
186.4 / 0.1	0.1 m **TOPSOIL**				
	EMBANKMENT FILL: Silty Sand to Sandy Silt, trace to some clay, trace gravel, greyish brown, moist to damp; very loose	1	SS	5	
	compact to dense	2	SS	20	
		3	SS	19	5 53 24 18
		4	SS	32	
		5	SS	23	
	loose to compact	6	SS	6	
		7	SS	12	3 47 32 18
		8	SS	22	
		9	SS	7	
178.9 / 7.6	**SAND** some silt, some organic pockets, greyish brown, wet	10	SS	100/25	sampler wet @ 7.6 m
178.5 / 8.0	**BEDROCK** Granite Gneiss, greyish / pink, sound; fractured	11	RC	T.C.R.=98% R.Q.D.=75%	Auger refusal and start of NQ coring at 8 m
	black, dark grey	12	RC	T.C.R.=100% R.Q.D.=100%	
		13	RC	T.C.R.=98% R.Q.D.=98%	
174.5 / 12.0	End of Borehole				

Elevation scale: 186, 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175

End of Borehole
Piezometer installed to 8.0 m. Water level in piezometer @ 0.9 m upon installation.
(not reliable due to water used for coring).
Water level in piezometer @5.3 m (El. 181.7 m) on June 17, 2013

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 19

METRIC

GWP	W.P. 2376-09-00	LOCATION	22+439, 13.4 m Lt C/L (N 4962242.619, E 287096.899)
ORIGINATED BY	LG		
DIST	5	HWY	400
BOREHOLE TYPE	Hollow Stem Auger	COMPILED BY	SSH
DATUM	Geodetic	DATE	15/05/2013 15/05/2013
CHECKED BY	ZO		

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
187.8	GROUND SURFACE					
187.6 / 0.2	190 mm **ASPHALT**					
187.4 / 0.4	**PAVEMENT FILL:** Gravel & Sand, brown, compact	1	SS	28		24 62 (14)
186.9 / 0.9	**PAVEMENT FILL:** Gravelly Sand, brown, compact	2	SS	22	187	2 58 22 18
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, compact, damp to moist	3	SS	23	186	
	loose	4	SS	7	185	
		5	SS	13	184	
		6	SS	27		
		7	SS	20	183	
	compact to dense	8	SS	40	182	2 50 30 18
		9	SS	26	181	
		10	SS	32	180	3 56 24 17
178.7 / 9.1		11	SS	100/8	179	spoon bouncing
178.6 / 9.2	**TOPSOIL** black, wet					

End of Borehole
Auger refusal @ 9.2 m
Probable Bedrock
Borehole dry and open upon completion

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

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

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Severn River Boat Channel Bridge

RECORD OF BOREHOLE No 20

METRIC

GWP W.P. 2376-09-00	LOCATION 22+438, 4.2 m Lt C/L (N 4962251.1, E 287100.708)		ORIGINATED BY LG
DIST 5 HWY 400	BOREHOLE TYPE Hollow Stem Auger		COMPILED BY SSH
DATUM Geodetic	DATE 16/05/2013	16/05/2013	CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.5	GROUND SURFACE						
186.5 / 0.1	0.05 m **TOPSOIL**						
	dense	1	SS	31		186	
	very loose	2	SS	3		185	
	loose	3	SS	9			5 53 23 17
	compact to dense	4	SS	16		184	
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, damp to moist	5	SS	44		183	
		6	SS	19		182	
		7	SS	34			3 36 41 20
		8	SS	27		181	
		9	SS	27		180	
					*	179	
178.9 / 7.6	**EMBANKMENT FILL:** Gravelly Fine Sand occ. silt zones grey, compact, wet	10	SS	16		178	Sampler wet @ 7.5 m
177.7 / 8.8	**FINE SAND** some organics, grey, wet						
177.3 / 9.3		11	SS	100/15			spoon bouncing

End of Borehole
Auger refusal @ 9.3 m
Probable Bedrock
*Borehole dry and caved @ 7.5 m (El. 179.0 m) upon completion (not stabilized)

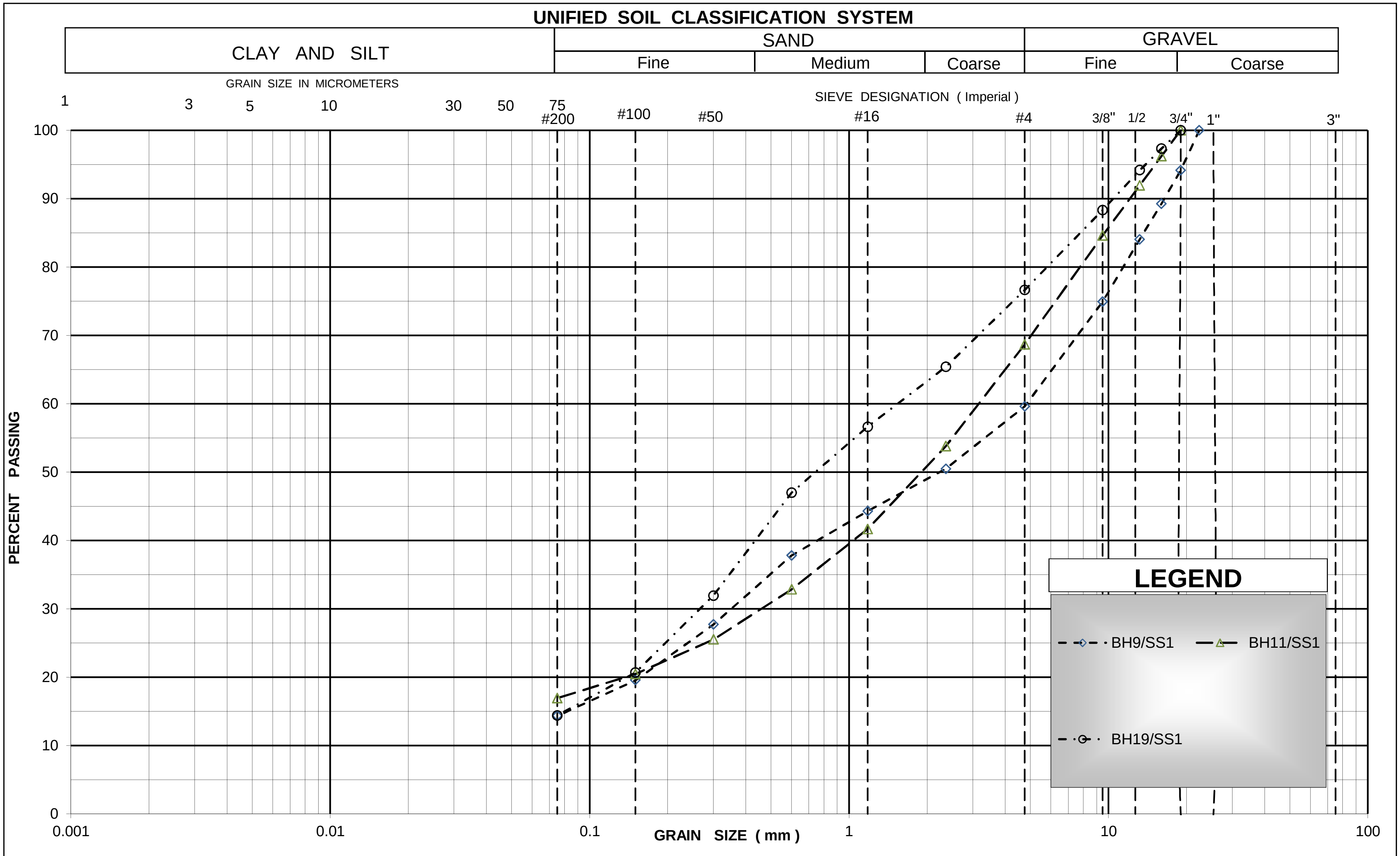
Chart columns (no data plotted): DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Test Results



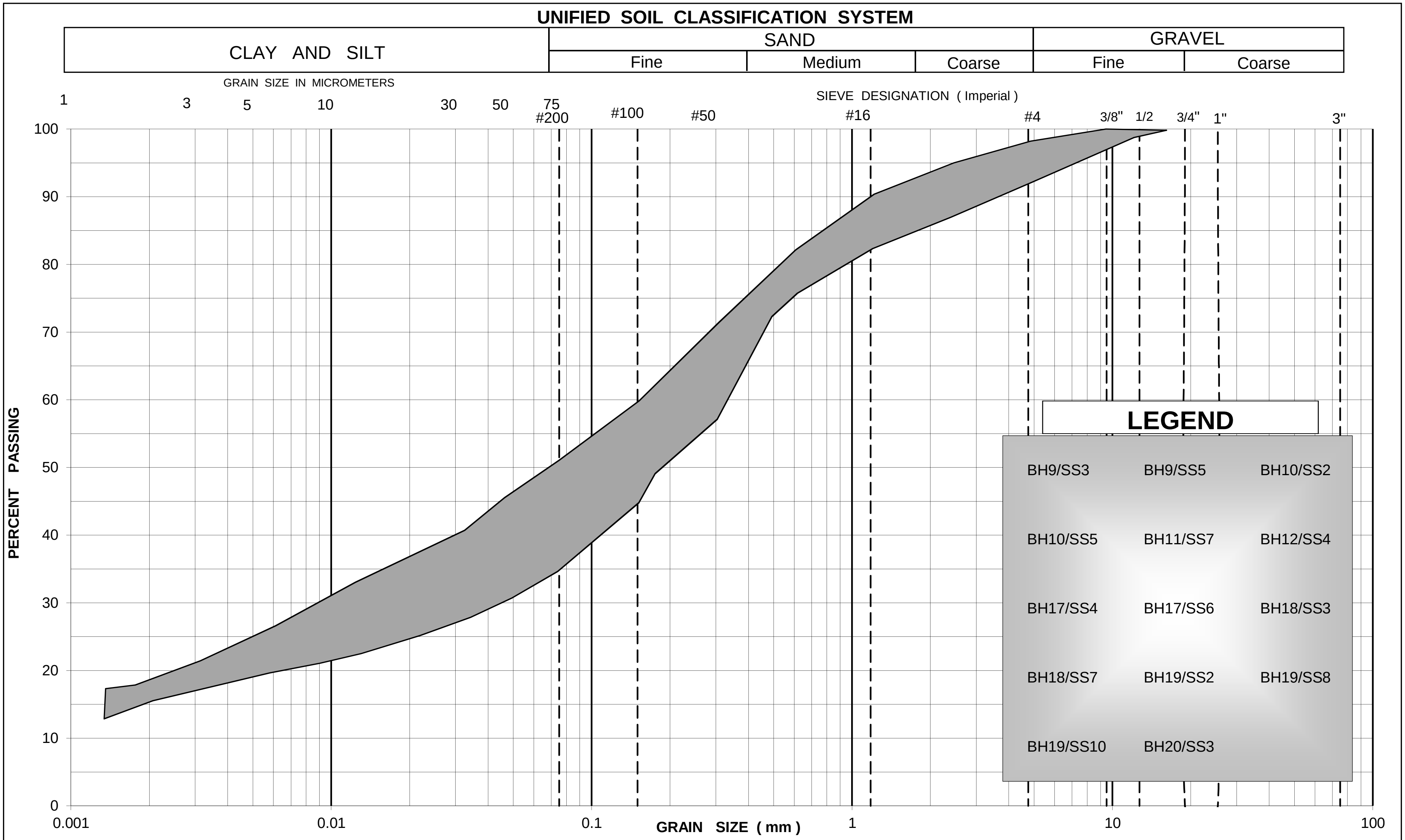
GRAIN SIZE DISTRIBUTION

PAVEMENT FILL

Figure: B-1

PROJECT # : TRANETOB20462AA

DATE : JULY, 2013



coffey

GRAIN SIZE DISTRIBUTION

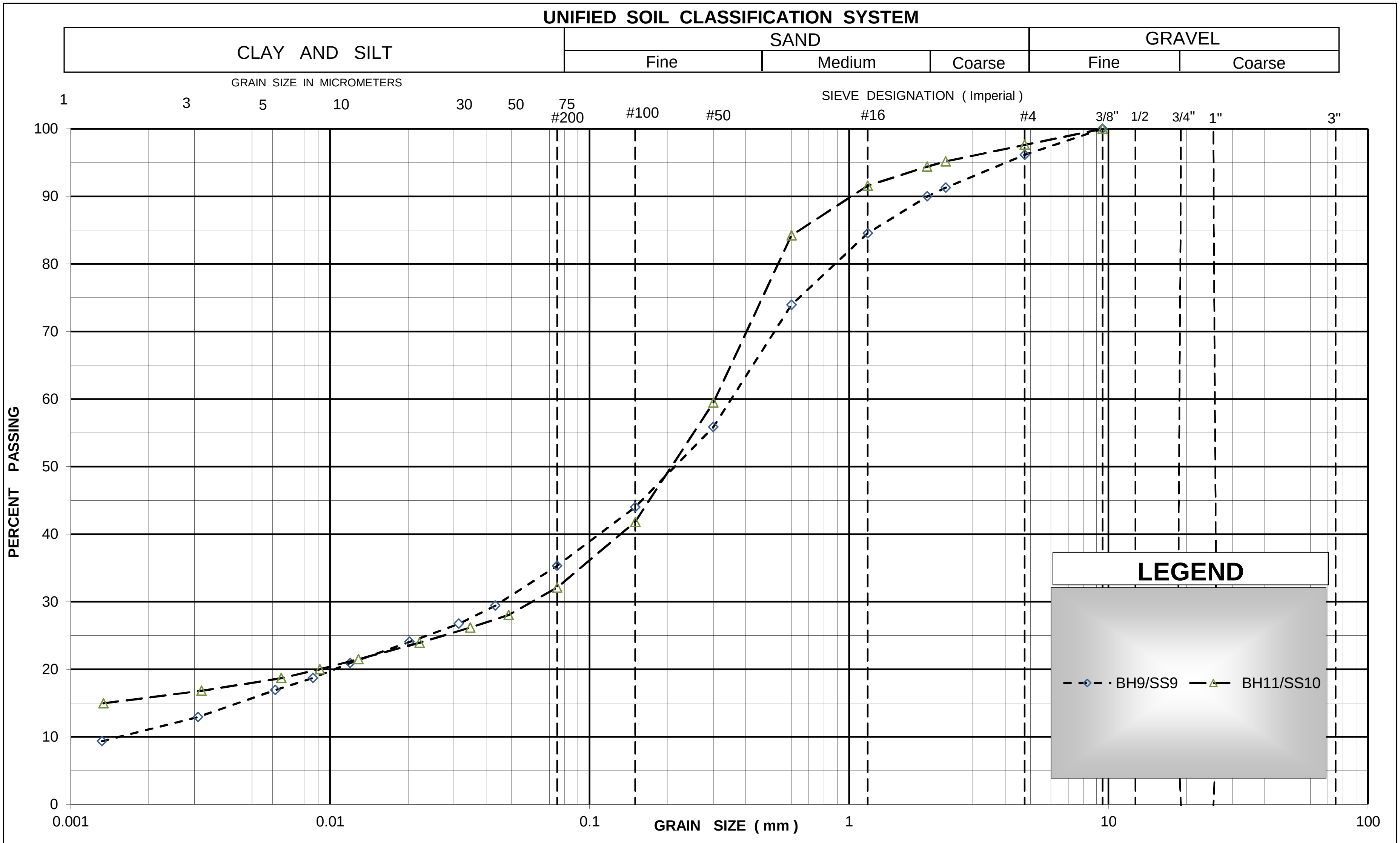
EMBANKMENT FILL

Silty Sand, trace to some clay, trace gravel

Figure: B-2

PROJECT # : TRANETOB20462AA

DATE SEPTEMBER, 2013



coffey

GRAIN SIZE DISTRIBUTION

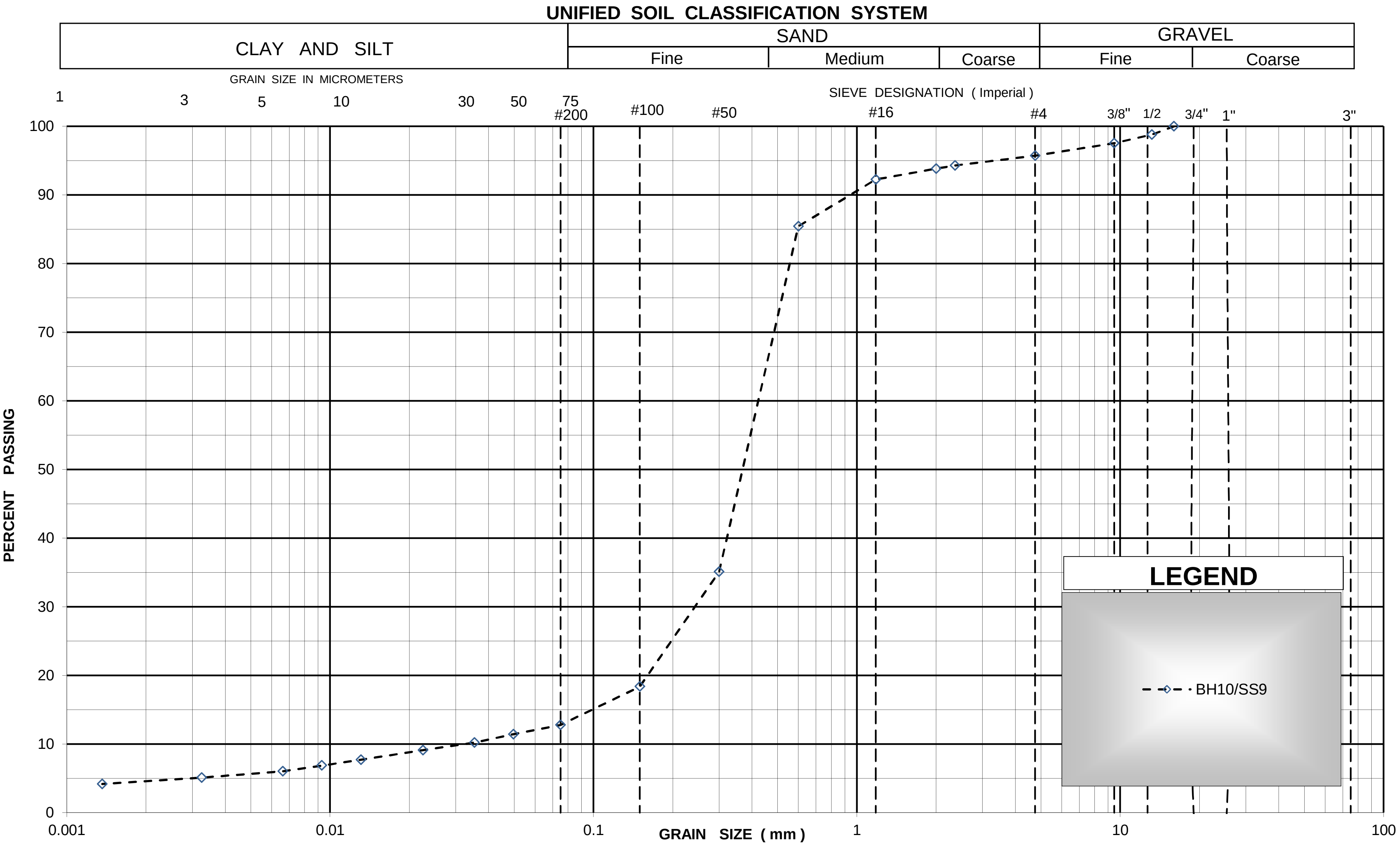
EMBANKMENT FILL

Sand, some silt and clay, trace gravel

Figure: B-3

PROJECT # : TRANETOB20462AA

DATE : SEPTEMBER, 2013



coffey

GRAIN SIZE DISTRIBUTION

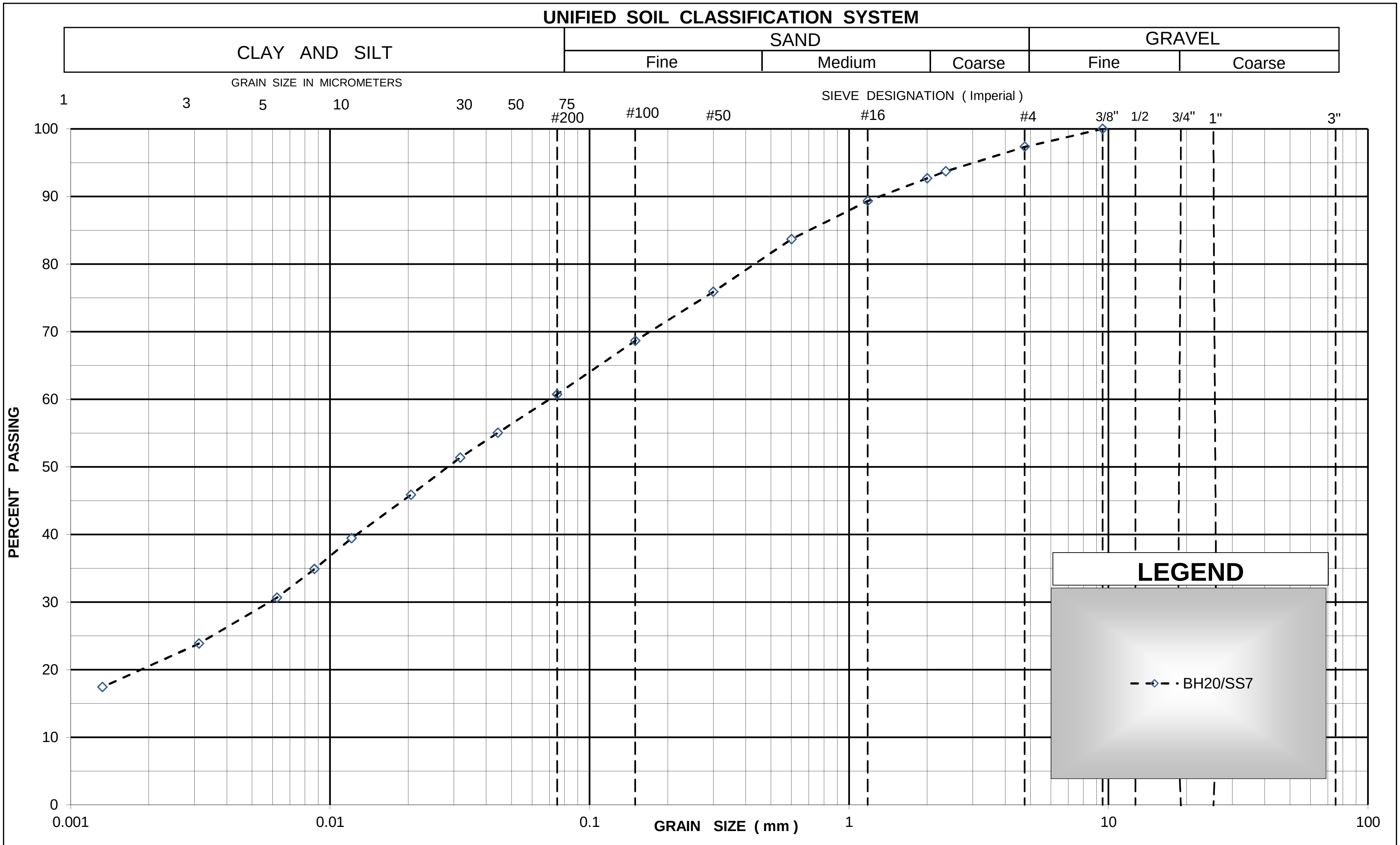
EMBANKMENT FILL

Sand, trace silt, clay and gravel

Figure: B-4

PROJECT # : TRANETOB20462AA

DATE : SEPTEMBER, 2013



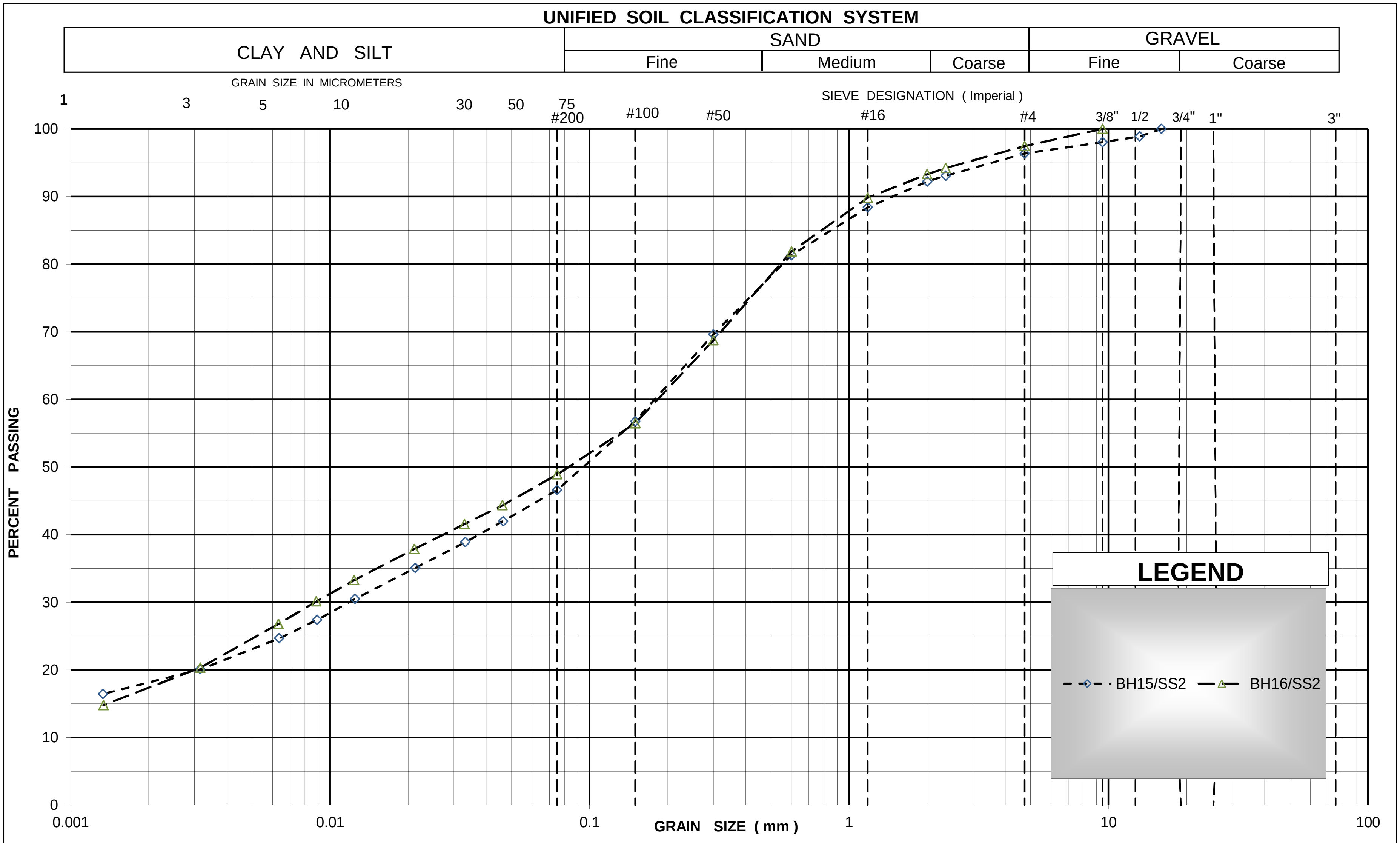
coffey

GRAIN SIZE DISTRIBUTION

EMBANKMENT FILL

Sandy Silt, some clay, trace gravel

Figure: B-5	
PROJECT # : TRANETOB20462AA	
DATE : SEPTEMBER, 2013	



coffey

GRAIN SIZE DISTRIBUTION

FILL

Silty Sand, some clay, trace gravel

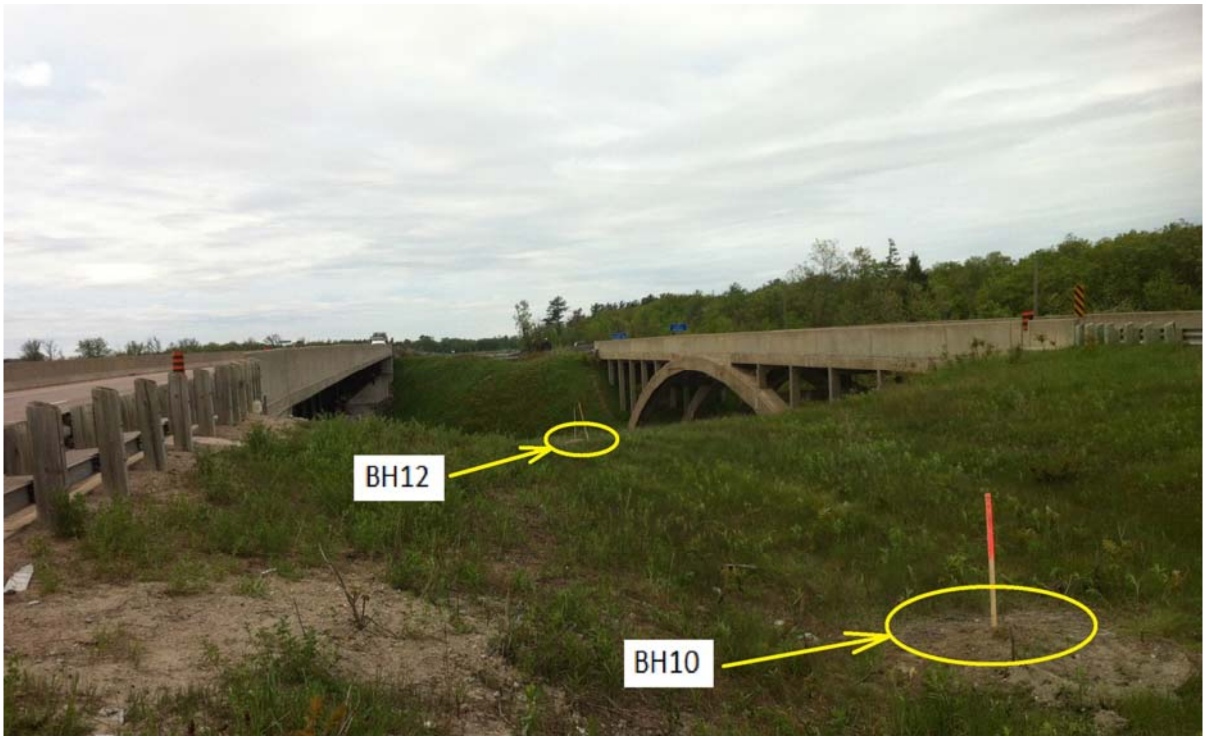
Figure: B-6

PROJECT # : TRANETOB20462AA

DATE : SEPTEMBER, 2013

Appendix C

Site Photographs



Photograph 1. Boreholes 10 and 12,looking west (construction north)

Photograph 2. Boreholes 13 and 14, looking east (construction south)

Photograph 3. Boreholes 15 and 16, looking west (construction north)

Photograph 4. Borehole 18, looking east (construction south)

Photograph 5. Borehole 18, looking east (construction south)

Appendix D

Rock Core Photographs and Test Results

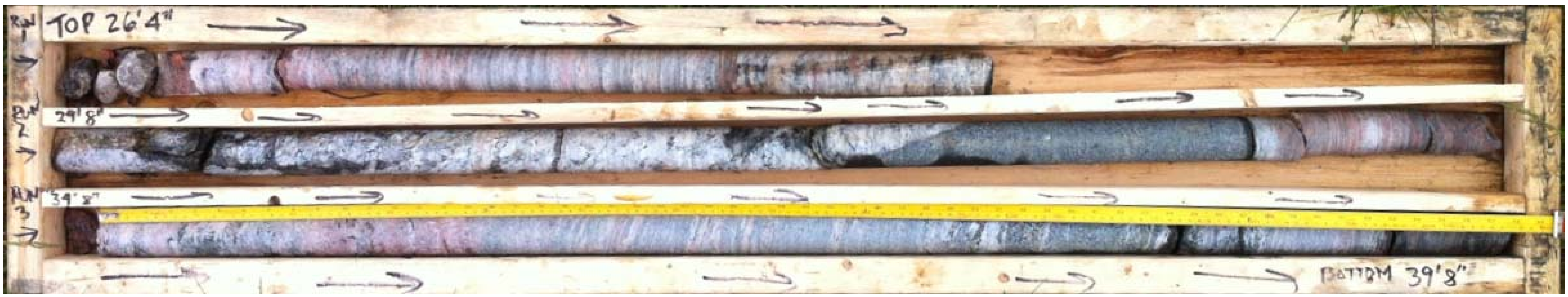
BH 12 (wooden box is 5 feet long)

BH 13 (wooden box is 5 feet long)

BH 14 (wooden box is 5 feet long)

BH 15 (wooden box is 5 feet long)

BH 16 (wooden box is 5 feet long)



BH 18 (wooden box is 5 feet long)

BH 11 (wooden box is 26 inches long)

BH 17 (wooden box is 26 inches long)

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	11	SAMPLE DEPTH, m	11.1-11.4

TEST CONDITIONS

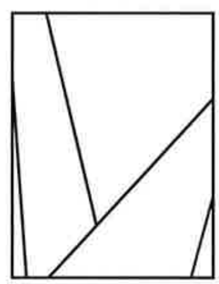
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.65	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.73	UNIT WEIGHT, kN/m^3	26.08
SAMPLE AREA, cm^2	17.60	DRY UNIT WT., kN/m^3	26.06
SAMPLE VOLUME, cm^3	187.40	SPECIFIC GRAVITY	-
WET WEIGHT, g	498.52	VOID RATIO	-
DRY WEIGHT, g	498.12		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	101.1

REMARKS:

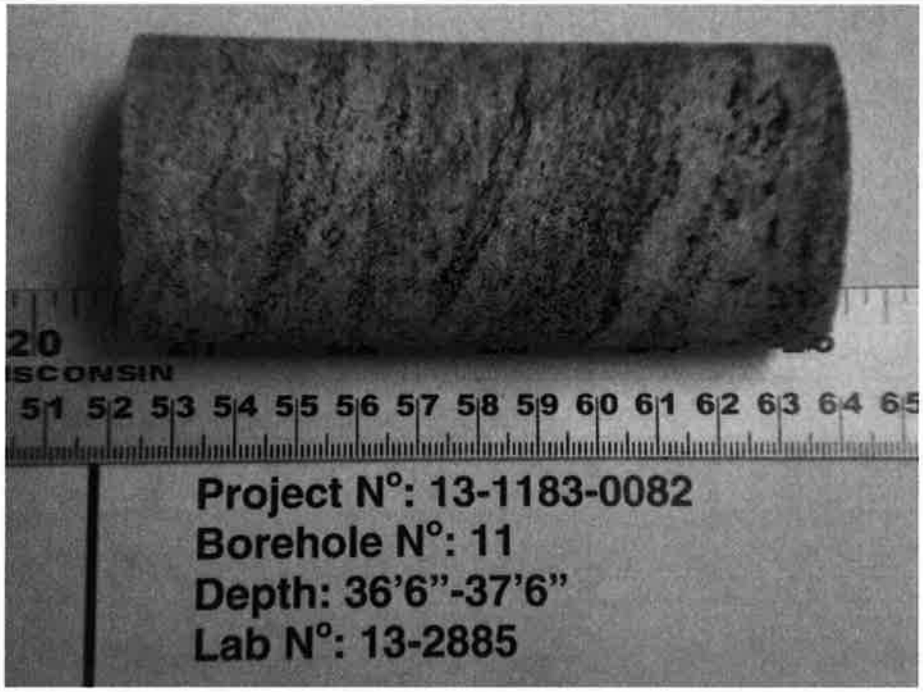
DATE: 7/24/2013

Checked By: RO

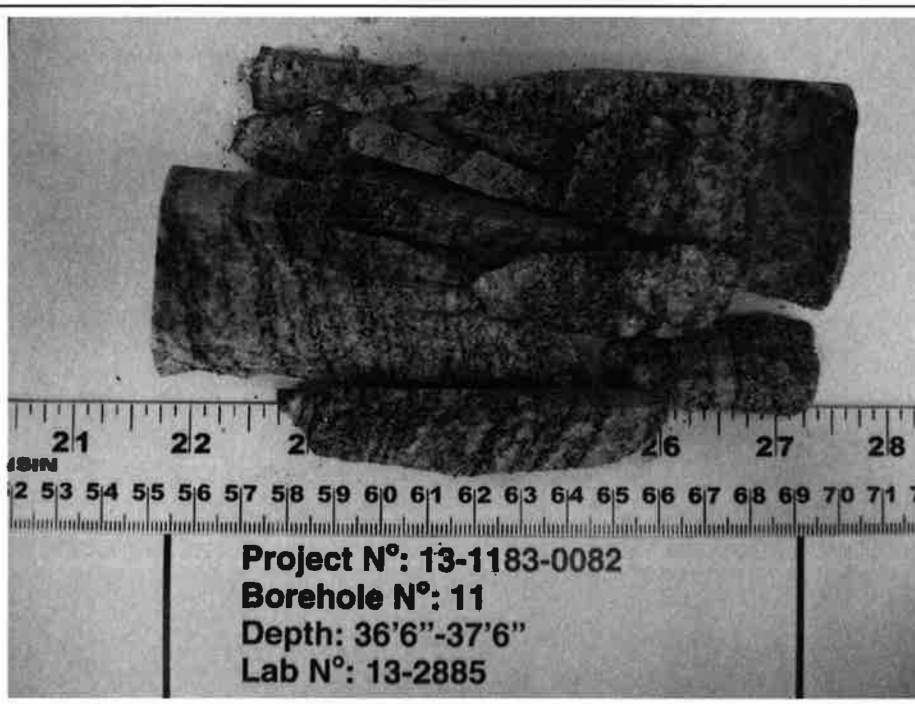
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RO

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	13	SAMPLE DEPTH, m	0.9-1.2

TEST CONDITIONS

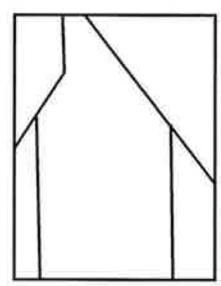
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.67	WATER CONTENT, (specimen) %	0.08
SAMPLE DIAMETER, cm	4.78	UNIT WEIGHT, kN/m^3	26.88
SAMPLE AREA, cm^2	17.92	DRY UNIT WT., kN/m^3	26.86
SAMPLE VOLUME, cm^3	191.12	SPECIFIC GRAVITY	-
WET WEIGHT, g	524.11	VOID RATIO	-
DRY WEIGHT, g	523.69		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	49.7

REMARKS:

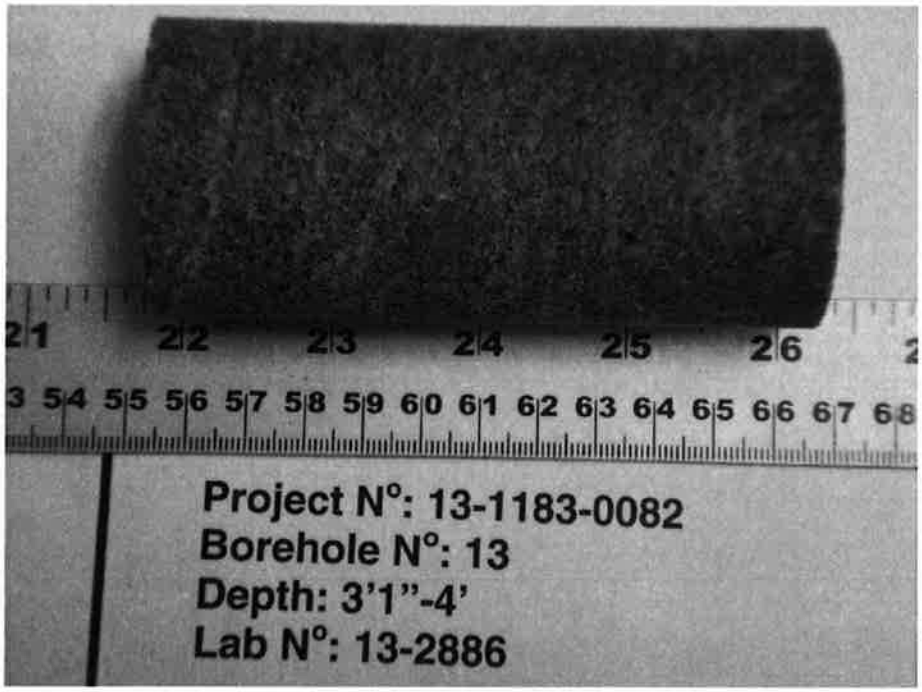
DATE: 7/24/2013

Checked By: RO

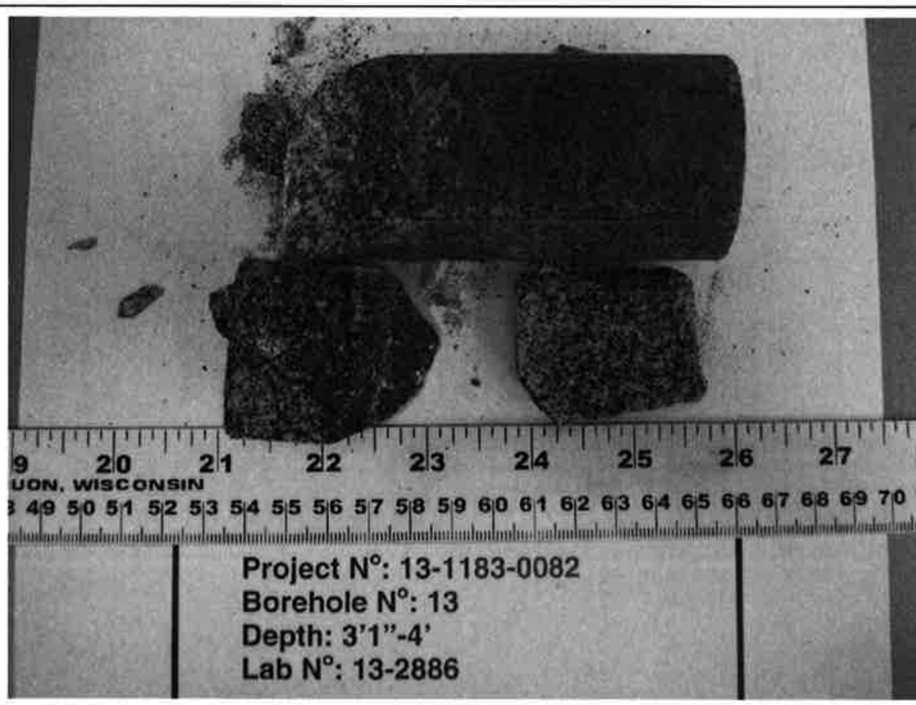
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RO

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	15	SAMPLE DEPTH, m	2.8-3.4

TEST CONDITIONS

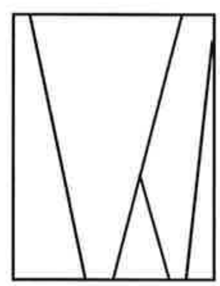
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	7.89	WATER CONTENT, (specimen) %	4.36
SAMPLE DIAMETER, cm	3.52	UNIT WEIGHT, kN/m^3	26.40
SAMPLE AREA, cm^2	9.70	DRY UNIT WT., kN/m^3	25.30
SAMPLE VOLUME, cm^3	76.51	SPECIFIC GRAVITY	-
WET WEIGHT, g	206.06	VOID RATIO	-
DRY WEIGHT, g	197.45		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

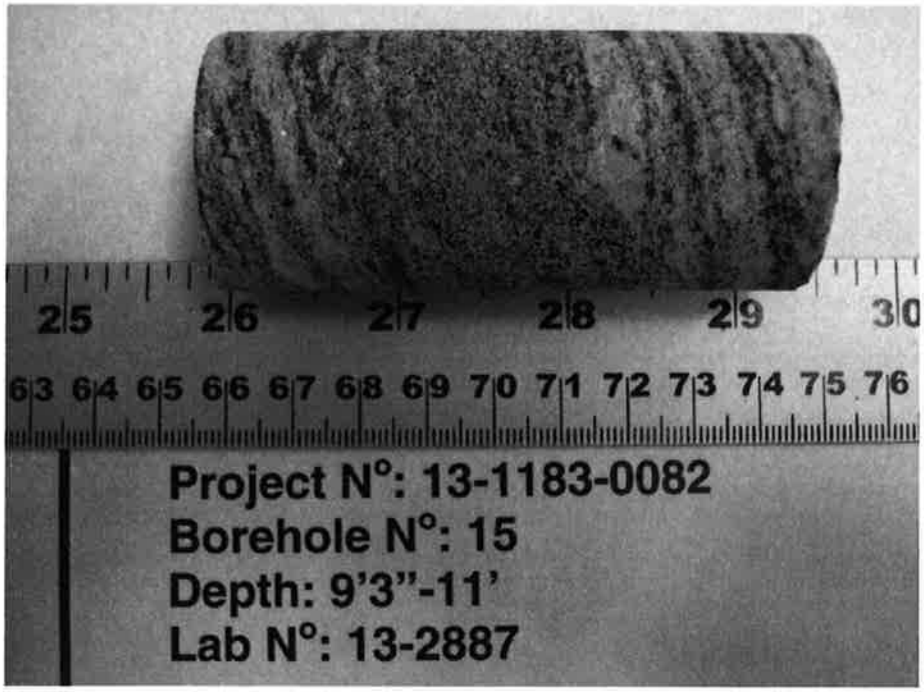
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	64.2

REMARKS:		DATE:	7/24/2013

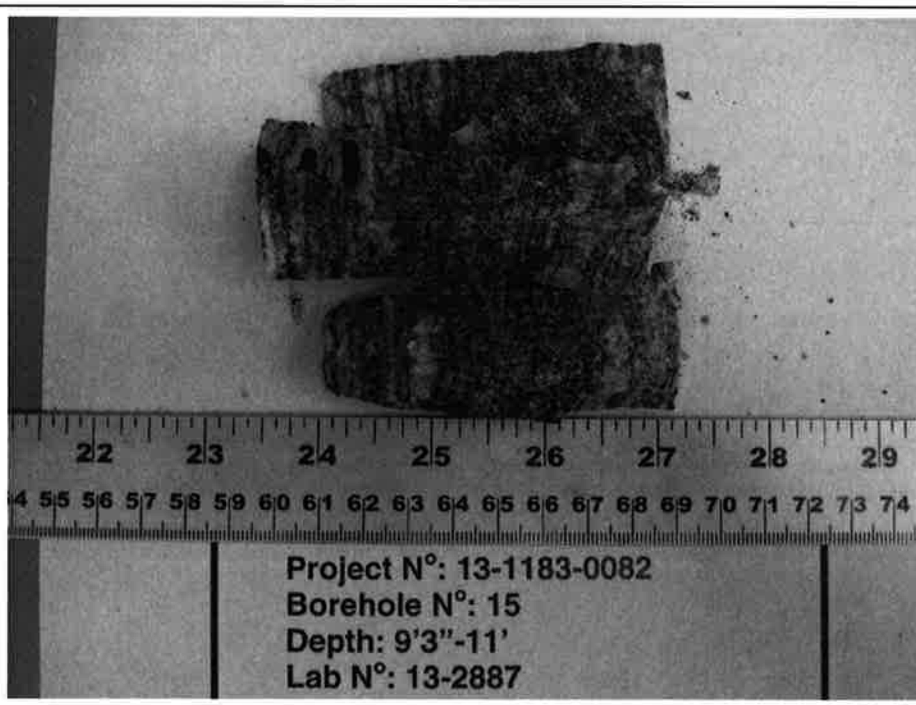
Checked By: Ro

Golder Associates

UNCONFINED COMPRESSION TEST ASTM D7012-07	FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. Ro

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	18	SAMPLE DEPTH, m	8.1-8.4

TEST CONDITIONS

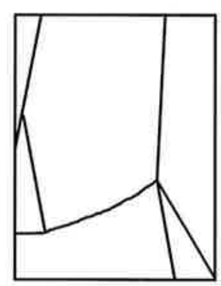
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.49	WATER CONTENT, (specimen) %	0.05
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	25.70
SAMPLE AREA, cm^2	17.71	DRY UNIT WT., kN/m^3	25.69
SAMPLE VOLUME, cm^3	185.80	SPECIFIC GRAVITY	-
WET WEIGHT, g	487.14	VOID RATIO	-
DRY WEIGHT, g	486.90		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	109.3

REMARKS:

DATE: 7/24/2013

Checked By: *[initials]*

Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE

BEFORE COMPRESSION

AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RO

UNCONFINED COMPRESSION TEST (UC)

ASTM D 7012-07

SAMPLE IDENTIFICATION

PROJECT NUMBER	13-1183-0082	SAMPLE NUMBER	-
BOREHOLE NUMBER	18	SAMPLE DEPTH, m	9.4-9.8

TEST CONDITIONS

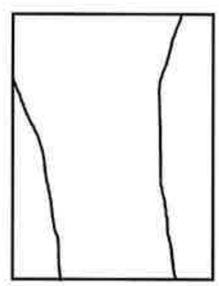
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.12

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.06	WATER CONTENT, (specimen) %	0.07
SAMPLE DIAMETER, cm	4.75	UNIT WEIGHT, kN/m^3	26.99
SAMPLE AREA, cm^2	17.71	DRY UNIT WT., kN/m^3	26.97
SAMPLE VOLUME, cm^3	178.21	SPECIFIC GRAVITY	-
WET WEIGHT, g	490.66	VOID RATIO	-
DRY WEIGHT, g	490.32		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	65.3

REMARKS:

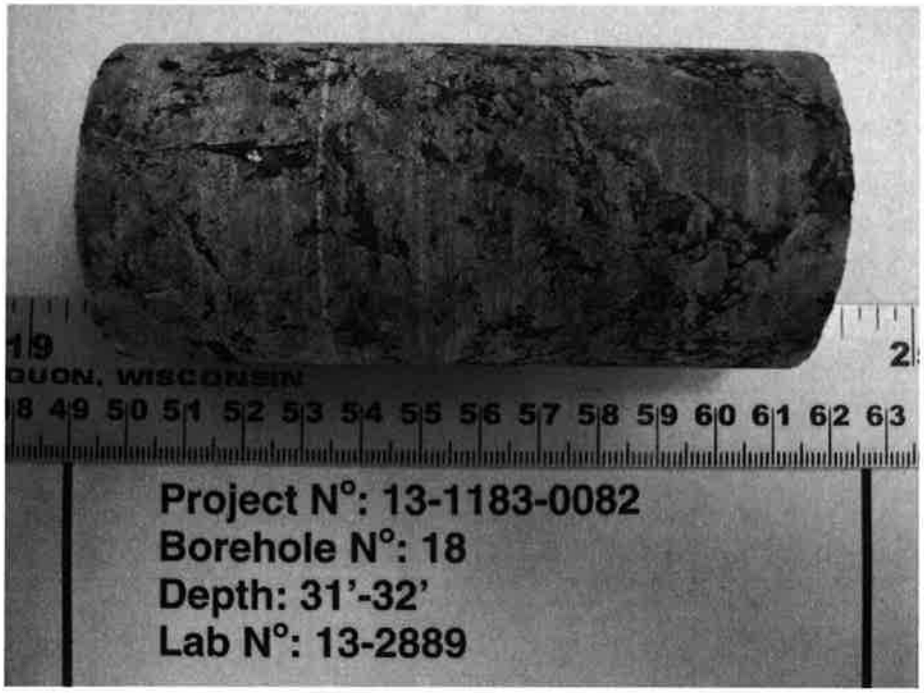
DATE: 7/24/2013

Checked By: Ro

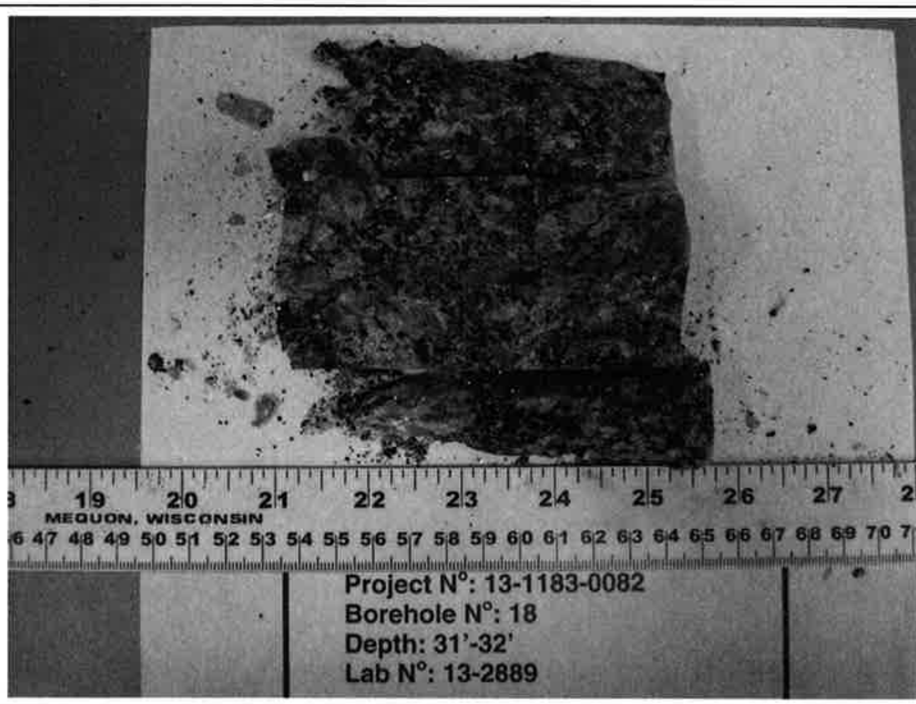
Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

Date 7/26/2013
Project 13-1183-0082

Golder Associates

Drawn Frank
Chkd. RO

Borehole No.	Run No.	Depth (ft)	Depth (m)	Test Type	Length (mm)	Core Diameter (mm)	Force (kN)	Rock Type	Is (MPa)	Is(50) (MPa)	Equivalent UCS (MPa)
BH11	1	37.8	11.5	A	41.0	48.0	20.8	GNEISS	8.3	8.3	199.0
	1	38.3	11.7	D		48.0	18.2	GNEISS	7.8	7.8	186.4
	2	42.5	13.0	A	41.0	48.0	21.5	GNEISS	8.6	8.6	206.3
	2	42.7	13.0	D		48.0	14.9	GNEISS	6.3	6.3	152.2
BH13	2	3.5	1.1	A	44.0	48.0	19.2	GNEISS	7.1	7.2	173.8
	2	4.0	1.2	D		48.0	10.8	GNEISS	4.6	4.6	110.8
	3	8.5	2.6	A	50.0	48.0	22.9	GNEISS	7.5	7.8	188.3
	3	8.7	2.6	D		48.0	17.6	GNEISS	7.5	7.5	179.8
BH15	2	9.3	2.8	A	33.0	37.0	13.8	GNEISS	8.9	8.0	191.2
	2	9.5	2.9	D		37.0	6.5	GNEISS	4.2	4.2	99.9
	3	14.7	4.5	A	26.0	37.0	10.7	GNEISS	8.7	7.4	178.5
	3	10.8	3.3	D		37.0	4.9	GNEISS	3.1	3.1	75.4
	4	19.8	6.0	A	26.0	37.0	7.7	GNEISS	6.3	5.4	129.0
	4	20.0	6.1	D	26.0	37.0	11.3	GNEISS	7.2	7.2	173.1
BH18	1	27.8	8.5	A	40.0	48.0	16.4	GNEISS	6.7	6.7	160.0
	1	27.9	8.5	D		48.0	17.8	GNEISS	7.6	7.6	181.6
	2	34.0	10.4	D		48.0	18.6	GNEISS	7.9	7.9	190.2
	2	34.5	10.5	A	48.0	48.0	22.9	GNEISS	7.8	8.1	194.1
	3	38.9	11.9	A	31.0	48.0	12.3	GNEISS	6.5	6.1	146.5
	3	39.2	11.9	D		48.0	5.6	GNEISS	2.4	2.4	57.7

Appendix E

Explanation of Terms Used in the Report

EXPLANATION OF TERMS USED IN REPORT

N-VALUE: THE STANDARD PENETRATION TEST (SPT) N-VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5 kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N-VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N-VALUE IS DENOTED THUS $\overline{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINT AND BEDDING:

SPACING	50mm	50 – 300mm	0.3m – 1m	1m – 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
Φ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
Φ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	W_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	W_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	W_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_P)/ I_P$	i	1	HYDAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(W_L - W) / 1_P$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION DESIGN REPORT - PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE OVER THE SEVERN RIVER BOAT CHANNEL, TOWNSHIP OF BAXTER, MTO CENTRAL REGION, W.P. 2376-09-00, SITE 42-87/1&2, GEOCRES 31D-566

McCormick Rankin

Project: TRANETOB20462AA
January 07, 2014

REPORT

Coffey
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

CONTENTS

FOUNDATION DESIGN REPORT
PROPOSED WIDENING OF SOUTHBOUND HIGHWAY 400 BRIDGE OVER THE SEVERN RIVER BOAT CHANNEL, TOWNSHIP OF BAXTER, MTO CENTRAL REGION, W.P. 2376-09-00, SITE 42-87/1&2

5 DISCUSSION AND RECOMMENDATIONS

McCormick Rankin (MRC) has been studying the feasibility of replacing/rehabilitation of the existing Highway 400 Bridges over the Severn River Boat Channel in the Township of Baxter.

Existing bridge information, based on available bridge drawings, is summarized in Table 5.1.

Table 5.1 Bridge Information

Title	Site Number	Year Built	Length (m)	Width (m)	Existing Structure Type	Proposed Structure Strategy
Severn River Boat Channel Bridge, NB	42-87/1	1957	93.7	10.2	Open Spandrel Deck Arch	Replacement
Severn River Boat Channel Bridge, SB	42-87/2	1992	118	12.0	3 Span – Slab on Steel I Girder Abutments and Two Concrete Piers	Rehabilitation

In 2012, Coffey prepared preliminary geotechnical investigation reports based on existing information (i.e. desk top study – no boreholes drilled) to aid MRC in their study. More recently, MTO and MRC decided to demolish the existing circa 1957 northbound open spandrel deck arch bridge and to accommodate the northbound traffic by widening the existing southbound bridge.

This report deals with the proposed widening of Highway 400 southbound Severn River Boat Channel Bridge to accommodate realigned northbound traffic. The foundation design report for the proposed Highway 400 northbound Severn River Bridge is presented under separate report cover.

As mentioned before, it is our understanding that the realigned northbound traffic will be carried by widening the existing circa 1992 southbound bridge. It will be located on the median side of the existing bridge. After the construction of the new structure, the existing northbound bridge will be demolished.

During this investigation, the subsurface conditions were explored at twelve borehole locations. Boreholes 9 to 12 and 17 to 20, which were drilled from the top of the existing highway embankment, show that the embankment fill extends to depths ranging between 7.6 and 11.1 m or to El. 179.6 to 176.0 m and consists of typically silty sand to sandy silt with traces to some clay and traces of gravel. Of the remaining four boreholes, Boreholes 15 and 16 contacted an approximately 2 m deep fill while in Boreholes 13 and 14, which were drilled from the Channel, no overburden was found (i.e. bedrock exposed below the water in the Channel). In Boreholes 10, 11 and 17, the embankment fill extends to the surface of proven/inferred bedrock, while in Boreholes 9, 12, 15, 16, 18, 19 and 20 some shallow native overburden was contacted, immediately overlying the bedrock.

In Boreholes 13 and 14 the surface of the bedrock was exposed at channel bottom at El. 174.9 and 175.1 m, respectively. The remaining boreholes contacted bedrock/inferred bedrock at El. 179.6 to 175.8 m.

5.1 Foundations

The existing structure, which carries the southbound traffic, will be widened to accommodate the realigned northbound traffic. The existing bridge is a three span structure supported on integral abutments and two piers are supported on shallow foundations (set minimum 200 mm into sound bedrock).

5.1.1 Abutment Support Elements

As the existing bridge abutments are supported on H-piles driven to refusal on bedrock, the most logical approach would be to duplicate this. This approach will enable the implementation of integral abutment design which is desirable from a structural perspective to duplicate the existing structural behaviour, but the proximity of the bottom of the abutment to the surface of the bedrock will generally render the pile lengths to be very short, which would be of concern.

The use of spread footing foundations resting on bedrock, to support the abutments would be another option, but this would require relatively deep excavations and shoring, leading to increased costs as well as possible problems with the existing pile foundations.

Drilled caissons (i.e. drilled and cast-in-place concrete piles) may be an attractive option but socketing the caissons into the bedrock may present some construction difficulties.

Finally, the use of micropiles is another, albeit expensive, option.

These are discussed in the following paragraphs.

5.1.1.1 Spread Footing Foundations

The new structure widening can be supported on spread footing foundations set about 0.2 m into the sound bedrock. This would translate into about 0.4 m into the bedrock.

Of the boreholes drilled, Boreholes 9, 10 and especially 11 and 12 were advanced closest to the proposed south (Barrie side) abutment location. In these boreholes, the surface of the bedrock was contacted at between El. 176.3 and 175.8 m (i.e. relatively level, but requiring rather deep excavations).

At the north (Parry Sound side) abutment site at Boreholes 17 and 18, the bedrock was contacted at El. 179.6 and 178.5 m, while in Boreholes 19 and 20, drilled somewhat further away, it was encountered at El. 178.6 m and 177.3 m, respectively. From these findings, it appears that at this location, the surface of the bedrock is relatively higher and undulating, but the excavations to construct the footings can be expected to be shallower.

In general, foundations bearing on the surface of the bedrock should be set at least 0.2 m into the sufficiently sound bedrock.

The following geotechnical resistances are available for footings bearing on level, sound bedrock:

- Factored Bearing Resistance at U.L.S. = 10,000 kPa
- Bearing Resistance at S.L.S will not govern

If the foundations are to be constructed adjacent to sloping ground, stability must be assured by socketing/keying-in the foundations sufficiently into the bedrock and/or doweling/anchoring into the bedrock. In addition, the footing must be placed on sufficiently level rock surface. If necessary, the bedrock surface can be flattened by levelling or making benches or the problem may be alleviated by providing dowels. As well, it should be ensured that the rock beneath the footing level will not be subject to detrimental scour or frost effects which might jeopardize the footings.

For inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with the Canadian Highway Bridge Design Code (CHBDC CAN/CSA, S6-06).

For the evaluation of the sliding resistance of the foundations, the interface friction factor (ultimate) between the underside of the concrete footing and the clean and sufficiently roughened bedrock surface can be taken as 0.6. Horizontal shear resistance can be supplemented by keying-in to the bedrock and utilizing the passive rock resistance and/or shear in grouted dowels and/or rock anchors. We recommended a minimum dowel length of 1.2 m (minimum 0.6 m into sound bedrock).

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 1000 kPa and resistance at S.L.S. need not be considered. The upper 0.5 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.2 m into the sound rock (embedded length in the sufficiently sound rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor ground resistances should also be checked.

For spread footing foundations, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer appointed by a QVE who is familiar with the findings of this investigation. This is important for this site and especially at the north abutment, as the surface of the bedrock appears to be sloping/variable and that the upper 0.2 to 0.3 m is generally shattered.

Normally for frost protection in this geographic area, the footings should have a permanent earth cover of not less than 1.6 m. If the footings are placed on sufficiently massive rock (i.e. no jointing, cracks, fissures, etc.,) it may be possible to reduce the thickness of frost protection or even eliminate it. For this purpose the following approach can be taken. The surface of the bedrock on which the footing is to be supported should be made level and carefully inspected by a Geologist or a Geotechnical Engineer. The surface of the rock to receive the footing must be free of open fractures, jointing, cracks, fissures or bedding planes, or any other defects which water can get into and cause problems due to frost. This is also applicable to rock surrounding the footing footprint. These areas must also be defect free or made so, such that water could not enter to cause problems with the rock supporting the footing (i.e. further opening the existing defects or causing heave due to frost action). From the borehole data and the anticipated founding depths, it is unlikely that frost will present a problem for footings placed on bedrock, but the above statements regarding frost protection are included herein for the sake of completeness and in case the rock surface at the footing locations is found within frost depth.

The rock must also be checked for any planes or other defects which may cause the footings to slide towards the channel. These are standard field features which are normally evaluated by a Geologist or Geotechnical Engineer, provided they are experienced enough.

If rock blasting is required/permitted (especially so close to the existing bridge) for excavations, it should be controlled in order to avoid over-breaking of bedrock. In our opinion, however, rock blasting should not be permitted. Wherever rock is over-excavated it should be inspected and approved by a Geotechnical Engineer and filled up with same class concrete as foundation concrete.

In addition, the bearing surface should be cleaned and made free from any loose debris prior to constructing the foundations.

5.1.1.2 Driven Steel H-piles

As mentioned before, the existing bridge abutments are supported on steel H-piles, driven to refusal on the bedrock. This approach would be suitable except for the fact that, especially at the north abutment, the surface of the bedrock was contacted at high elevations and thus, piles may be too short.

According to the GA drawing provided to us (dated June 2013, see Appendix F), the anticipated pile top elevations (i.e. bottom of abutment) at the south (Barrie side) and north (Parry Sound side) abutments are 182.0 m and 182.7 m, respectively, thus duplicating the existing bridge support elevations.

In the boreholes drilled near the north abutment location, the surface of the bedrock was contacted at the following elevations:

BH 18* – El. 178.5 m (4.2 m below the underside of the proposed abutment)

BH 17 – El. 179.6 m (3.1 m below the underside of the proposed abutment)

BH 19 – El. 178.6 m (4.1 m below the underside of the proposed abutment)

BH 20 – El. 177.3 m (5.4 m below the underside of the proposed abutment)

*closest to the proposed abutment location

This indicates that the length of the piles will likely be of the order of 4 m, but may be shorter.

At the south abutment location, the bedrock surface was recorded at the location of Boreholes 11 and 12 at El. 176.0 and 176.3 m, thus the length of the piles can be expected to be about 6 m. At boreholes 9 and 10 located nearby, the surface of the bedrock was contacted at similar elevations (i.e. El 175.8 and 176.3 m, respectively).

These short piles can be expected to 'walk' (i.e. slide) over the rock surface when driven to refusal, especially since there is little or no competent overburden above the rock surface. This can be expected to present a comparatively bigger problem at the north abutment location where the pile lengths are expected to be very short and the rock surface appears to be sloping.

In our experience, the minimum acceptable pile length is 5 m to provide a suitable fixity, but over sudden and strong (hard) bedrock surface this may not be sufficient. To rectify this situation, the following approach can be taken. At each pile location pre-augering into the bedrock can be effected. In essence, this would consist of a 0.6 m diameter hole which is extended into the bedrock and filled with concrete (below the flex zone) after dropping/driving the pile into the hole. The diameter of the hole may need to be increased to 0.76 m, depending on the site conditions, to facilitate cleaning of the base of the hole, if required. We recommend that this possibility be included in the contract. A larger diameter hole with a temporary casing may be needed in the overburden to prevent caving-in of the overburden. This temporary casing may need to be extended (screwed) into the bedrock sufficiently to provide a seal from water ingress. The hole into the bedrock (minimum 0.6 m diameter) may then be extended by coring/chopping into the bedrock. However, these aspects be left to the Contractor, while specifying the end results (i.e. minimum hole diameter, depth/elevation, clean base, and the withdrawal of any temporary casing, etc).

The required penetration of the hole into the bedrock also depends on the fixity requirements and this aspect should be decided by the Structural Engineer. This may lead to an increase in the required penetration of the hole into the bedrock. In addition, short pile lengths would not provide much resistance to uplift and this aspect will also play a role in choosing the depth of penetration into the bedrock. In our opinion, however, at the south abutment location an approximately 0.9 m penetration into the bedrock would suffice. This would bring the pile lengths to about 7 m. The entire hole below the rock surface would be filled with concrete (of suitable mix) after dropping/driving the pile into the hole and ensuring that it sits on bedrock and not on spoils from augering/coring/percussion operations and that the sides of the hole are sufficiently clean.

At the north abutment location, the situation appears to be more complex. Here the bedrock at BH 18 (closest to the abutment location) was contacted at El. 178.5 m and in BH 17 located some 7 m from the abutment, it was contacted at El. 179.6 m. As the proposed pile top (bottom of abutment) elevation is 182.7 m, the anticipated pile length is about 4.2 m (BH 18) but could be as short as 3.1 m (BH 17) or even shorter, if rock has a higher peak elevation. In this instance, the minimum recommended hole depth below the top of rock surface would be 1.2 m. However, to avoid possible unpleasant surprises during the construction, a bottom of hole elevation can also be specified, but not less than 1.2 m into the bedrock. In this instance, this elevation would be 177.7 m. This would ensure a minimum pile length of 5.0 m. It would also provide a flex zone of 3 to 4 m for implementing an integral abutment design. The hole within the bedrock (below the flex zone) would be filled with concrete after the installation of the pile.

It should be noted that in order to prevent cave-ins, the pre-auger holes may need to be cased in the overburden during the installation (i.e. above the rock level) until the concrete is poured. It should also be noted that with this approach, the installation of battered piles will be difficult and is not recommended, although battered piles will unlikely be used, as an integral abutment type bridge is being proposed.

We recommend that a heavy section such as HP 310 x 110 be used. MTO's standard design value for geotechnical resistance for piles driven to refusal on bedrock is normally 2000 KN/pile for ULS (factored) and SLS will not govern. However, since in this instance pile lengths are quite short, we recommend that this aspect be taken into consideration by the structural engineer in design by increasing applied load factors.

The following procedure can be followed.

After extending the minimum 0.6 m diameter hole to the required depth into the bedrock, the hole need to be properly cleaned and the steel H-pile can be placed in the hole centered and tapped gently into place with pile driving equipment (i.e. no hard driving). With this procedure, since no hard driving will take place, if desired, a lighter section, such as HP 310 x 79 can be used, rather than 310 x 110 (if lower resistance can be used such as factored ULS=1500 kN/pile). If necessary, pile flange reinforcement (OPSD 3001 Type I) can be used to minimize a pile tip damage. The hole can then be filled with concrete to provide fixity and adhesion below the flex zone. Any temporary steel casing that was used in the overburden to facilitate construction would then be slowly withdrawn, after the concrete has sufficiently set, ensuring that the pile and the surrounding concrete is not adversely affected (i.e. lifted, moved sideways, moved off-plumb, etc). The hole above the bedrock (concrete section) can be filled with a sufficiently uniform and fine sand, as the casing is being withdrawn.

The minimum spacing between the caisson/pile holes should be 1.5 m centre to centre.

If an increase in the fixity zone is required for integral abutments, the penetration of the 0.6 m diameter (or larger) holes into the bedrock can be increased. After filling the hole with concrete to the required height, the balance of the hole below the surface of bedrock can be filled with uniform sand, as specified by MTO convention for integral abutments.

The General Arrangement Drawing for the existing southbound bridge (proposed by Morrison Hershfield Limited – no date) shows an approximate anticipated bedrock elevation of 173 m at the north abutment location. This elevation represents an anomaly compared to the findings of this investigation. It would therefore be prudent to look into the as built drawings (if available), correspondence, etc. that may be available from the construction record regarding this aspect. At the south abutment, the bedrock elevation is shown as 175± m, which is only about 1 m lower than the average elevations contacted during this investigation (i.e. 176± m). It is also of interest to point out that the notes on the aforementioned GA Drawing include the following statement 'Pre-auger pile holes to bedrock level. Backfill hole with MTO Class 4 Aggregate prior to driving piles'.

5.1.1.3 Caisson Foundations

Drilled and cast-in-place concrete foundations (drilled caissons) can be considered. The caissons need to be socketed into the bedrock to carry axial and lateral loads.

Geotechnical resistances of cast-in-place concrete piles (caissons) increase with socket depth into the bedrock. For caissons which extend not less than 0.3 m into the relatively sound bedrock (i.e. typically 0.6 m into bedrock), 10,000 kPa can be used for end bearing resistance at ULS (factored). SLS will not govern. The minimum caisson penetration depth below the sufficiently sound bedrock surface may need to be increased depending on the degree of sloping of the bedrock surface to avoid sliding of the caisson due to unbalanced horizontal forces.

The minimum spacing of the caissons centre to centre should normally not be less than three diameters as per CHBDC S6-06. As well, a minimum caisson diameter of 0.76 m is recommended to enable the base and side inspection and cleaning, if required. However, if there is a compelling reason for the use smaller diameter caissons, this requirement can be looked into.

As was mentioned before, if the rock surface in front of the caisson is sloping and the caisson is located close to the sloping surface, this geometry may adversely affect the resistance, in particular the horizontal resistance (as well as rendering the installation of the caisson into the bedrock more difficult). In addition, if the rock around the caisson is shattered during the construction, this too will adversely affect the resistances and as such excessive shattering of the rock in the vicinity of the caissons must be avoided. As per OPSS 903, the caisson bottom may if necessary be stepped on sloping bedrock condition, with each step not greater than ¼ of the diameter of the bearing area.

Excavation methods shall be such that the sides and bottom of the hole are straight and free of loose material that might prevent intimate contact of the concrete with bedrock. While excavating, rock adjacent to caisson should not be shattered (i.e. damage to bedrock should be minimized). The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'.

Vibrations should not present major problems, except possibly when extending the caissons into the bedrock (i.e. while socketing the caisson into the bedrock), or if rock fill is encountered in the overburden. Vibration monitoring is discussed later in this report, under the heading 'construction'.

Some dewatering may be required to advance the basically cohesionless overburden and to seal water from entering into the excavation from the overburden/bedrock interface, if the temporary steel casing cannot be sufficiently advanced into the bedrock to provide a reasonably water tight seal. Tremmie concreting can be considered. A NSSP may be needed to alert the contractor, of the above, as well as the presence of hard (strong) nature of the bedrock, possible presence of rock fill/shattered rock and possible dewatering issues during the installation of caissons.

5.1.1.4 Micropiles

Another alternative would be to use micropiles. Similar to the use of driven steel H-piles and caissons, this method can be expected to reduce the extent of excavations and shoring in comparison with spread footing foundations.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can be installed in most soil and rock types, ground conditions as well as through existing mass or reinforced concrete (i.e. reinforcing steel bars should not present problems). A permanent steel casing is typically used to avoid the grout loss into the voids in the rock fill and to protect the micropile from being exposed to environments. Micropiles can withstand axial and/or lateral load. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, ground, and the environment. They can be installed in access-restrictive environments as well. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the micropiles are often used to enhance the support of existing structures. Micropile structural capacities, by comparison, rely on high capacity steel element to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout and ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors.

Geotechnical resistances for design purposes will depend on the type and installation methods used. For preliminary estimating purposes a factored bonding resistance between 600 and 1000 kPa at ULS (between the sound granite gneiss and grout) can be used but the upper 0.5m of the bedrock should be ignored. If the use of micropiles is to be considered, this should be further discussed with us.

The use of micropiles can be expected to be more costly than spread footing and caisson options. However, this and other details can be discussed with a specialized contractor; we will be pleased to facilitate this if requested.

5.1.1.5 Summary of Foundation Options for Abutments

From foundation engineering point of view (i.e. reliability) all four options discussed in the preceding paragraphs are considered to be equally acceptable. However, considering the fact that the existing bridge is supported on steel H-piles, which affords an integral abutment type design, the use of H-piles is considered to be the preferred option. In addition, this option is believed to be the most cost effective one, while the use of micropiles would likely be the least economical.

5.1.2 Pier Foundations

The south pier can be supported on spread footing foundation, bearing on sound bedrock, similar to the existing bridge south pier. The use of driven piles is technically not feasible, as the bedrock is exposed at the south pier location and is covered with only little native overburden and embankment fill at the north pier location. The use of drilled caisson and micropile foundations is technically feasible, if desired.

We understand that during the construction of spread footing foundations at the north pier location of the existing bridge, some construction difficulties were experienced due to the sloping nature of the bedrock surface. Because of this reason, consideration can also be given to the use of caisson (drilled and cast-in-place concrete pile) foundations at the north pier location.

5.1.2.1 Spread Footing Foundations on Bedrock

The piers for the proposed widening can be supported on similar type foundations (i.e. spread footings set at least 0.2m into sound bedrock) as the existing bridge.

At the borehole locations, the surface of the sound bedrock was contacted at the following elevations.

Table 5.1.2.1.1 Top of Sound Bedrock Elevation at Boreholes Drilled Near the Proposed Pier Locations

Support Location	Borehole No.	Top of Bedrock Elevation at Borehole Location (m)	Top of Sound Bedrock Elevations at Borehole Location (m)
South Pier	13	174.9	174.3
	14	175.1	174.7
North Pier	15	176.8	176.5
	16	177.2	177.1

The following geotechnical resistances are available for footings bearing on level, sound bedrock:

- Factored Bearing Resistance at U.L.S. = up to 10,000 kPa
- Bearing Resistance at S.L.S. will not govern

If the foundations are to be constructed adjacent to sloping ground (as may be the case for this project), stability must be assured by socketing/keying-in the foundations sufficiently into the bedrock and/or doweling/anchoring into the bedrock. In addition, the footings must be placed on sufficiently level rock surface. If necessary, the bedrock surface can be flattened by leveling or making benches or the problem may be alleviated by providing dowels. As well, it should be ensured that the rock beneath the footing level will not be subject to detrimental scour or frost effects which might jeopardize the footings.

For inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with the Canadian Highway Bridge Design Code (CHBDC CAN/CSA, S6-06).

For the evaluation of the sliding resistance of the foundations, the friction factor (ultimate) between the underside of the concrete footing and the clean and sufficiently roughened bedrock surface can be taken as 0.6. Horizontal shear resistance can be supplemented by keying-in to the bedrock and utilizing the passive rock resistance and/or shear in grouted dowels and/or rock anchors. We recommend a minimum dowel length of 1.2 m, but not less than 0.6m into sound bedrock.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 1000 kPa and resistance at S.L.S. need not be considered. The upper 0.5 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 1.2 m into the sound rock (embedded length in the sufficiently sound rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor ground resistances should also be checked.

For spread footing foundations, all footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer appointed by a QVE and who is familiar with the findings of this investigation. This is important for this site, since the surface of the bedrock appears to be sloping/variable and that the upper 0.1 to 0.6m appears to be shattered.

Normally, for frost protection in this geographic area, the footings should have a permanent earth cover of not less than 1.6 m. If the footings are placed on sufficiently massive rock (i.e. no jointing, cracks, fissures, etc.) it may possible to reduce the thickness of frost protection or even eliminate it. For this purpose, the following approach can be taken. The surface of the bedrock on which the footing is to be supported should be made level and carefully inspected by a Geologist or a Geotechnical Engineer. The surface of the rock to receive the footing must be free of open fractures, jointing, cracks, fissures or bedding planes, or any other defects which water can get into and cause problems due to frost. This is also applicable to rock surrounding the footing footprint. These areas must also be defect free or made so, such that water could not enter to cause problems with the rock supporting the footing (i.e. further opening the existing defects or causing heave due to frost action). This would not be applicable to footings in water, if it can be ensured that freezing will not occur at the surface of rock level. From the borehole data and the anticipated founding depths, it is unlikely that frost will present a problem for footings placed on bedrock, but the above statements regarding frost protection are included herein for the sake of completeness and in case the rock surface at the footing locations is found within frost depth.

The rock must also be checked for any planes or other defects which may cause the footings to slide towards the River. These are standard field features which are normally evaluated by a Geologist or Geotechnical Engineer, provided they are experienced enough.

If rock blasting/splitting is required/permitted for excavation, it should be controlled in order to avoid over-breaking of bedrock and also to prevent any damage to the existing bridge and its support element. In our opinion, however, rock blasting should not be permitted. Wherever rock is over-excavated, it should be inspected and approved by a Geotechnical Engineer and filled up with same class concrete as the foundation concrete.

Bearing surfaces should be cleaned and made free from any loose debris prior to concreting of foundations.

Any mass concrete used to raise the grade to the underside of the footings should be of sufficiently good quality to resist possible erosional forces that may exist in the Channel.

5.1.2.2 Deep Foundations

As mentioned before, owing to the presence of no overburden at the south pier location, the use of driven piles is considered unsuitable to support the piers for the proposed widening.

As the surface of the bedrock at the north pier location and appears to be sloping, the construction of normal spread footing foundations at the north pier location may present some difficulties. In addition, from MTO correspondence during the construction of the existing bridge (in the early 1990’s), this appears to be the case. The use of caisson foundations may therefore be preferable from risk management point of view and can be considered for the north pier support.

The use of micropiles is technically feasible, albeit expensive. This option is also discussed in the following paragraphs.

5.1.2.2.1 Cast-in-Place Concrete Pile (Caisson) Foundations

Drilled caisson foundations would be a less cost effective option than normal spread footing foundations and their use would not be normally recommended. However, in this instance, it may somewhat reduce shoring efforts, as follows. It appears from the GA Drawing, shoring will likely be required for the north pier construction on the east side. As the use of caisson foundations can be expected to reduce, to a certain extent, the shoring effort, this option can be looked into as a possible option. This option was discussed in Section 5.1.1.3 of the report but is essentially repeated here for the sake of expediency.

For this project, caisson foundations will need to be socketed into sound bedrock. For caissons which extend at least 0.3 m into the sound bedrock (i.e. generally about 0.6m below the rock surface). An exception to this is BH 13 location where the top 0.6m of the bedrock was found to be shattered (possibly due to blasting in the channel) and here an at least 0.9 m penetration would be required.

Factored Geotechnical Resistance at ULS*=10,000 kPa

Geotechnical Resistance at SLS will not govern

*end bearing resistance

Geotechnical resistance of caissons increases with increased socket depth into the bedrock but it is believed that higher resistances are not required for this project.

Due to the fact that existing north pier foundation of the existing southbound bridge extends into the bedrock at about El. 172.5 m, caisson foundations should be extended to same or similar elevation to minimize potential risk (e.g. bedrock fracturing/shattering) from the previous deep bench cut immediately beside the proposed bridge north pier foundation.

The minimum caisson penetration depth below the sufficiently sound bedrock surface may need to be increased, depending on the degree of sloping of the bedrock surface to avoid sliding of the caisson due to unbalanced horizontal forces.

The minimum spacing of the caissons centre to centre should normally not be less than three diameters as per CHBDC S6-06. As well, a minimum caisson diameter of 0.76 m is recommended to enable the base inspection and cleaning. However, if there is a compelling reason for the use of smaller diameter caissons, this requirement can be looked into.

If the rock surface in front of the caisson is sloping (this appears to be the case at the north pier location) and the caisson is located close to the sloping surface, this geometry may adversely affect the resistance, in particular the horizontal resistance. As well, if the rock around the caisson is shattered during the construction, this too will adversely affect the resistances and as such excessive shattering of the rock in the vicinity of the caissons must be avoided. As per OPSS 903, the caisson bottom may if necessary be stepped on sloping bedrock condition, with each step not greater than ¼ the diameter of the bearing area.

Excavation methods shall be such that the sides and bottom of the hole are straight and free of loose material that might prevent intimate contact of the concrete with bedrock. While excavating, rock adjacent to caisson should not be shattered (i.e. damage to bedrock should be minimized). The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'.

Vibrations should not present major problems, except when extending the caissons into the bedrock (i.e. while socketing the caisson into the bedrock), or if rock fill is encountered in the overburden. Vibration monitoring to prevent damage to the existing bridge will need to be provided.

Some dewatering may be required to advance the caisson hole in the basically cohesionless overburden and to seal water from entering into the excavation from the overburden/bedrock interface. If the temporary steel casing cannot be sufficiently advanced into the bedrock to provide a reasonably watertight seal, tremmie concreting can be considered. A NSSP may be needed to alert the Contractor, including the presence of hard (strong) nature of the bedrock, possible presence of rock fill/shattered rock and possible dewatering issues during the installation of caissons.

Frost protection requirements were discussed before and will not be repeated here.

5.1.2.2.2 Micropile Foundations

Micropile foundations are less economical in comparison with caisson and particularly spread footing foundations. However, they can be practical in situations where equipment access is limited and/or less overhead conditions for constructed. They are discussed in section 5.1.1.4 but this section is repeated here for expediency.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can be installed in most soil and rock types, ground conditions as well as through existing mass or reinforced concrete (i.e. reinforcing steel bars should not present problems). A permanent steel casing is typically used to avoid the grout loss into the voids in the rock fill and to protect the micropile from being exposed to environments. Micropile foundations can be designed to withstand axial and/or lateral loads. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, ground, and the environment. They can be installed in access-restrictive environments as well. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing structures. Micropile structural capacities, by comparison, rely on high capacity steel elements to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout and ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors.

Geotechnical resistances for design purposes will depend on the type and installation methods used. For preliminary estimating purposes a factored bonding resistance between 600 and 1000 kPa (at ULS, between the sound granite gneiss and grout) can be used but the upper 0.5m of the bedrock should be ignored. If the use of micropiles is to be considered, this should be further discussed with us.

The use of micropiles can be expected to be more costly than spread footing and caisson options. However, this and other details can be discussed with a specialized contractor; we will be pleased to facilitate this, if requested.

5.1.2.3 Summary of Foundation Options for Piers

Supporting the south pier on spread footing, duplicating the existing south pier foundation, is the preferred option, considering costs.

The use of caisson or micropile foundations for the south pier are other feasible options but are considered less economical, especially the micropile option.

For the north pier, the use of spread footing foundation is a feasible option, duplicating the existing bridge foundation support. However, as the existing footings appear to be extend a considerable depth below the bedrock surface, deep bedrock excavation immediately beside the existing pier foundation may not be favourable option. Therefore, due to this and the reasons cited before, consideration can be given to a caisson foundation option at the north pier location.

5.1.3 Horizontal Resistance of Deep Foundations

According to the GA drawing provided to us, the anticipated pile top elevation elevations at the south abutment location is 182.0m and at the north abutment location it is 182.7m. Reference may be made to Section C6-8.7.1 of the Canadian Highway Bridge Design Code S6-06, for assessing lateral pile resistances for driven steel piles. In this instance however, as integral abutments are likely to be utilized, lateral resistance consideration will not be necessary for abutment support. The following paragraphs are provided for the sake of completeness only for abutments and for pier resistance.

In cohesionless soils, the ultimate coefficient of horizontal subgrade reaction can be estimated from:

$k_s=n_h z/d$

Where k_s = coefficient of horizontal subgrade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 5.1. 3.1.

Also as presented in the same table are estimated values for ultimate angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given. In this case,

$k_s=67\ c_u/d$

Where k_s = coefficient of horizontal subgrade reaction

c_u = undrained shear strength

d = width of pile

Table 5.1.3.1 Anticipated n_h and c_u Values

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (kN/m^3)	Recommended Undrained Shear Strength, c_u (kPa)	Groundwater Elevation (m)
South Abutment BH9	182.0-180.3 180.3-178.0 178.0-176.2 176.2-175.8	embankment fill, dense embankment fill, compact embankment fill, compact silty sand, v. dense	20.5 19.5 19.5 20.5	31 30 30 32	9000 6600 4000 11000	- - - -	178.0*
South Abutment BH10	182.0-181.2 181.2-179.7 197.9-178.7 198.7-176.8 176.8-176.3	embankment fill, v. dense embankment fill, compact embankment fill, loose embankment fill, compact embankment fill, v. dense	20.0 19.5 18.5 19.5 20.5	32 30 29 30 32	15000 6600 2200 4000 10000	- - - - -	178.5*
South Abutment BH11	182.0-178.0 178.0-176.5 176.5-176.0	embankment fill, compact to dense embankment fill, compact embankment fill, v. dense	19.5 19.0 20.0	31 30 32	5000 4000 10000	- - -	178.0*
South Abutment BH12	182.0-179.0 179.0-177.0 177.0-176.3	embankment fill, v. loose to compact embankment fill, compact	18.5 19.0	28 29	1500 3000	- -	179.0

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (kN/m^3)	Recommended Undrained Shear Strength, c_u (kPa)	Groundwater Elevation (m)
		silty sand, dense to v. dense	19.5	31	8000	-	
North Abutment BH17	182.7-180.5 180.5-179.6	embankment fill, compact embankment fill, dense	19.5 20.0	30 31	6000 8000	- -	180.5*
North Abutment BH18	182.7-181.7 181.7-178.9 178.9-178.5	embankment fill, loose to compact embankment fill, loose to compact sand	19.0 19.0 19.5	30 30 31	3000 2000 4000	- - -	181.7
North Abutment BH19	182.7-180.0 180.0-178.7 178.7-178.6	embankment fill, compact to dense embankment fill, dense topsoil	20.0 20.0 17.0	31 31 26	8000 9000 1200	- - -	180.0*
North Abutment BH20	182.7-180.0 180.0-177.7 177.7-177.3	embankment fill, compact to dense embankment fill, compact fine sand	20.0 20.0 19.0	31 30 28	8000 4000 2000	- - -	180.0

* Estimated

For preliminary estimating, the following horizontal resistances can be used for HP 310 x 110 and HP 310 x 79 steel H-piles, respectively:

Factored Horizontal Resistance at U.L.S. = 110 kN/pile; 100 kN/pile

Horizontal Resistance at S.L.S. = 40 kN/pile; 35 kN/pile

These values are for an embedded pile length greater than 5m and the SLS condition is based on a horizontal deformation of 10mm.

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone.

MTO structural office requirements (Report SO-96-01) indicate that the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and filling with uniform sand. A special provision should be included in the contract specifying the supply and installation of the CSP's, including the gradation of the sand. The special provision is given in Appendix H; the required gradation of the uniform sand is presented in the following Table.

Table 5.1.3.2 Sand Gradation required for the Flex Zone

Sieve Size	Percentage Passing
2 mm	100 %
600 µm	80-100 %
425 µm	40-80 %
250 µm	4-25 %
150 µm	0-6 %

For the determination of horizontal resistance of caisson foundations, the sliding resistance between the concrete caisson and underlying clean bedrock surface can be utilized. As discussed in Section 5.1.1 of this report, the interface friction factor (ultimate) between the underside of concrete and the clean bedrock surface can be taken as 0.6. This is an ultimate value and some sliding may be necessary to fully mobilize it.

Horizontal shear resistance can also be provided by keying in the caisson into the bedrock and utilizing the shear resistance of the bedrock in front.

Another alternative would be using dowels and utilizing the shear in grouted dowels/rock anchors.

If there is overburden in front of the caisson, horizontal resistance can be mobilized based on passive resistance. However, mobilizing passive resistance in the overburden requires some movement (i.e. horizontal deformation) and should be used with caution. Furthermore, resistance from the overburden and the bedrock should not be added to each other, as they would require differing degrees of deformation. Hence, only one should be utilized in assessing horizontal resistance. In any event, caissons will unlikely be used to support abutments while at the pier locations little or no overburden is anticipated (i.e. probable exposed bedrock).

Using a conservative approach, the ultimate lateral resistance of the caissons socketed into the granite gneiss bedrock can be determined from the following expression:

$$P_u = 3\ B\ c\ z$$

Where P_u = the net ultimate lateral resistance

z = penetration depth of caisson (ignore top 0.3 m of bedrock)

B = diameter of caisson (0.76 m)

c = assume 1000 kPa for the bedrock (a conservative value)

Using this expression, a lateral resistance of 2280 kN can be obtained for a 0.76 m diameter caisson with 1.3 m penetration into the bedrock (ignoring top 0.3 m). Resistance factor 0.5 should be applied to the above expression to determine the factored lateral resistance value at ULS.

The horizontal resistance of micropiles depends on the type, size, installation method, etc. of the micropile used. We will be pleased to further discuss this, if the details of the micropiles are known. In our opinion, however, the use of micropiles for this project is a remote possibility, based on cost factor.

5.2 Lateral Earth Pressures

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150 and OPSD 3101.200.

Free-draining backfill materials (i.e. Granular ‘A’ or Granular ‘B’ Type I or Type II, with minus 0.075mm sieve size material not exceeding 5%) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C. For design purposes, the following static parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, ϕ = 35° (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

K_a = 0.27 K_b = 0.35

K_o = 0.43 K* = 0.45

Compacted Granular 'B' Type I

Angle of Internal Friction, ϕ = 32° (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

K_a = 0.31 K_b = 0.41

K_o = 0.47 K* = 0.57

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure.

K* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding (e.g. when supported on bedrock as is the case for this project), then at rest pressures should be used in accordance with Canadian Highway Bridge Design Code (CHBDC S6-06). The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the CHBDC Commentary can be consulted. K* is typically used when the retaining structure is supported on unyielding foundations, such as spread footings on bedrock. We recommend that where the lateral yield of the retaining structure may render the use of active soil pressure (i.e. the use of K_a may be possible), the intermediate pressure coefficient K_b be adopted to allow for future changes in the pressure distribution due to vibrations induced by the highway traffic.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

5.3 Seismic Design

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-00). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S, for the site is 1.0. Table A3.1.1 of the CHBDC provides that the general area has a Zonal

Acceleration Ratio of 0.05 and Velocity Related Seismic Zone (Z_v) of zero. As site coefficient (S) is 1.0, and the zonal acceleration is 0.05, the design zonal acceleration ratio for the site can be taken as A=0.05. This bridge site can be classified as Seismic Performance Zone 1 or 2 based on the above values and the intended use (e.g. lifeline structure designation or not). Subsection 4.4.5.3 and Table 4.2 of the CHBDC indicate that seismic analysis is not required for bridges in Seismic Performance Zone 1. These should be reviewed by the Structural Engineer.

5.3.1 Seismic Earth Pressures

If required, seismic (earthquake) loading (earth pressure) should be taken into account in the design in accordance with Section 4.6 of the CHBDC.

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as k_h=0.05. The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration coefficient, k_v. Three discrete values of vertical acceleration coefficient are typically selected analysis, corresponding to k_v = +2/3 k_h, k_v = 0, and k_v = -2/3 k_h.

The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h, and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

Table 5.3.1.1
Seismic Active Pressure Coefficients

Active Earth Pressure Coefficient	Granular 'A' (ϕ = 35° - unfactored)	Granular 'B' Type II (ϕ = 32° - unfactored)
Non-Seismic, K_a	0.27	0.31
Seismic, K_{AE}	0.28	0.32

In the calculation of K_{AE}, friction between the wall and the soil was considered δ=0.5 x ϕ.

5.3.2 Liquefaction Potential

If the proposed structures are supported on deep foundations (driven piles, caissons, or micropiles) or spread footings founded in/on the sound bedrock, the foundation materials are considered not liquefiable.

5.4 Approach Embankments

The existing Hwy. 400 southbound lanes embankment will be widened to accommodate the realigned northbound traffic. The final embankment grade at the widening will match the existing southbound lanes embankment grade. The widening is expected to entail less than 2m grade raise. This is because, as shown in the photographs presented in Appendix C, the median space between the existing northbound and southbound embankments was filled during the 1990's construction of the southbound embankments. The existing grade difference between the northbound and southbound road levels is generally about 3m (northbound being lower) and thus the grade raise immediately adjacent to the existing southbound lanes embankment should typically be between 1 and 2 m.

Grade raises of this magnitude are not expected to cause a foundation failure. They will however cause some settlements. Assuming that the embankments along the median and immediately adjacent to the southbound lanes median side shoulder were properly constructed (i.e. using suitable materials on which a systematic compaction was applied) and based on the available borehole data, settlements due to a 2m grade raise should not exceed 30mm. In addition, since the southbound and median embankments were build some two decades ago, any foundation settlements and settlements of the existing embankments due to their own weight should have been substantially completed. For these reasons, settlements due to about 2m grade raise should not cause major problems (i.e. up to about 30mm settlement), especially since some of this settlement will take place during the construction period. We recommend however, as a precaution against material and degree of compaction differences in the existing embankment fill (which would cause differential settlements), the paving of the new lanes be delayed by about three weeks after the grade is raised to the base level of the pavement, if possible.

For embankment construction, the existing grade should be stripped of all vegetation, topsoil and of any other unsuitable materials.

After stripping, the exposed subgrade should be inspected and approved. After approval, the approved subgrade should be properly compacted from the surface, using a suitably heavy compactor, in the presence of geotechnical personnel. If weak or unsuitable zones become evident during this process, the unsuitable materials should be removed and replaced with suitable soils.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized 2H:1V side slopes can be used for the construction of the approach fills, provided that the founding subgrade is prepared as discussed earlier in this section. Proper erosion control measures should be implemented by prompt seed and cover (OPSS 803) and sodding (OPSS804).

The existing embankment side slopes should be properly benched as per MTO standard (OPSD 208.010) where the embankment widening is proposed.

The material used for the construction of the embankment fills should consist of approved, acceptable earth fill (eg. Selected Subgrade Materials – OPSS 1010). Fill used for construction of the embankment should be in accordance with OPSS 212 and fill placement should meet or exceed the requirement of OPSS 501 and OPSS 206. Construction should be in accordance with SP206S03. Quality assurance should be provided as per MTO standard 501.08 (OPSS 501).

From the drawings available to us, the forward slopes will not require any new filling, except for possible removal of some excess material. Boreholes drilled closet to the existing and proposed forward slopes show, below the embankment fill, the presence of some native soils of limited thickness, underlain by bedrock. On the south side, the surface of the bedrock appears to be essentially level, but on the north side, the bedrock surface appears to be sloping and the fill and native soil overlying bedrock were found to be of very loose to compact relative density. These are not favourable conditions and may lead to a slope failure towards the channel, by sliding over the bedrock surface. However, based on the fact that the existing forward slopes are stable and from the borehole data, the surface of the rock appears to be dipping mildly (i.e. about 2 m over a horizontal distance of about 32 m), a sliding type failure is an unlikely scenario and thus we do not envisage a slope stability problem with the current design.

5.5 Construction Comments

All excavations, shoring and backfilling should be carried out in conformance with the Occupational Health and Safety Act (OHSA) 213/91, as well as the following specifications.

- OPSS 539 Construction Specification for Temporary Protection System

- OPSS 902 Construction Specification for Excavation and Backfilling

The boreholes show that the excavations in the overburden can be expected to extend through fill material at some locations, to the surface of the bedrock, while at other locations, the fill is underlain by some basal native overburden immediately above the bedrock. The fill generally consists of silty sand to sandy silt with traces to some clay and gravel. The composition of the native overburden at the borehole locations was found to range from silty sand to sand. Silty clay and gravel till were not contacted at the borehole locations, but these deposits are known to exist in the general area. These soils can be classified as follows:

Granular Pavement Fill	Type 3 soil
Topsoil (overlying embankment fill)	Type 3 soil
Embankment Fill	Type 3 soil above groundwater table
(typically silty sand to sandy silt	Type 4 soil below groundwater table
with traces to some clay and gravel)	
Silty Sand to Sand	Type 4 soil
Topsoil (beneath fill, overlying bedrock)	Type 4 soil
Glacial Till (dense to very dense)	Type 2 soil above groundwater table
	Type 4 soil below groundwater table
Silty Clay (stiff to hard)	Type 3 soil above groundwater table
	Type 4 soil below groundwater table
Silty Clay (very soft to firm)	Type 4 soil

The south pier foundation is expected to be supported on the bedrock. Therefore, cofferdam will be required to facilitate the rock excavation and to enable inspections to verify the condition of the bedrock, as well as to facilitate mass concrete pour to raise the grade to the underside of the proposed footing (if needed) and for the construction of the pier footing.

Drilled cast-in-place concrete piles (caissons) or spread footings extending into the bedrock are expected to be used at the north pier location.

Bedrock was contacted in Boreholes 13 and 14, located near the proposed south pier location, while in Boreholes 15 and 16 near the north pier location some fill underlain by 0.1 to 0.8m thick native overburden was encountered. Hence it is expected that at least some excavation will be carried out in the overburden below the water level in the Channel. The severity of the unwatering can possibly be reduced by regulating the level of the water (i.e. lowering) in the Channel by means of the existing upstream control structure. Regardless, however, some sort of cofferdam will be required to prepare the foundations on the bedrock, for concrete pour, etc. Tight interlocking steel sheet piling extending to the surface of the bedrock can be considered. This may however not provide a sufficiently tight enclosure (especially if the rock surface is not level) and may need to be sealed with tremie around base.

Sand bagging and pumping from within a cofferdam enclosure can also be considered. There are also other methods used by some contractors such as plastic bladder enclosure, etc. to provide easier working environment within the Channel. These decisions are however generally left to the discretion of the Contractor.

Some dewatering will also be required to facilitate stripping and the construction of the new embankment fills during rainy periods and this can normally consist of gravity drainage and pumping from strategically placed sumps.

Shoring will be required to construct the new abutments (abutting into the existing abutments) and the approach fills.

In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing/rakers). In this instance, the use of tiebacks may also be required. The soldier piles can be expected to extend into the bedrock. Tiebacks would extend, through the fill and some shallow overburden, into the bedrock. Tiebacks should be assumed to derive their resistance from the bedrock only (i.e. resistance from the overburden should be ignored). For preliminary design purposes, the factored rock/grout bond resistance at U.L.S. can be taken as 800 kPa and resistance at S.L.S. need not be considered.

The shoring system should be designed so that the lateral movement of any portion of the shoring system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work. As mentioned before all shoring should be in accordance with OPSS 539.

Table 5.5.1 Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	Ka	Ko	Kp	γ (kN/m3)
Granular Embankment Fill	0.32	0.49	3.1	21.0
Embankment Fill (typical)	0.36	0.53	2.8	20.0
Silty Sand/Sand	0.33	0.50	3.0	19.0
Bedrock	0.20	0.40	5.0	24.0

It should be pointed out that the presence of cobbles and boulders can be expected within the fill and the overburden, immediately above the bedrock. The presence of cobbles and especially boulders may present problems, if encountered, during the installation of driven piles as well as caissons and shoring. As well, their removal may present some difficulties during excavation and may lead to claims for extrs by the Contractor. We recommend that the possible presence of cobbles and boulders in the fill or in the natural overburden, as well as the presence of rock fill be ‘red-flagged’ in the Contract Documents.

It is recommended that the vibrations should be monitored during the installation of piles, caissons or for spread footing foundations (if rock excavation may include percussion type rock penetration or other methods causing vibration), and demolition of the existing structure. Special provision for vibration monitoring is given in Appendix H. An NSSP should be issued in this respect.

5.6 Frost Protection

Design frost protection depth for the general area is 1.6m. Therefore, a permanent soil cover of 1.6m or its thermal equivalent of artificial insulation is required for frost protection of foundations, placed on overburden or shattered/fractured rock. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

The Limitations of Report, as quoted in Appendix I, are an integral part of this report.

For and on behalf of Coffey.

Gwangha Roh, P.Eng., Ph. D.

Senior Geotechnical Engineer

Zuhtu Ozden, P.Eng.

Senior Principal

Appendix F

GA Drawings

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

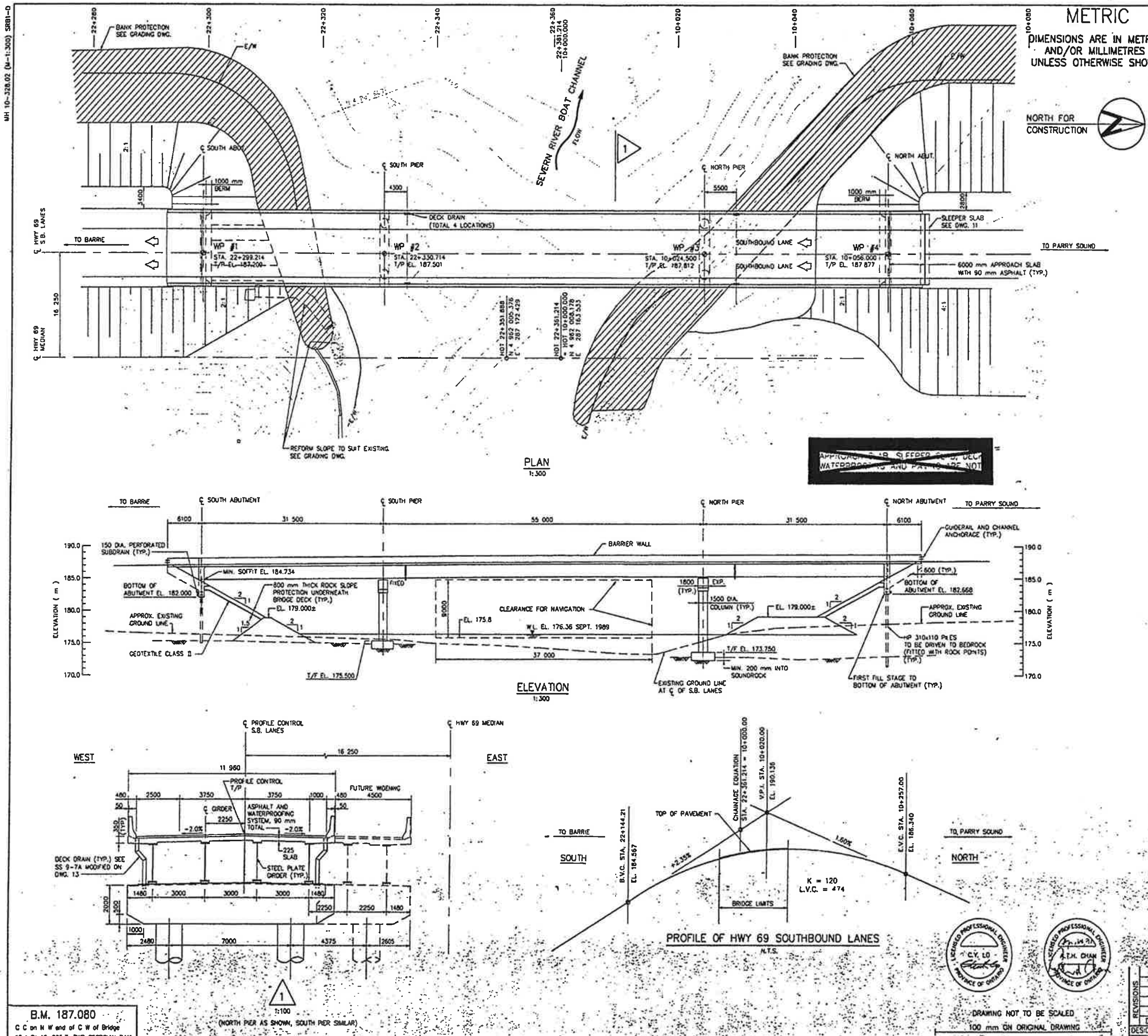
DIST. No. 5
CONT. No. 92-63
WP. No. 37-80-05

SEVERN RIVER BOAT CHANNEL BRIDGE
HIGHWAY 69 – SOUTHBOUND LANES
GENERAL ARRANGEMENT

SHEET J51C2

NORTH FOR CONSTRUCTION

Morrison Hershfield Limited
Consulting Engineers



B.M. 187.080
C C on N W end of C W of Bridge
10.4 Rt 10+035.3, TWP GEORGIAN BAY

GENERAL NOTES

1. CLASS OF CONCRETE

ALL CONCRETE	30 MPa

2. CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100±25
PIER COLUMNS AND CAP BEAMS	80±20
ABUTMENTS AND WINGWALLS	
FRONT FACE	80±20
BACK FACE	70±20
DECK SLAB	
TOP SURFACES	70±20
BOTTOM SURFACES	40±10
REMAINDER	70±20 UNLESS OTHERWISE NOTED

3. REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS

4. CONSTRUCTION NOTES:

IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCED STEEL TO SUIT THE ACTUAL HEIGHTS.

COMPACTED FILL, MAXIMUM GRAIN SIZE 75 mm SHALL BE PLACED UP TO THE BOTTOM OF THE ABUTMENT WALL FOOTING ELEVATION PRIOR TO DRIVING PILES.

PRE-AUGER PILE HOLES TO BEDROCK LEVEL. BACKFILL HOLE WITH MTO CLASS 4 AGGREGATE PRIOR TO DRIVING PILES.

BACKFILL BEHIND ABUTMENTS TO BE PLACED SIMULTANEOUSLY. THE DIFFERENCE IN THE LEVELS OF BACKFILL SHALL NOT EXCEED 500 mm.

FOOTINGS SHALL BE SET 200 mm INTO SOUND BEDROCK.

ROCK SURFACES IN OVER-EXCAVATED AREAS SHALL BE SUBJECT TO APPROVAL BY THE ENGINEER.

OVER-EXCAVATION SHALL BE REPLACED WITH CONCRETE OF SAME CLASS AS FOOTING CONCRETE.

5. CONSTRUCTION SEQUENCE:

DECK CONCRETE POUR SHALL FOLLOW THE SEQUENCE AS STATED ON DRAWING 6.

UPPER PORTION OF THE ABUTMENTS AND THE DECK ARE CAST INTEGRALLY AT THE GIRDER LOCATIONS IN ONE POUR.

ROCK BERMS AS SPECIFIED ON GRADING DRAWINGS SHALL BE IN PLACE BEFORE ABUTMENT CONSTRUCTION.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATA
3. FOOTING LAYOUT AND DETAILS
4. ABUTMENTS AND WINGWALLS
5. PIERS AND BEARING DETAILS
6. DECK LAYOUT AND SCREED ELEVATIONS
7. DECK REINFORCING DETAILS
8. STRUCTURAL STEEL – I
9. STRUCTURAL STEEL – II
10. BARRIER WALLS
11. 6000 mm APPROACH SLAB
12. AS CONSTRUCTED ELEVATION AND DIMENSION
13. STANDARD DETAILS
14. QUANTITIES STRUCTURE – I
15. QUANTITIES STRUCTURE – II

LEGEND

T/F	TOP OF FOOTING
T/C	TOP OF CONCRETE
T/P	TOP OF PAVEMENT
WP	WORKING POINT
E/W	EDGE OF WATER
W.L.	WATER LEVEL

APPLICABLE STANDARD DRAWINGS

DD-3503 MINIMUM GRANULAR BACKFILL REQUIREMENTS (MODIFIED, SEE DWG. 13)

REVISIONS		
DATE	BY	DESCRIPTION

DESIGN KL	CHK. CL	CODE OHBDC-83	LOAD CLASS A	DATE	FEB./91
DRAWN HT	CHK. ATC	SITE 42-675	STRUCT.	SCHEME	DWG. 1

DISTRICT
CONT. No.
WP No. 2376-09-00

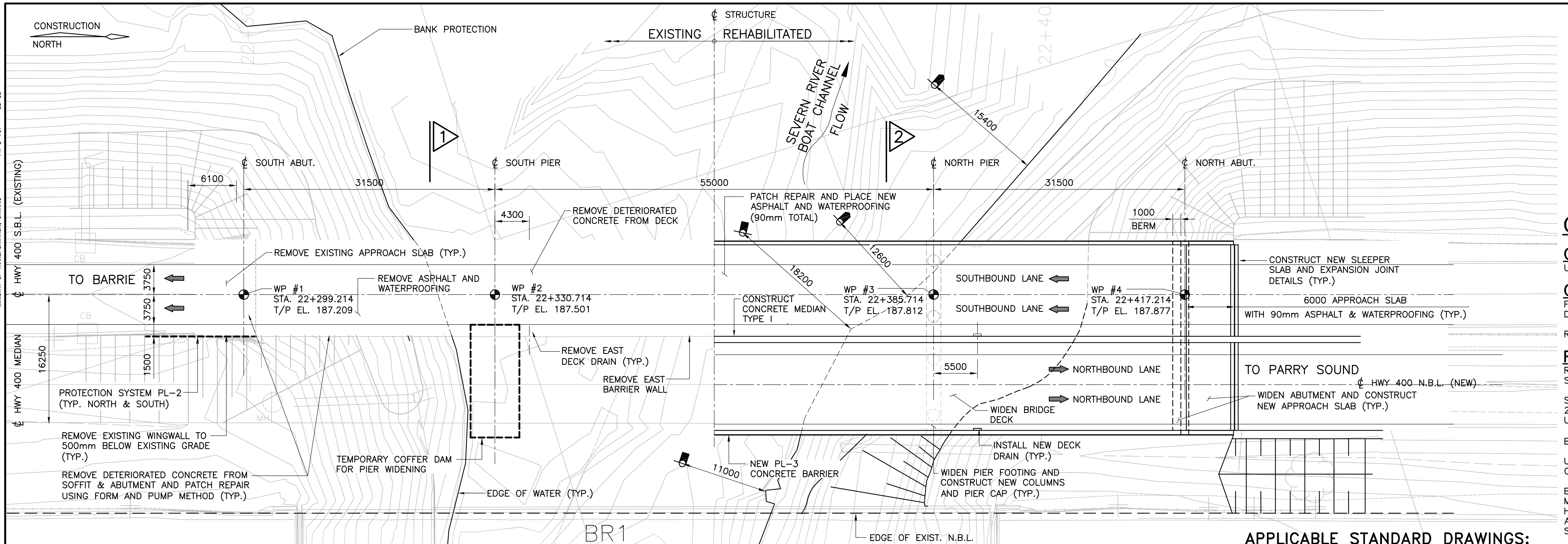
HIGHWAY 400 NBL & SBL
SEVERN RIVER BOAT CHANNEL BRIDGE
BRIDGE REHABILITATION

GENERAL ARRANGEMENT

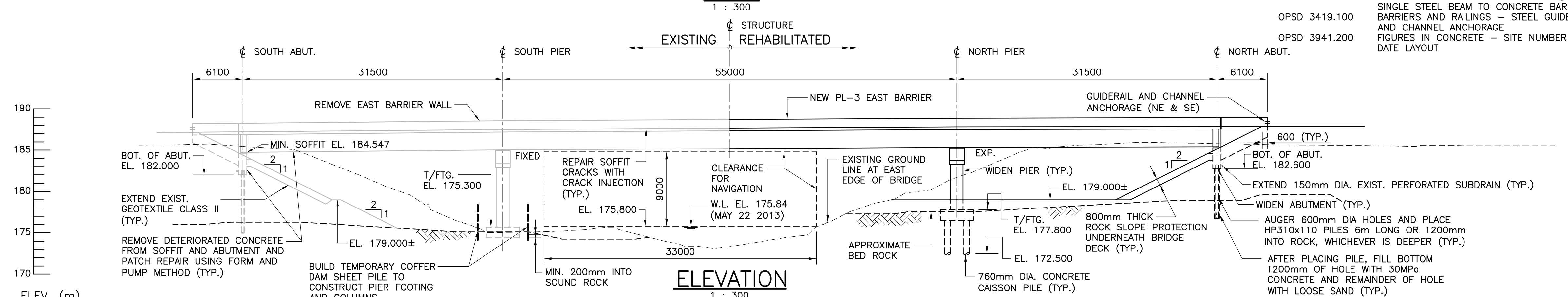
SHEET 21

METRIC

McCORMICK RANKIN
A member of MMM GROUP



PLAN
1 : 300



ELEVATION
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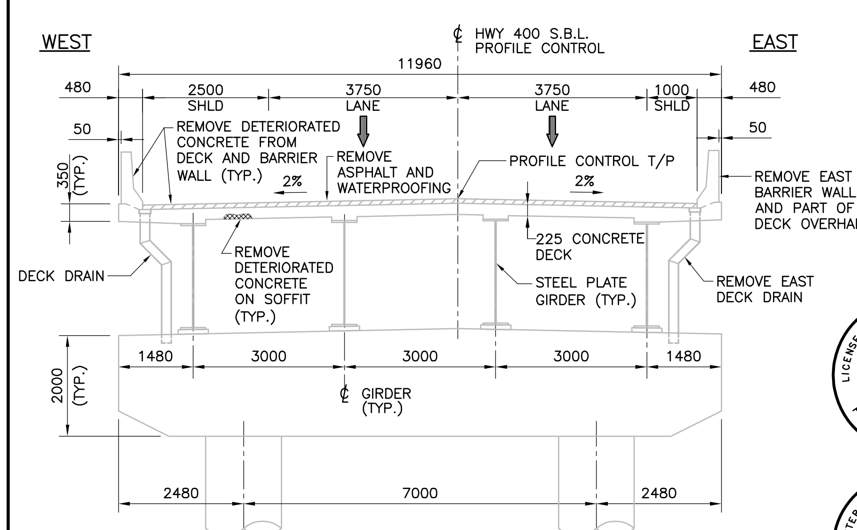
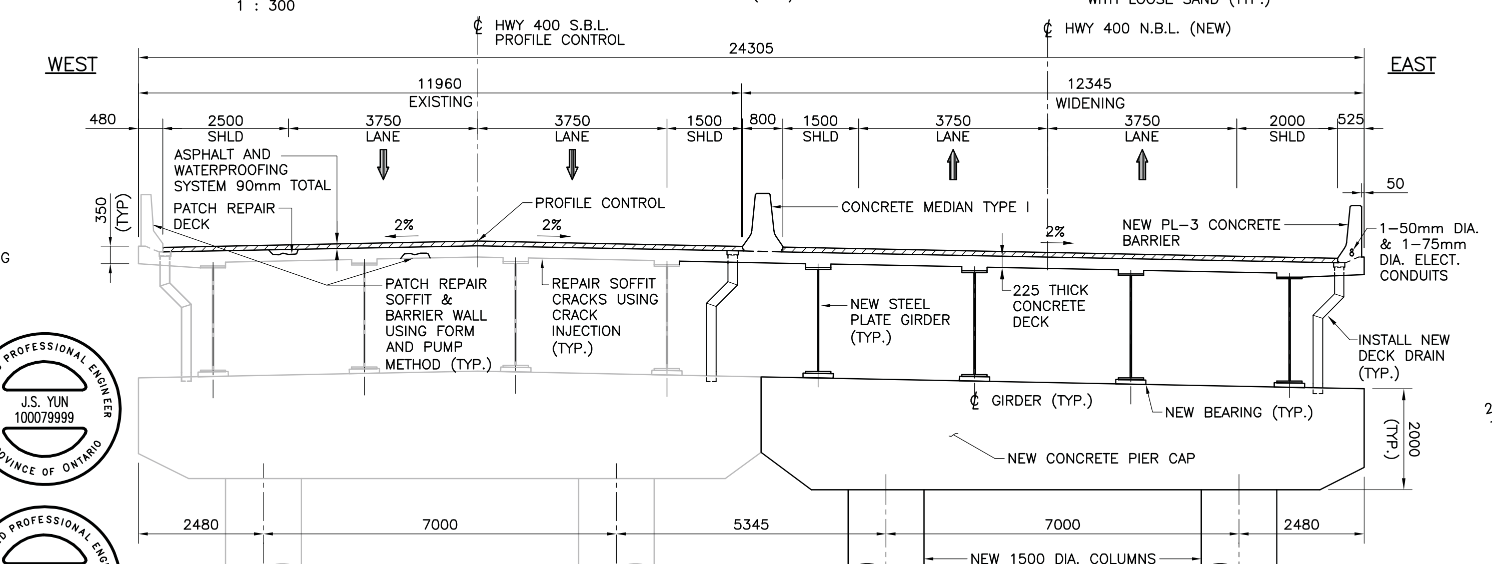


1 EXISTING
1 : 75

LICENSED PROFESSIONAL ENGINEER
J.S. YUN
100079999
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
B. R. BRIDGES
PROVINCE OF ONTARIO



2 REHABILITATED
1 : 75

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

GENERAL NOTES

CLASS OF CONCRETE:

UNLESS OTHERWISE NOTED 30MPa

CLEAR COVER TO REINFORCING STEEL:

FOOTINGS		00 ± 25
DECK	TOP	70 ± 20
	BOTTOM	40 ± 10
REMAINDER		70 ± 20 UNLESS OTHERWISE NOTED

REINFORCING STEEL:

REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.

STAINLESS REINFORCING STEEL SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa, UNLESS OTHERWISE SPECIFIED.

BAR MARKS WITH PREFIX 'S' DENOTE STAINLESS STEEL BARS.

UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.

BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWINGS SS12-1 AND SS12-2, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, DETAILS AND ELEVATIONS OF EXISTING STRUCTURE THAT ARE RELEVANT TO THE WORK SHOWN ON THE DRAWINGS PRIOR TO COMMENCEMENT OF WORK. ANY DISCREPANCIES SHALL BE REPORTED TO THE CONTRACT ADMINISTRATOR AND THE PROPOSED ADJUSTMENT OF THE WORK REQUIRED TO MATCH THE EXISTING STRUCTURE SHALL BE SUBMITTED FOR APPROVAL.

SAWCUTS WHERE INDICATED SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL WHICHEVER IS LESS.

FOR TRAFFIC STAGING AND MAINTENANCE OF TRAFFIC SEE CONSTRUCTION STAGING DRAWINGS.

PROTECTION SYSTEMS REQUIRED TO COMPLETE THE WORK SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA BY CONTRACTOR. LIMITS OF PROTECTION SYSTEM TO BE DETERMINED BY CONTRACTOR. PROTECTION SYSTEMS SHALL BE SUFFICIENT FOR ALL ACCESS AND WORKING PLATFORMS.

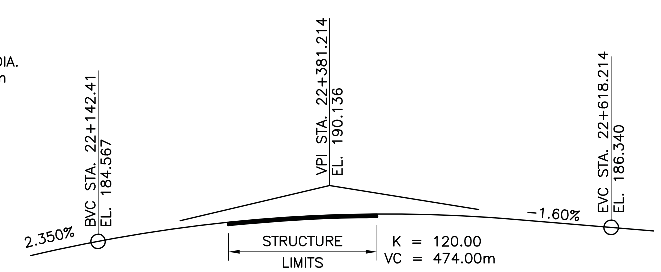
BACKFILL SHOULD NOT BE PLACED UNTIL THE DECK HAS REACHED 75% OF ITS SPECIFIED STRENGTH. BACKFILL SHOULD BE PLACED SIMULTANEOUSLY AT BOTH ENDS OF THE STRUCTURE KEEPING THE HEIGHT OF BACKFILL THE SAME. AT NO TIME SHALL THE DIFFERENCE IN HEIGHT OF BACKFILL BE GREATER THAN 500mm.

APPLICABLE STANDARD DRAWINGS:

OPSD 912.4800	GUIDE RAIL SYSTEM, CONCRETE BARRIER, PERMANENT CONNECTION INSTALLATION, SINGLE STEEL BEAM TO CONCRETE BARRIER
OPSD 3419.100	BARRIERS AND RAILINGS – STEEL GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD 3941.200	FIGURES IN CONCRETE – SITE NUMBER AND DATE LAYOUT

LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGING



PROFILE OF HWY 400 S.B.L.
N.T.S.

REVISIONS					
	DESCRIPTION				
DESIGN AY	CHK BB	CODE CHBDC 06	LOAD CL-625-ONT	DATE NOV/13	
DRAWN CA	CHK AY	SITE 42-87/1&2	STRUCT	SCHEME	DWG 1

MINISTRY OF TRANSPORTATION, ONTARIO PR-D-707 88-05

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DATE PLOTTED: 11/5/2013 9:20:45 AM BY: RICHIE RAPISTA

S:\2012\32\3212093\310-2 BOAT CHANNEL SBL\3212093-310-001GA.dwg, 11/5/2013 9:20:02 AM, Bluebeam PDF

Appendix G

Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Table G-1

Foundation Options for Severn River Boat Channel Bridge Widening - Abutments

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	-Will not allow integral abutment design -Dewatering and unwatering required -Will require extensive shoring	-Greater shoring effort will be needed in comparison with driven steel H-pile, caisson and micropile options -Will not match existing foundations -Sloping bedrock condition observed during the existing SBL bridge construction which may cause problems during foundation construction	Medium	-Feasible but not recommended -Temporary support system is required
Driven H-pile foundations	-Will reduce shoring effort in comparison with shallow foundations -No dewatering is required -Pre-augering into bedrock will be required -Feasible for integral abutment design -Matches the existing foundations	-Possible presence of rock fill or boulders -Pre-augering into bedrock is required which will increase cost	Medium	-Recommended option considering cost
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-Will reduce shoring effort in comparison with shallow foundations -not suitable for integral abutment design -feasible only for semi-integral abutment design	-Possible presence of rock fill or boulders -Socketing into bedrock will be difficult	High to Medium	-Can be considered but not recommended based on cost
Micropiles	-May reduce shoring effort -Equipment easier to operate under low overhead and restricted access conditions -Can be installed through mass concrete if encountered	-Rock fill, if encountered, may create problems during installation but to a lesser extent than caisson option	Higher in comparison with other options	-Not recommended based on cost consideration

Table G-2

Foundation Options for Severn River Boat Channel Bridge Widening – South Pier

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	-Lower cost than other options -Will match the existing bridge supports -Dewatering and unwatering required	-Possible sloping bedrock surface	Low	-Feasible -Recommended option
Driven H-pile foundations	-Technically not feasible due to high bedrock surface		Not feasible	Not feasible
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-More costly than shallow foundations	-Socketing into bedrock will be difficult -Possible sloping bedrock surface will render construction of caisson difficult and will increase required caisson depths	Medium to high	--Not recommended based on cost
Micropiles	-Equipment easier to operate under low overhead and restricted access conditions		Higher in comparison with other options	-Not recommended based on cost

Table G-3

Foundation Options for Severn River Boat Channel Bridge Widening – North Pier

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	-Lower cost than other options -Will match the existing bridge supports -Dewatering and unwatering required -May require some minor shoring	-Sloping bedrock surface	Low	-Feasible
Driven H-pile foundations	-Technically not feasible due to high bedrock surface		Not feasible	Not feasible
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-May reduce shoring effort -More costly than shallow foundations	-Possible presence of rock fill or boulders -Socketing into bedrock will be difficult -Sloping bedrock surface will render construction of caisson difficult and will increase required caisson depths	Medium to high	-Feasible and possibly the best option for the north pier based on risk management viewpoint (i.e. damage to existing foundations)
Micropiles	-May reduce shoring effort -Equipment easier to operate under low overhead and restricted access conditions -Can be installed through mass concrete if encountered	-Rock fill, if encountered, may create problems during installation but to a lesser extent than caisson option	Higher in comparison with other options	-Not recommended based on cost

Appendix H

List of OPSS, OPSD and Non-standard Specifications

List of OPSDs, OPSSs and Non-standard Specifications

OPSDs

OPSD 208.01 Benching of Earth Slopes

OPSSs

OPSS206 - Construction Specification for Grading

OPSS212 - Construction Specification for Borrowing

OPSS 501 - Construction Specification for Compacting

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 803 - Construction Specification for Sodding

OPSS804 - Construction Specification for Seed and Cover

OPSS 903 – Construction Specification for Deep Foundations

OPSS.PROV 1010 – Material Specification for Aggregates-Base, Sub base, Select Subgrade, and Backfill Material

SP

SP206S03 – Earth Excavation, Grading

NSSP Wording

Special Provision

Vibration Monitoring

The vibration monitoring equipment shall be placed on the existing and newly widened structure such that it will not be disturbed. The location should be as close as possible to the construction works.

The vibrations at the existing structure shall not exceed 100 mm/s (peak particle velocity).

The Contractor shall take readings during the construction. The results shall be submitted to the Contract Administrator at the end of each day.

If the readings are not within the limits stated above, the Contractor must alter his/her construction procedures until the vibrations on the existing and newly built structure are within acceptable levels.

<u>CSP FOR INTEGRAL ABUTMENT</u> - Item No.

Special Provision

SCOPE

This specification covers the requirements for the installation of the Corrugated Steel Pipe (CSP), including augering and sand fill at the abutments.

REFERENCES

This specification refers to the following standards, specification or publications:

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Material:

OPSS 1801 Corrugated Steel Pipe Products

Canadian Standards Association Standards:

CSA G164-M Galvanizing of Irregularly-Shaped Articles

Ministry of Transportation Publications:

MTO Manual of Designated Sources of Materials

DEFINITIONS

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

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

SUBMISSION AND DESIGN REQUIREMENTS

Submissions

The Contractor shall submit three (3) sets of the workings drawings to the Contract Administrator at least two (2) weeks prior to the commencement of installation of the CSP for information purposes only. Prior to making a submission, an Engineer's seal and signature shall be affixed on the working drawings verifying that the drawings are consistent with the Contract Documents. Where multi-discipline engineering work is depicted on the same working drawings and a single engineer is unable to seal and sign the working drawings for all aspects of the work, the working drawings shall be sealed and signed by as many additional engineers as necessary.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Location of reference points, and location of the centroid of each pile with respect to the reference points at the level of the bottom face of the abutment stem;
3. Source of the sand fill, and description of placing method and equipment;
4. Location and details of all temporary bracing, for the piles, CSP's, and abutment stems;
5. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of all temporary bracing, required to maintain the piles, CSP's, abutment stems, and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing shall be removed.

MATERIAL

Corrugated Steel Pipe

CSP shall be accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Sand Fill

The sand fill for backfilling in the CSP shall meet the gradation requirements of Table A below:

Table A – Sand Fill Gradation Requirements		
MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 µm	# 30	80 % to 100 %
425 µm	# 40	40 % to 80 %
250 µm	# 60	5 % to 25 %
150 µm	# 100	0 % to 6 %

CONSTRUCTION

General

The sequence of construction for augering and installing the CSP's, sand fill, abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

CSP's shall be supplied in the lengths and with the end treatments, either square or skewed, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall ensure the full perimeters of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.

The gap between the auger holes and the CSP's shall be filled with granular material.

After the CSP's have been set into position, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the CSP and the pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and/or displace the CSP's.

After the sand fill has been placed to the top of each CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and holes filled with non-shrink grout.

Tolerances

The CSP's at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of CSP from pile centroid	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified elevation	± 10 mm

QUALITY ASSURANCE

Prior to augering the holes for installation of CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with, the installation of the abutments stems, has been given by the Contract Administrator.

BASIS OF PAYMENT

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

Appendix I

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

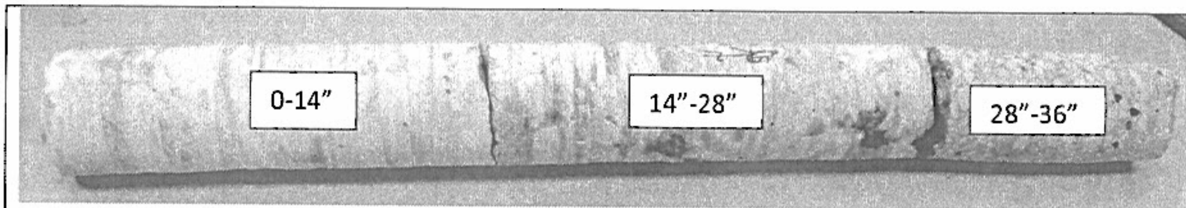
Appendix C
Concrete Test Results
Rock Profile at Test Pit

DRAFT

THURBER

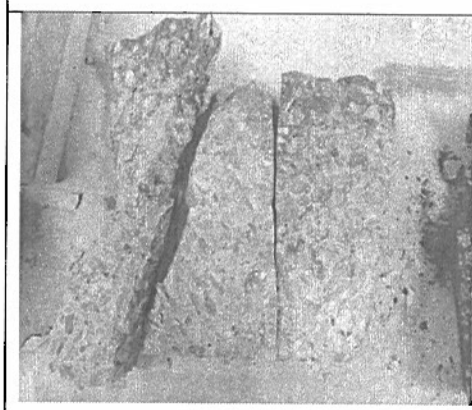
Port Severn Concrete Core Strength Test Results Project No. 19-5161-194

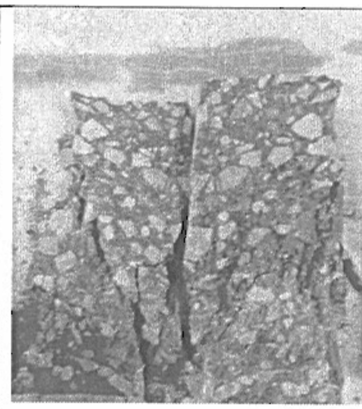
Cores on the south side

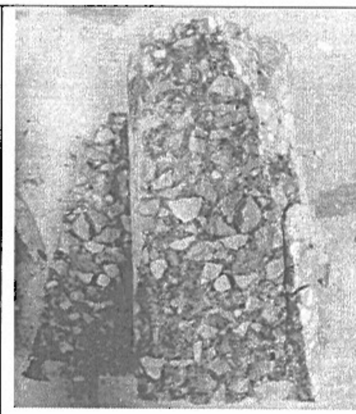


Core #1 – South abutment, 11.6 m o/s from south bound bridge (0-36")

Photos of the concete cores after compressive stength test







Results of concrete compressive strength test:

Core ID	Concrete Diameter (mm)	Concrete Height (mm)	Compressive Strength (MPa)
Core #1A – 0.0 - 0.36m	89.0	178.0	25.7
Core #1B – 0.38 - 0.71m	89.0	178.0	45.0
Core #1C – 0.71 - 0.91 m	89.0	178.0	36.2

Client: MMM
Project: Port Severn Bridge
File: H\19\5161\194\Foundation\Core Test Results

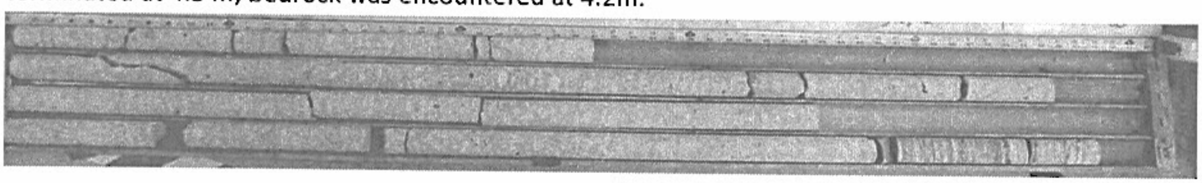
THURBER

Results of concrete compressive strength test:

Core ID	Concrete Diameter (mm)	Concrete Height (mm)	Compressive Strength (MPa)
Core #2A - 0-100 mm	89.0	89.0	32.0
Core #2B- 200-380 mm	89.0	178.0	44.2

Core #1 – 50 mm diameter core

Concrete was cored at 10.4m from southbound bridge, 0.6m from the retaining wall. Core hole was terminated at 4.5 m; bedrock was encountered at 4.2m.



Results of concrete compressive strength test:

Core ID	Concrete Diameter (mm)	Concrete Height (mm)	Compressive Strength (MPa)
Core #1A- 0.2-0.27 m	35.0	70.0	26.4
Core #1B- 0.7-0.77 m	35.0	70.0	33.8
Core #1C- 1.3-1.37 m	35.0	70.0	40.2

Client: MMM
Project: Port Severn Bridge
File: H\19\5161\194\Foundation\Core Test Results

THURBER

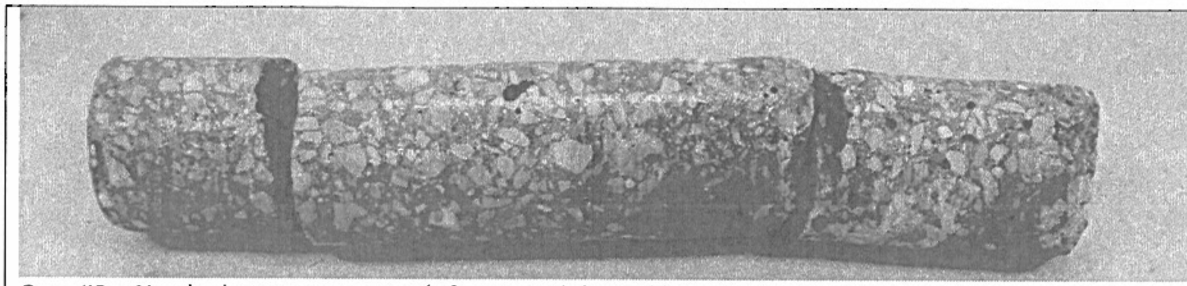
Port Severn Concrete Core Strength Test Results

Project No. 19-5161-194

Cores on the north side

Core #2 – 100 mm diameter core

Concrete was cored at 9.1m from south bound bridge and 0.6m from the retaining wall



Core #2 – North abutment, 9.1 m o/s from south bound bridge (0-24")

Photos of the concete cores after compressive stength test

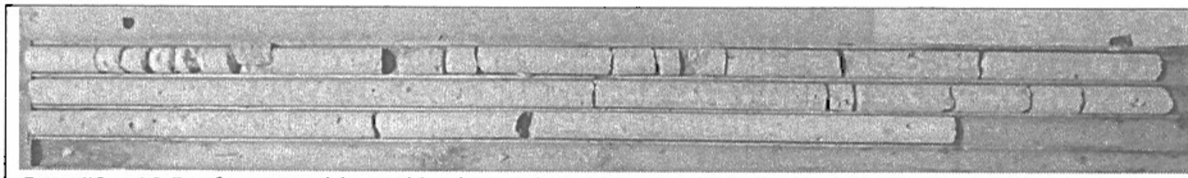
Core 2A – 0-100 mm

Core #2B – 250-430 mm

Client: MMM
Project: Port Severn Bridge
File: H\19\5161\194\Foundation\Core Test Results

THURBER

Core #2 – 50 mm diameter core



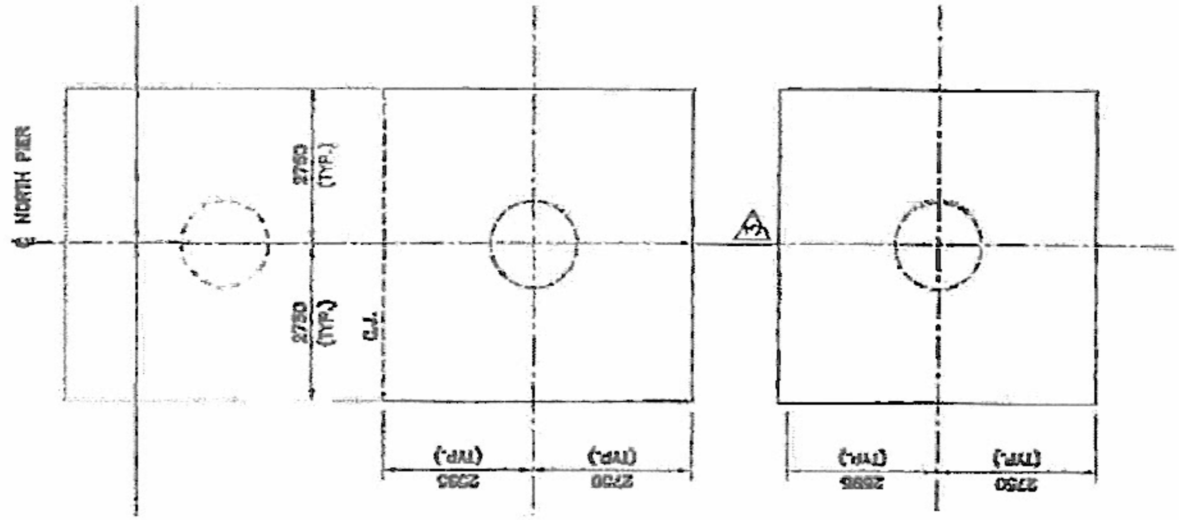
Core #2 – 10.5m from southbound bridge and 0.8 m from the edge
Core hole was terminated at 5.1m. Bedrock was encountered at 4.8m.

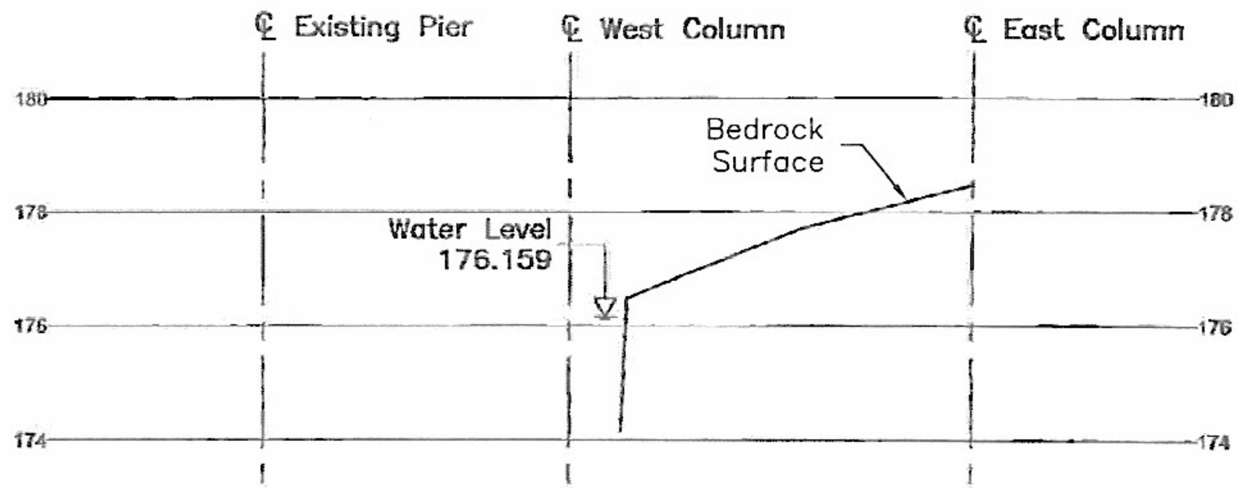
Results of concrete compressive strength test:

Core ID	Concrete Diameter (mm)	Concrete Height (mm)	Compressive Strength (MPa)
Core #2A - 0.4 - 0.47m	35.0	70.0	26.4
Core #2B - 0.6 - 0.67m	35.0	70.0	31.7
Core #2C - 0.97-1.04m	35.0	70.0	37.0

Client: MMM
Project: Port Severn Bridge
File: H\19\5161\194\Foundation\Core Test Results

THURBER





SEVERN RIVER BOAT CHANNEL TEST PIT OBSERVATIONS

(N.T.S. SCHEMATIC ONLY)

SK.1