

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, GEOCREC 31D-564

TRANETOB20462AA

07 January, 2014



When you
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get smaller



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January 07, 2014

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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.

A handwritten signature in blue ink, appearing to read "Zuhtu Ozden", written in a cursive style.

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

CONTENTS

1	INTRODUCTION	1
2	SITE DESCRIPTION AND GEOLOGY	1
3	INVESTIGATION PROCEDURES	2
4	SUBSURFACE CONDITIONS	3
4.1	Asphalt	4
4.2	Topsoil	4
4.3	Pavement and Embankment Fill	4
4.4	Native Overburden	5
4.4.1	Silty Sand, Sand, Gravelly Sand and Sand & Gravel	5
4.4.2	Silty Sand Till	6
4.4.3	Silty Clay	7
4.5	Bedrock	7
4.6	Groundwater Conditions	9

Drawings

Drawing 1: Borehole Location Plan and Stratigraphic Profile

Drawing 2: Stratigraphic Section

Drawing 3: Stratigraphic Section

Appendices

Appendix A: Record of Borehole Sheets

Appendix B: Laboratory Test Results

Appendix C: Site Photographs

Appendix D: Rock Core Photographs and Test Results

Appendix E: Explanation of Terms Used in Report

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%
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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.

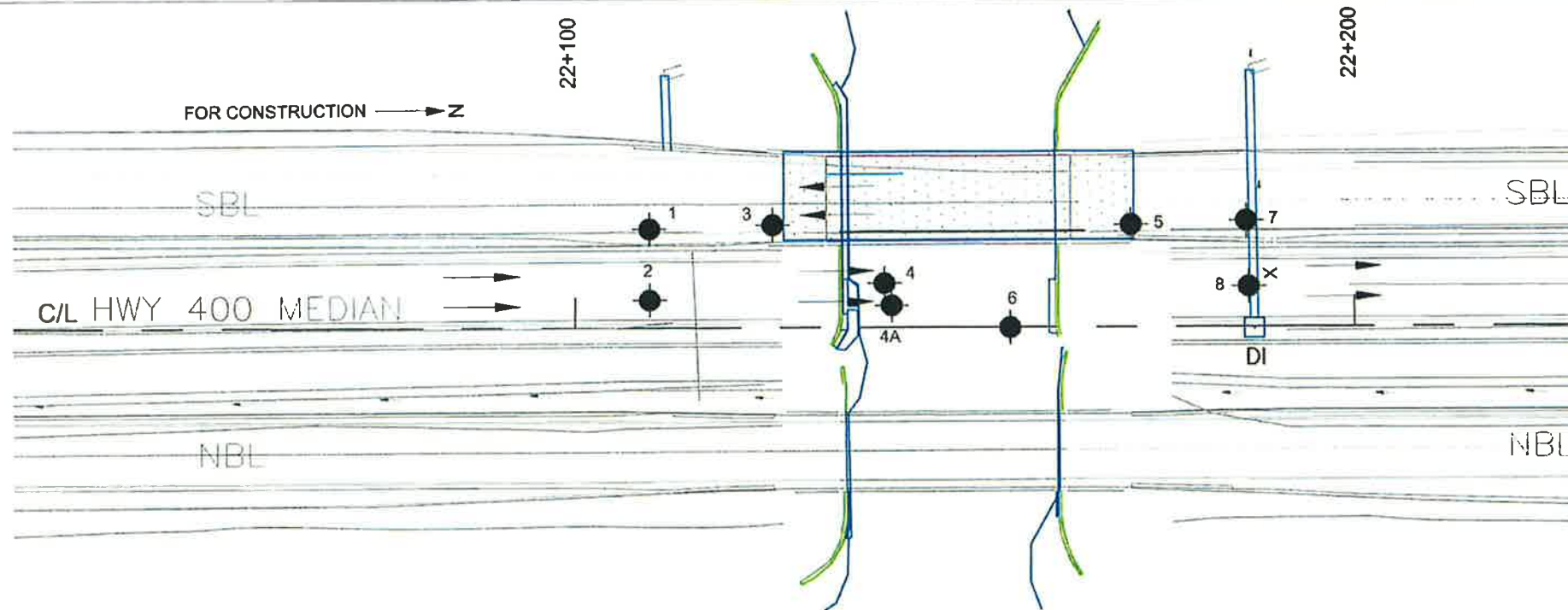

Gwangha Roh, P.Eng., Ph. D.
Senior Geotechnical Engineer




Zuhtu Ozden, P.Eng.
Senior Principal



Drawings



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.

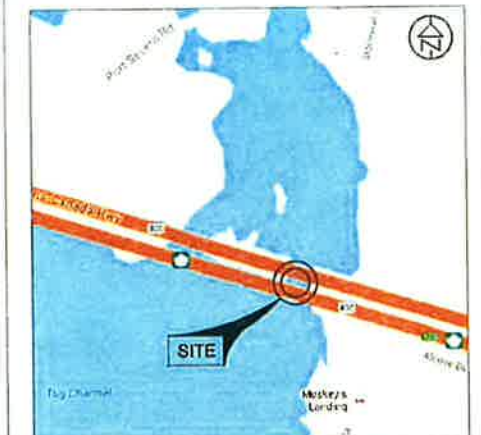
CONT No. -
W.P.: 2360-06-00

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

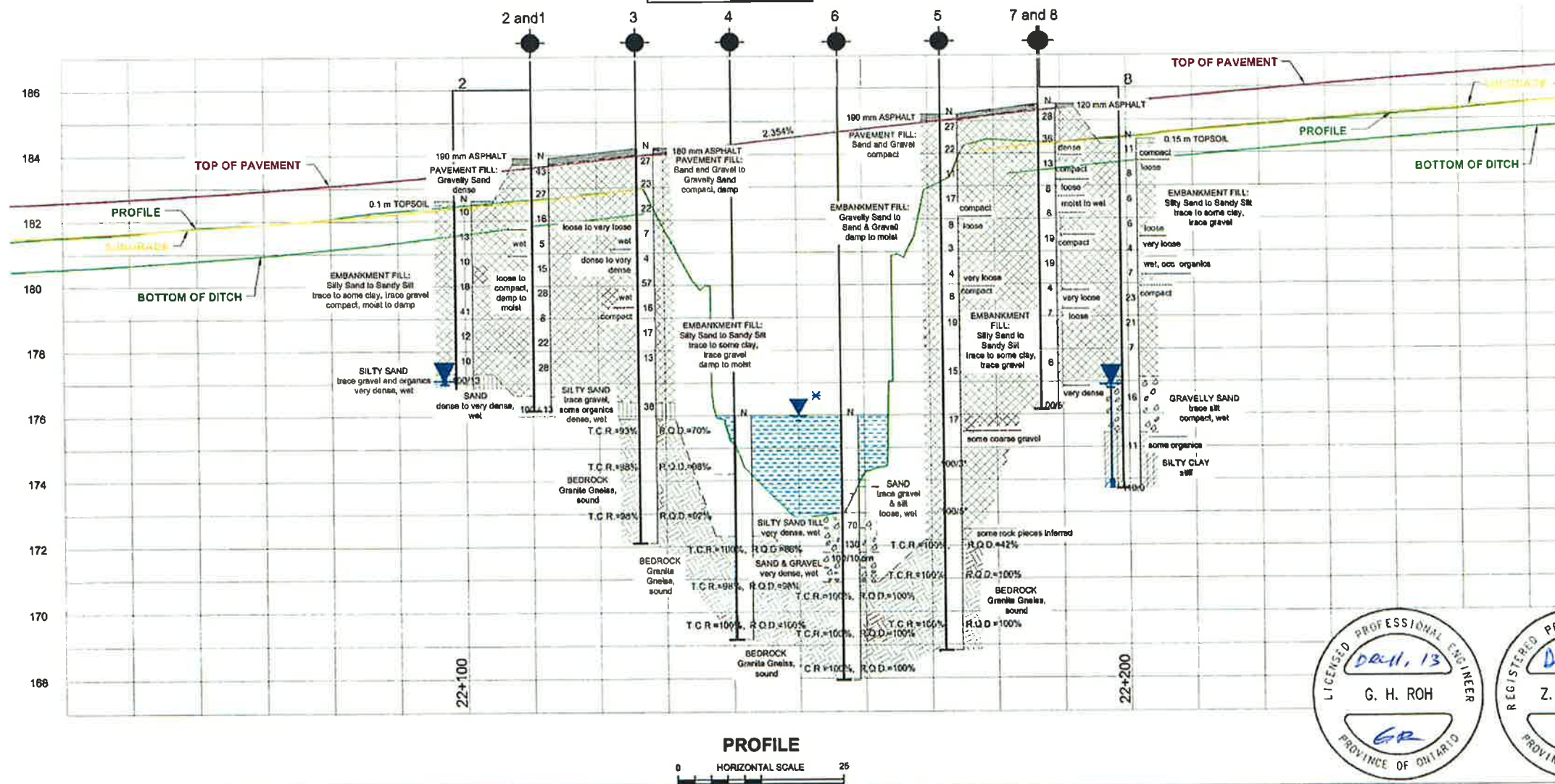


SHEET

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 LI C/L
BH2	182.8	22+110	3.8m LI C/L
BH3	184.2	22+125	13.3m LI C/L
BH4	178.0	22+140	5.7m LI C/L
BH5	185.2	22+172	13.2m LI C/L
BH6	178.0	22+158	@ C/L
BH7	185.5	22+187	13.7m LI C/L
BH8	184.4	22+187	5.3m LI C/L
BH4A	178.0	22+141	2.9m LI C/L

-NOTE-

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

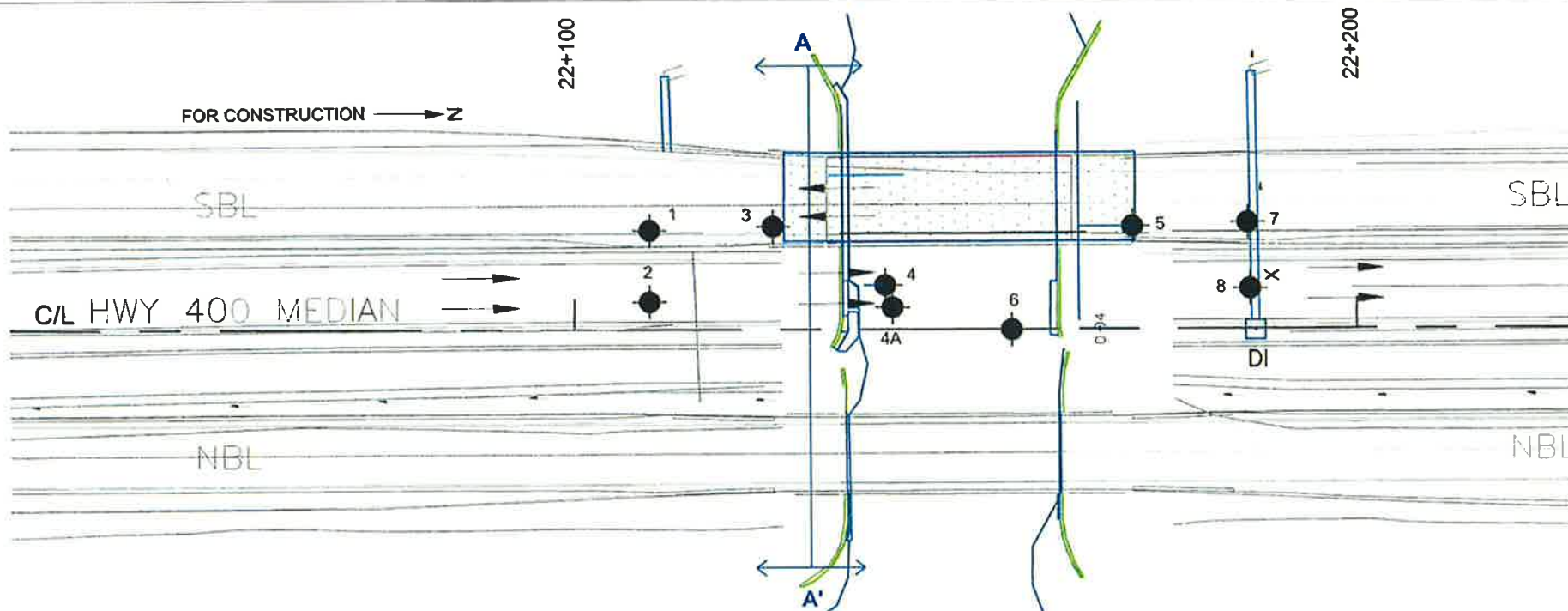
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.



REVISIONS	DATE	BY	DESCRIPTION

Geocore No. 310-584

SUBMIT	CHECKED	DATE	APPROVED	SITE	DIST
SSH	SS	December, 2013	ZO	42-88/1&2	1



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.

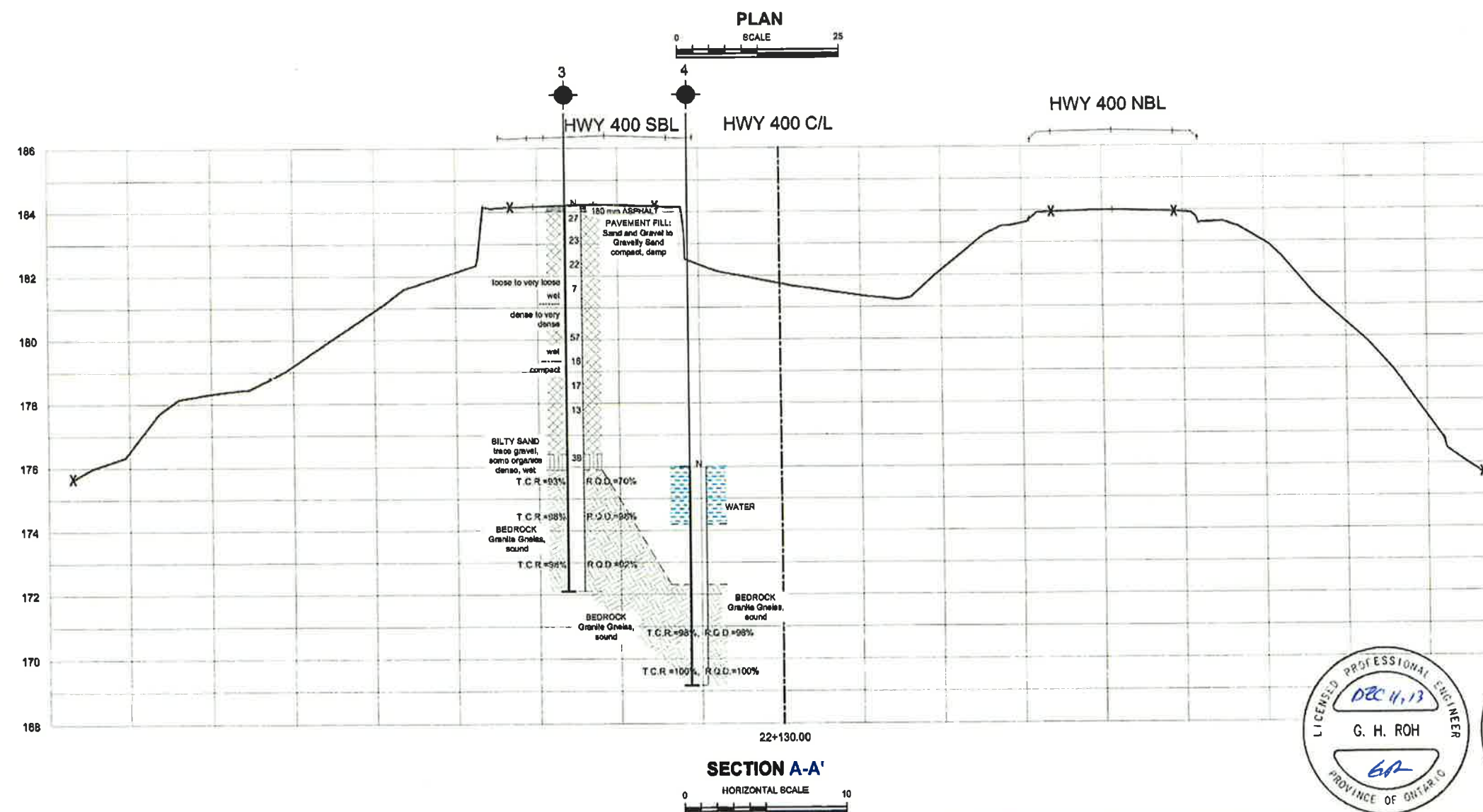
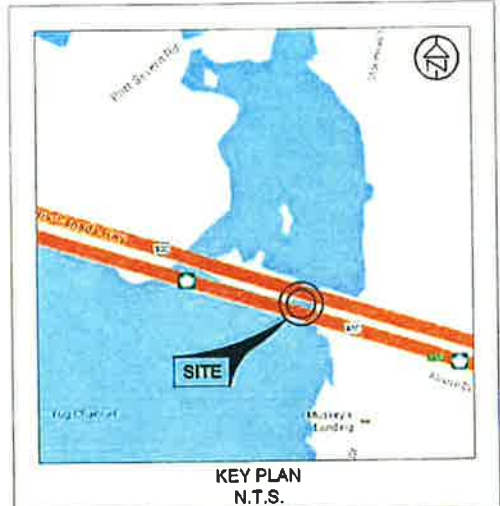
CONT No. -
W.P.: 2360-06-00

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



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



LEGEND

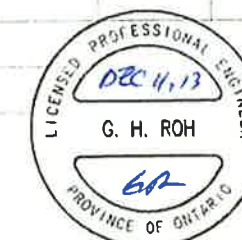
- Borehole
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- A-A' Section

No	ELEVATION	STATION	OFFSET
BH3	184.2	22+125	13.3m LI C/L
BH1	178.0	22+140	8.7m LI C/L

-NOTE-

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

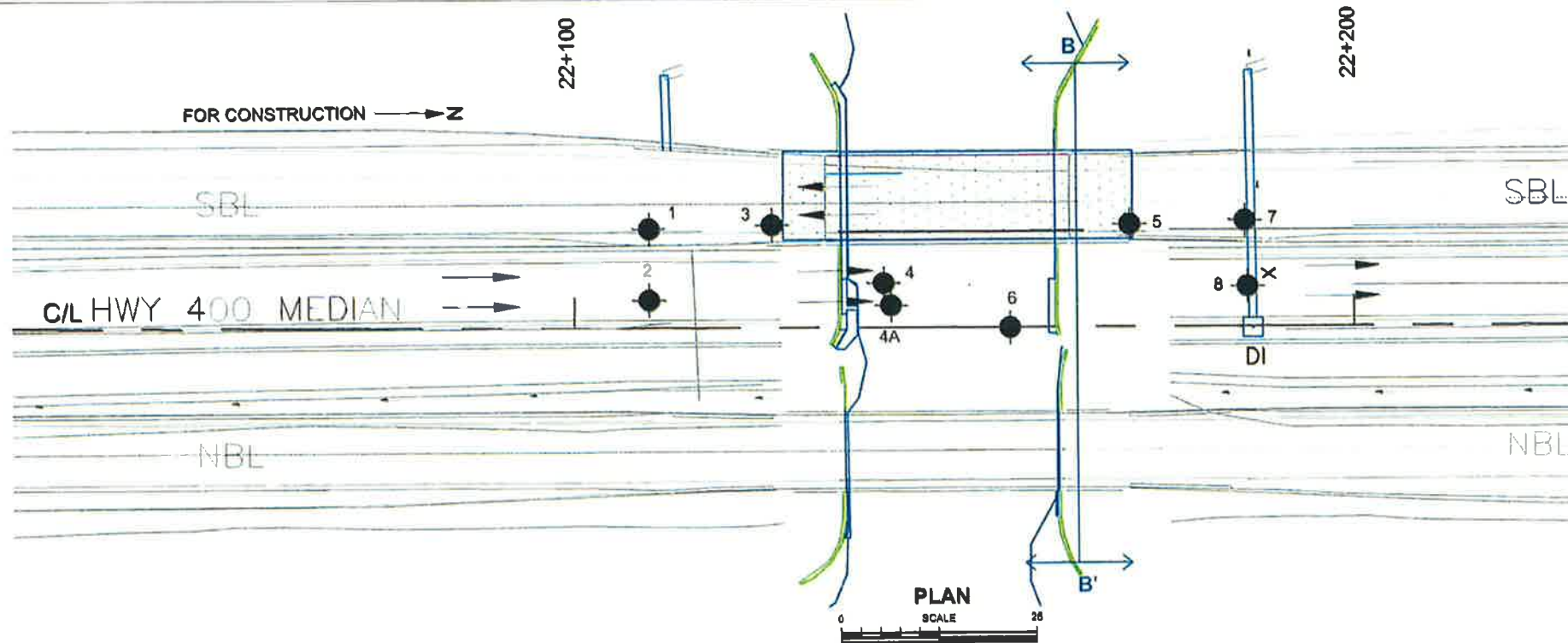
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.



DATE	BY	DESCRIPTION

Geocres No. 31D-564

SUBMIT	CHECKED	DATE	SITE
DRAWN	SSH	CHECKED	OR
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



KEY PLAN
N.T.S.

LEGEND

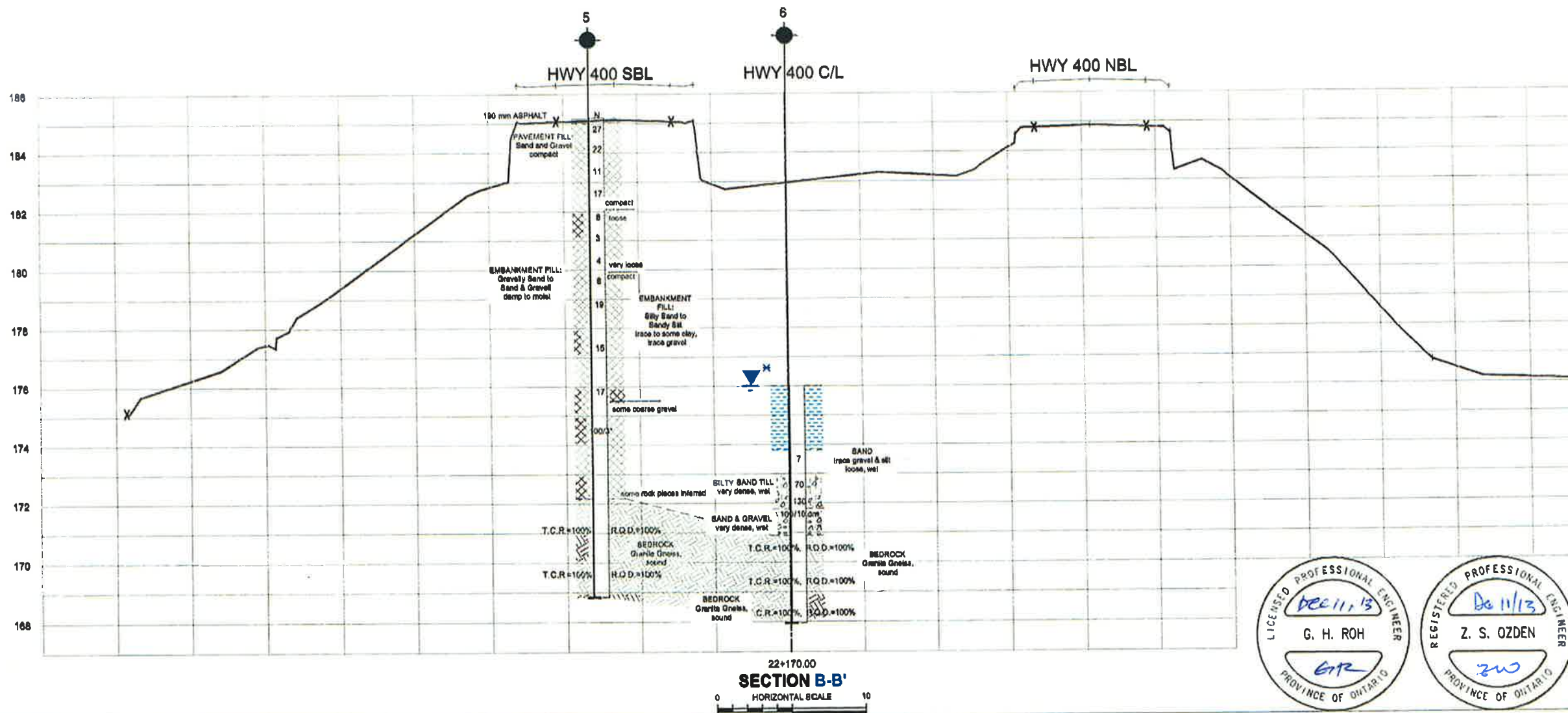
- Borehole
- Blows/0.3m (Std Pen Test, 475 Jblow)
- Water Level at Time of Investigation (W.L. NOT STABILIZED)
- Section

No	ELEVATION	STATION	OFFSET
BH5	185.2	22+172	13.2m LI C/L
BH6	178.0	22+168	@ 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

Geotechnical No. 310-584				TRANETOB20462AA		DIBT	
SUBMIT	CHECKED	DATE	December, 2013	BITE	42-96/182		
DRAWN	BSH	CHECKED	GR	APPROVED	ZO	DWG	3

Appendix A

Record of Borehole Sheets

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 1

1 OF 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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
183.9	GROUND SURFACE													
183.7	190 mm ASPHALT		1	SS	43									32 56 (12)
0.2	PAVEMENT FILL: Gravelly Sand brown, dense		2	SS	27									3 58 22 17
183.2			3	SS	16									
0.7			4	SS	5									
			5	SS	15									
			6	SS	28									2 34 46 18
			7	SS	8									
			8	SS	22									
			9	SS	28									
			10	SS	100 / 13 cm									spoon wet and bouncing
176.6														
7.3	SAND													
176.2	grey, dense to very dense, wet													
7.8	End of Borehole Auger refusal @ 7.8 m Probable Bedrock Borehole open & dry on completion (not stabilized)													

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 2

1 OF 1

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
182.6	GROUND SURFACE																
182.5	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		180										
			4	SS	18		179										
			5	SS	41		178										
			6	SS	12		177										
			7	SS	10												
			8	SS	100/13												
177.3	SILTY SAND																
5.3	trace gravel and organics																
176.8	grey, very dense, wet																
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																

TRANETOB20462AA: Hwy 400, Port Severn

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) ORIGINATED BY LG
DIST 5 HWY 400 BOREHOLE TYPE Hollow Stem Auger + NQ Coring COMPILED BY SSH
DATUM Geodetic DATE 23/05/2013 23/05/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
184.2	GROUND SURFACE																
184.0	180 mm ASPHALT																
0.2																	
	PAVEMENT FILL: Sand and Gravel to Gravelly Sand brown, compact, damp		1	SS	27		184										
			2	SS	23		183										36 48 (16)
			3	SS	22		182										40 46 (14)
181.9							181										
2.3	loose to very loose wet brown		4	SS	7		180										10 55 19 16
			5	SS	4		179										
	greyish brown dense to very dense wet compact		6	SS	57		178										3 40 39 18
			7	SS	16		177										
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel damp to moist		8	SS	17		176										8 65 (27)
			9	SS	13		175										Auger refusal and Start of NQ coring at 8.3 m
	grey						174										
176.4							173										
7.8	SILTY SAND		10	SS	38												
175.9	trace gravel, some organics grey / black, dense, wet																
8.3	fractured		11	RC	T.C.R.=93% R.Q.D.=70%												
	BEDROCK Granite Gneiss greyish / pink, sound		12	RC	T.C.R.=98% R.Q.D.=98%												
			13	RC	T.C.R.=98% R.Q.D.=92%												
172.1																	
12.1	End of Borehole Borehole open and dry upon completion (prior to coring), not stabilized																

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 4

1 OF 1

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
176.0 0.0	WATER SURFACE						176							GR SA SI CL
	WATER						175							
174.2 1.8	River Bottom						174							
	wash bored through overburden without sampling due to the presence of rockfill (see Record of Borehole 4A for details of overburden)						173							
172.3 3.7	fractured						172							
	BEDROCK Granite Gneiss sound		1	RC	T.C.R.=100% R.Q.D.=86%		171							
	mainly pink		2	RC	T.C.R.=98% R.Q.D.=98%		170							
	greyish / pink		3	RC	T.C.R.=100% R.Q.D.=100%									
169.2 6.8	End of Borehole													

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 4A

1 OF 1

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
176.0	WATER SURFACE						176										GR SA SI CL
0.0	WATER																
174.3							175										
1.7	SAND AND GRAVEL		1	SS	6		174										
173.8	grey, loose, wet																
2.2	SILTY CLAY		2	SS	4												
173.2	reddish grey, soft																
2.8	SAND		3	SS	22		173										
172.6	some silt and gravel grey, compact, wet																
3.4	End of Borehole See Record of Borehole 4 for continuation of stratigraphy Auger refusal @ 3.4 m Probable Bedrock																

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 5

1 OF 2

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
185.2	GROUND SURFACE													
185.0	190 mm ASPHALT						185							
0.2	PAVEMENT FILL: Sand and Gravel brown, compact		1	SS	27									
184.1			2	SS	22		184							
1.1	EMBANKMENT FILL: Gravelly Sand to Sand and Gravel brown, damp to moist		3	SS	11									41 44 13 2
			4	SS	17		183							
	compact													
	loose		5	SS	8		182							
181.5			6	SS	3		181							9 41 33 17
3.7			7	SS	4									
	very loose													
	compact		8	SS	8		180							
	compact													
	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		9	SS	19		179							
			10	SS	15		178							6 58 20 16
			11	SS	17		176							
	some coarse gravel						175							* Nvalue influenced by coarse gravel
			12	SS	100/3*									
							174							
			13	SS	100/5*		173							
172.1	some rock pieces inferred													
13.1	fractured		14	RC	T.C.R.=100% R.Q.D.=42%		172							Auger refusal and Start of NQ coring at 13.1 m
	BEDROCK Granite Gneiss greyish / pink, sound		15	RC	T.C.R.=100% R.Q.D.=100%		171							

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE


TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 5

2 OF 2

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
170.2																	
	BEDROCK Granite Gneiss greyish / pink, sound		16	RC	T.C.R.=100% R.Q.D.=100%		170										
168.7							169										
16.5	End of Borehole Borehole open and dry upon completion (prior to coring), not stabilized																

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 6

1 OF 1

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
176.0	WATER SURFACE						176							12 62 (26)
0.0	WATER						175							
							174							
173.8	River Bottom						173							
2.2	SAND trace gravel and silt grey, loose, wet		1	SS	7		172							
173.0	SILTY SAND TILL grey, very dense, wet		2	SS	70		171							
3.0			3	SS	130		170							
171.8	SAND AND GRAVEL grey, very dense, wet broken rock pieces contacted below 4.8 m (possible shattered bedrock)		4	SS	100/10 cm		169							
4.2			5	RC T.C.R.=100% R.Q.D.=100%			168							
170.9			6	RC T.C.R.=100% R.Q.D.=100%										
5.1	BEDROCK Granite Gneiss greyish / pink, sound		7	RC T.C.R.=100% R.Q.D.=100%										
168.0	End of Borehole													
8.1														

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 7

1 OF 1

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

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)									WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR.	× LAB VANE							
185.5	GROUND SURFACE						20	40	60	80	100		10	20	30	GR SA SI CL		
185.4	120 mm ASPHALT																	
0.1																		
185.2	PAVEMENT FILL: Sand and Gravel, brown, compact		1	SS	28								○			22 63 (15)		
0.3																		
184.9	PAVEMENT FILL: Gravelly Sand, brown, compact		2	SS	36								○			7 58 20 15		
0.6																		

TRANETOB20462AA: Hwy 400, Port Severn

RECORD OF BOREHOLE No 8

1 OF 1

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			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
184.4	GROUND SURFACE													
184.3	0.15 m TOPSOIL													
0.2	compact		1	SS	11		184							
	loose		2	SS	8									3 61 21 15
			3	SS	6		183							
	EMBANKMENT FILL: Silty Sand to Sandy Silt trace to some clay, trace gravel greyish brown, moist to damp		4	SS	6		182							
	loose		5	SS	4		181							
	very loose		6	SS	7		180							
	wet, occ. organics		7	SS	23		179							7 49 27 17
	loose		8	SS	21		178							
	compact		9	SS	7		177							sampler wet
177.1	GRAVELLY SAND trace silt brown, compact, wet (possible fill)		10	SS	16		176							26 65 (9)
7.3														
175.4	some organics		11	SS	11		175						43	
9.0	SILTY CLAY brown, stiff						174							spoon bouncing
173.7			12	SS	110/0									
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													

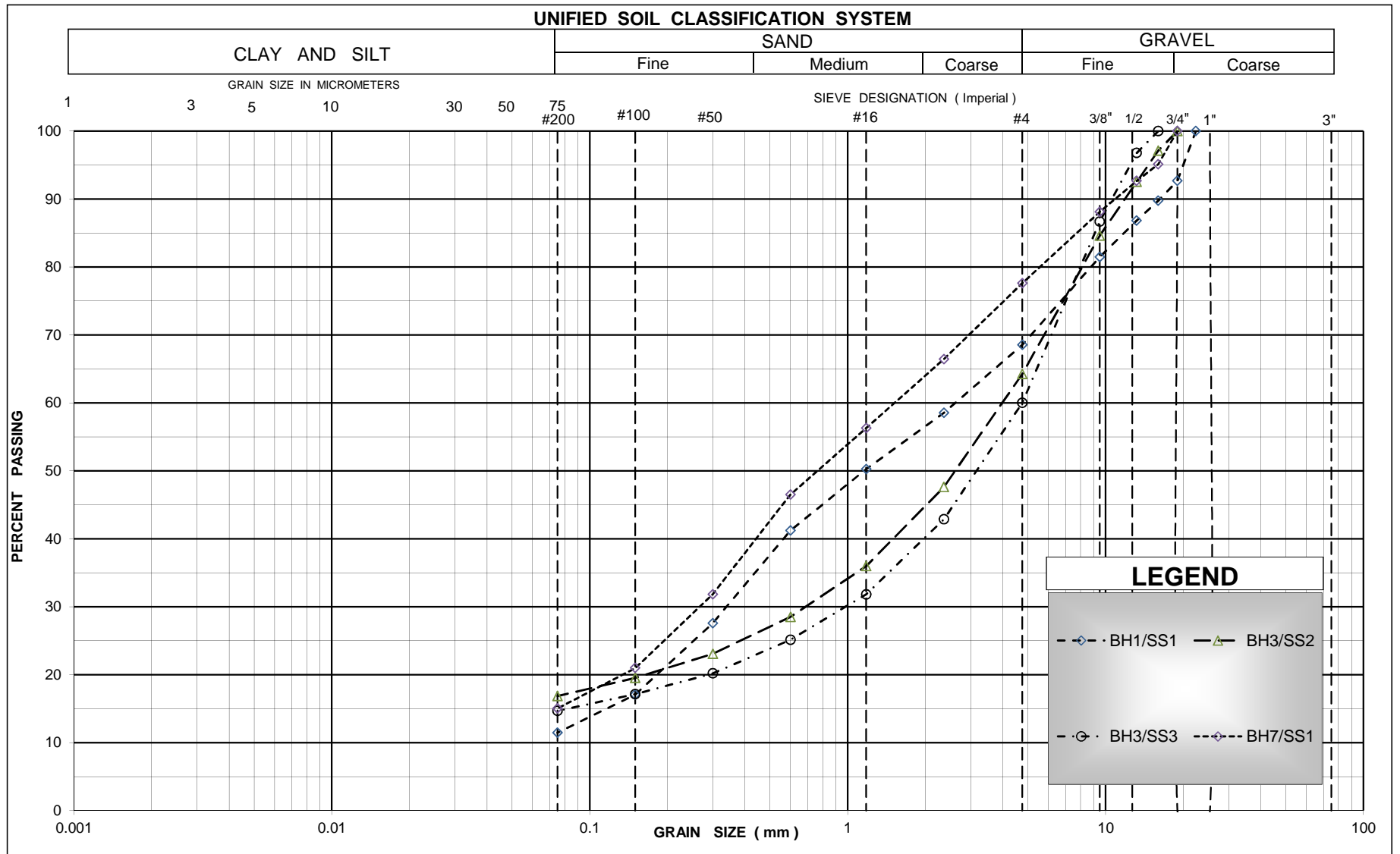
+³, ×³: Numbers refer to
Sensitivity

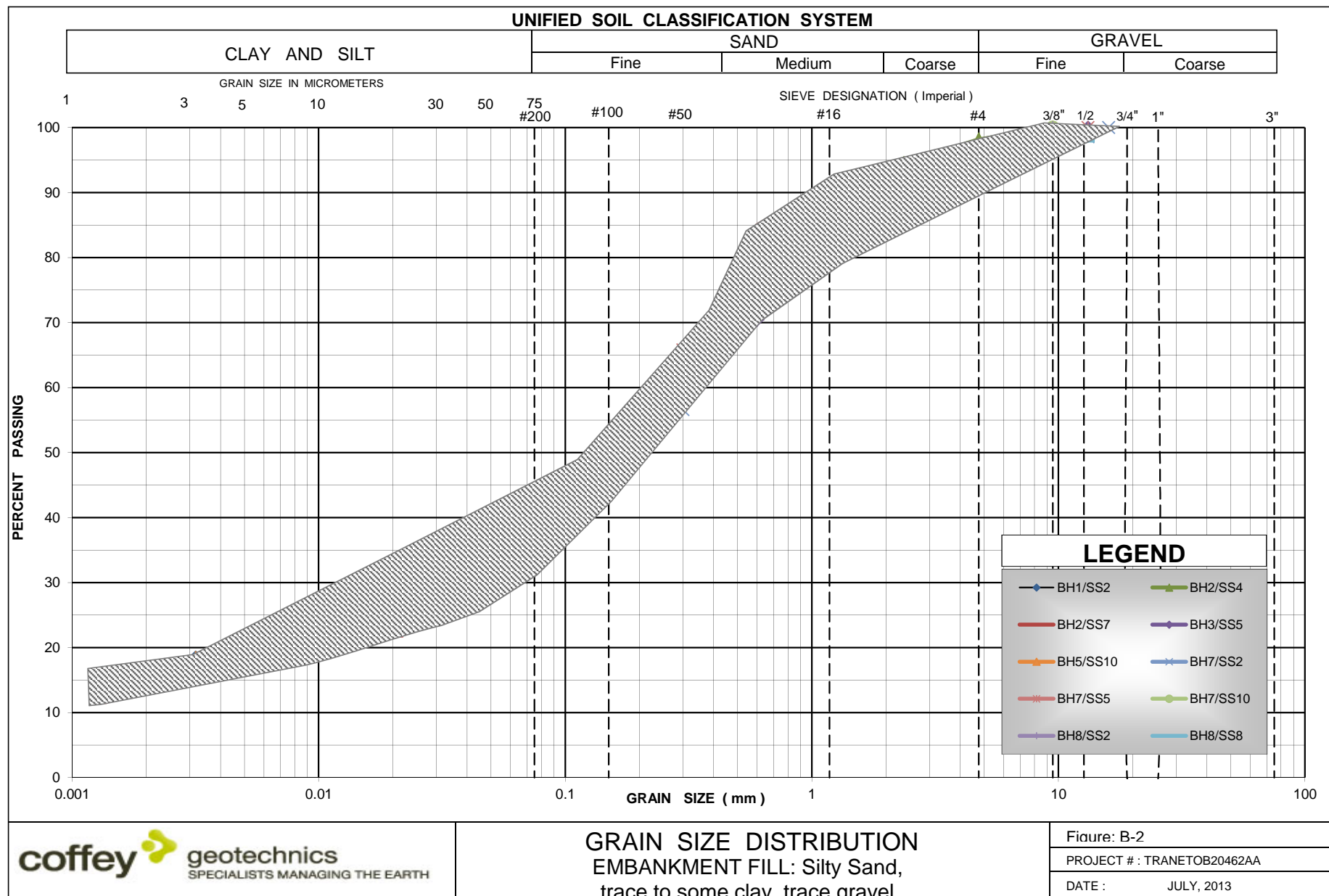
20
15
10

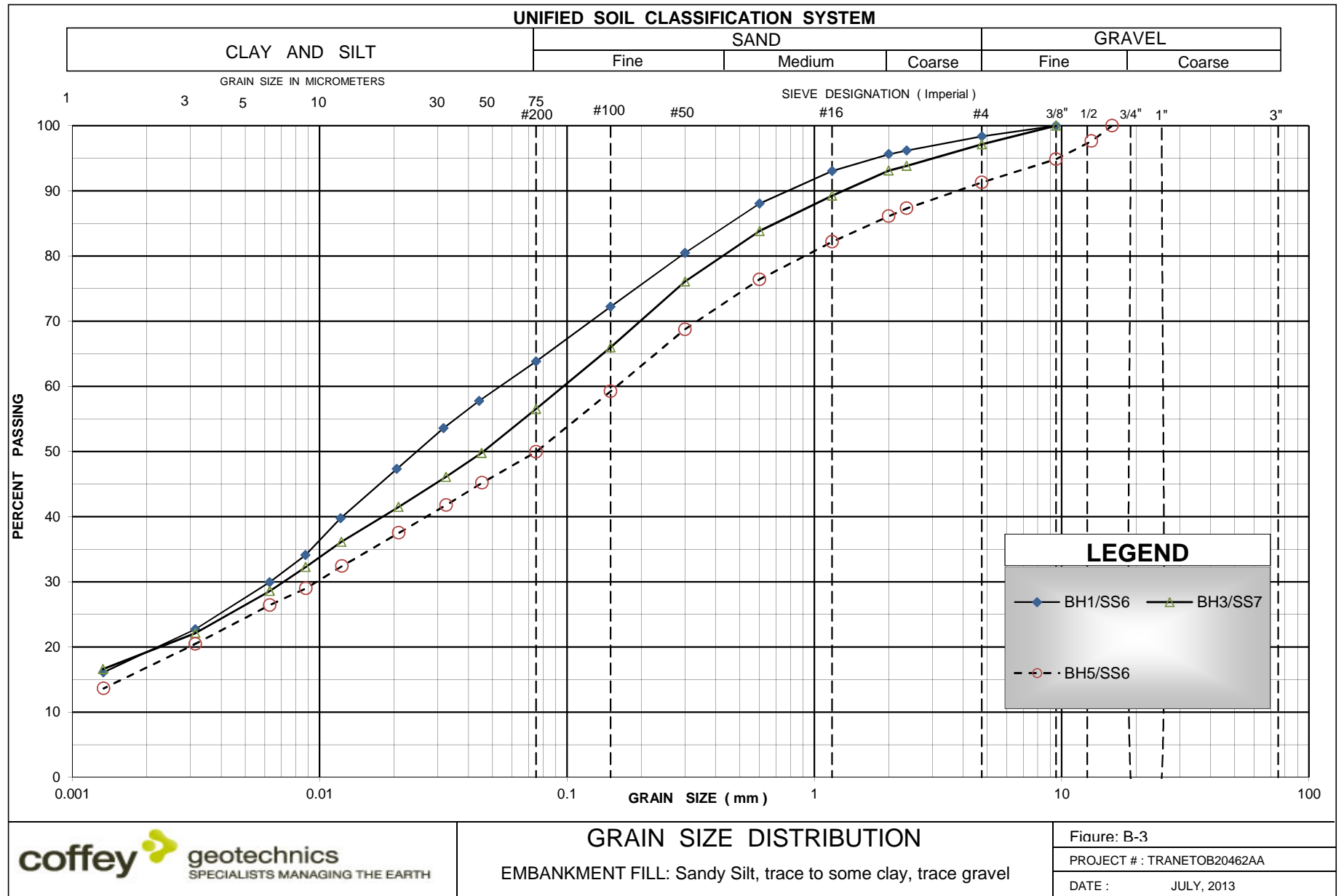
(%) STRAIN AT FAILURE

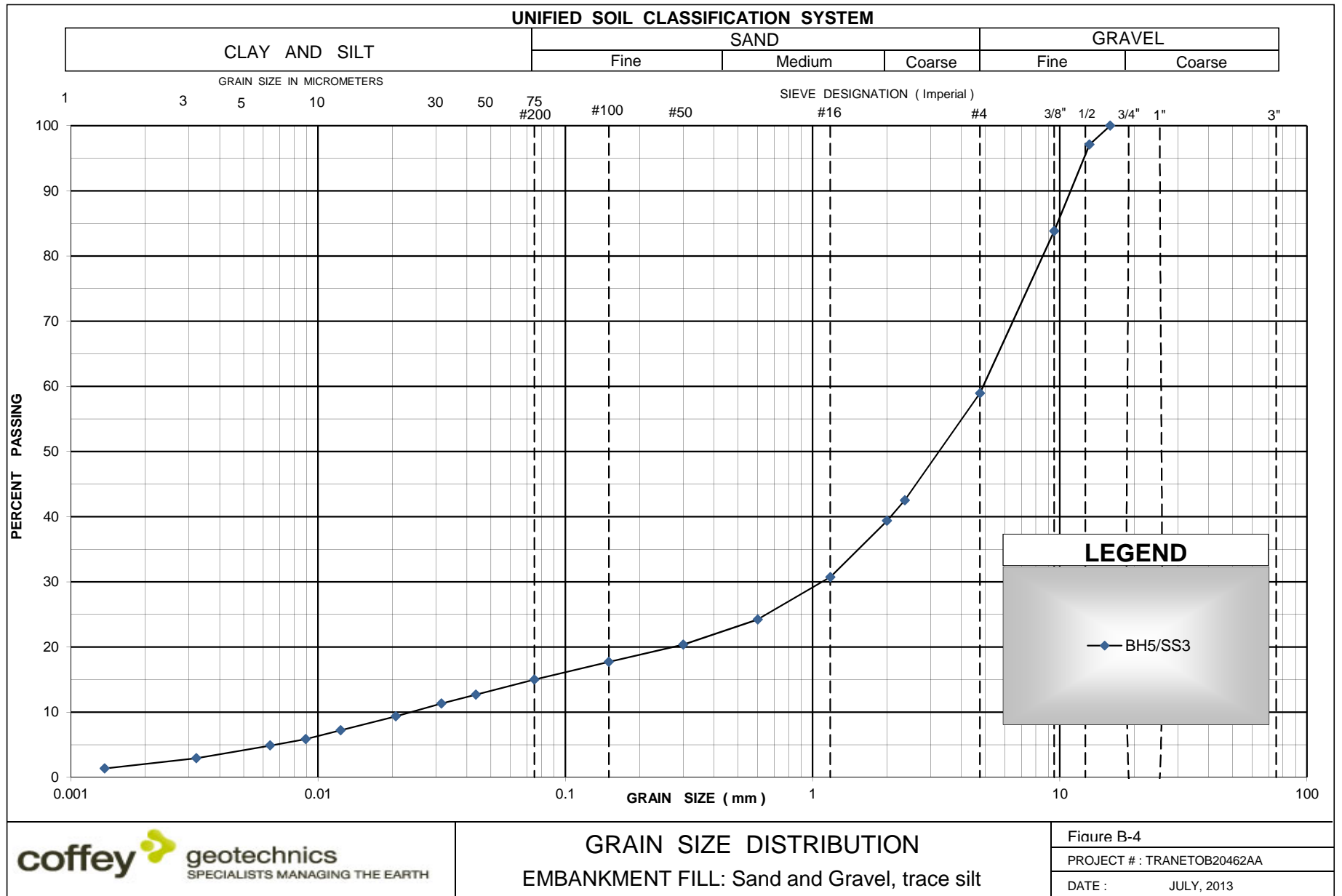
Appendix B

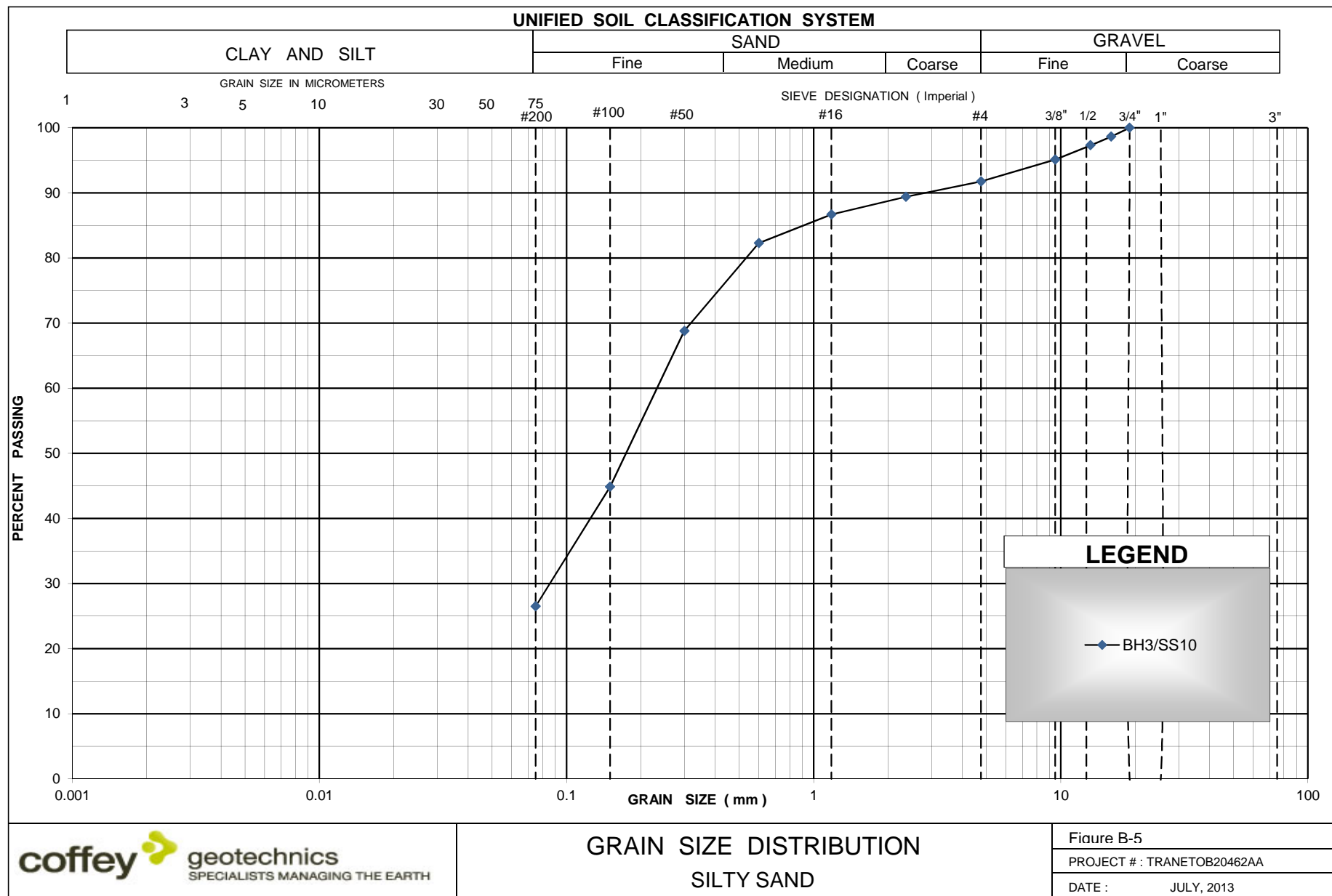
Test Results

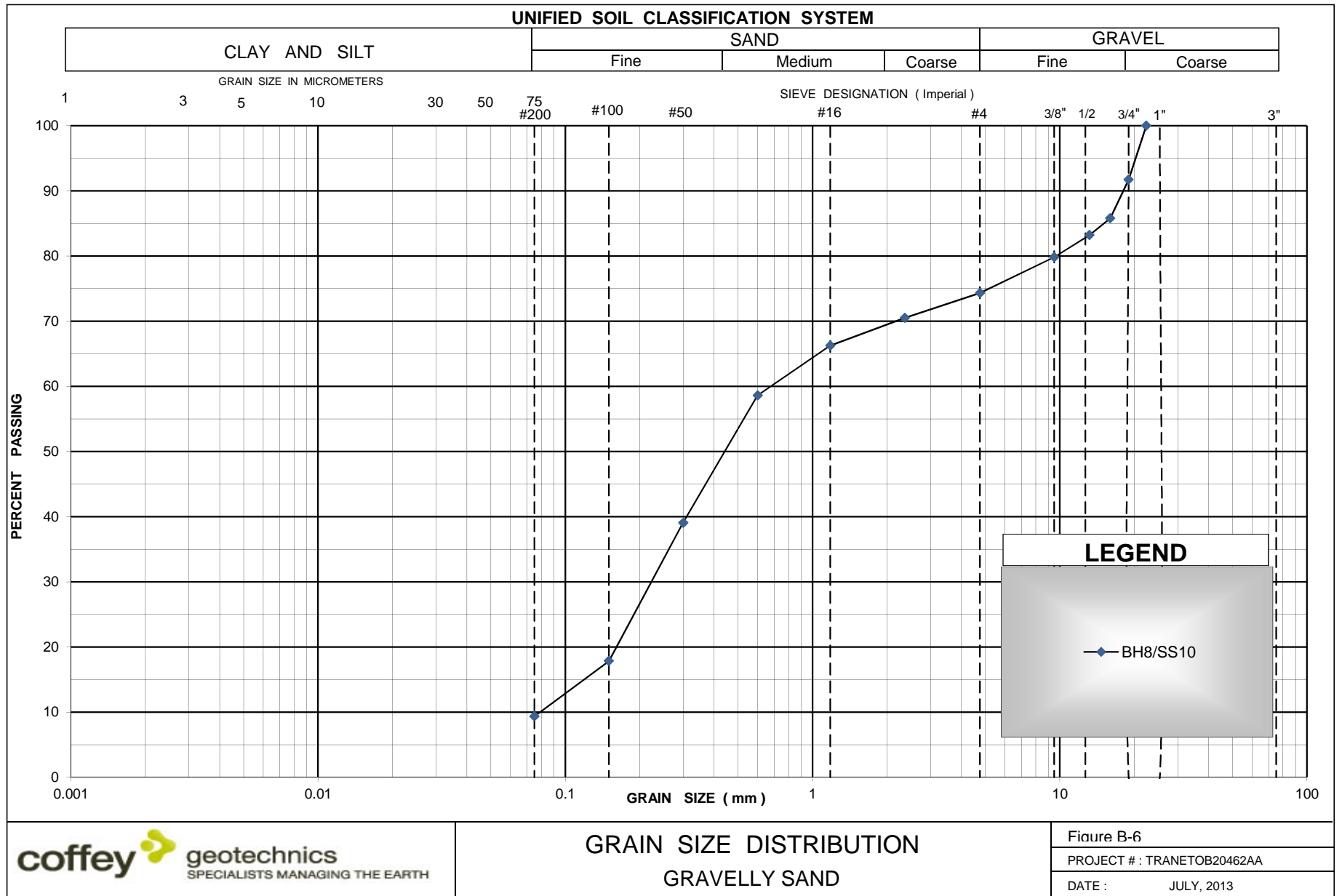


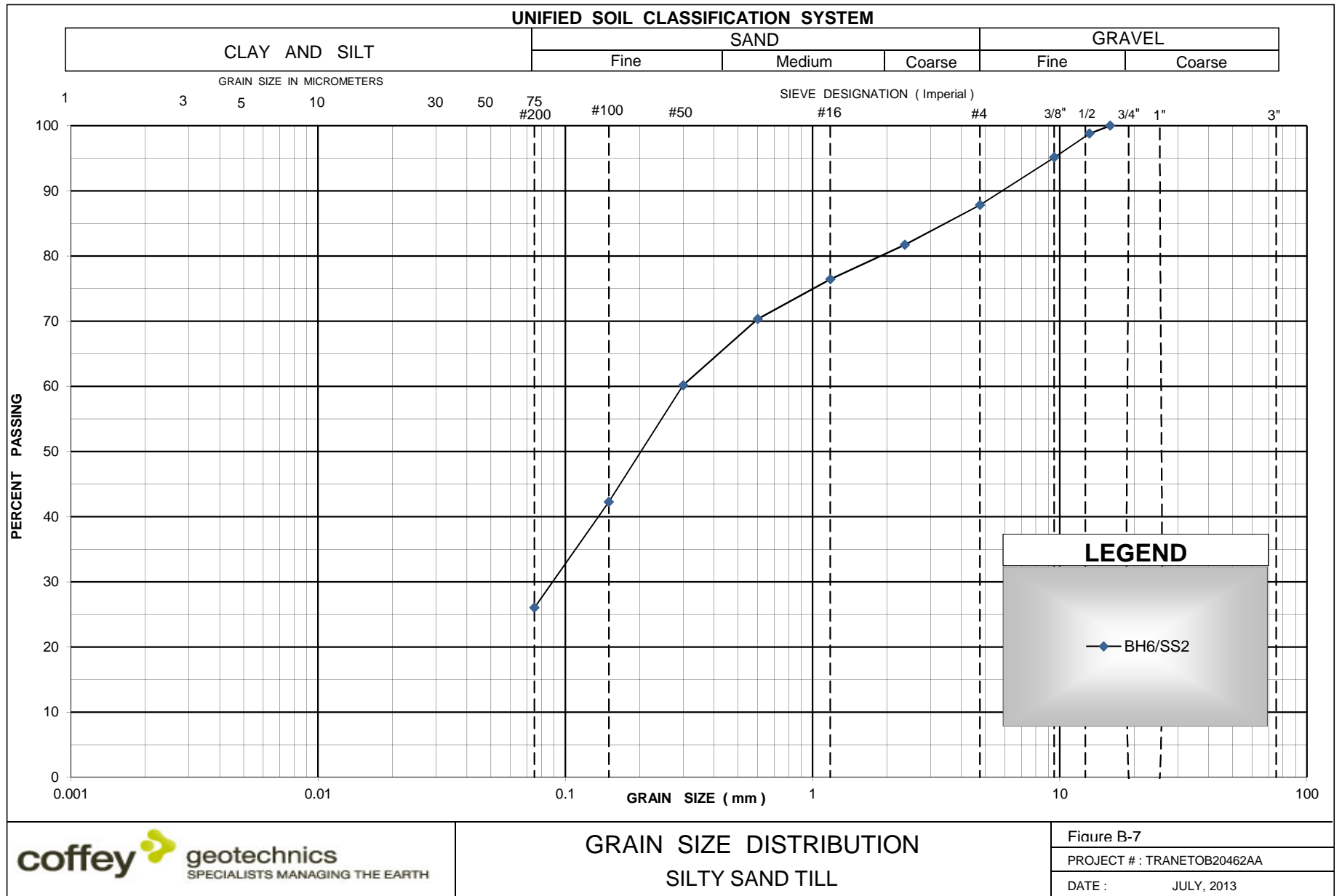


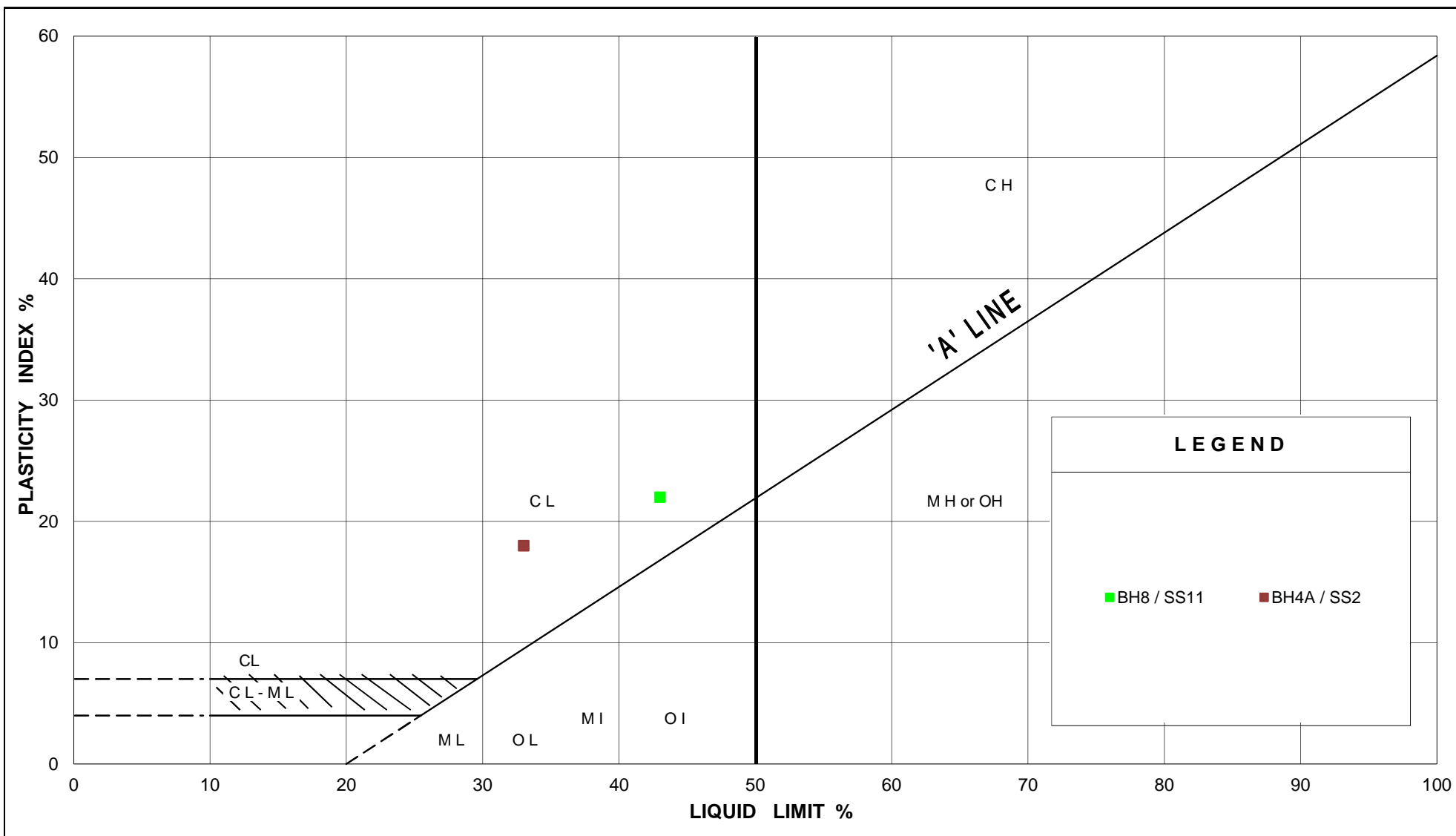












Appendix C

Site Photographs



Photograph 1. Borehole 3 looking east (south)



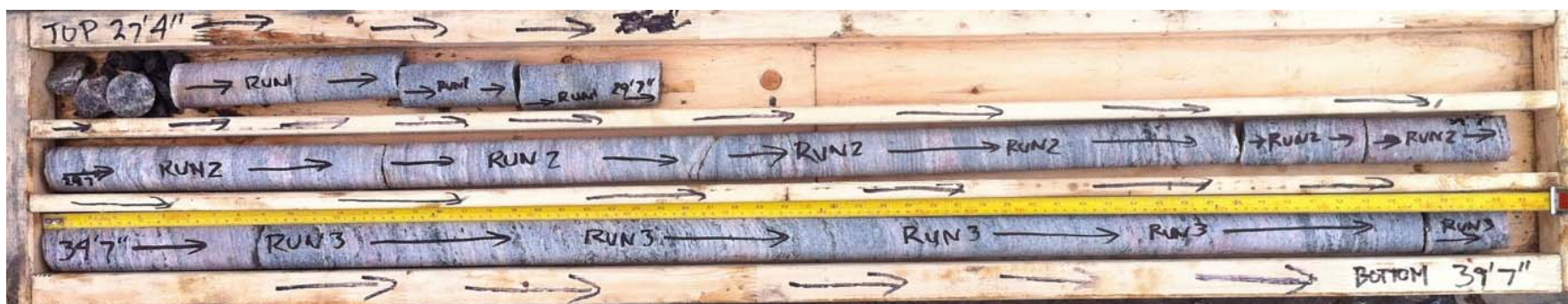
Photograph 2. Borehole 2 looking west (north)



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

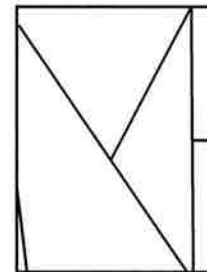
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.24

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
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REMARKS:

DATE:

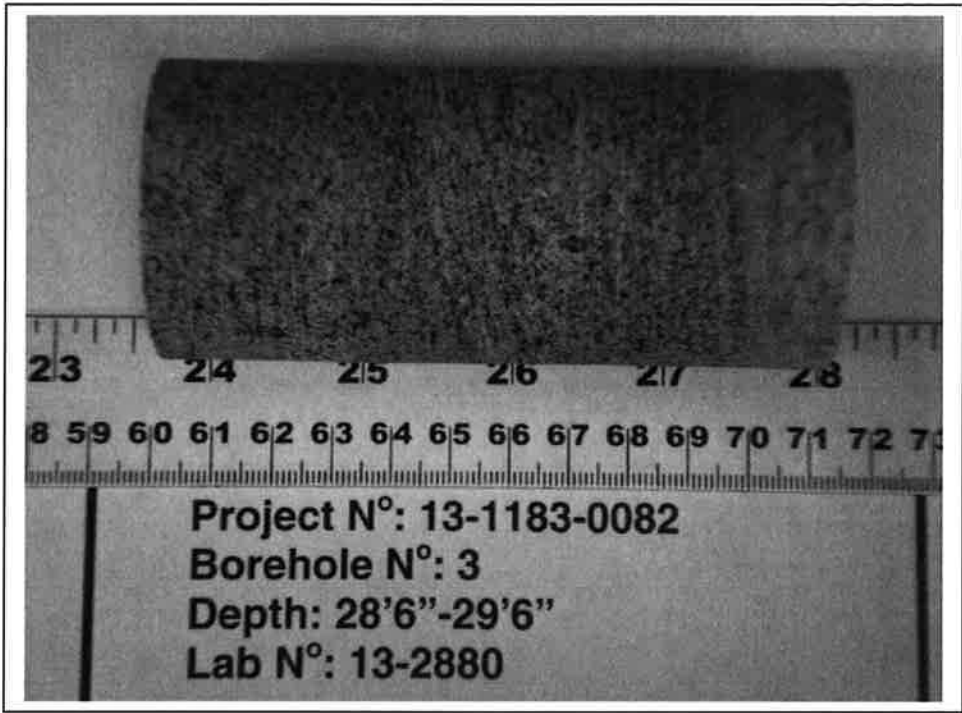
7/24/2013

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Golder Associates

UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

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

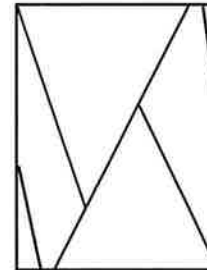
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST, min	>2 <15	L/D	2.23

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
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REMARKS:

DATE:

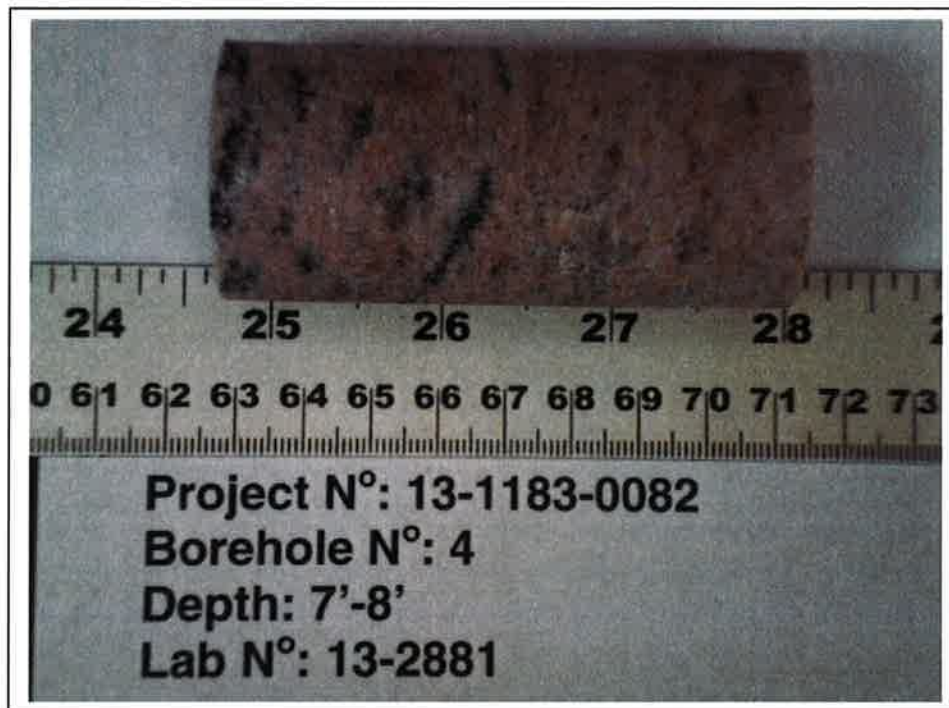
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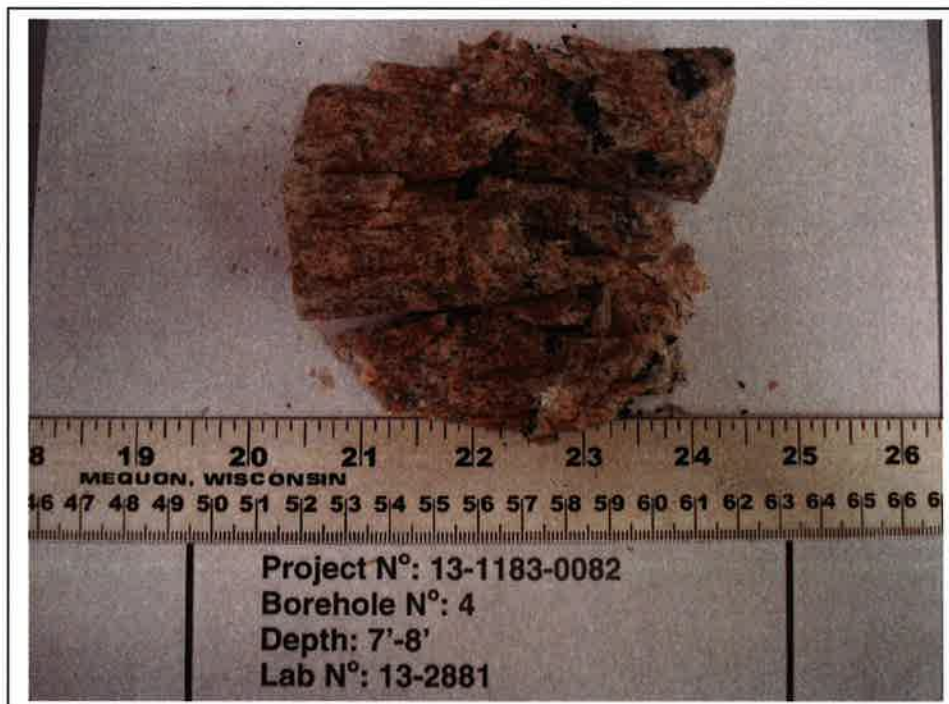
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UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

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

Golder Associates

Drawn Frank
Chkd. fo

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

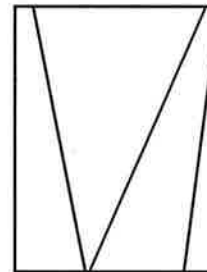
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.20

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
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REMARKS:

DATE:

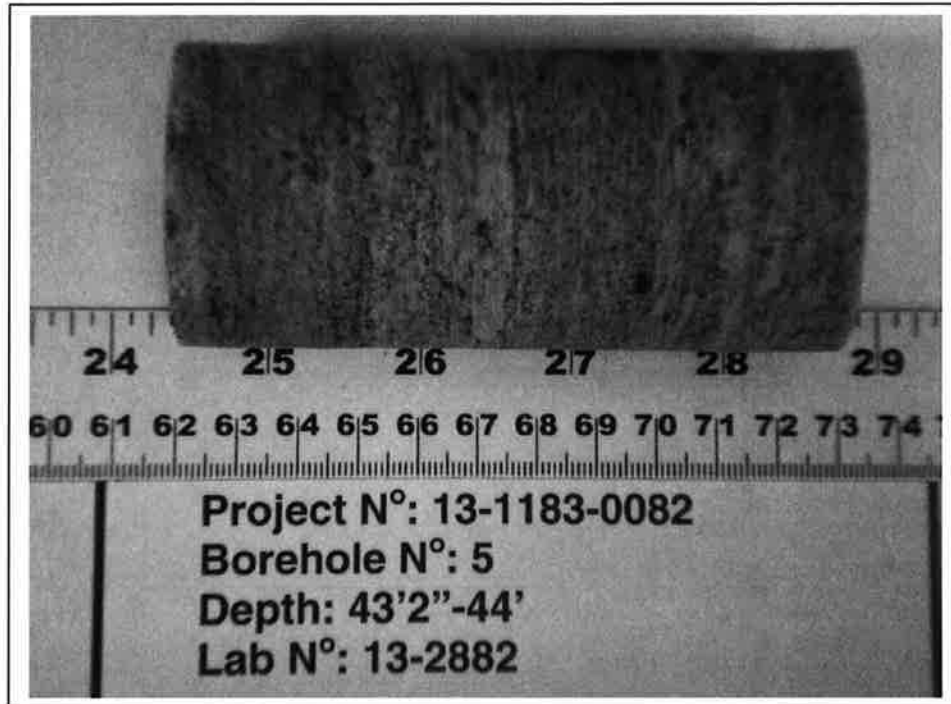
7/24/2013

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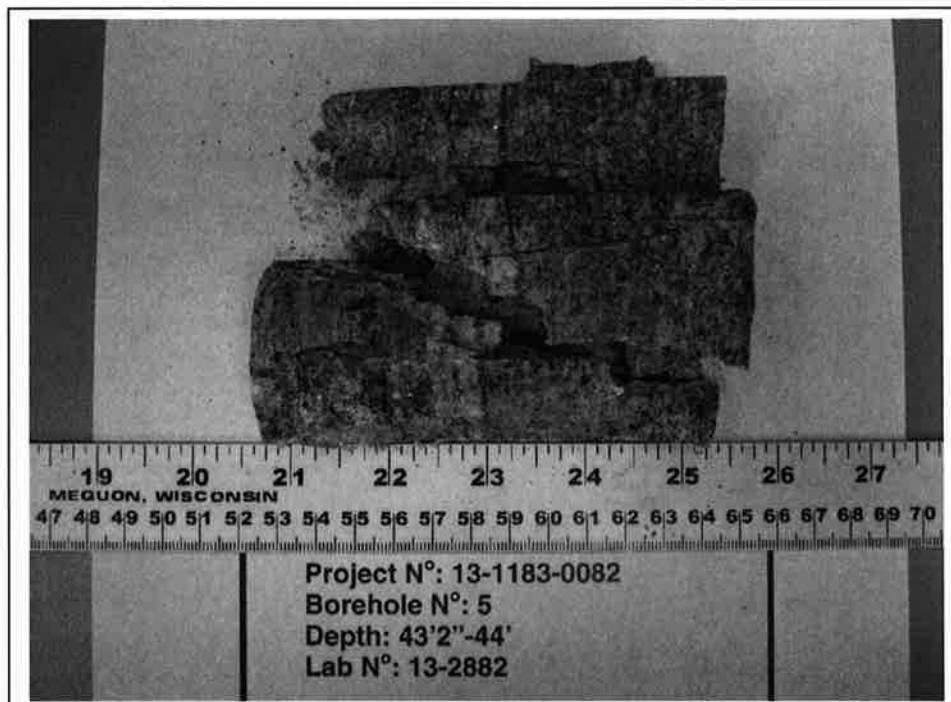
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UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

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

Golder Associates

Drawn Frank
Chkd. la

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

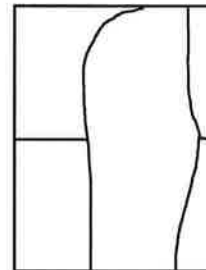
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.23

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
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REMARKS:

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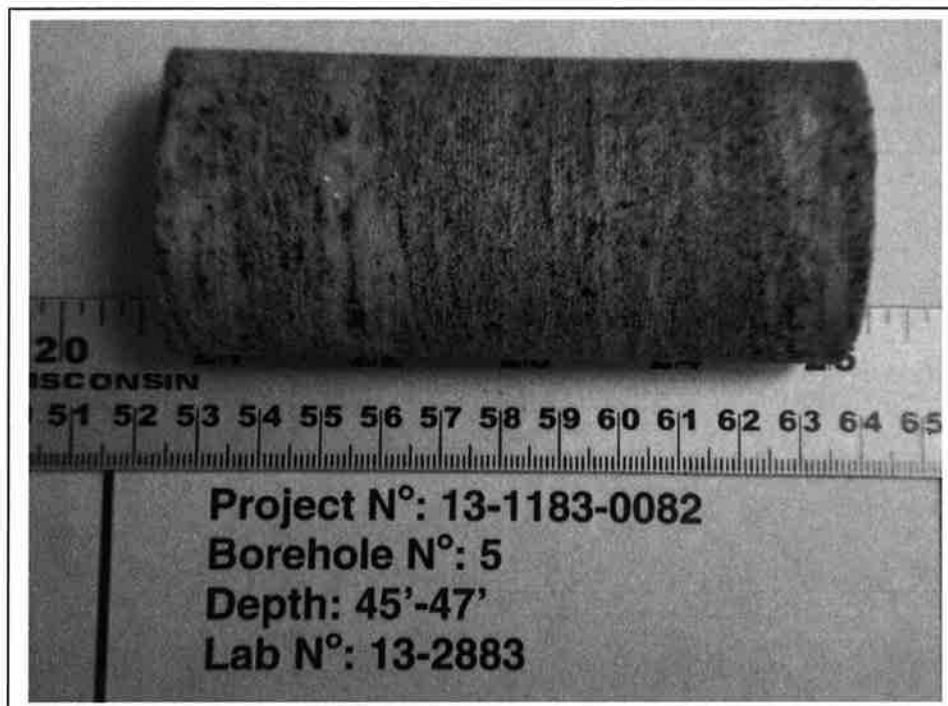
7/24/2013

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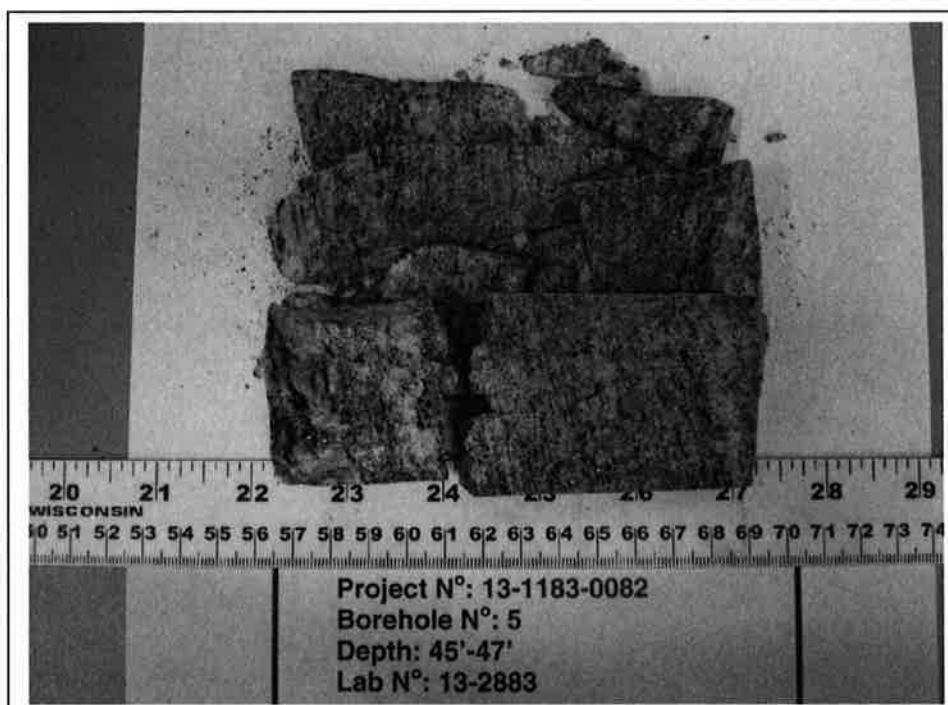
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UNCONFINED COMPRESSION TEST
ASTM D7012-07

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

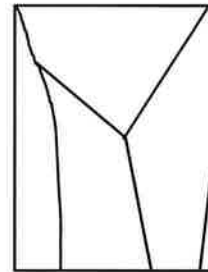
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.21

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
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REMARKS:

DATE:

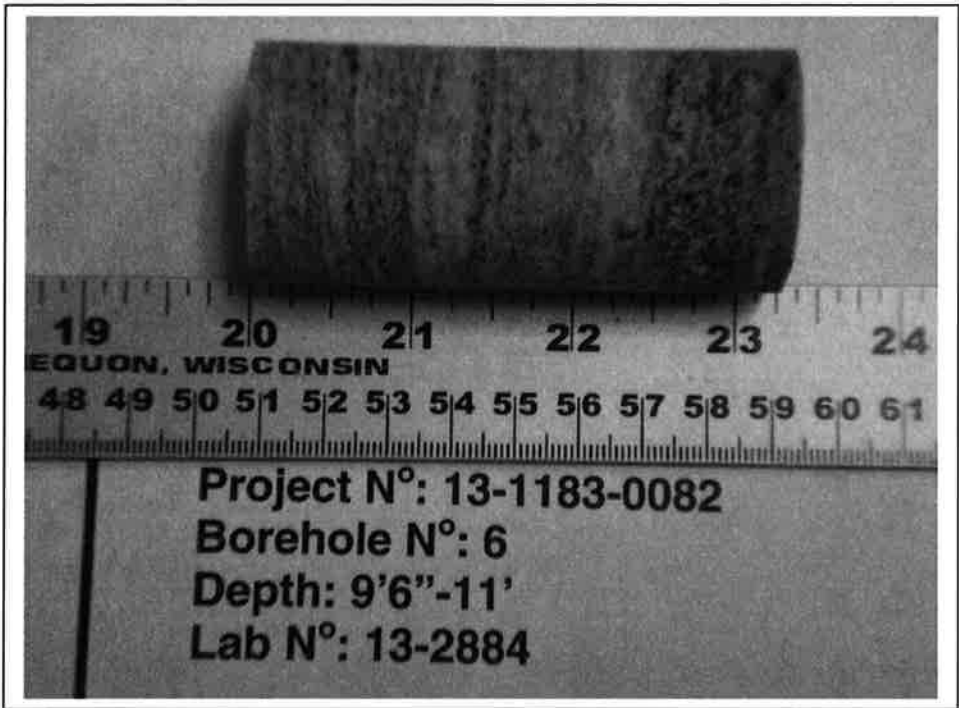
7/24/2013

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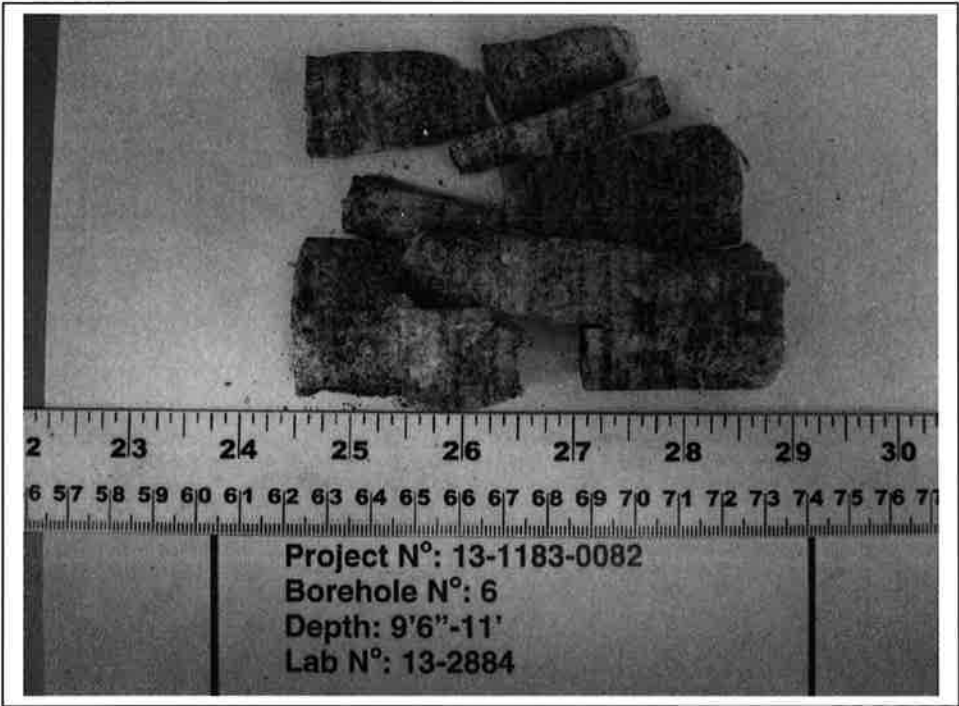
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UNCONFINED COMPRESSION TEST
ASTM D7012-07

FIGURE



BEFORE COMPRESSION



AFTER COMPRESSION

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
	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 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 COMPOSITION AND STRUCTURAL 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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	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_e	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p) / I_p$	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 = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED 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

CONTENTS

5	DISCUSSION AND RECOMMENDATIONS	10
5.1	Foundations	11
5.1.1	Spread Footing Foundations	11
5.1.2	Deep Foundations	13
5.1.2.1	Cast-In-Place Concrete Pile (Caisson) Foundations	13
5.1.2.2	Micropiles	14
5.1.3	Summary of Foundation Options	15
5.2	Approach Embankments	15
5.2.1	Retaining Wall	18
5.2.1.1	Conventional Cast-in-place Reinforced Concrete Retaining Wall	18
5.2.1.2	Contiguous Caisson Retaining Wall	18
5.2.1.3	Retained Soil System (RSS)	18
5.2.1.4	Retaining Wall Backfill	19
5.3	Lateral Earth Pressures	19
5.4	Seismic Design	20
5.5	Construction Comments	21
5.6	Scour and Erosion Protection	23
5.7	Frost Protection	23
6	CLOSURE	23

Appendices

Appendix F: GA Drawings and Cross Sectional Drawings

Appendix G: Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives

Appendix H: List of OPSDs, OPSSs, SP and Non-standard Specifications

Appendix I: Limitations of Report

**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 accommodate 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 $\frac{1}{4}$ 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 and 7, 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, $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

$$K_a = 0.27 \qquad K_b = 0.35$$

$$K_o = 0.43 \qquad K^* = 0.45$$

Compacted Granular 'B' Type I

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

$$K_a = 0.31 \qquad K_b = 0.41$$

$$K_o = 0.47 \qquad K^* = 0.57$$

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 with traces to some clay and gravel)	Type 4 soil below groundwater table
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 from 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

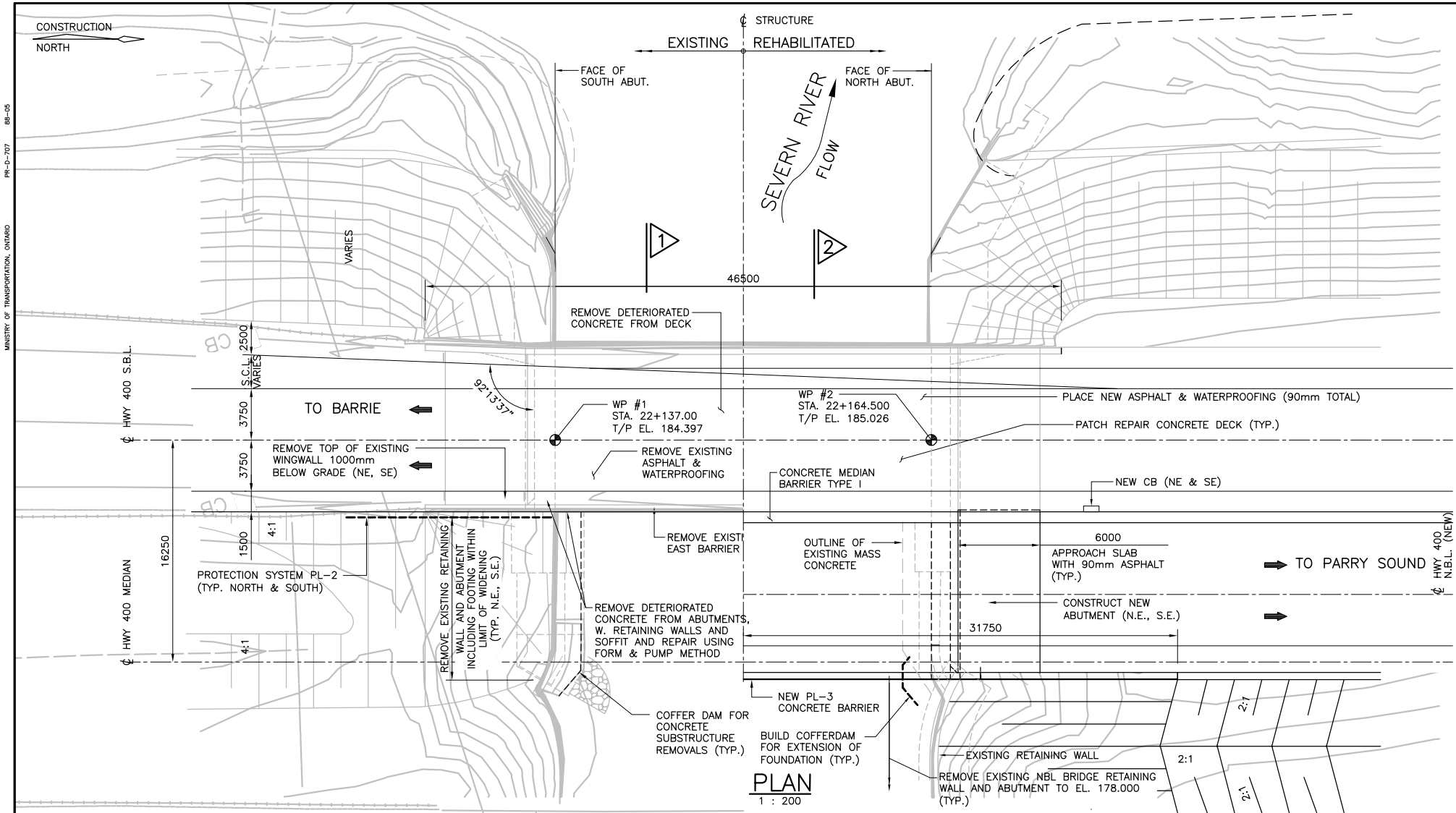


Zuhtu Ozden, P.Eng.
Senior Principal



Appendix F

GA Drawings and Cross Sectional Drawings



LIST OF DRAWINGS:

1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGING

APPLICABLE STANDARD DRAWINGS:

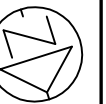
- | | |
|---------------|---|
| OPSD 912.4800 | GUIDE RAIL SYSTEM, CONCRETE BARRIER, PERMANENT CONNECTION INSTALLATION, SINGLE STEEL BEAM TO CONCRETE BARRIER |
| OPSD 3419.100 | BARRIERS AND RAILINGS - STEEL GUIDE RAIL AND CHANNEL ANCHORAGE |
| OPSD 3941.200 | FIGURES IN CONCRETE - SITE NUMBER AND DATE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

DISTRICT
CONT. No.
WP No. 2360-09-00

HIGHWAY 400 NBL & SBL
SEVERN RIVER BRIDGE
BRIDGE REHABILITATION

PRELIMINARY GENERAL ARRANGEMENT

MRC **McCORMICK RANKIN**
A member of **MMM GROUP**



SHEET

METRIC

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.

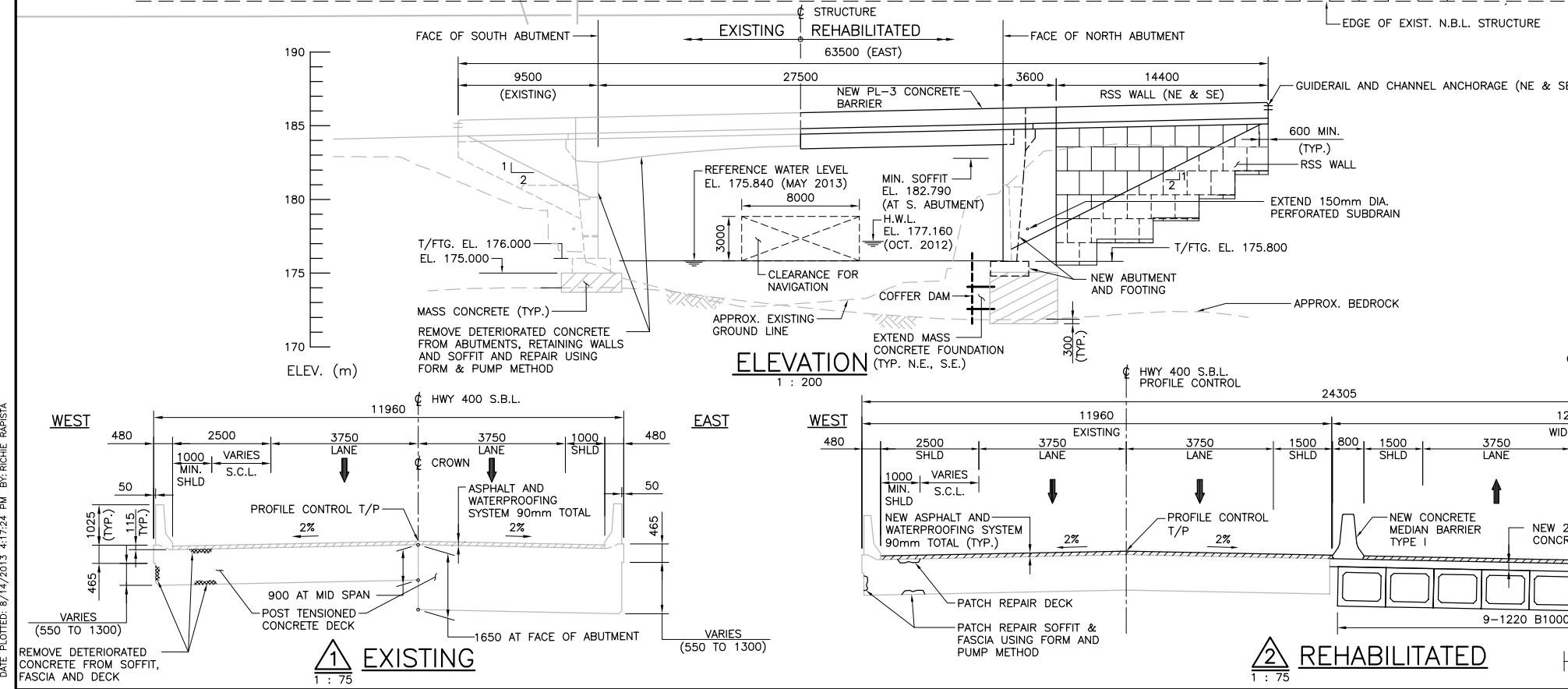
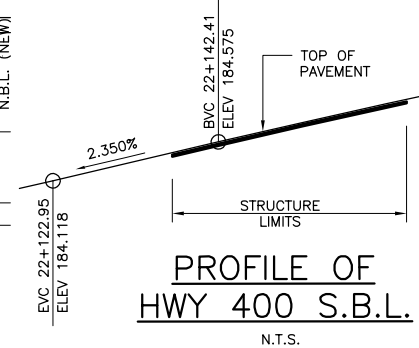
PROTECTION SYSTEMS REQUIRED TO COMPLETE THE WORK SHALL BE DESIGNED TO PERFORMANCE LEVEL 2 CRITERIA BY CONTRACTOR. LIMITS OF PROTECTION SYSTEM TO BE DETERMINED BY CONTRACTOR. PROTECTION SYSTEMS SHALL BE SUFFICIENT FOR ALL ACCESS AND WORKING PLATFORMS.

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



REVISIONS		DESCRIPTION					
DESIGN	AY	CHK	BB	CODE	CHBDC-06	LOAD	CL-625-ONT
DRAWN	CA	CHK	AY	SITE	42-86/1&2	STRUCT	SCHEME
						DWG	P1

Drawing E-6

DIST. No. 5
CONT. No. 91-35
WP. No. 37-80-04



SEVERN RIVER BRIDGE
HIGHWAY 69 - SOUTHBOUND LANES
GENERAL ARRANGEMENT

SHEET
32

Morrison Hershfield Limited
Consulting Engineers

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



NOTE:
APPROACH SLAB, DECK WATERPROOFING
AND PAVING ARE NOT PART OF THIS
CONTRACT

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 80±20
FRONT FACE 70±20
DECK SLAB 70±20
TOP 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 300 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 TEMPORARILY CLOSED TO BOAT TRAFFIC DURING CONSTRUCTION WITH THE MINISTRY APPROVAL.

LIST OF DRAWINGS

1. GENERAL ARRANGEMENT
2. BORDOIRLE 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 SLOPED 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

LEGEND

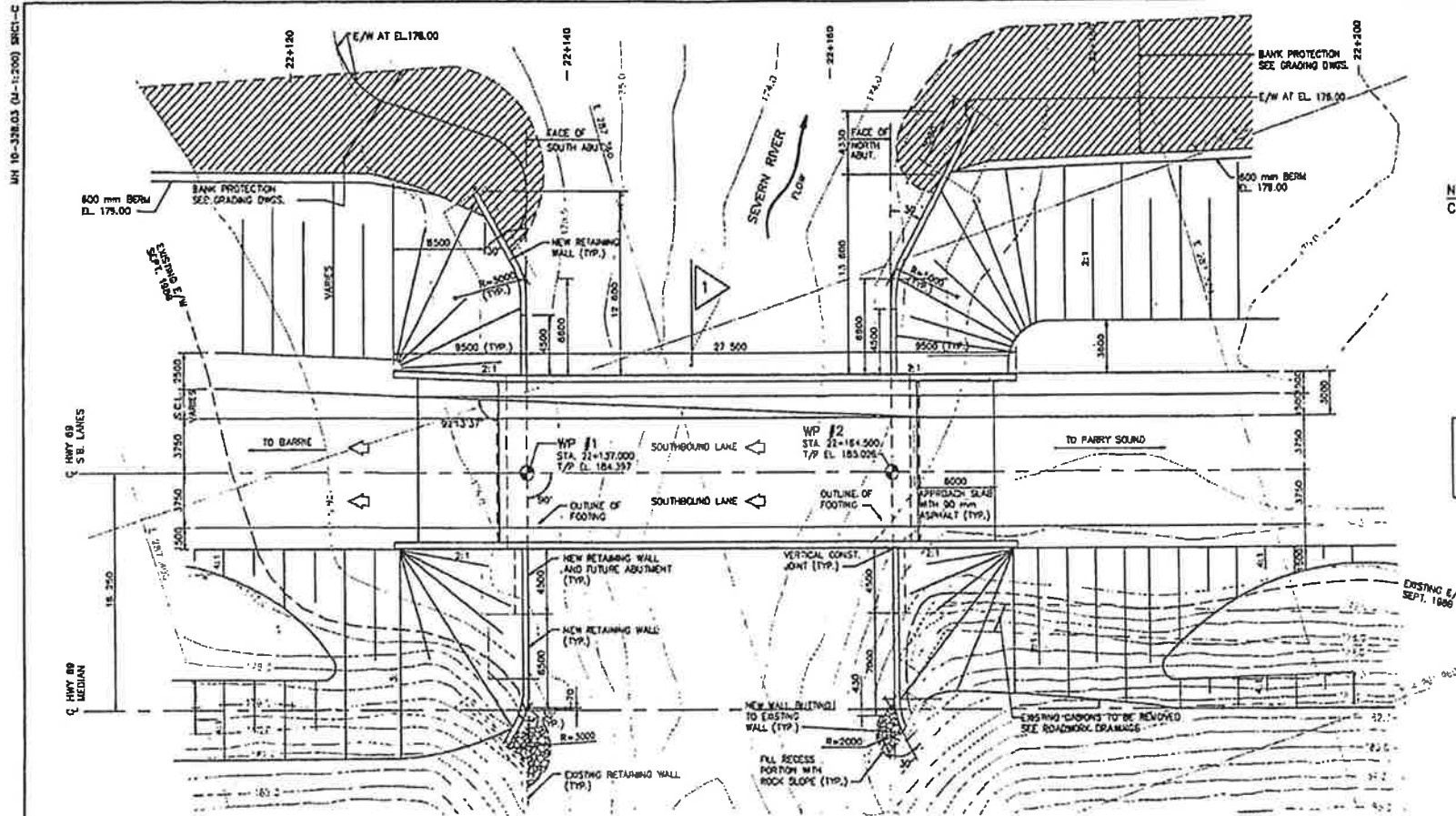
- T/T - TOP OF FOOTING
T/C - TOP OF CONCRETE
T/P - TOP OF PAVEMENT
WP - WORKING POINT
E/W - EDGE OF WATER
S.C.L. - SPEED CHANGE LANE
H.W.L. - HIGH WATER LEVEL
C.J. - CONSTRUCTION JOINT

APPLICABLE STANDARD DRAWINGS

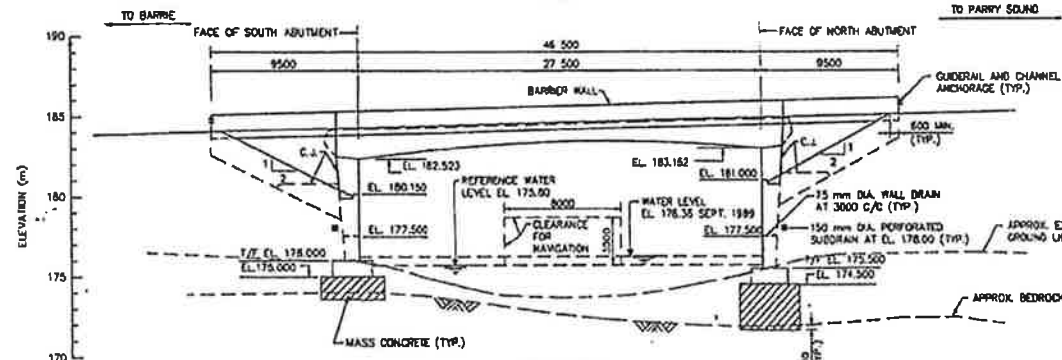
- DO-3501 MINIMUM GRANULAR BACKFILL REQUIREMENTS
DO-3504 RETAINING WALL BACKFILL REQUIREMENTS



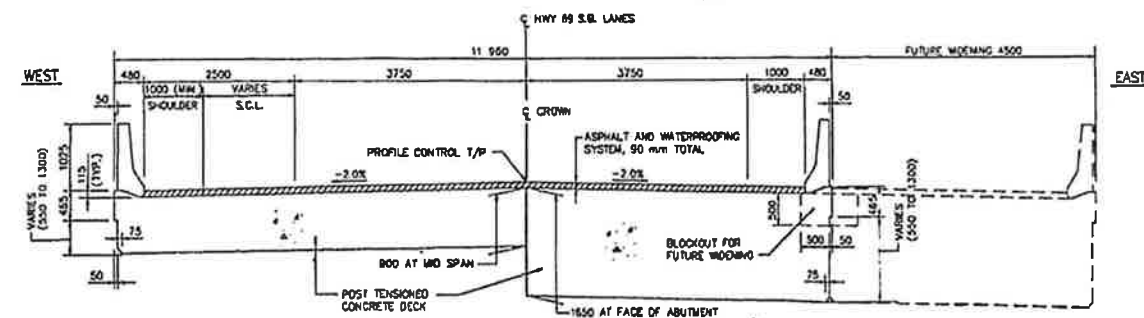
DATE	BY	DESCRIPTION
DESIGN	CL	CHK. ATC CODE 0400-83 LOAD CLASS A1 DATE DEC/90
DRAWN	MT	CHK. ATC SITE 42-545 STRUCT. SCHEME DWG. 1



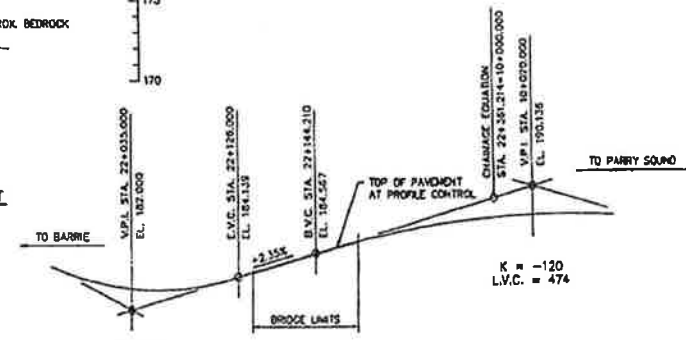
PLAN
1:200



ELEVATION
1:200

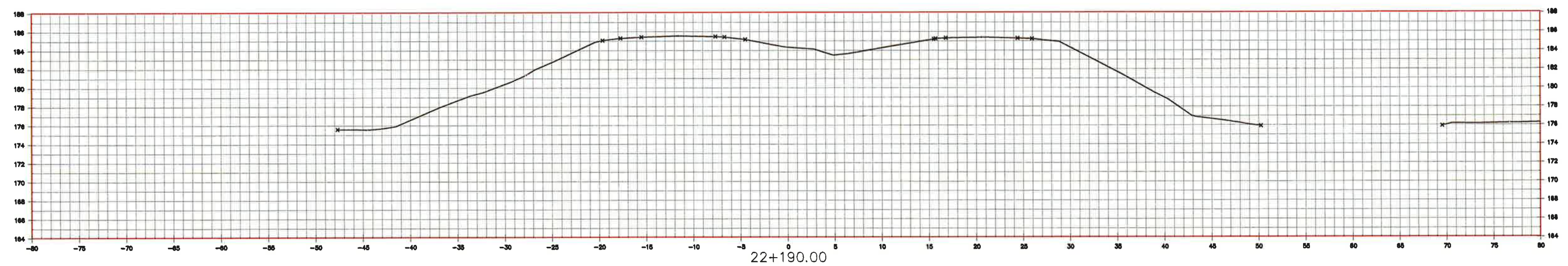
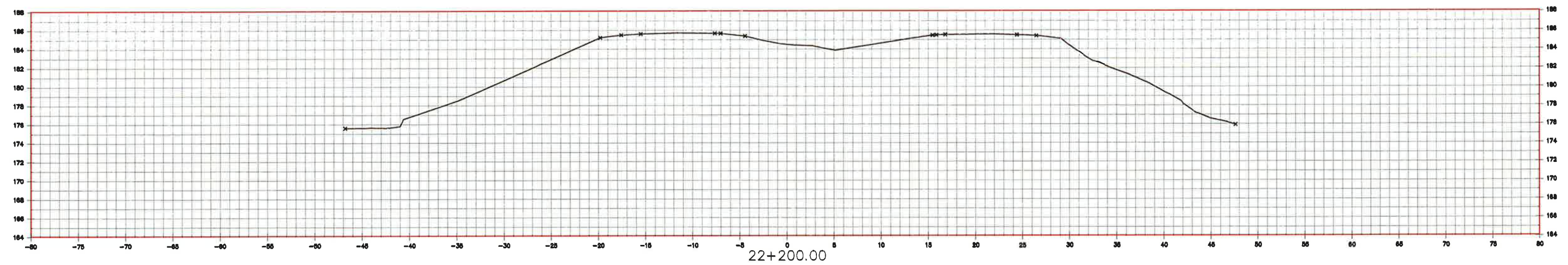


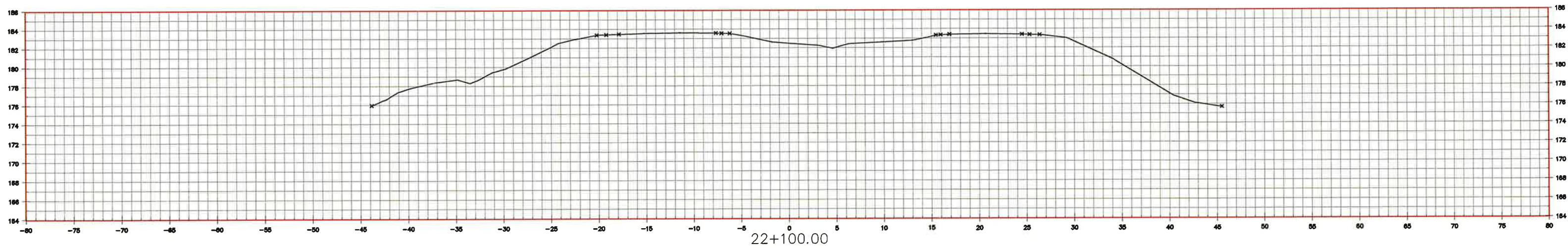
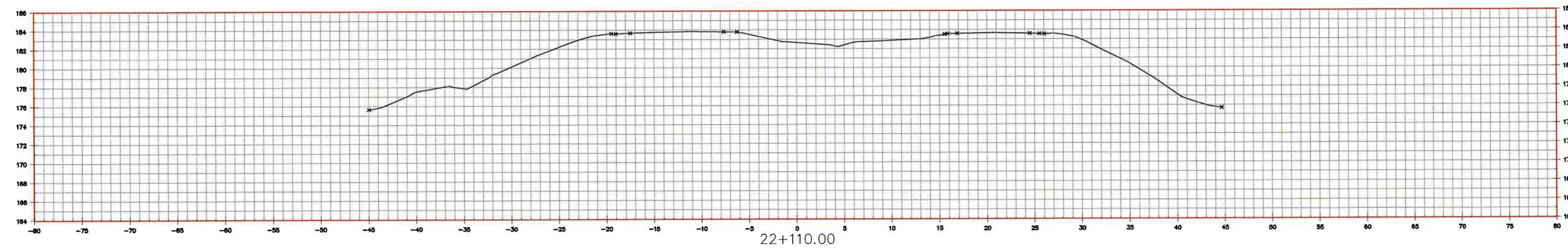
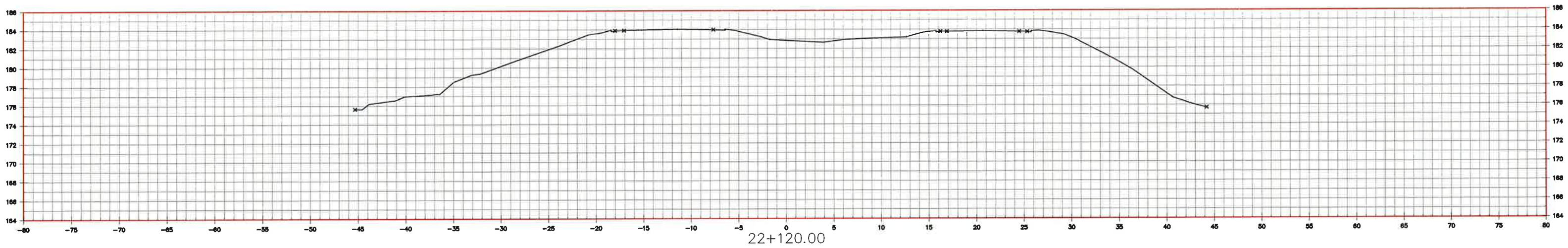
BM 184.008
Cut Cross Dd M of S E End
of Bridge on Conc. Ret. Wall
21.0 M 22+130.4



PROFILE OF HWY 69 SOUTHBOUND LANES
N.T.S.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING





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)	<ul style="list-style-type: none"> -Lower cost than deep foundation options -Dewatering and unwatering required -May require extensive shoring 	<ul style="list-style-type: none"> -Greater shoring effort will likely be needed in comparison with caisson and micropile options 	Low to Medium	<ul style="list-style-type: none"> -Feasible -Temporary support system is required
Shallow Foundations with re-use of the existing mass concrete	<ul style="list-style-type: none"> -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 	<ul style="list-style-type: none"> -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	<ul style="list-style-type: none"> -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	<ul style="list-style-type: none"> -May reduce shoring effort 	<ul style="list-style-type: none"> -Existing mass concrete may create problems 	Medium	<ul style="list-style-type: none"> -Not feasible for the prevailing subsurface conditions
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	<ul style="list-style-type: none"> -May reduce shoring effort 	<ul style="list-style-type: none"> - Existing mass concrete may create problems, but to a lesser degree than driven piles 	Medium	<ul style="list-style-type: none"> -Can be considered if shoring is expected to be extensive
Micropiles	<ul style="list-style-type: none"> -May reduce shoring effort -Equipment easier to operate under low overhead and restricted access conditions -Can be installed through mass concrete if encountered 	<ul style="list-style-type: none"> -Rock fill, if encountered, may create problems during installation but to a lesser extent than caisson option 	Higher in comparison with other options	<ul style="list-style-type: none"> -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.