



**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORTS
REPLACEMENT AND REHABILITATION OF
FOUR EXISTING BRIDGES OVER THE
SEVERN RIVER AND SEVERN RIVER BOAT
CHANNEL, TOWNSHIP OF BAXTER,
MTO CENTRAL REGION
G.W.P. 2360-09-00 & G.W.P. 2376-09-00
GEOCRES NO. 31D-555**

McCormick Rankin Corporation
Project: TRANETOB20462AA
July 31, 2013

REPORT



July 31, 2013

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

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

Dear Mr. Hui:

**RE: Preliminary Foundation Investigation and Design Reports
Replacement and Rehabilitation of Four Bridges over the Severn River and Severn River
Boat Channel, Township of Baxter, MTO Central Region,
G.W.P. 2360-09-00 & G.W.P. 2376-09-00, GEOCRE 31D-555**

Please find attached our preliminary 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".

Zuhtu Ozden, P.Eng.
Senior Principal



**PRELIMINARY FOUNDATION
INVESTIGATION REPORT -
REPLACEMENT AND REHABILITATION
OF FOUR EXISTING BRIDGES OVER THE
SEVERN RIVER AND SEVERN RIVER
BOAT CHANNEL, TOWNSHIP OF BAXTER
MTO CENTRAL REGION
G.W.P. 2360-09-00 & G.W.P. 2376-09-00
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**PRELIMINARY FOUNDATION INVESTIGATION REPORT
REPLACEMENT AND REHABILITATION OF FOUR BRIDGES
OVER THE SEVERN RIVER AND SEVERN RIVER BOAT CHANNEL,
TOWNSHIP OF BAXTER, MTO CENTRAL REGION,
G.W.P. 2360-09-00 & G.W.P. 2376-09-00**

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by McCormick Rankin Corporation (MRC) to carry out a preliminary foundation investigation for the proposed replacement and rehabilitation of the existing Highway 400 bridges over the Severn River and the Severn River Boat Channel in the Township of Baxter, Ontario.

Existing bridge information, based on available bridge drawings, are summarized in the table below.

**Table 1.1
Bridge Information**

Title	Site Number	Year Built	Length (m)	Width (m)	Existing Structure Type	Proposed Structure Strategy
Severn River Bridge, NB	42-86/1	1957	27.4	10.4	Single Span Rigid Frame	Replacement
Severn River Bridge, SB	42-86/2	1991	27.5	12.5	Single Span Rigid Frame	Rehabilitation
Severn River Boat Channel Bridge, NB	42-87/1	1957	87.3	11.3	3 Rib Deck Arch	Replacement
Severn River Boat Channel Bridge, SB	42-87/2	1992	118	12.0	3 Span Slab on Steel I girder - Integral Abutments and Piers	Rehabilitation

This preliminary investigation (Phase 1) was carried out based on the available information only (desktop study) and no borehole investigation was performed for this study.

2 SITE DESCRIPTION AND GEOLOGY

The site is located on Highway 400 at the mouth of Little Lake to 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. Overburden, where present, consists of silty sands, either surficial loose deposits or as dense glacial till above the bedrock. An over-consolidated stiff to hard silty clay is also present in areas where bedrock is relatively deeper in occurrence. Organic mucks are also common in marshy areas.

3 SUBSURFACE CONDITIONS

3.1 Past Reports

The existing subsurface information from MTO GEOCREST information system was used to prepare this report. It should be noted that some old bridges' names do not match with the existing bridge names and

the relevance of some of the previous foundation investigations is not clear on existing GEOCREs information. As a consequence, this report is prepared based on our best approximation. A number of previous geotechnical investigations has been conducted at the site and below is a brief summary of the available GEOCREs information.

❖ **GEOCREs 31D-116, Foundation Investigation for Two Bridge Sites, Highway 103 Revision, Port Severn Area, Ontario, Racey, MacCallum & Associates Limited, 1956**

The purpose of this investigation was to assess the subsurface conditions at the then proposed two structure locations. Nine boreholes were carried out at the site and two of which were advanced in the marshy areas (these boreholes are numbered 1 through 9 in the Racey, MacCallum report but are designated as A-series boreholes, A1, A2 for the purposes of this report). Boreholes A4 and A5 were advanced in the marshy areas, across the boat channel.

The subsoil conditions at the site were found to be relatively uniform in a horizontal direction over the site, although the clay deposits were thicker and softer in marsh areas to the south of the proposed bridge. The organic muck, comprising the upper few feet of the overburden, was in an extremely soft condition and it was underlain by stiff to very stiff silty clay. In the marshy area this silty clay became softer below a depth of 15 feet (4.5 m). About 4 feet (1.2 m) dense fine sand was encountered immediately above the granite and biotite gneiss bedrock. For the channel crossing, shallow foundation seated on the bedrock was recommended. For the Little Lake overflow crossing, pile foundations driven to bedrock were recommended due to the relatively thicker overburden.

❖ **GEOCREs 31D-323, Foundation Investigation Report, Proposed Crossing, Highway 69 SBL/Severn River Boat Channel, District 5, Owen Sound, W.P. 37-80-05, Strata Engineering Corp., 1990**

The purpose of this investigation was to assess the subsurface conditions for the proposed new southbound water crossing structure foundations. Eight boreholes (including two hand auger holes and three probe test holes) were carried out using a raft-mounted drill at the site (re-designated as B-series boreholes, B1, B2 etc. for the purposes of this project). The predominant feature of this site is the shallow water and bedrock, and bedrock outcrops. The overburden, where present, consists of lake bed silty sand (0 to 1.1m thick) and underlying sandy glacial till (0.3 to 2.6 m thick). Due to the shallowness of bedrock, all abutments and piers were recommended to be founded on bedrock.

❖ **GEOCREs 31D-326, Foundation Investigation Report, Proposed Crossing, Highway 69 SBL/Severn River, District 5, Owen Sound, W.P. 37-80-04, Strata Engineering Corp., 1990**

The purpose of this investigation was to assess the subsurface conditions for the proposed new southbound water crossing structure foundations. Seven boreholes (including one probe test hole and three dynamic cone penetration tests) were carried out at the site (re-designated as C-series boreholes, C1, C2 etc. for the purposes of this project). The predominant features of this site are the shallow water and bedrock, and bedrock outcrops. The overburden, where present, consisted of a stiff to hard silty clay, overlying a thin veneer of dense silty sand till. The depth of the overburden to the surface of the proven or inferred bedrock at the location of the borehole/probe locations ranged from near nil to 4.6 m. Shallow foundations placed on sound bedrock were recommended but consideration was also given to short caisson foundations due to the relatively thicker overburden, unwatering requirements and the sloping nature of bedrock towards the river.

3.2 Compiled Subsurface Conditions Based on Previous Investigations

3.2.1 Northbound Severn River Bridge Subsurface Conditions

Based on the available information (GEOCRES 31D-116, GA drawings, recent MTO bridge inspection report and site photographs), it is assumed that Little Lake Overflow crossing (at old chainage 212+50) may be the current northbound Severn River Bridge. In 1956, Racey, MacCallum and Associates Limited carried out a site investigation consisting of seven boreholes (i.e. Boreholes A1, A2, A3, A6 through A9) for the northbound Severn River Bridge, as a part of Highway 103 (currently Highway 400) revision. Exact locations of the boreholes cannot be defined due to the quality of data (e.g. no reference points and poor scanning quality). Boreholes A3 and A8 were drilled in the marshy area to get information on the stability of soil underlying the marshy ground to the south of the Little Lake Overflow. Boreholes A1, A6 and A7 were drilled at Little Lake Overflow bridge location. Borehole location plan and subsurface profile drawings prepared for northbound Severn River bridges are included in this report as Drawings 2 and 3. The table below summarizes the available boreholes.

Table 3.2.1.1 Borehole Information

Company / Year of Investigation (Geocres #)	Borehole No.	Surface condition (test from)	Top Elevation of Investigation (m)	Bottom Elevation of Investigation (m)	Remark
Racey, MacCallum and Associates Limited/ 1956 (Geocres 31D-116)	A1	water	176.8*	166.3	rock coring
	A2	water	176.8*	168.4	-
	A3	swamp	176.8*	166.4	-
	A6	water	176.8*	167.9	rock coring
	A7	water	176.8*	168.4	rock coring
	A8	swamp	176.8*	168.2	-
	A9	water	176.8*	171.5	-

*at all boring locations on water the water elevation has been taken as El. 580 feet, although water level fluctuated over short period (Geocres 31D-116).

Note that the geotechnical data used in this study were logged and prepared by Racey, MacCallum and Associates Limited. Therefore, some limited refinement of their interpretations was made by Coffey when compiling the data.

The following assumptions were made for our interpretation:

- Elevations were assumed to be based on the geodetic datum.
- Imperial elevations were directly converted to metric and no correction factor was used.
- Penetration test, field vane test, Atterberg Limits tests and other test results presented in this section are approximations.

The following provides a compiled overview of the subsurface conditions encountered at the bridge site. The following descriptions of the individual strata are provided to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site.

3.2.1.1 Organic Soils

At ground surface in the marshy areas and at the river bed (below water), an about 1.3 to 2.6 m thick blackish muck, sometimes peaty, was encountered in Boreholes A2, A3, A8 and A9. In general, relatively

higher organic contents are not uncommon within a shallow water deposits such as the present site. This organic rich soil appears to have a very soft consistency (cohesive soils) or is in a very loose compactness condition (cohesionless soils).

3.2.1.2 Silty Clay

From the river bottom and below the organic rich soil, a silty clay deposit was encountered in Boreholes A1, A2, A3, A6, A7, A8 and A9. This is a cohesive soil and its thickness at the borehole locations was 2.7 to 5.6 m and it increases up to about 6 m in the marshy area (Borehole A3). Some sand pockets/lenses/seams were found within the silty clay deposit.

Twelve Atterberg Limits tests were performed on the silty clay soil samples and results are given in Appendices A and B. The tests show the following index values;

Liquid Limit:	34 - 60 %
Plastic Limit:	17 - 26 %
Plasticity Index:	14 - 34
Natural Moisture Content:	29 – 55 % (extremely high moisture contents were excluded)

These results are characteristics of clayey soils of medium to high plasticity.

Based on the field test results (e.g. field vane tests and penetration tests), consistency of this silty clay can be described as stiff to very stiff. In the marshy area this silty clay became softer below a depth of about 4.5 m.

3.2.1.3 Sand

A sand layer comprised the basal soil layer overlying bedrock at the site. This granular soil layer consists of medium dense to dense fine to coarse sand. The thickness of this granular soil layer varies from very minimal to 2.4 m.

3.2.1.4 Bedrock

A granite and biotite gneiss bedrock was found at the relatively shallow depths at the bridge site. Its quality was determined by coring. Seams were observed within the depth of exploration but those will unlikely cause foundation problems under anticipated loading conditions. Table 3.2.1.4.1 summarizes the bedrock investigation and its quality.

Table 3.2.1.4.1 Bedrock Conditions

Company / Year of Investigation (Geocres #)	Borehole No.	Ground Elevation (m)	Bedrock Elevation/Depth (m)	Coring Length (m)	Recovery (%)	RQD
Racey, MacCallum and Associates Limited/ 1956 (Geocres 31D-116)	A1	175.4	169.5/7.3	3.2	100	-
	A2	176.5	168.4/8.1	No rock coring		
	A3	176.8*	166.4/10.4	No rock coring		
	A6	175.1	170.7/4.4	2.7	100	-
	A7	176.0	171.4/4.6	3.0	100	-
	A8	176.8*	168.2/8.6	No rock coring		
	A9	176.3	171.5/4.9	No rock coring		

*from swamp surface

Based on the recorded total rock core recovery value, the rock can be considered to be in a good condition.

3.2.1.5 Groundwater and Lake Water Conditions

Some of the boreholes were drilled below the water level while in the others groundwater measurements were not made at the time of the investigation (1956).

At all boring locations on water, the water elevation has been taken as El. 176.8 m (580 feet), although water level fluctuated over short period of time as the result of wind action, between El. 176.7 m (579.7 feet) and 176.9 m (580.5 feet) during the field work.

3.2.2 Northbound Severn River Boat Channel Bridge Subsurface Conditions

For the purposes of this report, we assumed that the Channel Crossing at old chainage 190+00 presented in GEOCRE 31D-116 is the current northbound Severn River Boat Channel Bridge based on the available information (GEOCRE 31D-116, GA drawings, recent MTO bridge inspection report and site photographs). Racey, MacCallum and Associates Limited performed a site investigation in 1956 consisting of two boreholes plus 49 soundings at the bridge location as a part of Highway 103 (currently Highway 400) revision. Borehole and sounding location plan shown on GA drawing is included in this report as Drawing 4. Sounding depths from water to bedrock or gravel were also shown on the drawing.

The table below summarizes the available boreholes.

Table 3.2.2.1 Borehole Information

Company / Year of Investigation (Geocres #)	Borehole No.	Surface condition (test from)	Top Elevation of Investigation (m)	Bottom Elevation of Investigation (m)	Remark
Racey, MacCallum and Associates Limited/ 1956 (Geocres 31D-116)	A4	ground	178.6	168.9	rock coring
	A5	water	176.8*	166.4	rock coring

*at boring location on water the water elevation has been taken as El. 580 feet (176.8 m), although water level fluctuated over short period (Geocres 31D-116).

Note that the geotechnical data used in this study were logged and prepared by Racey, MacCallum and Associates Limited. However, some refinement of their interpretations was made by Coffey when compiling the data.

The following assumptions were made for our interpretation:

- Elevations were assumed to be based on the geodetic datum.
- Imperial elevations were directly converted to metric and no correction factor was used.

The following provides a compiled overview of the subsurface conditions encountered at the bridge site. The following descriptions of the individual strata are provided to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site.

3.2.2.1 Overburden

About 2 inches (50 mm) of organic sandy topsoil was found at the ground surface in Borehole A4 which was drilled from land (as opposed to Borehole A5, which was drilled from water in the channel). Below this

the Borehole 4 contacted thin layers of slab rock and some sand to a depth of about 1.9 m below the ground surface or to El. 174.7 m (based on the borehole data sheet). This may represent a highly weathered rock at the surface.

3.2.2.2 Bedrock

A granite and biotite gneiss bedrock was found at a shallow depth at the bridge site and its quality was determined by coring. Seams or cracks were observed within bedrock but those were found to be tight in general and thus of a nature that would unlikely cause foundation problems under anticipated loading conditions. Borehole A4, drilled on the north bank of the channel crossing, contacted alternate thin layers of slab rock with some sand to a depth of 1.9 m (6 feet 2 inches on the borehole data sheet). This material possibly represents highly weathered surface rock. Bedrock in this borehole was contacted below this material at El. 176.7 m. Relatively sound bedrock was encountered at a depth of 2.4 m (8 feet in the report) or at El. 176.2 m in Borehole A4. Borehole A5 was advanced from the water surface in the channel. This borehole contacted bedrock immediately below the water in the channel, at the ground surface. The table presented below summarizes the bedrock investigation and quality.

Table 3.2.2.2.1 Bedrock Conditions

Company / Year of Investigation (Geocres #)	Borehole No.	Drilling top Elevation (m)	Bedrock Elevation/Depth (m)	Coring Length (m)	Recovery (%)	RQD
Racey, MacCallum and Associates Limited/ 1956 (Geocres 31D-116)	A4	178.6	176.7/1.9	7.9	100	N/A
	A5	176.8	174.7/2.1	8.2	100	N/A

Based on the recorded total rock core recovery value of 100%, the rock mass quality at the boreholes can be estimated to be in a good condition.

3.2.2.3 Groundwater and Lake Water Conditions

No groundwater measurements were made at the time of investigation (1956).

At boring location on water the water elevation has been taken as El. 176.8 m (580 feet), although water level fluctuated over short period as the result of wind action, between El. 176.7 m (579.7 feet) and 176.9 m (580.5 feet) during the field work.

3.2.3 Southbound Severn River Boat Channel Bridge Subsurface Conditions

Based on the available information (GEOCRE 31D-323, GA drawings, recent MTO bridge inspection report and site photographs), it is assumed that Highway 69 Bridge over Severn River Boat Channel is the existing Highway 400 southbound Severn River Boat Channel Bridge. The following paragraphs are based on these premises. In 1989 and 1990, Strata Engineering Corp. carried out a foundation investigation for Highway 69 southbound Severn River Boat Channel Bridge. The report indicates that “the geology of the area is dominated by “granite gneiss” bedrock outcrops with shallow overburden. The Boat Channel was blasted into bedrock which is clearly visible below the water. Overburden, where present, consists of silty sands, either as surficial loose deposits or as a dense glacial till above the bedrock. Further to the south of this site, and beyond the limits of the investigation, a desiccated hard silt clay is also present as overburden”. Borehole location plan and soil strata (subsurface profile) drawings prepared by Strata

Engineering Corp for the proposed Highway 69 southbound Severn River Boat Channel Bridge are included in this report as Drawing 5.

Table 3.2.3.1 is summary of the available boreholes, probe holes and dynamic cone penetration tests at the bridge locations. It should be noted that the letter B was added in front of the borehole numbers to distinguish them from other boreholes advanced at other sites considered in this report.

Table 3.2.3.1 Borehole Locations and Drilling Elevations

Company / Year of Investigation (Geocres #)	Borehole No.	Surface condition (test from)	Top Elevation of Investigation (m)	Bottom Elevation of Investigation (m)	Remark
Strata Engineering Corp./1990 (Geocres 31D-323)	B1	ice	176.3	175.7	hand drilling
	B2	water	176.0	173.6	rock coring
	B3	water	176.0	174.8	probe testing
	B4	water	176.0	173.3	rock coring
	B5	water	176.1	168.0	rock coring
	B6	water	175.9	175.7	probe testing
	B7	water	176.0	174.5	probe testing
	B8	ground	177.1	176.3	hand drilling

The following provides a compiled overview of the subsurface conditions encountered at the southbound Severn River Boat Channel Bridge sites. The descriptions of the individual strata are provided to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site.

3.2.3.1 Surficial Silty Sand

The investigation indicates that the lake bottom is covered with a thin layer of silty sand containing organics and some gravel. The thickness of this material varies from 0 to 1.1 m. Where the river enters the lake, this deposit is absent and bedrock is clearly visible in the boat channel. A shallow veneer of sandy topsoil at the ground surface was overlying the surficial silt sand in Borehole B8 drilled on the west bank of the bridge site.

A grain size analysis was performed on a selected soil sample from the surficial silty sand layer (Borehole B2) and results are presented in Appendices A and B. The results are as follows;

Gravel:	18 %
Sand:	56 %
Silt and Clay Size Particles:	26 %

The soil is in a very loose condition based on recorded N values of 2 blows/0.3 m in two of the boreholes.

3.2.3.2 Silty Sand Till

A silty sand till was encountered in Boreholes B1, B2, B5 and B8. Its thickness ranged from 0.3 m in Borehole B1 to 2.6 m in Borehole B5. The silty sand till directly overlies granite gneiss bedrock.

Four grain size analyses were performed on the silty sand till samples from southbound bridge locations (Borehole B5) and results are presented in Appendices A and B. The results are as follows;

Gravel:	8 - 39 %
Sand:	41 - 55 %

Silt and Clay Size Particles: 20 - 37 %

The N-values in this granular soil (i.e. non-cohesive) ranged between 59 and 88 blows/0.3 m indicating a very dense condition

In Boreholes B1 and B8 the thickness of the glacial till could not be established, as refusal to auger penetration was reached after initial penetration into the till stratum. However, it is believed that at these locations, the glacial till deposit is fairly thin ($1.0 \pm m$), and directly overlies the bedrock.

3.2.3.3 Bedrock

A pink granite gneiss bedrock was encountered at shallow depths at the project site along the centreline of proposed structure. The quality of the bedrock was determined by coring. It should be noted that in 1990 investigation, some shattered rocks were found at the boat channel area (See Borehole B5) due to the previous channel construction (e.g. blasting). In general, the bedrock is sound, except locally near the boat channel.

The table below summarizes the bedrock investigation and quality.

Table 3.2.3.3.1 Bedrock Conditions

Company / Year of Investigation (Geocres #)	Borehole No.	Drilling top Elevation (m)	Bedrock Elevation/Depth (m)	Overburden Thickness (m)	Coring Length (m)	Recovery (%)	RQD
Strata Engineering Corp./1990 (Geocres 31D-323)	B1	176.3	N/A	>0.6	N/A		
	B2	176.0	174.5/1.5	0.7	0.9	90/95	73/95
	B3	176.0	174.8/1.2*	0	Probe test (bedrock is visible)		
	B4	176.0	175.0/1.0*	0	1.7	99	65
	B5	176.1	170.9/5.2	3.7	2.9	55/96/64	0/29/47
	B6	175.9	175.7/0.2*	0	Probe test		
	B7	176.0	174.5/1.5*	0	Probe test		
	B8	177.1	N/A	>0.8	N/A		

* Indicates depth of water

From the above table it can be surmised that the surface of the bedrock at the site is generally at El. 175.7 to 174.5 m, except at Borehole B5 location where it appears to be deeper (i.e. maybe gouged out) at El. 170.9 m.

Based on the recorded RQD values, bedrock at the site can be classified as being generally in a fair to excellent condition except at Borehole B5 location where RQD values of between 0 and 47% were recorded indicating a very poor to fair condition.

3.2.3.4 Groundwater and Lake Water Conditions

The groundwater level in Boreholes B1 and B8 was the same as the water level in the lake. The lake water level fluctuates depending on the need of the Trent-Severn waterway system.

Due to the flat topography of the area, the shoreline configuration (including bedrock outcrops in the water course) changes considerably with changing water levels.

3.2.4 Southbound Severn River Bridge Subsurface Conditions

Based on the available information (GEOCRE 31D-326, GA drawings, recent MTO bridge inspection report and site photographs), it is considered that Highway 69 Bridge over the Severn River is the existing Highway 400 southbound Severn River Bridge. Strata Engineering Corp. carried out a foundation investigation for Highway 69 southbound Severn River Bridge in 1989 and 1990. Borehole location plan and soil strata (subsurface profile) drawings, prepared by Strata Engineering Corp for the proposed Highway 69 southbound Severn River Bridge are included in this report as Drawing 6. The report states that "The geology of the area is dominated by paleozoic granitic gneiss bedrock outcrops with shallow overburden. Overburden, where present, consists of silty sands, either as surficial loose deposits or as a dense glacial till above the bedrock. An over-consolidate stiff to hard silty clay is also present as overburden in areas where the bedrock is relatively deeper in occurrence".

Table 3.2.4.1 is a summary of the available boreholes, probe holes and dynamic cone penetration tests at the site. It should be noted the prefix letter C was added to the borehole numbering to enable the reader to distinguish the boreholes drilled at this site from the adjacent sites.

Table 3.2.4.1 Borehole Locations and Drilling Elevations

Company / Year of Investigation (Geocres #)	Borehole No.	Surface condition (test from)	Top Elevation of Investigation (m) (water)	Bottom Elevation of Investigation (m)	Remark
Strata Engineering Corp./1990 (Geocres 31D-326)	C1	water	176.1	175.5	probe testing
	C2	water	176.1	171.5	rock coring
	C3	water	176.3	171.9	DCPT
	C4	water	176.2	169.8	rock coring
	C5	water	176.3	174.1	overburden drilling
	C6	water	175.9	173.7	DCPT
	C7	water	175.9	173.5	DCPT

The following provides a compiled overview of the subsurface conditions encountered at the southbound Severn River Bridge site. The descriptions of the individual strata are provided to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site.

3.2.4.1 Surficial Silty Sand

The lake bottom was found to be covered with a thin layer of silty sand containing trace to some organics and some gravel (Boreholes C2 and C5). The thickness of this granular material (i.e. non-cohesive) varies from 25 mm to 0.5 m.

One grain size analysis was performed on a selected soil sample from this layer (Borehole C2) and results are presented in Appendices A and B. The results are as follows;

Gravel:	13 %
Sand:	57 %
Silt and Clay Size Particles:	30 %

The soil appears to be in a loose condition based on a recorded N value of 6 blows/0.3 m.

3.2.4.2 Silty Clay

From the river bed and below the surficial silty sand, a silty clay deposit was encountered at the site. This is a cohesive soil. At Boreholes C2, C4 and C5, where this deposit was sampled, the thickness of the silty clay deposit ranged from 1.0 to 1.7 m. Elsewhere, at the Dynamic Cone Penetration test holes (Boreholes C3, C6 and C7), its thickness is estimated to range between 1.0 and 2.5 m.

Four Atterberg Limits tests were performed on selected silty clay soil samples and the results are given in Appendices A and B. The tests show the following index values;

Liquid Limit:	36 - 43 %
Plastic Limit:	16 - 19 %
Plasticity Index:	17 - 24
Natural Moisture Content:	24 – 37 %

These results are characteristics of clayey soils of medium plasticity.

Based on the recorded N values ranging from 9 to in excess of 100 blows/ 0.3 m, consistency of this silty clay can be described as stiff to hard.

3.2.4.3 Silty Sand Till

In general, the silty clay is underlain by a thin stratum of silty sand with some gravel (glacial till). This stratum was sampled only at Borehole C4 where its thickness was found to be about 200 mm. It is inferred to have been encountered also at Boreholes C3, C6 and C7 in the dynamic cone penetration resistance testing conducted at these locations, and would appear from the data to be less than 300 mm thick at these locations as well.

One grain size analysis was performed on the silty sand till sample from Borehole C4 and results are presented in Appendices A and B. The results are as follows;

Gravel:	20 %
Sand:	62 %
Silt and Clay Size Particles:	18 %

Based on the recorded N-values and dynamic cone penetration tests results, this granular soil layer can be described as being in a compact to very dense condition.

3.2.4.4 Bedrock

A pink granite gneiss bedrock underlies the silty clay stratum directly, or the silty sand (glacial till) deposit. It was cored in Boreholes C2 and C4. Elsewhere at this site, the bedrock is inferred to have been contacted when refusal to the dynamic cone penetration testing was encountered.

The table below summarizes the bedrock investigation and quality.

Table 3.2.4.4.1 Bedrock Condition at Borehole Locations

Company / Year of Investigation (Geocres #)	Borehole No.	Drilling top Elevation (m)	Bedrock Elevation/Depth (m)	Overburden Thickness	Coring Length (m)	Recovery (%)	RQD
Strata Engineering Corp./1990 (Geocres 31D-326)	C1	176.1	175.5**/0.6**	*	Probe test 100/86		
	C2	176.1	173.7/2.4	1.5	2.2	100/100	100/86
	C3	176.3	171.9**/4.4**	1.9**	DCPT refusal on probable bedrock		
	C4	176.2	171.6/4.6	1.7	1.8	77/100	77/100
	C5	176.3	174.1**/2.2**	1.0**	No rock coring		
	C6	175.9	173.7**/2.2**	1.0**	DCPT refusal on probable bedrock		
	C7	175.9	173.5**/2.4**	1.1**	DCPT refusal on probable bedrock		

* a very thin layer of silty sand and organics on surface of inferred bedrock

** inferred

Based on the recorded RQD values, bedrock at the site can be generally classified as being in a good to excellent condition.

3.2.4.5 Groundwater and Lake Water Conditions

No groundwater observation was presented in the southbound Severn River Bridge report.

The lake water level fluctuates depending on the needs of the Trent-Severn waterway system. Maximum fluctuations noted in one day were 0.3 m during this investigation.

Due to the flat topography of the area, the shoreline configuration (including bedrock outcrops in the water course) changes considerably with changing water levels.

For and on behalf of Coffey

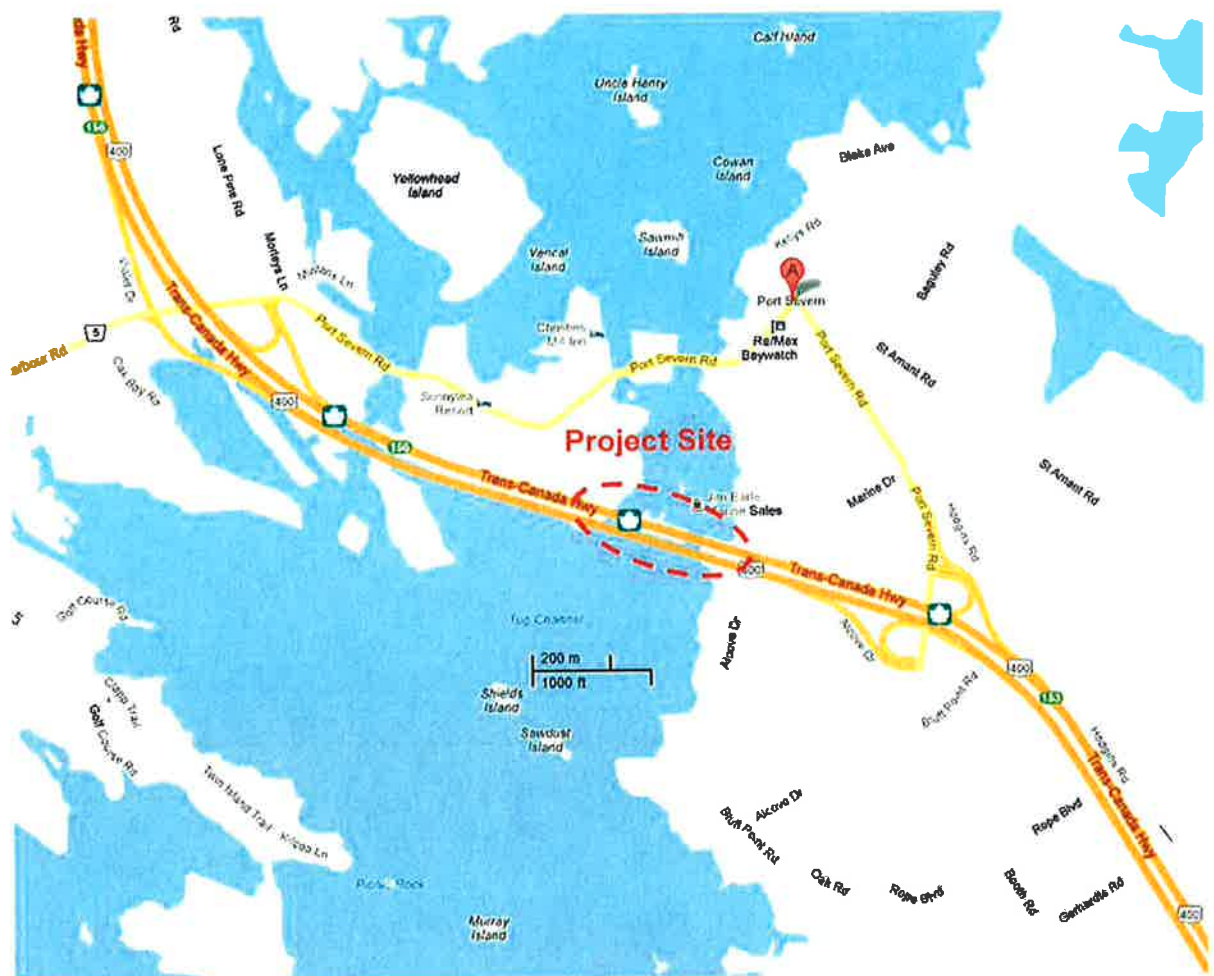

Gwangha Roh, P.Eng., Ph. D.
 Senior Geotechnical Engineer




Zuhtu Ozden, P.Eng.
 Senior Principal



Drawings

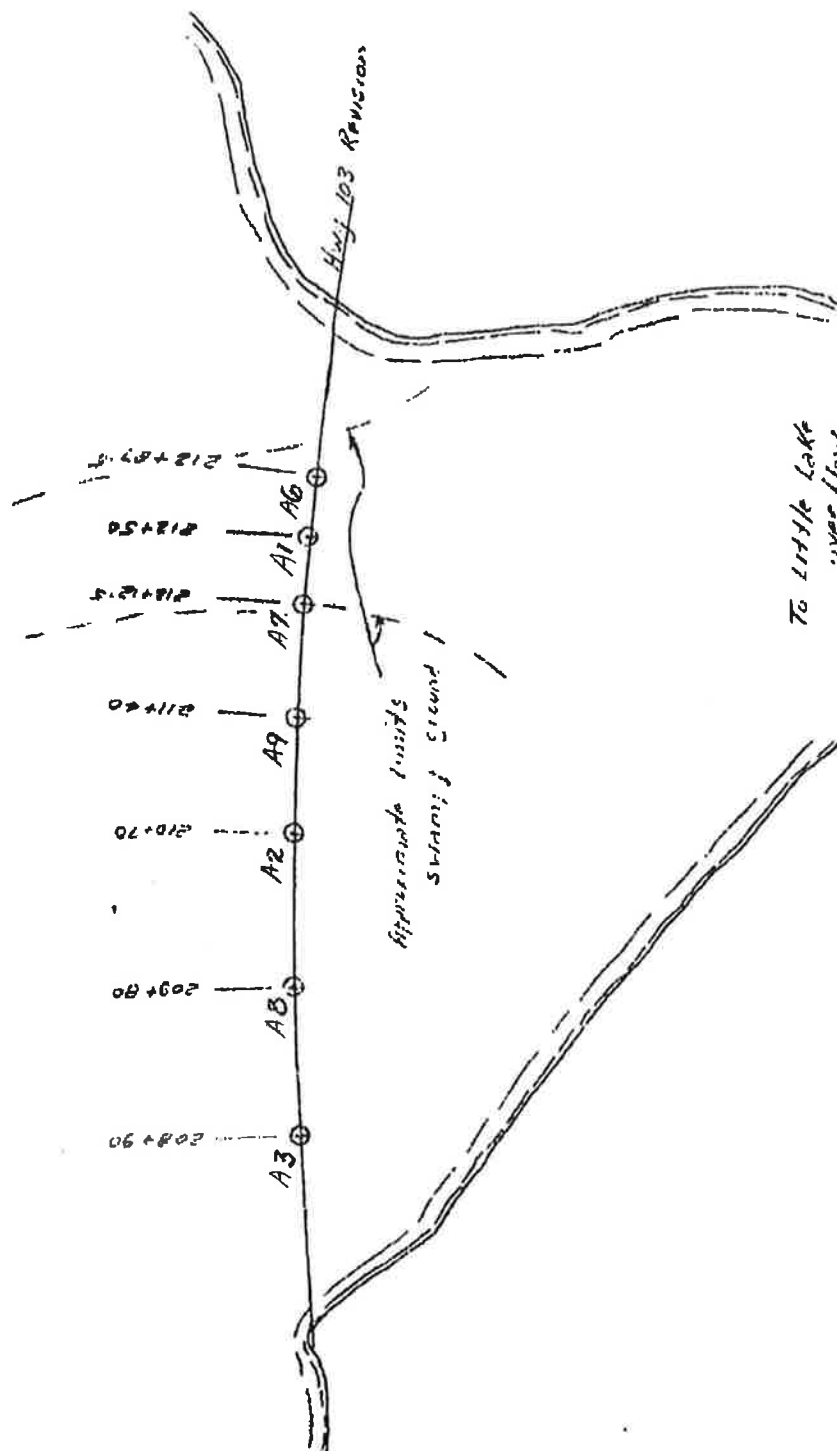


Drawing 1 Site Plan

Order No. S-500-501/7-383

Drawing 2

Prep. By B. F. W.



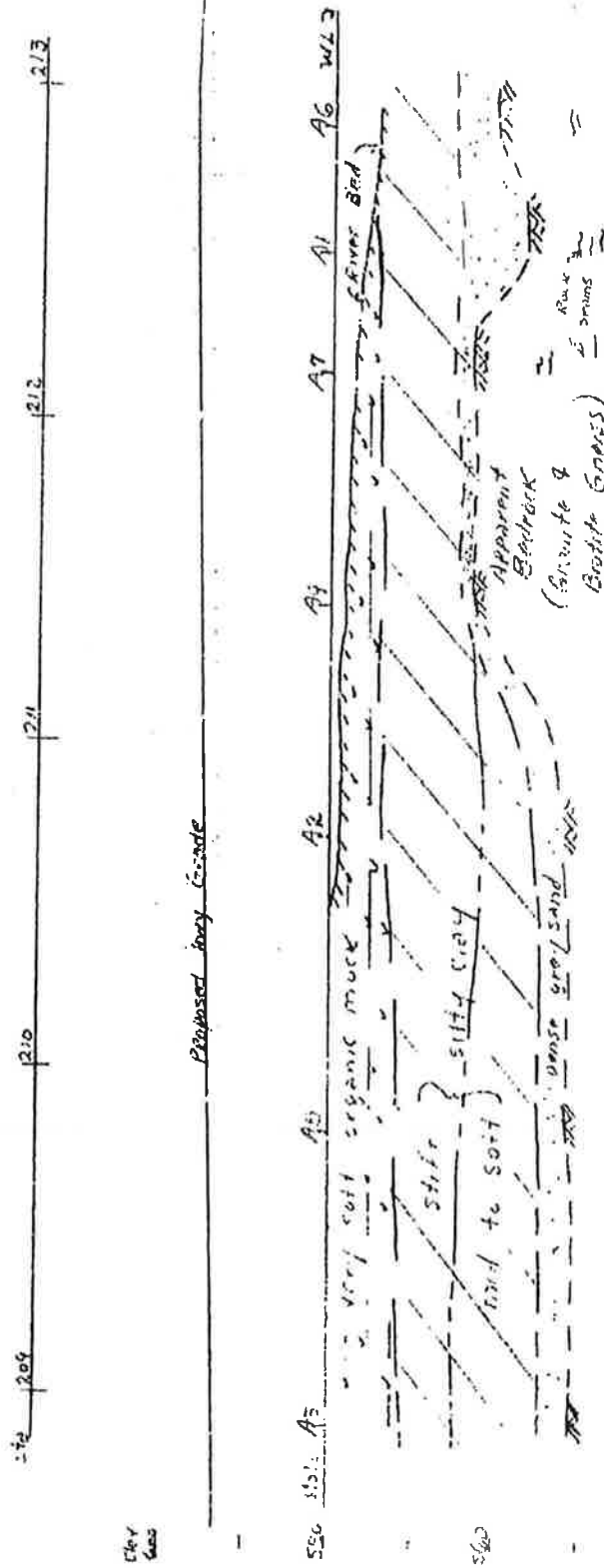
LOCATION OF BOREHOLES
Scale 1" = 100'

Racey, MacCallum & Associates Ltd.

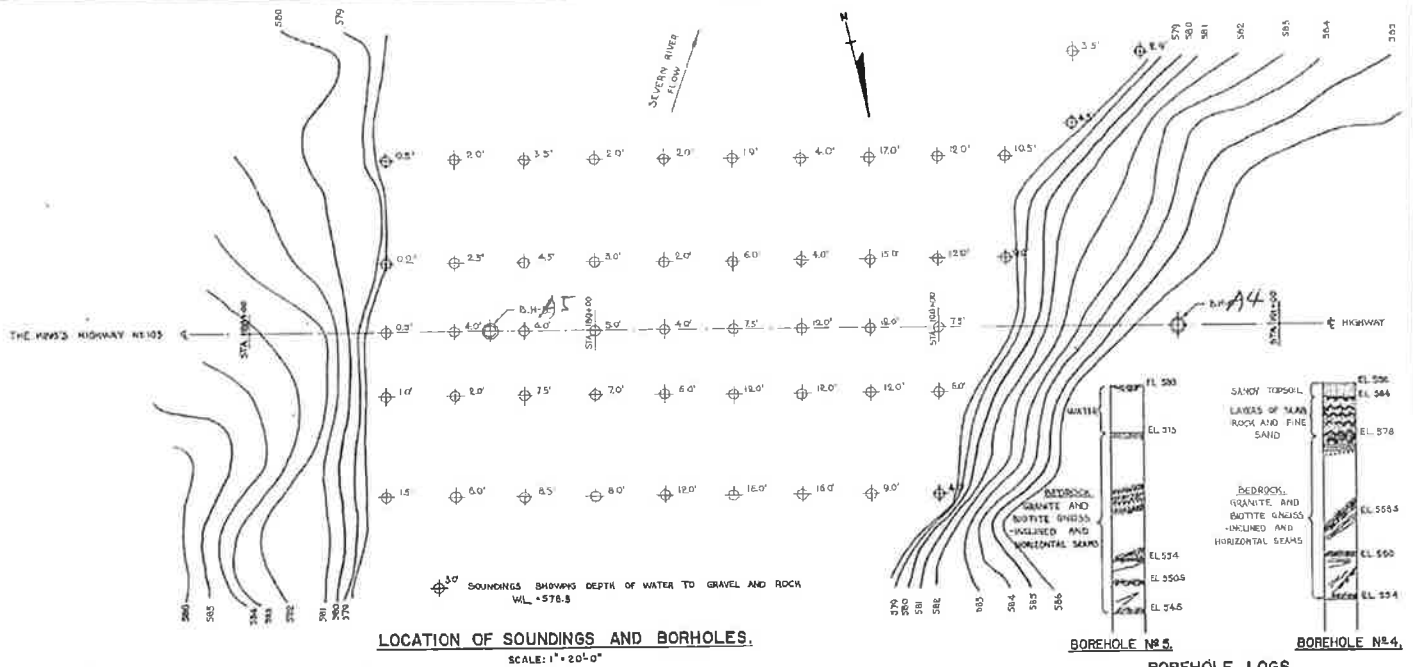
Order No. 5510 501 1995

Prep. By J.T.

Drawing 3

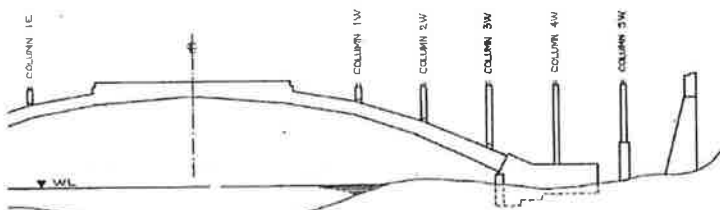


Racey, MacCallum & Associates Ltd.

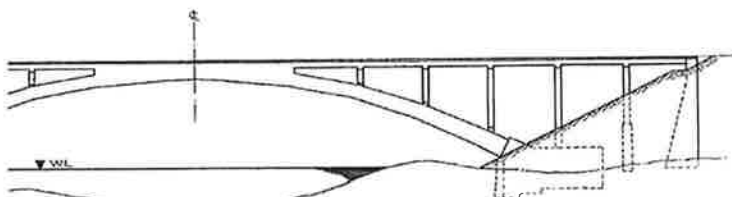


- NOTES:**
- THIS DRAWING TO BE READ IN CONJUNCTION WITH DWS 03783-3, 4 & 5.
 - SOUNDINGS ARE REPRODUCED FROM Q.M.D. PLAN E-347-1.
 - DRILLING IN BEDROCK OR CONCRETE WORK ON THIS STRUCTURE MUST NOT START BEFORE CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE ENGINEER.
 - SOUNDINGS AND ROCK ELEVATIONS ARE NOT GUARANTEED BY D.H.A.

Drawing 4



ICE AND CONCRETE HINGES AT CROWN AND THRUST BLOCKS.



IT CROSS BEAMS, STRINGERS AND DECK, WORKING SIMULTANEOUSLY FROM & ON BOTH SIDES
: SIDE SHALL NOT BE MORE THAN 20 FT. IN ADVANCE OF THE OTHER.
ICE APPROACH FILL AFTER SUPERSTRUCTURE HAS SUFFICIENTLY CURED.

REVISIONS	DATE	BY	DESCRIPTION

WP 56A-56

T.O. LAZARIDES, LOUNT AND PARTNERS
CONSULTING ENGINEERS

HEAD OFFICE: 205 DAVENPORT RD. TORONTO

DEPARTMENT OF HIGHWAYS, ONTARIO
BRIDGE OFFICE - TORONTO

SEVERN RIVER BRIDGE
BOAT CHANNEL

THE KING'S HIGHWAY No. 103 (T.G.M.) DIST. No. 11

DIST. OF MUSKOGA LOT 27 CON. 2

LAYOUT OF FOOTINGS, BOREHOLES
AND CONSTRUCTION SEQUENCE

APPROVED: *[Signature]* BRIDGE SUPERVISOR

DATE: JANUARY 23, 1957

REVISIONS: 57-83

D-3783-2

THIS PRINT SUPERSEDES ALL PRINTS BEARING A PREVIOUS LETTER

D-3783










CONT No
WP No 37-80-05

SEVERN RIVER BOAT CHANNE

WATER HOLE LOCATIONS & SOIL STRATA

1



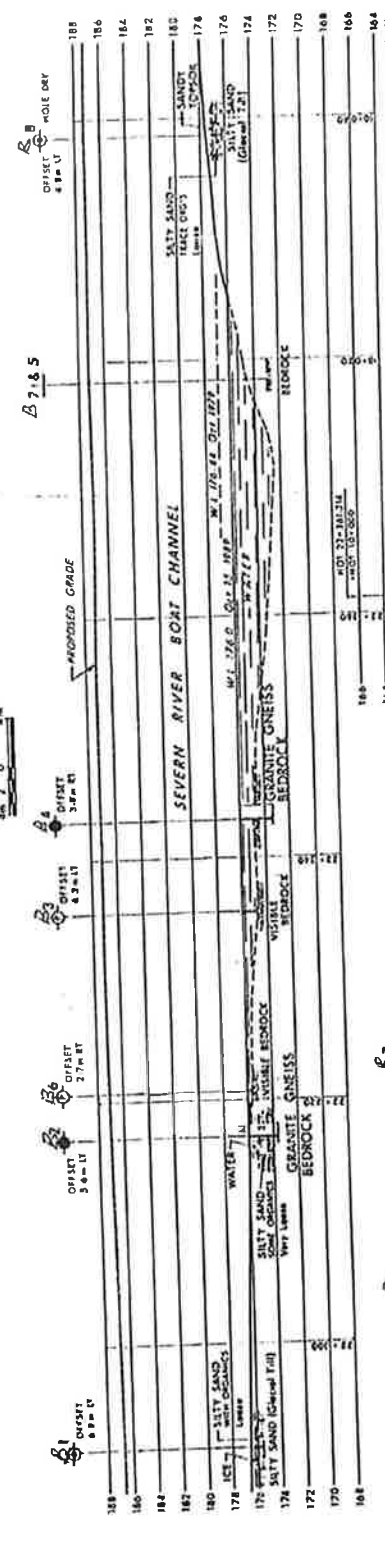
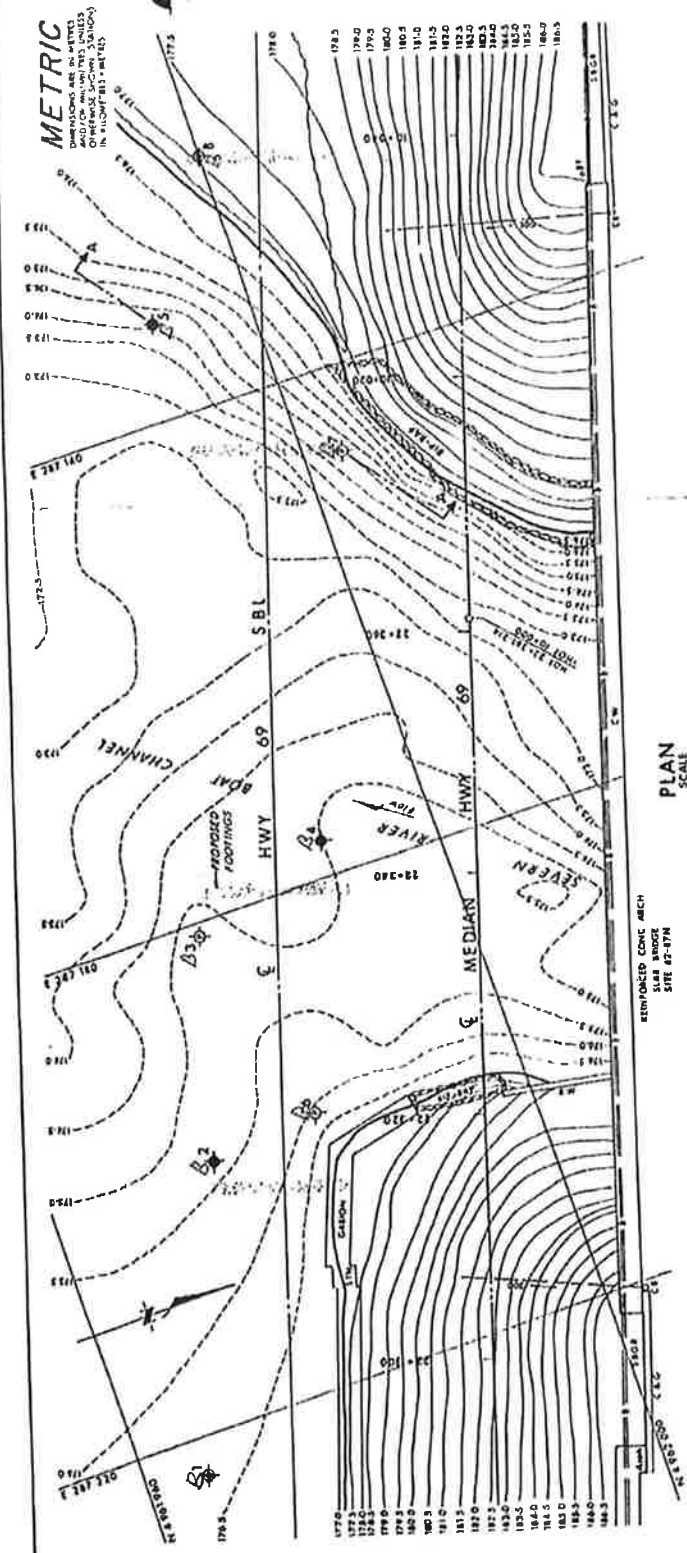
LEGEND	
	Bare Hole
	Drill Hole Core Penetration
	Bore Hole & Core
	N
	Core Bore 0.3 - 1.5m Pen Test
	Core Bore 1.0 - 3m (40% Core 23)
	N. at time of test 1980
	Hand Auger Hole
	Probe Hole

NO	REGISTRATION	CO-OP
21	176-3	4 961 095
22	176-0	4 980 074
23	176-0	4 951 939
24	176-0	4 961 991
25	176-1	4 961 982
26	175-9	4 961 983
27	176-0	4 962 003
28	177-1	4 962 000

Drawing 5

NOTE:-
The boundaries between the eight
one or more new locations. Several
boundaries are claimed from each
of the county boundaries mentioned
above and the various documents
concerning the various County boundaries
are not the same documents in each
county and the locations of the

SECRET No J1D-323



Appendix A

Record of Borehole Sheets

RACEY MACCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: A1

Project: Severn River Bridge
 Location: Vicinity Sta 212 Hwy 103 Revision: 1 B
 Hole Location: Hwy 103 Sta 212.50
 Hole Elevation and Datum: 580 MSL
 Field Work Begun: July 23/56 Ended: July 23/56 Date: W.T.

Field Supervision: F.C.

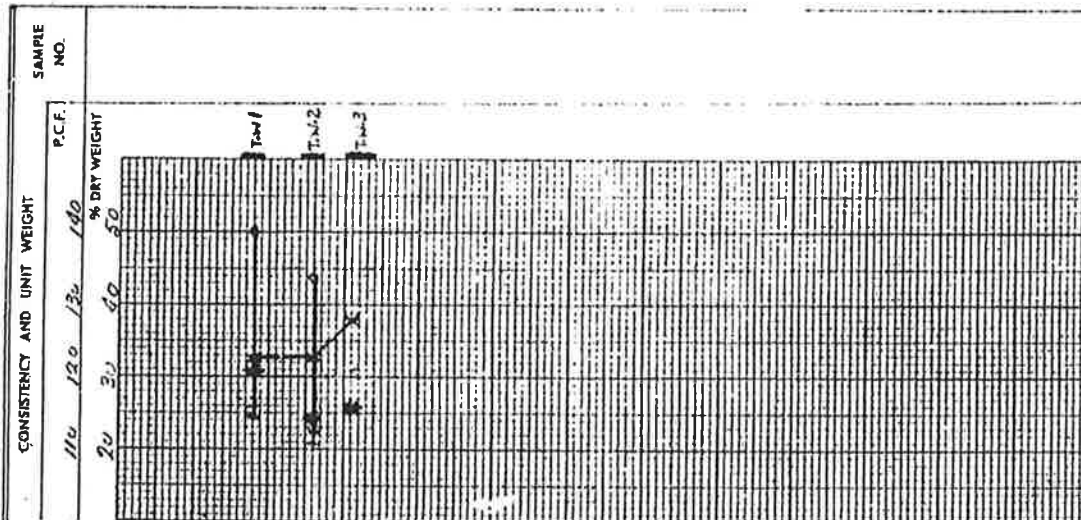
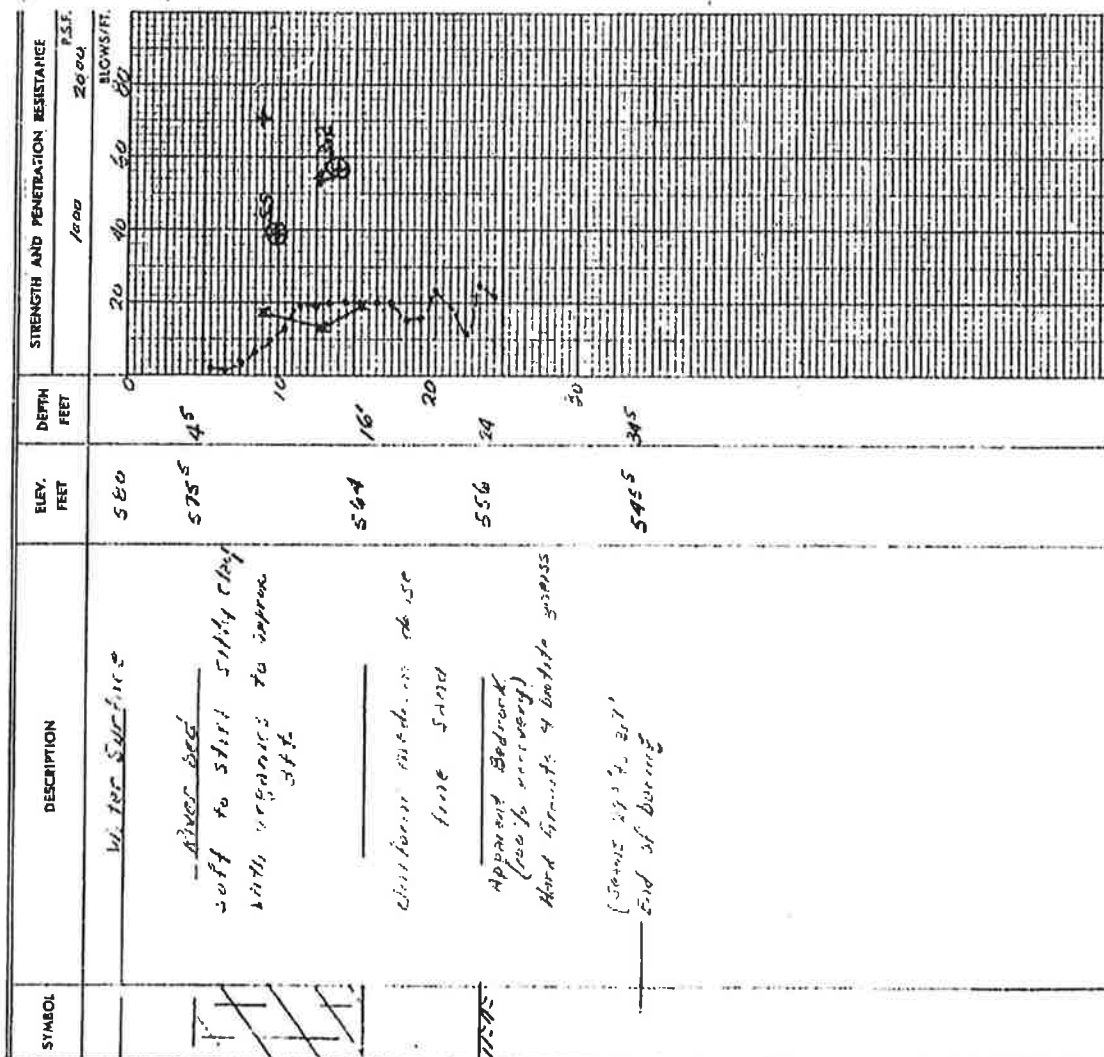
Driller: L.B.

Prep: M.T.

Checked: W.T.

Date: W.T.

Sampling Method
 2" Split Tube
 2" Shelby Tube
 Penetration Resistance
 2" Split Tube
 2" Dia. Cone
 Coaxial Anvil
 Strength
 Unconfined compression
 Vane test and sensitivity
 Consistency
 Natural moisture
 Liquid limit
 Plastic limit
 Natural Unit Weight



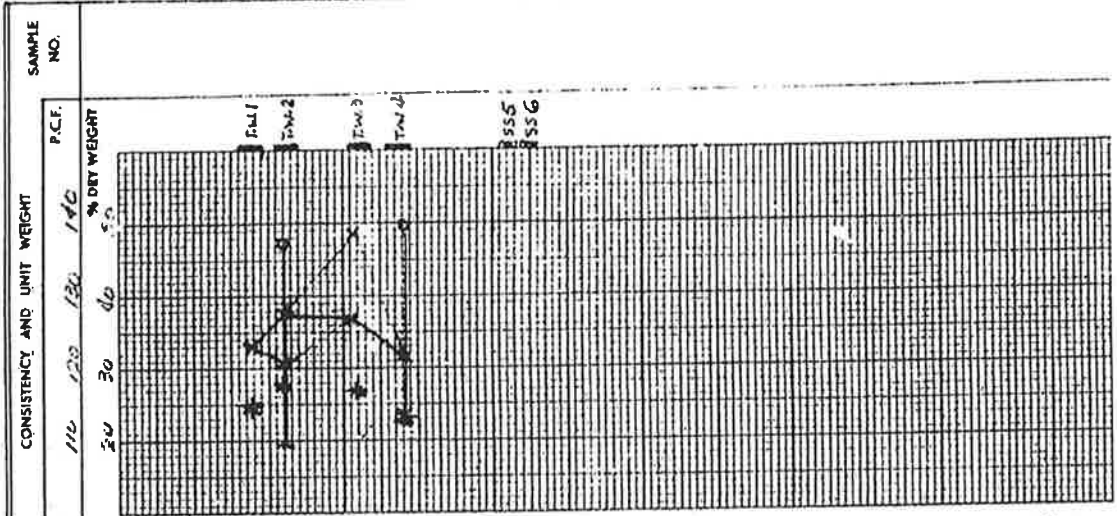
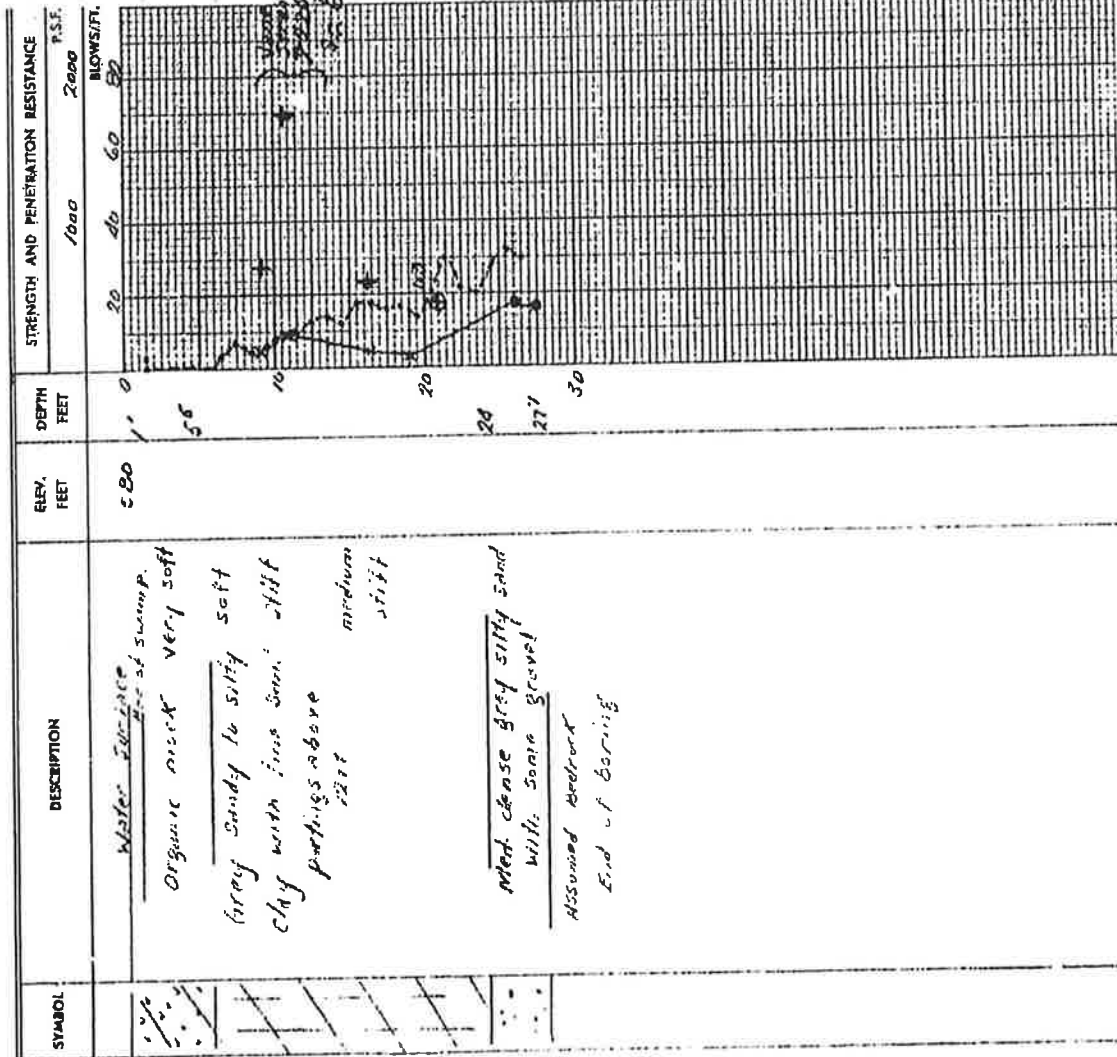
RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: A2

Project: *Savona River Bridge* Field Supervisor: *E. C.*
 Location: *Vicinity Sta. 211 Hwy 103 Dawson* Driller: *L.B.*
 Hole Location *Hwy 6 Sta. 210+70* Post *Survey, Cont* Prep: *W.T.*
 Hole Elevation and Datum: *520* Checked: *W.T.*
 Field Work Begun *July 21/56* Ended *July 21/56* Date:

2" Dia. split tube
 2" Shelby tube
 Penetration Resistance
 2" Split tube
 2" Dia. Cone
 Casing
 2" Shelby tube
 Strength
 Unconfined compression
 Vane test and sensitivity
 Consistency
 Natural moisture
 Liquid limit
 Plastic limit
 Moisture Unit Weight



RACEY MACCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: **A3**

Project: **Savern River Bridge**

Location: **May 103, Revision**

Hole Location: **May 103, Revision**

Hole Elevation and Datum: **580**

Field Work Begun: **Aug. 1**

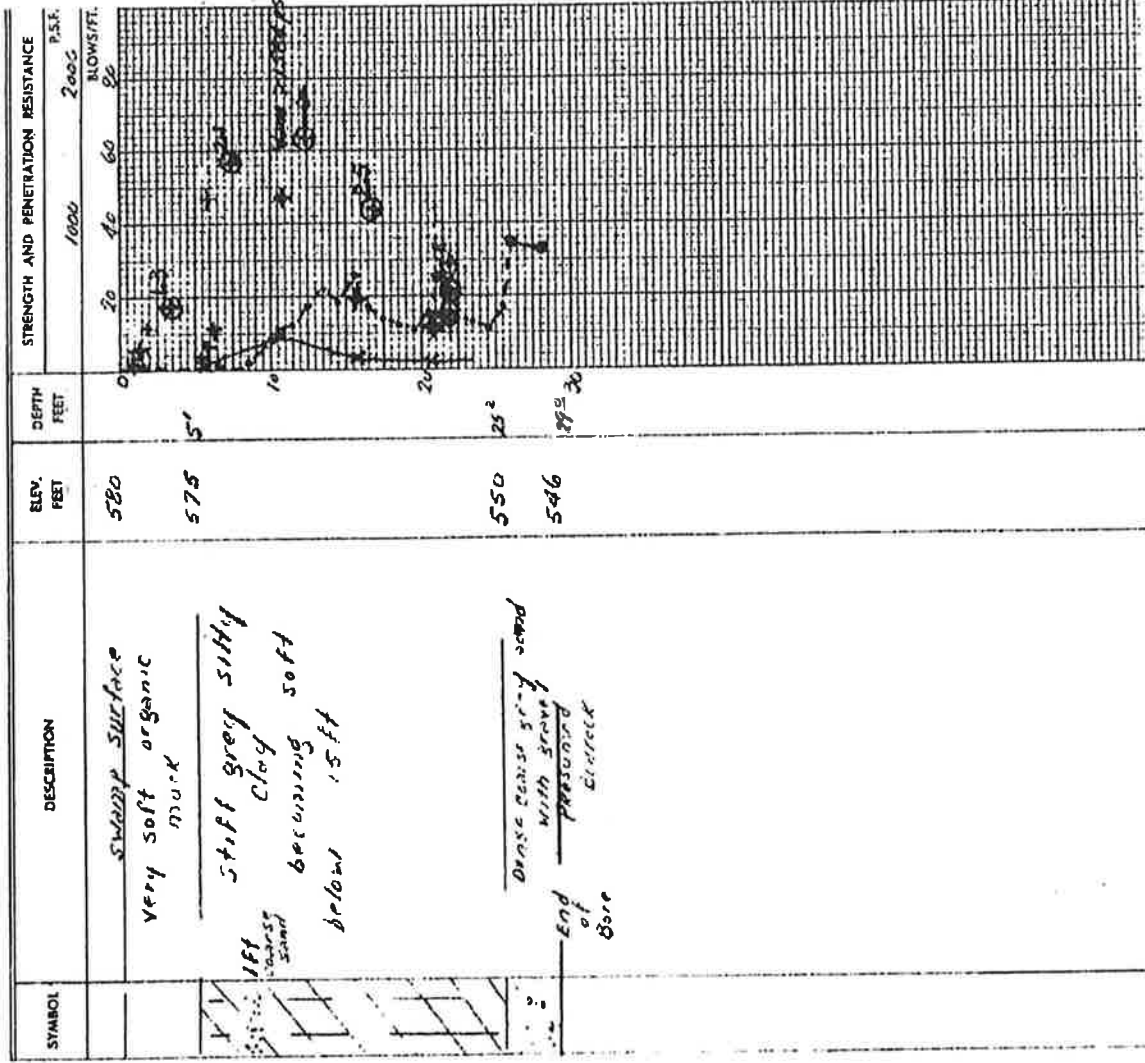
Field Supervisor: **E. C.**

Driller: **L. B.**

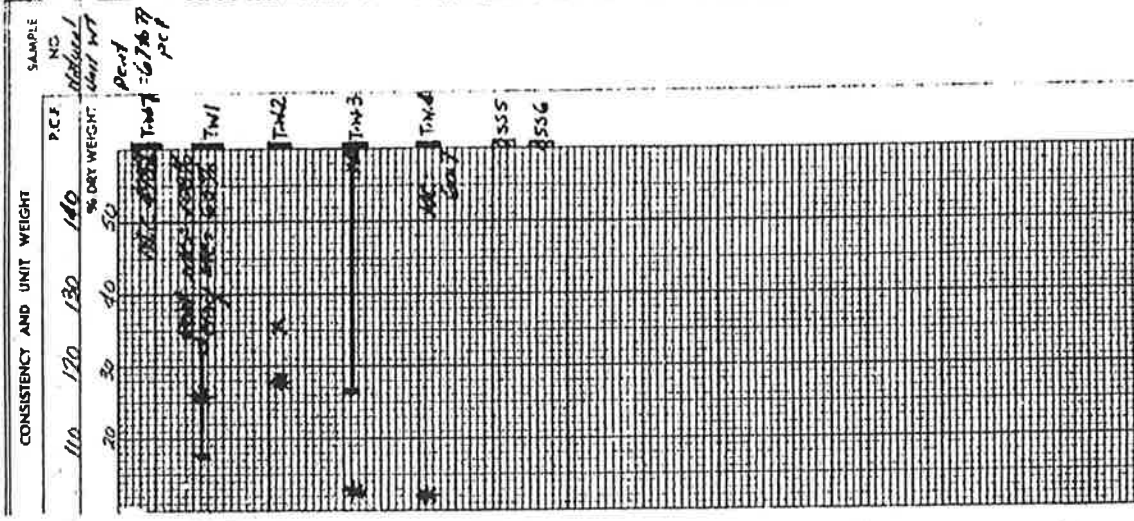
Prep: **w. T.**

Checked: **w. T.**

Date: **Aug. 1**



Penetration Resistance
 2" Split tube
 3/4" Dia. Cone
 Case
 3" Shelby tube
 Standard
 Unconfined compression
 Vane test and sensitivity
 Consistency
 Natural moisture
 Liquid limit
 Plastic limit
 Natural Unit Weight



Foundation Engineering Division

Engineering Data Sheet for Borehole: A4

Project: Severn River Bridge

Location: Vicinity Sta 139 May 103 Revison

Hole location Hwy 190+70
13% Severe Cont Prep:

Hole Elevation and Datum: 500

Field Work Begun 16-8 Ended 17-E-136 Date:

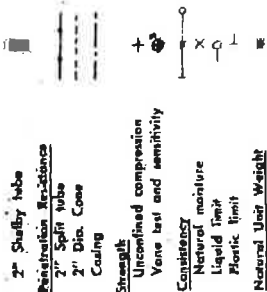
Field Supervisor: _____

Driller: L.B.

Prep:

Checked:

Dat:



SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P.S.I.	
	<p>cleaved surface organic sandy topsoil intermittent thin layers of shale and red fine sand</p> <p>Bedrock (100% recovery) Dense granite and basaltic gneiss Horizontal seams 7' to 8' H.</p> <p>Approx 45° - 50° to 175°</p> <p>Approx 10" first seams 25' to 26'</p> <p>Approx 20" seams 31' to 32'</p> <p>E. of 100' base</p>	<p>566'</p> <p>573'</p> <p>565'</p> <p>560'</p> <p>554'</p>	<p>2'</p> <p>62'</p> <p>10'</p> <p>19.5'</p> <p>32.5'</p> <p>31'</p> <p>32'</p> <p>40'</p>		
	<p>Note: All specimens are rock cracks No further rock present in the cracks</p>				

RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: A5

Project: *Sovereign River Bridge*
 Location: *Vicinity Sta 189 Highway Division*
 Hole Location: *Sta 188+70 5th right*
 Hole Elevation and Datum: *580'*
 Field Work Begun: *18-8* Ended: *20-8* Date:
 Field Supervision: *E.C.*
 Driller: *L.B.*
 Preparer: *Dr. J. J. J. J. J.*
 Checked:

1. 1/2" Spill Tube
 2. 2" Dia. Core
 3. Casing
 4. Straight
 5. Underreamed compression
 6. Vane test and sensitivity
 7. Consistency
 8. Natural moisture
 9. Liquid limit
 10. Plastic limit
 11. Natural Unit Weight

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE P.S.F.	BLOWS/FT.
	Water Surface	580'	0'		
	Water				
	Horizontal strain 75'				
	Bedrock (100% recovery)	573'	7'		
	Dense granite & basalt zones				
	17' interval strains 165 to 175'				
	18 ft 17.5, 25.5, 27.5 to 28.5 ft				
	Horizontal strain 26 ft and 29 ft				
	5' - 6' strain & horizontal 1.5' strain 33 ft				
	30' strain & 1.5' strain				
	End of bore	546'	94'		

CONSISTENCY AND UNIT WEIGHT	P.C.F.	SAMPLE NO.
% DRY WEIGHT		

Foundation Engineering Division

Engineering Data Sheet for Borehole: A6

Project: Severn River Bridge
Location: Vicinity Sta 212 Hwy 103 Peniston
Hole Location Hwy 50 Sta 212 1875
Hole Elevation and Datum: 500
Field Work Begun July 23/56
Field Work Ended July 25/56
Driller: L.B.
Prep: Post seven up.
Checked: WT.
Date: _____
Field Supervisor: E.C.

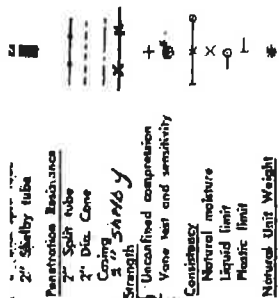
Field Supervisor: **F, C**

Driller: C.B.

Prep.:

Checked: W.T.

Date:



SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				1000	2000 P.S.F.
	Water Surface	580	0		
	River Bed	570.5	5.5		
	Soft to stiff Grey silty clay with small sand pebbles and partings changing to Medium dense fine to medium sand with fine gravel		10		
	Apparent Bedrock Hard Granite & Diolite gneiss 100% recovery.	560	14.5		
	seams in rock 28-29 ft End of boring		20.20		
			29.30		
			40		

CONSISTENCY AND UNIT WEIGHT				SAMPLE NO.	
110	120	130	140	P.C.F.	
% DRY WEIGHT					
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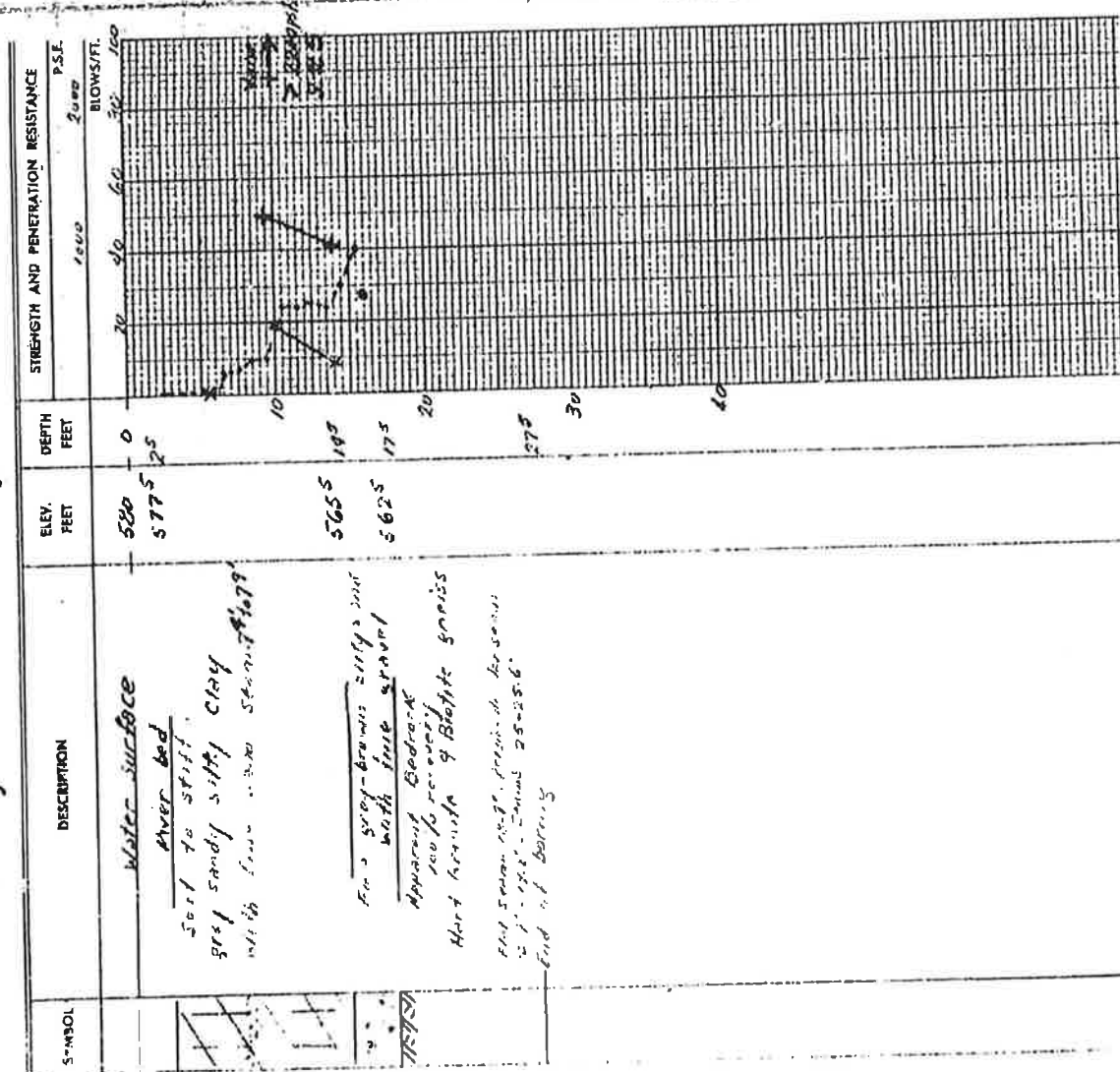
RACEY McCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: **A7**

Project: **Savannah River Bridge** Field Supervisor: **E. C.**
 Location: **Vicinity Sta 212 Hwy 103 Ravenna** Driller: **L. B.**
 Hole Location: **Hwy 212 212+72.5** Post Office: **Prep: B. F. W.**
 Hole Elevation and Datum: **520**
 Field Work Begun: **July 25/56** Ended: **July 26/56** Date:

1' - 100 lb. cone
 Penetration Resistance
 2" Split Tube
 2" Dia. Cone
 Casing
 2" - 100 lb.
 Standard
 2" - 100 lb.
 Unclassified compression
 2" - 100 lb.
 Vane test and sensitivity
 Consistency
 Natural moisture
 Liquid limit
 Plastic limit
 Natural Unit Weight



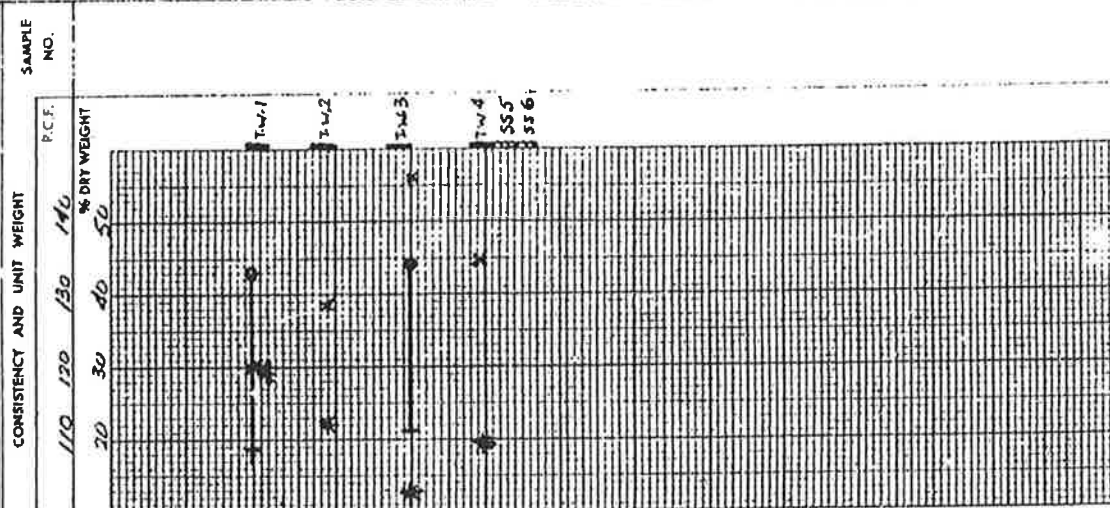
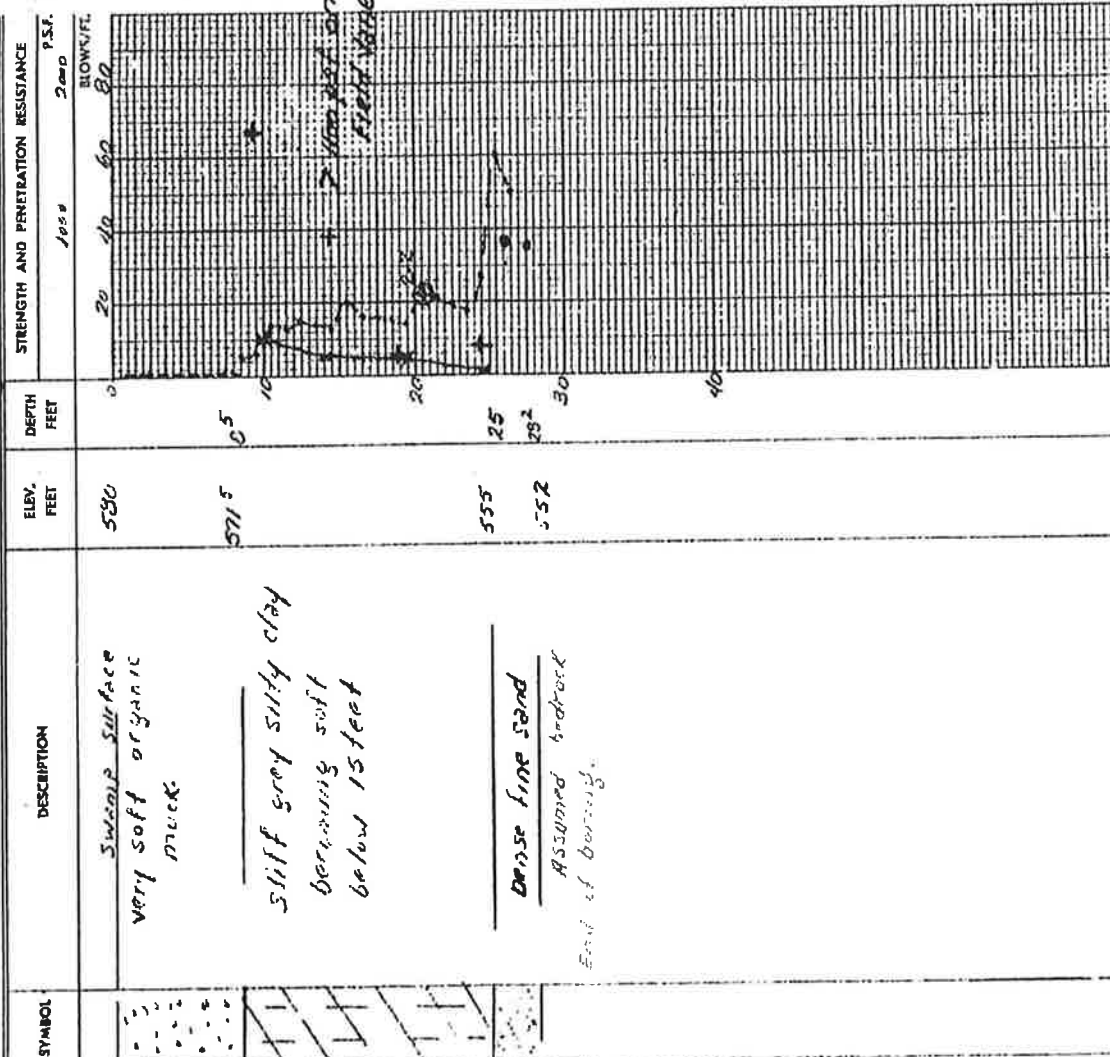
RACEY MACCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: **A8**

Project: *Sewern River Bridge*
 Location: *Hwy 103 Revision Port Severn Unit*
 Hole Location: *Hwy 103 Stn. 209180*
 Hole Elevation and Datum: *520*
 Field Work Begun: *Aug 1*
 Field Supervision: *E.C.*
 Driller: *L.B.*
 Prep: *W.T.*
 Checked: *W.T.*
 Date: *Aug 2/56*

- 3" Shelby tube
- Penetration Resistance
- 2" Split tube
- 2" Dia. Cone
- Coring
- 2" Shelby
- Strength
- 2" Unconfined compression
- Vane test and sensitivity
- Consistency
- Natural moisture
- Liquid limit
- Plastic limit
- Natural Unit Weight



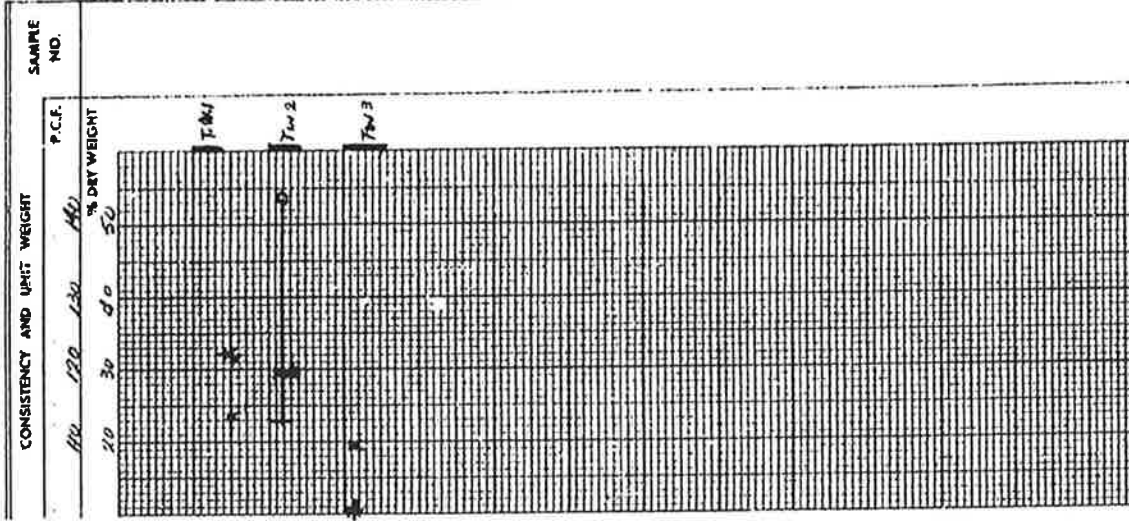
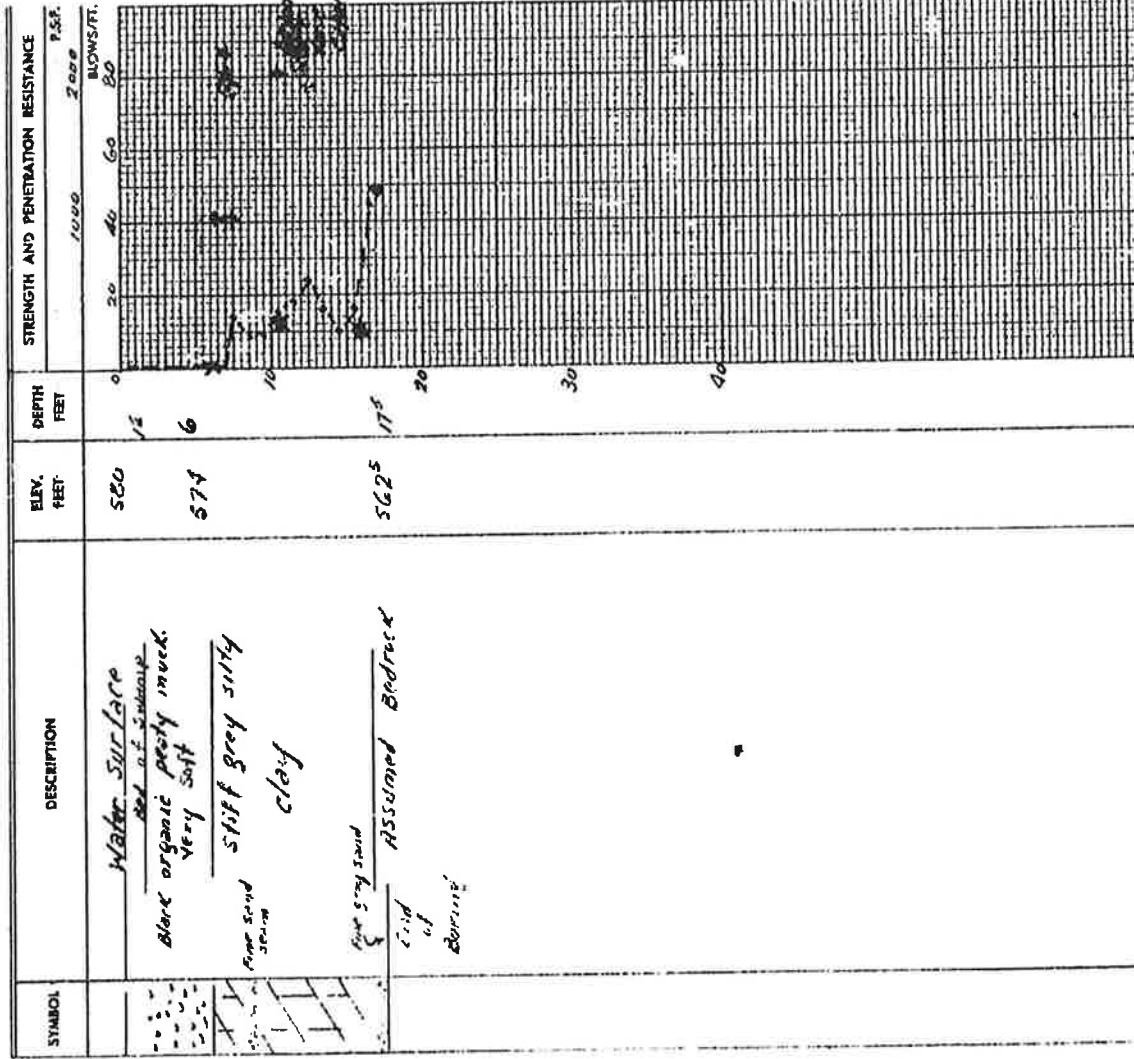
RACEY MACCALLEUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: **A9**

Project: **SEVERN RIVER BRIDGE**
 Location: **VICINITY STA. 211 HWY 103 KANSAS, MO. SMO. CO. PREP.**
 Hole Location **NW 1/4 211 + 40**
 Hole Elevation and Datum: **580**
 Field Work Begun **Aug. 9** Ended **Aug. 5/56** Date: **Aug. 5/56**
 Field Supervision: **E. C. Driller: I. B. Prep.: Checked:**

2" Shelby tube
 Penetration Resistance
 2" Split Tube
 2" Dia. Cone
 Casing
 2" 36" 18" 7"
 Strength
 Unconfined compression
 Vane test and sensitivity
 Consistency
 Natural moisture
 Liquid limit
 Plastic limit
 Natural Unit Weight





RECORD OF BOREHOLE No B1

METRIC.

W P 37-80-05 LOCATION Co-ords: N 4 961 965; E 287 223 ORIGINATED BY JM
 DIST 5 HWY 69 BOREHOLE TYPE Hand Auger COMPILED BY AK
 DATUM Canadian DATE 1990.01.31 CHECKED BY CM

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	γ _w VALUES			20	40	60	80	100					
176.3	Ground Surface																
176.0	10cm Ice loose Si, Sand, Org. Gt						176										
175.2	Si. Sand, Tr. Clay (Glacial Till) v. Dense																
0.6	End of Borehole No further penetration possible with hand auger. Probable Till.																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity 20 15 10 (+5) (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No B2

METRIC

W P 37-80-05 LOCATION Co-ords: N 4 961 974; E 287 199 ORIGINATED BY KC
 DIST 5 HWY 69 BOREHOLE TYPE Raft mounted diamond drill & washboring COMPILED BY AK
 DATUM Geodetic DATE 1989 10 14 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT (LIQUID LIMIT)			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
176.0	Water Level																
0.0	Water																
175.2							175										18 56 (26)
0.8	Silty Sand, some organics, occ. gravel		1	SS	2												
174.5	Grey, V. loose																RQD=73%
1.5	Granite Gneiss		2	BQ	DOY BQc												
173.6	Bedrock		3	BQ	95% fac.		174										RQD=95%
2.4	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

+3, κ^5 ; Numbers refer to Sensitivity 15 ± 8 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No B3

METRIC

W P 37-80-05 LOCATION Co-ords: N 4 961 979; E 287 181
 DIST 5 HWY 69 BOREHOLE TYPE Probe Hole
 DATUM Geodetic DATE 1989 10 15

ORIGINATED BY NT
 COMPILED BY AK
 CHECKED BY CM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	Wp	W	W _L		
176.0	Water Level																
0.0	Water																
174.8							175										
1.2	Bedrock visible Granite Gneiss																

OFFICE REPORT ON SOIL EXPLORATION

+3, x⁵: Numbers refer to Sensitivity 20
 15 5 [%] STRAIN AT FAILURE
 10



RECORD OF BOREHOLE No B4

METRIC

W P 37-80-05 LOCATION Co-ords N 4 961 991; E 287 177 ORIGINATED BY KC
 DIST 5 HWY 69 BOREHOLE TYPE Raft mounted diamond drill & washboring COMPILED BY AK
 DATUM Geodetic DATE 1989 10 15 CHECKED BY CH

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	VALUES		20	40	60	80	100					
176.0	Water Level															
0.0	Water															
175.0						175										
1.0	Granite Gneiss Bedrock; Fractured		1	BQ RC	99% Rec	174										R ₁₀ = 65%
173.3																
2.7	End of Borehole															

OFFICE REPORT ON SOIL EXPLORATION

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



RECORD OF BOREHOLE No B5

METRIC

W P 37-80-05 LOCATION Co-ords N 4 961 992; E 287 132 ORIGINATED BY R
 DIST 5 HWY 69 BOREHOLE TYPE Raft mounted diamond drill & washboring COMPILED BY AA
 DATUM Geodetic DATE 1989 10 15, 16 CHECKED BY CM

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 A GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	W VALUES			20	40	60	80	100					
176.1	Water Level																GR SA SI CL
0.0	Water						176										
174.6							175										
1.5	Silty sand with org. occ. gravel v. Loose		1	SS	2		174										39 41 (20)
173.5			2	SS	9		173										16 51 (37)
2.6	Silty sand with some Gravel Tr Clay (Glacial Till) very Dense Grey		3	SS	59		172										10 54 (36)
			4	SS	88		171										8 55 (37)
			5	SS	70		170										RQD=0
170.9			6	BQ	55% Rec		169										RQD=25%
5.2	Granite Gneiss Bedrock, Fractured		7	BQ	96% Rec		168										RQD=47%
			8	BQ	64% Rec												
168.0																	
8.1	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

METRIC

W P 37-80-05 LOCATION Co-ords N 4 961 983; E 287 198 ORIGINATED BY KR
DIST 5 HWY 69 BOREHOLE TYPE Probe Hole COMPILED BY AK
DATUM Geodetic DATE 1989 10 25 CHECKED BY CH

[illegible]

+1, x5 : Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No. B7										METRIC						
W P 37-80-05		LOCATION Co-ords N 4 962 003; E 287 147				ORIGINATED BY KC										
DIST 5 HWY 69		BOREHOLE TYPE Probe Hole				COMPILED BY AK										
DATUM Geodetic		DATE 1989 10 16				CHECKED BY CM										
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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	<div style="text-align: center;"> 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED * YIELD VANE ● QUICK TRIAXIAL x LAB VANE </div>								
176.0	Water Level															
0.0	Water					175										
174.5	Refusal to Probe															
1.5	Fractured Bedrock or Shot Rock End of Probe Hole															Fractures due to blasting of R/R in nearby Boil Channel

OFFICE REPORT ON SOIL EXPLORATION

$\cdot 2, \times 5$: Numbers refer to Sensitivity

13 \pm 5 [%] STRAIN AT FAILURE



RECORD OF BOREHOLE No B8

METRIC

W P 37-80-05 LOCATION Co-ords N 4 962 000; E 287 120 ORIGINATED BY IM
 DIST 5 HWY 69 BOREHOLE TYPE Hand Auger COMPILED BY AK
 DATUM GEodetic DATE 1990 01 31 CHECKED BY CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STAY PLOT	NUMBER	TYPE	N° VALUES		20	40	60	80	100	Wp	W	W _L		
177.1	Ground Surface															
0.0	Sandy Topsoil															
176.7	Grey SI Sa Tr Org Loose		1	CS												
0.4	Si Sand w/ some gravel		2	CS												
176.3	(Glacial Till) v. Dense															
0.8	End of Borehole No further penetration possible with hand auger. Probable Glacial Till															

OFFICE REPORT ON SOIL EXPLORATION

*3, *5; Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE



RECORD OF BOREHOLE No C1

METRIC

W P 37-80-04 LOCATION Co-ords: N 4 961 917; E 287 374 ORIGINATED BY KC
 DIST 5 HWY 69 BOREHOLE TYPE Probe Hole COMPILED BY AR
 DATUM Geodetic DATE 1989 10 11 CHECKED BY CM

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
176.1	Water Level															
0.0	Water					176										
175.5	see * below															
0.6	End of Probe Hole Probable Bedrock * very thin layer of silty sand and organics on surface of bedrock															

OFFICE REPORT ON SOIL EXPLORATION

* 3, x²; Numbers refer to Sensitivity 20 15 10 (% STRAIN AT FAILURE



RECORD OF BOREHOLE No C2

METRIC

W P 37-80-04 LOCATION Co-ords: N 4 961 932; E 287 372 ORIGINATED BY KI
 DIST 5 HWY 69 BOREHOLE TYPE Raft mounted Diamond Drill & Washboring BW Casing COMPILED BY AK
 DATUM Geodetic DATE 1989 10 11-12 CHECKED BY CH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				NATURAL MOISTURE CONTENT			UNIT WEIGHT	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES		SHEAR STRENGTH kPa				PLASTIC LIMIT Wp	W	LIQUID LIMIT Wl		
176.1 0.0	Water Level					176	20 40 60 80 100				WATER CONTENT (%)			γ	GRAIN SIZE DISTRIBUTION (%)
175.2 0.9	Water					175	○ UNCONFINED • FIELD VANE ● QUICK TRIAXIAL x LAB VANE				15	30	45		
174.7 1.4	Silty Sand, Grey, Loose Tr. Org & Gravel		1	SS	6	174									13 57 (30)
173.7 2.4	Silty Clay, Tr. Sand Stiff Reddish Grey		2	SS	13	173									RQD=100%
	Granite Gneiss Bedrock		3	BQ RC	100% Rec.	172									RQD=86%
	Sound		4	BQ RC	100% Rec.										
171.5 4.6	End of Borehole														

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to 20 15 10 (% STRAIN AT FAILURE Sensitivity



RECORD OF BOREHOLE No C3

METRIC

W P 37-80-04 LOCATION Co-ords: N 4 961 930; E 287 352 ORIGINATED BY KC
 DIST 5 HWY 69 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY AK
 DATUM Geodetic DATE 1989 10 12 CHECKED BY CM

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL * LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE						
176.3 0.0	Water Level									
174.8 1.5	Water									
	Probable Silty Clay (stiff - hard)									
171.9 4.4	Prob. Sil. Sand (Glacial till) End of Cone Test. Refusal to Further Penetration. Probable Bedrock									

OFFICE REPORT ON SOIL EXPLORATION

+3, +5 : Numbers refer to Sensitivity
 20
 15
 10
 5
 0
 (% STRAIN AT FAILURE)



RECORD OF BOREHOLE No C4

METRIC

W P 37-80-04 LOCATION Co-ords: N 4 961 924; E 287 350 ORIGINATED BY ..KC..
 DIST 5 HWY 69 BOREHOLE TYPE Reft mounted Diamond Drill & Washboring BH Casing COMPILED BY AK
 DATUM Geodetic DATE 1989 10 12 CHECKED BY CM

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
176.2	Water Level													GR SA SI CL
0.0	Water													
173.3														
2.9	Silty Clay, Tr. Sa. Stiff-Hard		1	SS	9									
	Reddish - Grey		2	SS	20									
171.6	Si. Sand (Glacial Till)		3	SS	20/8	cm								
4.6	Granite Gneiss Bedrock Sound		4	BQ RC	77% Rec.									RQD=77%
			5	BQ RC	100% Rec.									RQD=100%
169.8	End of Borehole													
6.4														

+3, x5 : Numbers refer to Sensitivity
 20 15 10 (% STRAIN AT FAILURE



RECORD OF BOREHOLE No C5

METRIC

W P 37-80-04 LOCATION Co-ords: N 4 961 942; E 287 319 ORIGINATED BY KC
 DIST 5 HWY 69 BOREHOLE TYPE Raft mounted Diamond Drill & Washboring BW Casing COMPILED BY AK
 DATUM Geodetic DATE 1989 10 13 CHECKED BY CM

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100					
176.3	Water Level															
0.0	Water					176										
175.1						175										
1.2	Sl. Sand, some organics		1	SS	16											
	Silty Clay, fr. Sand															
	V. Stiff															
174.1	Reddish Grey		2	SS	018/32 cm											
2.2	End of Borehole															
	Probable Bedrock															

OFFICE REPORT ON SOIL EXPLORATION

+3, x⁶; Numbers refer to Sensitivity

20
15 + 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No C6

METRIC

W P 37-A0-04 LOCATION Co-ords: N 4 961 937; E 287 322 ORIGINATED BY JK
 DIST 5 HWY 69 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY AK
 DATUM Geodetic DATE 1989 10 24 CHECKED BY CM

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 (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40					
175.5 0.0	Water level													
174.7 1.2	Water													
173.7 2.2	Probable Silty Clay													
	End of Cone Test. Refusal to Further Penetration. Probable Bedrock													

OFFICE REPORT ON SOIL EXPLORATION

♦³, ♦⁵: Numbers refer to Sensitivity 13 ♦⁵ 20 5 (%) STRAIN AT FAILURE 10



RECORD OF BOREHOLE No. C7

METRIC

W P 37-BO-04 LOCATION Co-ords: N 4 961 940; E 287 313 ORIGINATED BY KC
DIST 5 HWY 69 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY AK
DATUM Geodetic DATE 1989 10 24 CHECKED BY CM

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	'N' VALUES			20	40					
175.9 0.0	Water Level													
174.6 1.3	Water													
173.5 2.4	Probable Silty Clay													
	End of Cone Test. Refusal to Further Penetration. Probable Bedrock													

OFFICE REPORT ON SOIL EXPLORATION

+3, x³: Numbers refer to Sensitivity20
15 10 5 (%) STRAIN AT FAILURE

Appendix B

Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	
			MINISTRY SIEVE DESIGNATION (Metric)					

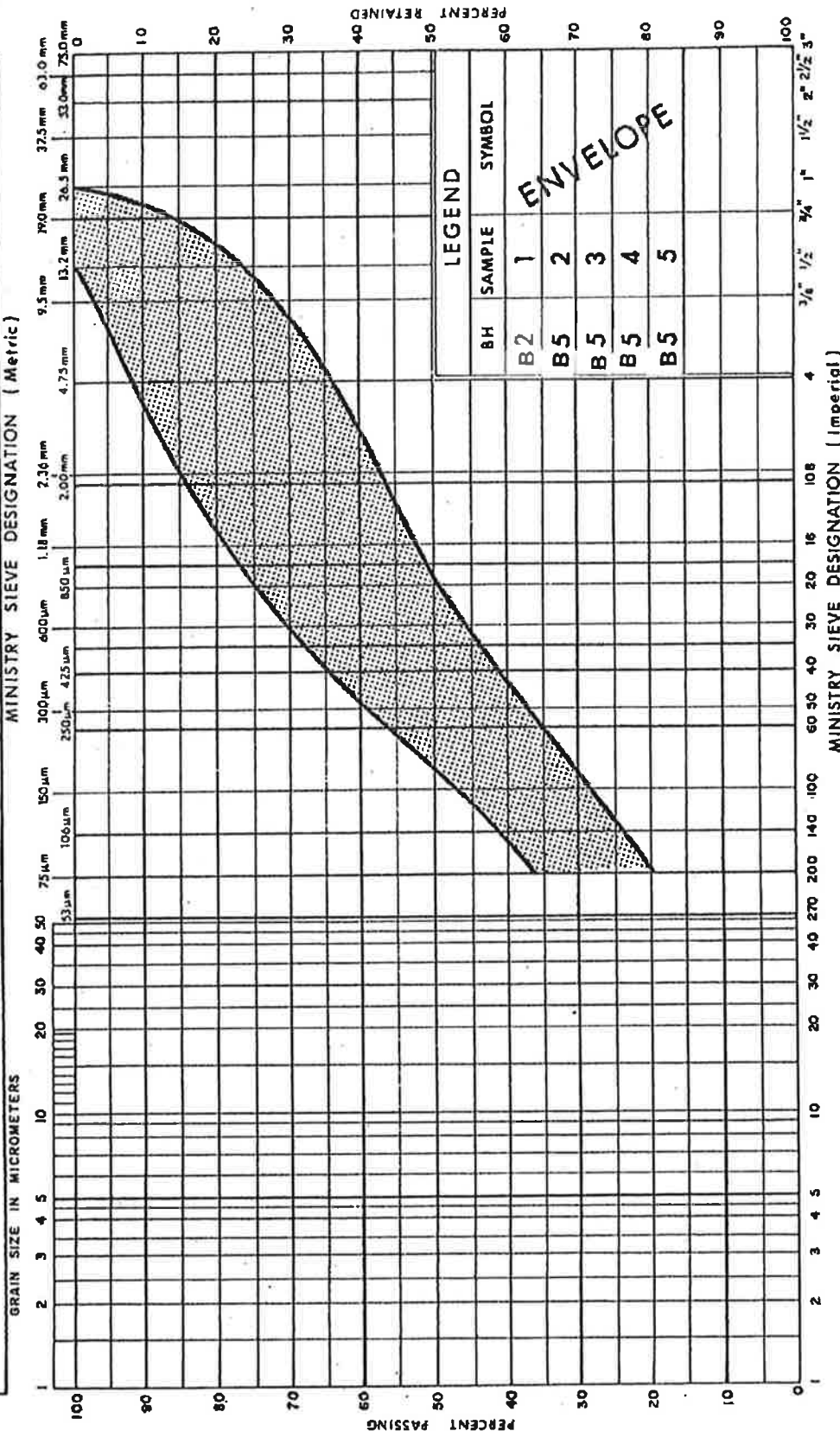


Figure B-1

W P 37-80-05

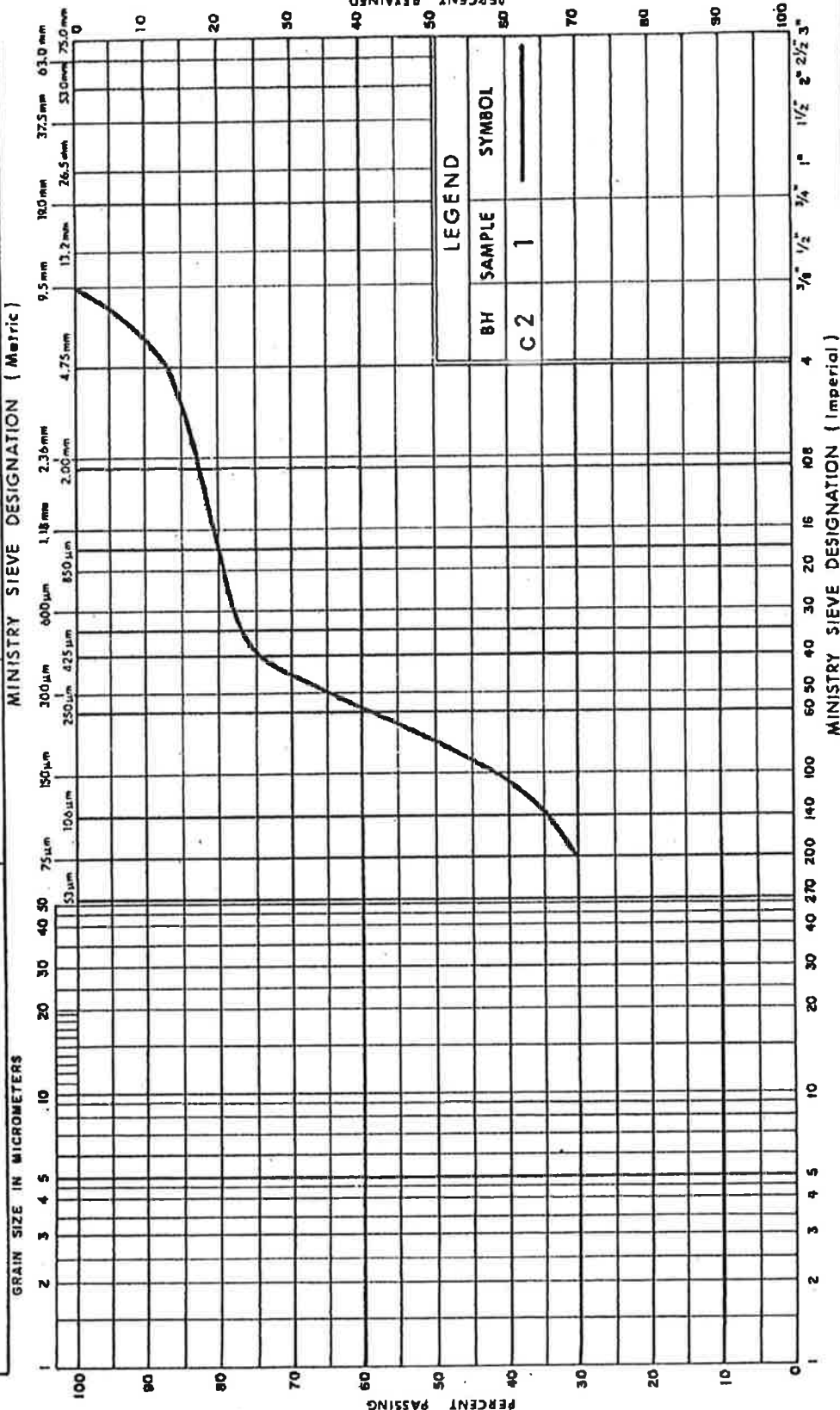
SEVERN RIVER BOAT CHANNEL

Ministry of
Transportation



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine			Coarse		



Oct 75, FF-S-21

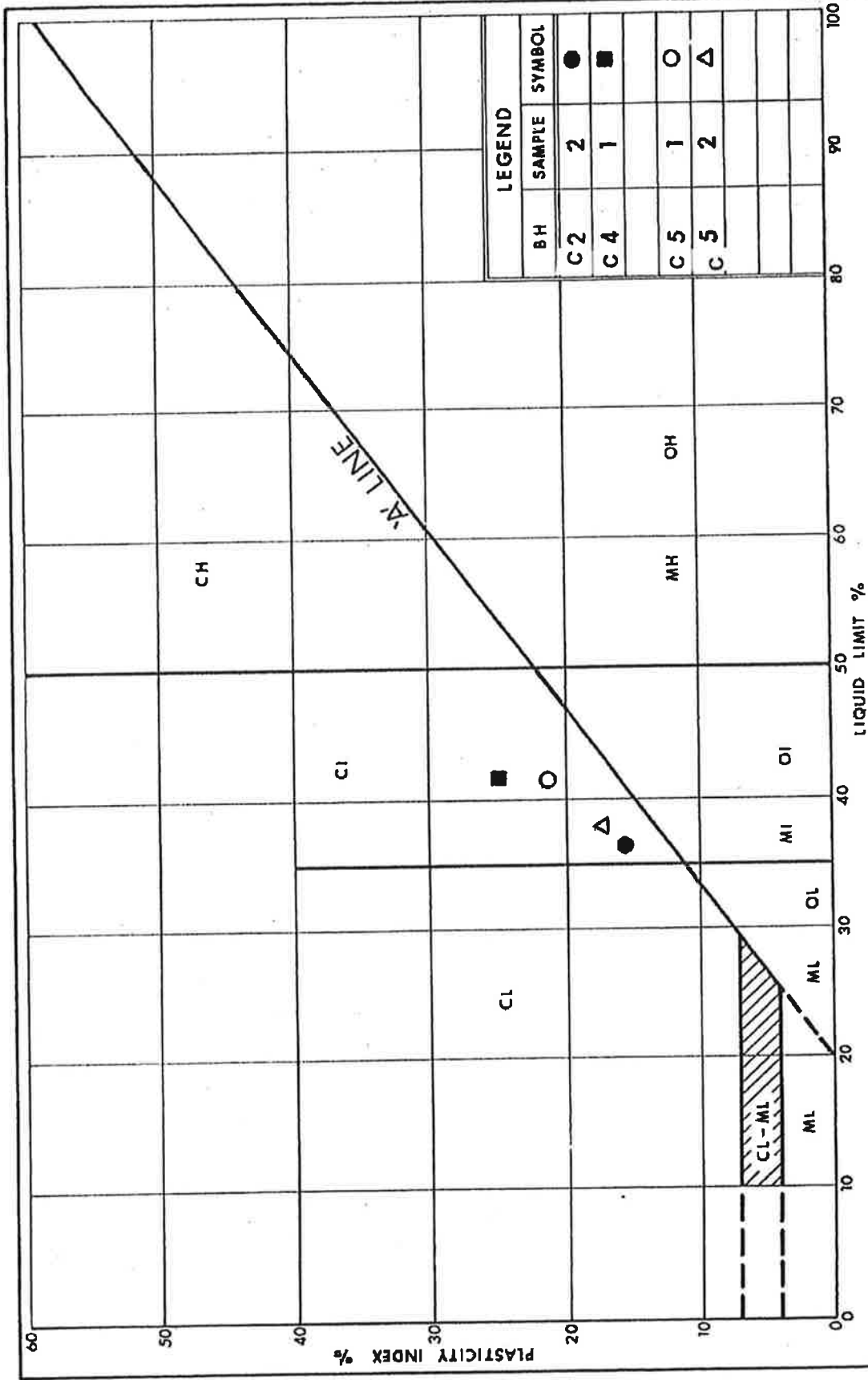


Figure B-3

W P 37-80-04

Savern River

PLASTICITY CHART
SILTY CLAY, TRACE SAND

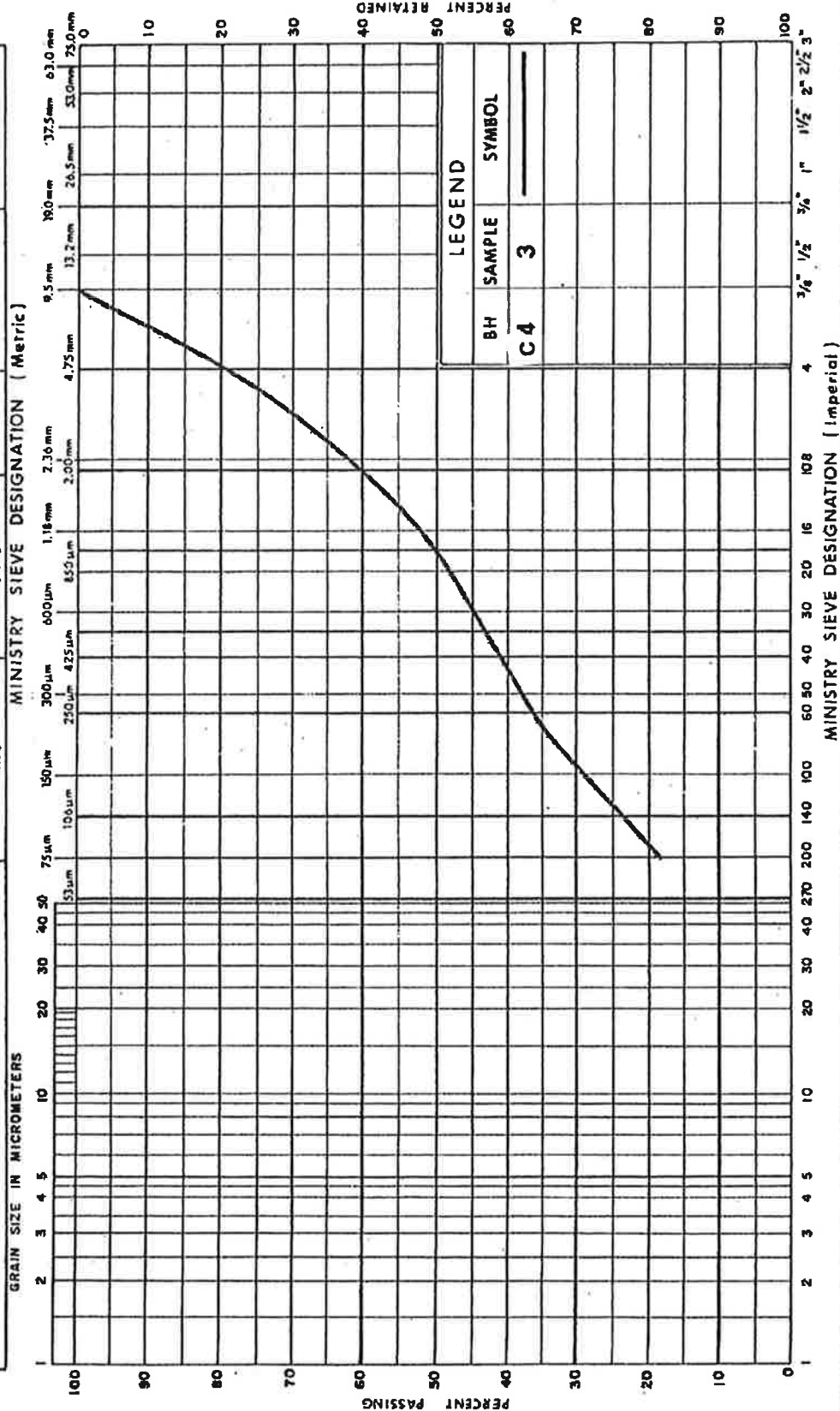
Ministry of
Transportation



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UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



Appendix C

Site Photographs



Photograph 1. NB Severn River Bridge



Photograph 2. NB Severn River Bridge looking south



Photograph 3. SB Severn River Bridge



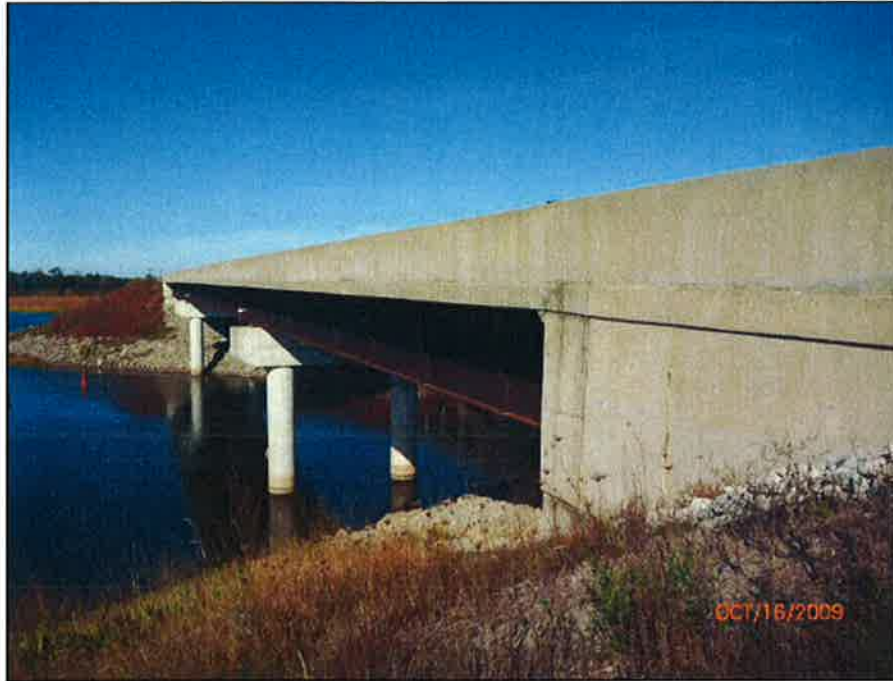
Photograph 4. SB Severn River Bridge looking south



Photograph 5. NB Severn River Boat Channel Bridge



Photograph 6. Looking east upstream (NB Severn River Boat Channel Bridge)



Photograph 7. SB Severn River Boat Channel Bridge



Photograph 8. SB Severn River Boat Channel Bridge (looking north)

Appendix D

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 = $(w_L - w) / 1_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						



**PRELIMINARY FOUNDATION DESIGN
REPORT - REPLACEMENT AND
REHABILITATION OF FOUR EXISTING
BRIDGES OVER THE SEVERN RIVER AND
SEVERN RIVER BOAT CHANNEL,
TOWNSHIP OF BAXTER
MTO CENTRAL REGION
G.W.P. 2360-09-00 & G.W.P. 2376-09-00
GEOCRES NO. 31D-555**

McCormick Rankin Corporation
Project: TRANETOB20462AA
July 31, 2013

REPORT

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**PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT AND REHABILITATION OF FOUR BRIDGES
OVER THE SEVERN RIVER AND SEVERN RIVER BOAT CHANNEL,
TOWNSHIP OF BAXTER, MTO CENTRAL REGION
G.W.P. 2360-09-00 & G.W.P. 2376-09-00**

4 DISCUSSION AND RECOMMENDATIONS

This project consists of replacement and rehabilitation of four bridges on Highway 400 in Township of Baxter, Ontario.

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

**Table 4.1
Bridge Information**

Title	Site Number	Year Built	Length (m)	Width (m)	Existing Structure Type	Proposed Structure Strategy
Severn River Bridge, NB	42-86/1	1957	27.4	10.4	Single Span Rigid Frame	Replacement
Severn River Boat Channel Bridge, NB	42-87/1	1957	93.7	10.2	3 Rib Deck Arch	Replacement
Severn River Boat Channel Bridge, SB	42-87/2	1992	118	12.0	3 Span – Slab on Steel I Girder Abutments and Central Pier	Rehabilitation
Severn River Bridge, SB	42-86/2	1991	27.5	12.0	Single Span Rigid Frame	Rehabilitation

This preliminary study (Phase 1) was carried out based on the available information only (desktop study) and detail structure strategy mentioned in Phase 2 of Request for Proposal (RFP) was not considered for this report.

The existing bridge general arrangement (GA) drawings were provided to us by McCormick Rankin (MRC) for this study (see Appendix E). Below table summarizes the existing bridges details.

**Table 4.2
Existing Bridge Information (based on GA drawings)**

Title	Foundation Type	Bridge Top Elevation (m)	Founding Elevation	Remark
Severn River Bridge, Northbound	Shallow foundations on rock	About 184.7	175.8 m (576.8 feet) at top of footing	Concrete footing placed 0.3 m into the bedrock
Severn River Boat Channel Bridge, Northbound	Thrust blocks (buttress)	About 187	North thrust block bottom level: 175.0 m (574 feet) South thrust block bottom level 173.4 m (569.0 feet)	Thrust blocks stepped into bedrock with dowels
Severn River Boat Channel Bridge, Southbound	Integral abutments with H piles and shallow foundations on bedrock for piers	About 187.9	South abutment: about 175 m (pile tip) North abutment: about 171.7 m (pile tip) South pier: about 174.2 m North pier: about 172.45 m	Shallow foundations placed 0.2 m into bedrock Rock points for H-piles

Title	Foundation Type	Bridge Top Elevation (m)	Founding Elevation	Remark
Severn River Bridge, Southbound	Shallow foundations on mass concrete	About 185	South abutment: 175.0 m at mass concrete top North abutment: 174.5 m at mass concrete top	Mass concrete placed on bedrock surface

Based on the finding of previous investigations at the bridge locations, the predominant feature of the site is the shallow water, shallow bedrock and bedrock outcrops. The overburden, where present, consists of organic rich surficial soil, surficial silty sand, silty clay and sand to silty sand till overlying bedrock. The thickness of silty clay increases towards the marshy areas adjacent to northbound Severn River Bridge and consistency of silty clay becomes soft below a depth of 4.5 m.

The construction of the new bridges may have locally modified the surface soils and also possibly the original rock surface. For example, it is evident that at the Boat Channel Structure locations, foreslope fill materials and/or rock fill have been placed.

Based on the finding of previous investigations at the site and provided GA drawings, bedrock surface at the site is not flat and localized depression of bedrock is also not uncommon. Bedrock at foundation locations are sloping towards watercourse/lake in a longitudinal direction of bridges (along centreline of the bridges) and also sloping in a transverse direction of the structures. Quality of bedrock within the project limits is relatively sound but localized weathering and shattered rock due to previous construction (e.g. boat channel) are also noted at the site.

Bedrock conditions and available information on bedrock quality at each bridge location are summarized in the following table.

Table 4.3
Bedrock Conditions at Each Bridge Location

Title	Borehole No.	Drilling top Elevation (m)**	Bedrock Elevation/Depth (m)	Coring Length (m)	Recovery (%)	RQD
Severn River Bridge, Northbound	A1	176.8	169.5/7.3	3.2	100	-
	A2	176.8	168.4/8.1	No rock coring		
	A3	176.8	166.4/10.4	No rock coring		
	A6	176.8	170.7/4.4	2.7	100	-
	A7	176.8	171.4/4.6	3.0	100	-
	A8	176.8	168.2/8.6	No rock coring		
	A9	176.8	171.5/4.9	No rock coring		
Severn River Boat Channel Bridge, Northbound	A4	178.6	176.7/1.9	7.8	100	-
	A5	176.8	174.7/2.1	8.3	100	-
Severn River Boat Channel Bridge, Southbound	B2	176.0	174.5/1.5	0.9	90/95	73/95
	B3	176.0	174.8/1.2	Probe test (visible bedrock)		
	B4	176.0	175.0/1.0	1.7	99	65
	B5	176.1	170.9/5.2	2.9	55/96/64	0/29/47
	B6	175.9	175.7/0.2	Probe test (visible bedrock)		
	B7	176.0	174.5/1.5	Probe test		
Severn River Bridge, Southbound	C1	176.1	175.5/0.6	Probe test		
	C2	176.1	173.7/2.4	2.2	100/100	100/86
	C3	176.3	171.9/4.4	DCPT refusal on probable bedrock		
	C4	176.2	171.6/4.6	1.8	77/100	77/100

Title	Borehole No.	Drilling top Elevation (m)**	Bedrock Elevation/Depth (m)	Coring Length (m)	Recovery (%)	RQD
	C5	176.3	174.1/2.2	No rock coring		
	C6	175.9	173.7/2.2	DCPT refusal on probable bedrock		
	C7	175.9	173.5/2.4	DCPT refusal on probable bedrock		

* Bridge foundation boreholes are in **bold**

** Some of the boreholes were advanced from the water or ice surface.

4.1 Bridge Foundations

Based on the information provided to us by MRC, bridge rehabilitation including widening is proposed for the southbound bridges, and bridge replacement is proposed for the northbound bridges.

4.1.1 Northbound Severn River Bridge (Bridge 42-86/1)

The existing northbound Severn River Bridge is a 27.4 m long single span, rigid frame structure. Based on the GA drawing (see Drawing E-1 and E-2 in Appendix E), it is supported on shallow foundations bearing on bedrock (inset 0.3 m).

Drawing E-2 (Appendix E) shows that for the retaining walls, mass concrete was used to raise the grade to place shallow foundations on top of the mass concrete. About 1.2 m (4 feet) long dowels, extending at least 0.6 m (2 feet) into the bedrock, were installed at the base of mass concrete.

Depending on the details of the new bridge, the most obvious choice is to support the new structure on bedrock, probably on spread footing foundations, similar to the existing bridge.

If the existing bridge will be replaced with a similar length bridge, consideration can be given to re-use, improvement or extension (for widening) of the existing foundations. In this case, the existing bridge foundation elements and the surrounding bedrock should be inspected to verify their condition and load carrying capacity.

If the existing bridge will be replaced with a longer bridge, new foundations will be required. The extent of the existing mass concrete may need to be verified to avoid unpleasant surprises and the existing embankment fill material should be investigated prior to new foundation selection. Since bedrock is deep below the existing embankment grade (about 14 m deep), for the support of abutments, shallow foundations bearing on bedrock may be difficult to construct. If new foundations are to be installed well beyond the existing mass concrete and rock fill is not found during detail investigation, consideration can be given to deep foundation options. Driven steel H-piles may be a feasible option if rock fill was not used for existing approach embankment. Integral bridge abutments can be constructed with driven H-piles with 3 m long flex zone. In the case of bridge and embankment widening, oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

Cast-in-place concrete (caisson) piles may also be considered (if rock was not used for the existing embankment construction), but this option may be more costly than driven steel piles because maintaining caisson hole stability and socketing caissons into the sound bedrock (stepping may be required as per OPSS 903 due to sloping bedrock) may cause problems during the installation.

If rock was used to construct the embankments, another alternative would be to use micropiles to avoid excavating rock fill. Micropiles can be advanced through rock fill, but in such instances a permanent steel casing is typically used to avoid the loss concrete into the voids in the rock fill and to protect the micropile from being exposed to elements. The micropile can be advanced through existing mass or reinforced

concrete used to support the existing foundations (i.e. reinforcing steel bars should not present problems). As can be expected, micropiles are costlier than driven steel piles or caissons. In addition, drilling through rock fill and placing the casing in the rock fill will add to the cost (perhaps as much as 50% over and above normal cases where permanent casing is not required, depending on the details). But in spite of this, we feel that this alternative merits consideration. More details can be obtained from a specialized contractor. The micropile will obtain its resistance to loads from anchoring into bedrock. For preliminary estimating purposes, a bond value of 1000 kPa between the relatively sound rock and concrete can be assumed. Resistance from any overburden should be ignored.

If rock fill was used for embankment construction, the extent of rock fill (both vertical and horizontal) should be confirmed to prepare foundation alternatives.

Details of deep foundations will be discussed in Section 4.2.

Advantages, disadvantages, cost, risk and consequences of typical foundation options are summarized in Table F-1 in Appendix F.

4.1.2 Northbound Severn River Boat Channel Bridge (Bridge 42-87/1)

The existing northbound Severn River Boat Channel Bridge is a three rib deck arch bridge supported by concrete thrust blocks and shallow foundations bearing on bedrock. Boreholes A4 and A5 were advanced in 1956 at the then proposed bridge site. Borehole A4 was advanced from land and contacted below about 2 m of sandy overburden and rock slabs and encountered the bedrock at EL. 176.7 m. From the notes on the borehole log, the bedrock appears to be more sound below at about EL. 174.0 m. Borehole A5 was put from water surface and contacted below 2.1 m of water, the surface of the bedrock at EL. 176.7 m. Based on GA drawings (See Drawings E-3 and E-4 in Appendix E), the base of the west thrust block was placed at EL. 175 m (574 feet) and dowels or steel bars were installed at the base of mass concrete. The east thrust block was placed at 173.4 m (568.9 feet). Columns 5E and 5W and abutments are supported by reinforced concrete columns which are founded on bedrock.

If the existing bridge will be replaced with a similar arch bridge, consideration can be given to re-use, improvement or extension of the existing thrust blocks. Removal of previously placed concrete thrust blocks (about 3 m thick on west side, thicker on the east side) may be impractical and it can cause damage to the surrounding bedrock during the removal. In this case, existing thrust blocks and surrounding bedrock may need to be inspected to verify their condition and load carrying capacity. Visual inspection of the other shallow foundation units will be difficult due to the existing embankment fill thickness over the other shallow foundation units. Consideration can be given to deep foundation options. The extent of existing shallow foundations and the nature of the embankment fill materials should be confirmed prior to selecting the new foundation option.

If the existing arch bridge will be replaced with a similar structure as the southbound Severn River Boat Channel Bridge (3 span bridge with piers), new foundations will be required. The extent of thrust blocks and shallow foundations, and the extent and nature of the existing embankment fill material should be investigated prior to new abutment foundation selection. If new abutments are to be installed beyond the existing shallow foundation units, deep foundations can be considered. Since elevation difference between the existing grade of the bridge approach embankment (about EL. 187 m) and the bedrock surface (average about EL. 175 - 176 m) is about 11-12 m, the use of shallow foundations may not be feasible. Alternatively, consideration can be given to deep foundation options. Driven steel H-piles may be a feasible option if rock fill was not used for existing approach embankments. Integral bridge abutments can be constructed with driven H-piles with 3 m long flex zone. In the case of bridge and embankment widening,

oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven. As was discussed in Section 4.1.1, cast-in-place concrete (caisson) piles may not be a cost-effective foundation option for this site because socketing caissons into the sound bedrock (maybe stepping as required as per OPSS 903 due to sloping bedrock) may be difficult. If rock fill was used to construct the existing embankments and approach fills, the use of caissons in our opinion will be an expensive option and is not recommended. If rock fill was used for embankment construction, the extent of rock fill (both vertical and horizontal) should be confirmed to prepare abutment foundation alternatives. For the piers, shallow foundations can be used, similar to the existing south bound Severn River Boat Channel Bridge and foundations should be set 0.2 to 0.3 m into the relatively sound and level bedrock. If the rock surface is inclined this may necessitate greater set depth. Cofferdams (e.g. clayey soil dam or jersey barriers with sand bags and vinyl cover) will likely be needed for pier foundation construction. Deep foundation options are not feasible for pier foundations due to the shallow bedrock.

Consideration can also be given to micropiles as was discussed in Section 4.1.1, especially if rock fill is present.

Details of shallow and deep foundations will be discussed in Section 4.2

Advantages, disadvantages, cost, risk and consequences of typical foundation options are summarized in Table F-2 in Appendix F.

4.1.3 Southbound Severn River Boat Channel Bridge (Bridge 42-87/2)

The existing southbound Severn River Boat Channel Bridge is a three span structure with integral abutments. Boreholes B1, B2, B3, B4, B5, B6 and B7 were advanced at the bridge site and bedrock was generally found in between EL. 174.5 and 175.7 m except for Borehole B5. Borehole B5, drilled on the west south quadrant of the bridge, encountered bedrock at EL. 170.9 m below very dense silty sand till.

Based on the available GA drawing (See Drawing E-5 in Appendix E), HP 310 x 110 steel H piles were to be driven to bedrock which was expected at about EL. 172 m on the west (construction north) abutment location and about EL. 175 m on the east (construction south) abutment location. According to construction notes in GA Drawing E-5, pre-augering to the bedrock surface was to be used prior to driving the piles for the southbound Severn River Boat Channel Bridge. Pre-augered holes were backfilled with MTO approved Class 4 aggregates prior to pile driving. The top of the piles appear to be at EL. 182 – 183 m and thus with the anticipated refusal elevations, the length of the piles are probably about 7 m at the east (construction south) abutment and about 11 m at the west (construction north) abutment.

North (west) and south (east) piers in the river are supported by shallow foundations installed at least 0.2 m into the sound bedrock at about EL. 172 and 174 m, respectively. An advantage of using the same type of foundation is that it will afford easier access and inspection of the existing pier foundations during the construction, if such is required, as dewatering will be carried out for the construction of the pier foundation in any event.

For a rehabilitation option, the existing bridge foundation elements may possibly need to be inspected to verify their load carrying capacity and the remaining service life but this is an essentially structural issue and may not be needed. If the widening of the existing structure is proposed with rehabilitation, the most logical and recommended option is to support the widening portion on the same type of foundations as the existing bridge foundations, especially if an integral abutment type bridge is required.

Other foundation options, such as shallow foundations for bridge abutments or driven H-piles foundations for pier foundations, are unlikely to be feasible due to integrity and long term performance of structure as

well as the prevailing subsurface conditions. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven. Cofferdams (e.g. compacted clayey soil dam or jersey barriers with sand bags and vinyl cover) will probably need to be provided during the construction.

The use of spread footing foundations at the abutment locations is not recommended, as this would require extensive shoring, as well as causing possible detrimental effects on the existing H-piles during the construction. As well, this type of structure would not behave as the existing integral abutment.

Driven piles to support the piers are neither recommended nor feasible due to the presence of bedrock with little or no overburden cover.

The use of drilled and cast-in-place concrete piles (caissons) may present an attractive alternative at the abutments, if there is no existing rock fill through which the caisson would be installed. As the existing piles are unlikely to be battered towards the new widening (i.e. integral abutments), there should be no interference. An advantage of this is that caisson foundations can be used for the widening at the pier locations. A disadvantage is that integral abutments cannot be duplicated and as mentioned before, this would be a major impediment, as the use of integral abutments would be desirable.

One other problem with caissons is sufficient socketing into the bedrock, as the bedrock at the site is expected to be a hard rock type (i.e. granitic gneiss, etc.). There may be some difficulties during the installation, which sometimes leads to claims for extras from the contractors, in spite of the fact that this aspect may be well documented in the contract. This aspect can perhaps be discussed with a caisson contractor and we will be pleased to assist you in this aspect should you wish us to do so.

Another alternative is to use micropiles. These are costlier than conventional foundation types, but this may be offset by reducing the amount of shoring required, as well as easier access for the equipment to install them in confined spaces. This too can be discussed with a specialized contractor and we will be pleased to assist on this, if we are asked to.

Details of shallow and deep foundations will be discussed in Section 4.2

Advantages, disadvantages, cost, risk and consequences of typical foundation options are summarized in Table F-3 in Appendix F.

4.1.4 Southbound Severn River Bridge (Bridge 42-86/2)

The existing southbound Severn River Bridge is a single span rigid frame structure supported by shallow foundations bearing on mass concrete block and bedrock (possibly adjacent to Borehole C1 location). Boreholes C1, C2, C3 and C4 were advanced at the bridge site. Bedrock was encountered at Boreholes C1 and C2 in between EL. 173.7 and 175.5 m at the east side of the bridge (construction south) while bedrock was encountered in between EL. 171.6 and 171.9 m in Boreholes C3 and C4 on the west (construction north) side of the bridge. Based on the GA drawing (See Drawing 6 in Appendix E), about 1.5 to 3 m thick mass concrete was placed to raise the grade to EL. 174.5 and 175.0 m on which strip footing foundations were constructed. Mass concrete was typically set 0.3 m into the bedrock.

For a rehabilitation option, the existing bridge foundation elements (including mass concrete and surrounding bedrock) may need to be inspected to verify their load carrying capacity and the remaining service life. As mentioned before, however, this is a primarily structural issue and may not be necessary, especially if no additional loads to the existing are to be applied. If widening of the existing structure is proposed with rehabilitation, widening portion can be supported on the same type of foundation as the existing bridge foundations. Alternatively, caissons or micropile foundations can be given consideration, as

discussed in the previous section of this report, covering the southbound Severn River Boat Channel Bridge. Temporary support system will be required for new shallow foundation construction immediately beside the existing structure foundations. The use of caissons and micropiles can be expected to reduce the effort required for shoring, in comparison with excavating to the bedrock level for the placement of mass concrete.

Advantages, disadvantages, cost, risk and consequences of typical foundation options are summarized in Table F-4 in Appendix F.

4.2 Foundation Details

4.2.1 Shallow Foundations

From the o.g. grade, bedrock is relatively shallow at this project site. In general, shallow foundations are the most economical and favourable option for shallow bedrock condition. However, due to the previous bridge construction, the grade was raised about 10 to 14 m at the site and thus, shallow foundation option may not be economical at some abutment locations, as extensive excavations would be required to reach bedrock. Spread footings are considered more economical at the following support elements (at other locations they may not be).

- Northbound Severn River Boat Channel Bridge piers
- Southbound Severn River Bridge (if widening is proposed)
- Southbound Severn River Boat Channel Bridge piers (if widening is proposed)

Where necessary, as shown on the GA drawings, the grade from the surface of the bedrock can be raised using mass concrete to the underside of the footings. Doweling into the sound bedrock may be necessary, as was done for the existing northbound bridges.

In general, shallow 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 dowelling/anchoring into the bedrock. In addition, the footing must be placed on sufficiently level rock surface. If necessary, the bedrock surface can be flattened by levelling or making benches or the problem may be alleviated by providing dowels. As well, it should be ensured that the rock beneath the footing level will not be subject to detrimental scour or frost effects which might jeopardize the footings.

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

For the evaluation of the sliding resistance of the foundations, the friction factor between the underside of the concrete footing and the clean, sound and sufficiently roughened bedrock surface can be taken as 0.70. Horizontal shear resistance can be supplemented by keying-in to the bedrock and utilizing the passive rock

resistance and/or shear in grouted dowels and/or rock anchors. We recommend a minimum dowel length of 1.2 m (minimum 0.6 m into sound bedrock).

The bearing surfaces should be inspected, evaluated and approved by the Geotechnical Engineer/Geologist appointed by the QVE. This is important for this project for the following reason.

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.

The rock must also be checked for any bedding 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.

Rock blasting may be required for shallow foundation construction. If rock blasting is required/permitted for excavation, it should be controlled in order to avoid over-breaking of bedrock. Wherever rock is over-excavated it should be inspected and approved by a Geotechnical Engineer and filled up with same class concrete as foundation concrete.

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

4.2.2 Deep Foundations

If the foundations are to be installed sufficiently beyond the existing shallow foundations or concrete thrust blocks and no rock fill is found within the new foundation footprint, consideration can be given to deep foundation options.

4.2.2.1 Driven H-piles

Piles driven to bedrock should be installed as per OPSS 903. Due to the sloping nature of bedrock at the site, rock injectors/points may be required. If the piles have to be driven through existing rock fill (if rock fill was used to build the existing embankments), the use of driven piles will neither be practical nor cost effective.

For 310x110 H-piles driven to bedrock, a factored geotechnical resistance of 2000 kN/pile can be used at ULS and SLS will not govern. The piles can be expected to be driven to the surface of the bedrock.

The lateral resistance of the piles can be assessed when detail subsoil conditions at the foundation locations become available. If required, lateral loading can be supported by horizontal components of battered piles.

If widening of the embankment at the site is proposed, oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

If integral abutments must be used for the new bridge, piles for integral abutments should have a 3 m long flex zone in accordance with MTO requirements (MTO Structural Office Standard).

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

Table 4.2.2.1.1
Sand Gradation required for the Flex Zone

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

When piles are driven onto bedrock, a shorter stroke shall be used. Piles should not be overdriven in any cases. When driving piles to bedrock, the Contractor shall adequately seat the pile on bedrock without damaging the pile. Where rock points are used, the rock points shall penetrate into the rock. Piles driven using rock points shall be driven to ensure adequate seating on the bedrock without damaging the pile. If the piles do not reach set at the anticipated tip elevation, the Contractor shall notify the Contract Administrator (CA).

Driving of piles on sloping bedrock shall be stopped when initial contact is made with the bedrock. The bedrock elevation shall be recorded. Driving shall then continue, commencing with energy of 10% of the maximum energy of the hammer. The pile shall be driven in sets of 20 blows at this energy until no penetration is observed. 20 additional blows shall be applied, and, if no penetration is observed, the energy shall be increased by an additional 10% and the above procedure repeated. Driving shall continue with these stepped increases in energy and with the same series of blows as described above, until the pile has been seated on the bedrock.

If unrealistic excessive penetration per blow is observed, driving shall be stopped and this excessive penetration immediately reported to the CA.

The Quality Verification Engineer shall determine when the hammer energy can be increased and when the driving is complete for each pile.

Retapping of piles driven to bedrock is not required. Driven pile tolerances would be as per OPSS 903.

Minimum clearance should be maintained for pile driving from the freshly poured concrete as per OPSS 903.

With the presently available data, the dip of the bedrock surface appears to be relatively mild and therefore the use of a suitable end bearing pile point, such as Titus H Bearing Pile Point or equivalent should be sufficient. However, should the detailed investigation reveal that the bedrock surface may dip more aggressively, then the use of Oslo Point rock points, as per OPSD 3000.21, may be required. Corrosivity may be another issue. If a corrosive environment is detected, then the required steel thickness may need to be increased (e.g. an HP310 x 110 pile may need to be substituted with an HP310 x 125 size H-pile).

In summary, the critical aspects of pile driving for this project are proper seating of the steel piles on the bedrock without causing “walking” of the piles due to lack of adequate overburden to prevent this condition, as well as avoiding improper seating of the piles which may cause sliding of the pile on the sloping bedrock surface.

4.2.2.2 Cast-in-place Concrete Piles

Cast-in-place concrete piles (drilled caissons) can be considered and piles socketed into the bedrock would be required to resist the axial and lateral loads. Vibrations and noise will be less in comparison with driven pile foundation option, except possibly when extending the caissons into the bedrock (i.e. while socketing into the bedrock). While excavating, rock adjacent to caisson should not be shattered (damage to the bedrock should be minimized). The use of drilled caissons are unlikely to present a practical, cost effective solution if the caissons are to be drilled through rock fill (e.g. if rock fill was used to build the existing embankments).

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 the imbalance horizontal forces.

The minimum spacing of the caissons centre to centre should 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 this is a compelling reason 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 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’.

4.2.2.3 Micropiles

Another alternative would be to use micropiles to avoid excavating existing foundations, mass concrete and possible rock fill.

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. For preliminary estimating purposes, a bond value of 1000 kPa between the sound granitic gneiss and concrete can be assumed. It may be possible to increase this value when details are known. Resistance offered by the overburden should be ignored. Minimum socket of 2 m into relatively sound bedrock should be considered for design of micropiles under compression. Structural steel and casing sizes should be selected based on the expected loading conditions but minimum 7" (178 mm) diameter casing is recommended for this project to ensure the minimum socketing of casing into the bedrock.

As can be expected, micropiles are costlier than driven steel piles or caissons. In addition, drilling through rock fill and placing the casing in the rock fill will add to the cost, (perhaps as much as 50% over and above normal cases where permanent casing is not required, depending on the details). But in spite of this, we feel that this alternative can be considered as an option. More details can be obtained from a specialized contractor. We will be pleased to facilitate this, if requested.

4.2.3 Summary of Foundation Options

This section presents a brief overview of recommended foundation options.

4.2.3.1 Northbound Severn River Bridge

The existing single span bridge is supported on spread footings bearing on and inset 0.3 m into bedrock. The existing approach embankments are rather high (about 14 m high) and there is some evidence that they are constructed using rock fill.

The new bridge can be supported on spread footings on mass concrete resting on bedrock, similar to the existing bridge.

The excavations extending to the surface of the bedrock required will however be rather deep (from the rock surface) and may possibly entail costly shoring support, and therefore in such a case consideration can be given to the use of deep foundations. This can consist of driven steel H-piles or caissons, if rock fill was not used. If rock fill was used, then consideration can also be given to the use of micropiles, as these can penetrate through the rock fill, existing concrete and into the bedrock.

4.2.3.2 Northbound Severn River Boat Channel Bridge

The existing northbound Severn River Boat Channel Bridge is supported by concrete thrust blocks. Columns 5E, 5W and abutments are supported by reinforced concrete columns and these are founded on bedrock. The approach embankments are likely to be constructed using earth fill rather than rock fill.

For pier support, the use of spread footings (possibly on mass concrete) may be an economical choice. For the abutments, depending on the distance from the shore, consideration can also be given to the use of deep foundations, namely driven steel H-piles, caissons and possibly micropiles, for integral or semi-integral abutment configuration.

4.2.3.3 Southbound Severn River Boat Channel Bridge

The existing 119 m long, three-span bridge may be widened. The existing integral abutments are supported on driven steel H-piles. These can be duplicated. If necessary, consideration can be given to the use of caisson foundations, or if access and overhead restrictions govern, then the use of micropiles can be considered but driven steel H-piles may be the preferred choice for integral abutment configuration.

The piers can be supported on spread footings bearing directly on bedrock, similar to the existing bridge.

4.2.3.4 Southbound Severn River Bridge

The existing bridge is a 27.5 m long single span, rigid frame structure. It is supported on spread footing foundations constructed on mass concrete embedded into the bedrock. The bridge may be widened.

The widening can be supported on spread footing foundations constructed on mass concrete bearing on bedrock, duplicating the existing structure.

Alternatively, in order to reduce shoring effort and cost, consideration can be given to the use of deep foundations. As the existing mass concrete may protrude beyond the existing bridge footprint and into the widening footprint, the use of driven steel piles may not be suitable. In addition, the overburden appears to be rather scanty, especially on the south side. For these reasons, caissons and especially micropiles may be more suitable in comparison with driven piles, if the use of deep foundations is preferred.

4.3 Lateral Earth Pressures

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

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B' Type I or Type II, with minus 0.075 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:

$K_a = 0.27$ $K_b = 0.35$

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

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$ $K_b = 0.41$

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

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

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

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding (e.g. when supported on bedrock), 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.

4.4 Approach Embankments

4.4.1 Northbound Bridges Approach Embankments

According to Racey MacCallum Report (GEOCRE 31D-116), a separate report for the embankment stability was issued but it was not available at the time of preparing our report. The Racey MacCallum Report (GEOCRE 31D-116) indicates that “the surcharge load imposed by the embankment is much lower than the compressive strength of the underlying clay and, therefore, the occurrence of slow plastic flow in this clay is extremely unlikely”. This sentence indicates that short term basal stability of rock fill embankment (rock fill was assumed for surcharge load calculation; top elevation of embankment was assumed at EL. 596 ft or 181.7 m and the existing embankment top elevation is about EL. 184 m) was confirmed by Racey, MacCallum and Associates Limited. Although embankment was constructed about 2.3 m higher than expected, no record of embankment instability appears to have been reported. In general, basal stability of embankment improves with time (e.g. dissipation of excess pore water pressure, consolidation, etc.) and therefore the clayey soil within the footprint of the existing embankment may not cause problem during new embankment construction. However, if widening and/or significant grade raise of the existing embankment is proposed, detail foundation investigation should be carried out to assess the stability of the new embankment with due consideration of the extent of marshy areas. These aspects will be pointed out again in the Section 5. recommendations for detail foundation investigation.

For widening of embankment, all organics and softened material should be stripped from the plan limits of the approach embankment prior to the placement of any fill. The new fill should be properly placed on the existing embankment as per OPSD 208.01.

If there is chance to induce additional loading on the clayey soil (e.g. widening and/or grade raise), settlement of founding soil and stability of the embankment should be assessed. If calculated settlements are excessive, either surcharging or preloading should be considered.

4.4.2 South Bound Bridges Approach Embankments

Details of proposed slope configurations, anticipated settlement of about 11 m high approach embankments were discussed in Strata Engineering Corp. reports (GEOCRE31D-323 and 326).

For the Severn River Bridge north (construction north, west) approach embankment, slope stability analyses were carried out in 1990 by Strata Engineering Corp. Based on the stability analyses results, no steeper than 2H:1V slope configuration was recommended for west side embankment slope (on the lake side) even if rock fill is used (rather than the then typical 1.5 H:1V side slopes for rock fill embankments) due to the possible translation instability towards the lake. A conventional slope configuration of 1.5H:1V was suggested for rock fill at the other side (east) embankment due to the existing highway fill on the east side. From the plasticity characteristics of the soil, settlement no more than 50 mm (founding soil only) was estimated for about 11 m high embankment.

No stability issue was reported for the rest approach embankments and conventional 2H:1V earth slopes and 1.5H:1V rock fill slopes were recommended.

For widening of embankments, all organics and soft materials must be stripped from the plan limits of the immediate approach embankment prior to the placement of any fill. New fill should be properly keyed into the existing embankment as per OPSD 208.01.

If there is chance to induce additional loading on the clayey soil (e.g. widening and/or grade raise), settlement of founding soil and stability of the embankment should be assessed. If calculated settlements are excessive, either surcharging or preloading should be considered. Additional boreholes should be advanced for settlement and stability assessment. These requirements are included in the Section 5. recommendations for detail foundation investigation.

4.5 Seismic Design

4.5.1 Single span bridges

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.

4.5.2 Multiple span bridges

Background Information

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-00). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S , for the site is 1.0. Table A3.1.1 of the CHBDC provides that the general area has a Zonal Acceleration Ratio of 0.05 and Velocity Related Seismic Zone (Z_v) of zero. As site coefficient (S) is 1.0, and the zonal acceleration is 0.05, the design zonal acceleration ratio for the site can be taken as $A=0.05$. This bridge site can be classified as Seismic Performance Zone 1 or 2 based on the above values and the intended use (e.g. lifeline structure designation or not). Subsection 4.4.5.3 and Table 4.2 of the CHBDC indicate that seismic analysis is not required for bridges in Seismic Performance Zone 1. These should be reviewed by the Structural Engineer.

Seismic Earth Pressures

Seismic (earthquake) loading should be taken into account in the design in accordance with Section 4.6 of the CHBDC.

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

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

Table 4.5.2.1
Seismic Active Pressure Coefficients

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

In the calculation of K_{AE} , the effect of the friction between the wall and the soil is not considered (i.e. $\delta=0.5\phi$).

4.5.3 Liquefaction Potential

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

4.6 Construction

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

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

Excavations within the existing fill and native soils should be possible using heavy equipment such as a hydraulic excavator. Contractor should keep in mind that rock fill can be contacted within the project limit. Rock blasting may be required for shallow foundation construction. If rock blasting is required (and permitted), it should be controlled carefully in order to avoid over-breaking of bedrock.

Extent of dewatering and unwatering should be assessed carefully with consideration of groundwater and lake water conditions at the time of construction. Trent-Severn waterway system discharge schedule should be also considered. If required, earthen cofferdams (such as compacted clayey soil or sand bags and vinyl cover with jersey barriers) should be implemented to maintain a relatively dry work platform for the bridge replacement or rehabilitation.

In the case of staged construction (if full road closure is not permitted), temporary shoring will be required to support the excavations. In Ontario, shoring typically consists of soldier pile and timber lagging or sheet

piling (with or without bracing / rakers). The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. The shoring system should be designed by a Professional Engineer, experienced in this type of work to meet performance level 2 criteria in OPSS 539. We will be pleased to assist temporary support design, if staged construction is proposed.

4.7 Vibration Monitoring

Due to the fact that four bridges and a residence are located close to each other, vibrations induced by possible blasting (or other rock removal methods) and pile driving should be monitored during construction.

4.8 Scour and Erosion Protection

On-going erosion of bridge forward slope was noted at northbound Severn River boat channel bridge site. Rock protection as per OPSS 511 should be considered. We recommend that channel and bridge scour protection and erosion control be designed by an experienced Hydraulic Engineer.

4.9 Frost Protection

Design frost protection depth for the general area is about 1.6 m. Therefore, a permanent soil cover of about 1.6 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. However, where footings are placed on massive rock (i.e. free of fissures, etc. where water can get into) and the recommended measures presented Section 4.2.1 can be carried out, the frost depth requirements can be reduced or eliminated, depending on the bedrock and site conditions.

In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection.

5 RECOMMENDATIONS FOR DETAILED FOUNDATION INVESTIGATION

Some of the salient features for the detailed foundation investigation are as follows:

- ❖ Minimum two boreholes per each foundation unit and one borehole per each approach embankment are required.
- ❖ Rock coring should be carried out at foundation borehole locations and unconfined compression strength (UCS) tests on selected rock core samples should be carried out. These can be supplemented by point load tests where desirable.
- ❖ Additional boreholes may be required depending on structure strategy (e.g. widening, staged construction, rehabilitation etc.).
- ❖ Existing embankment fill materials should be investigated (if necessary, instead of one deep hole, several shallow holes should be advanced to refusal depths on the rock fill).
- ❖ Field vane tests (MTO N vane or smaller vane for higher shear strength than 200 kPa) should be carried out during borehole investigation where clayey soil is encountered, especially where embankment widening and/or grade raise is proposed.
- ❖ Consolidation characteristics of the clayey soil should be investigated if additional loading is anticipated. Proper sampling and testing should be included in the investigation program.

6 CLOSURE

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

For and on behalf of Coffey



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

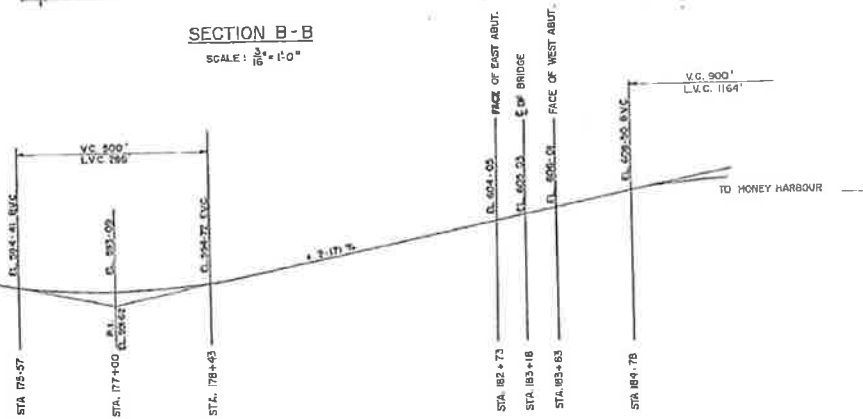
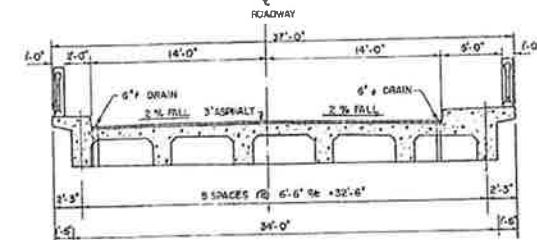
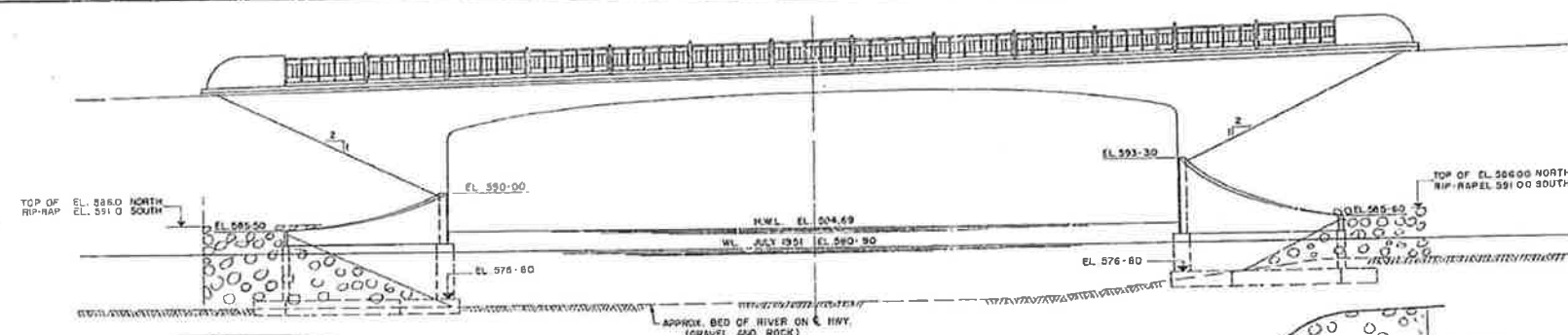


Zuhtu Ozden, P.Eng.
Senior Principal

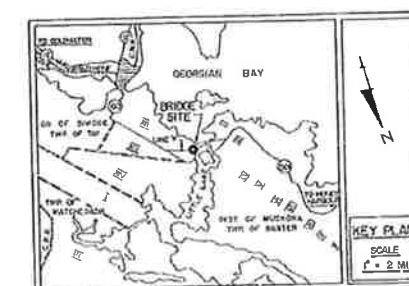
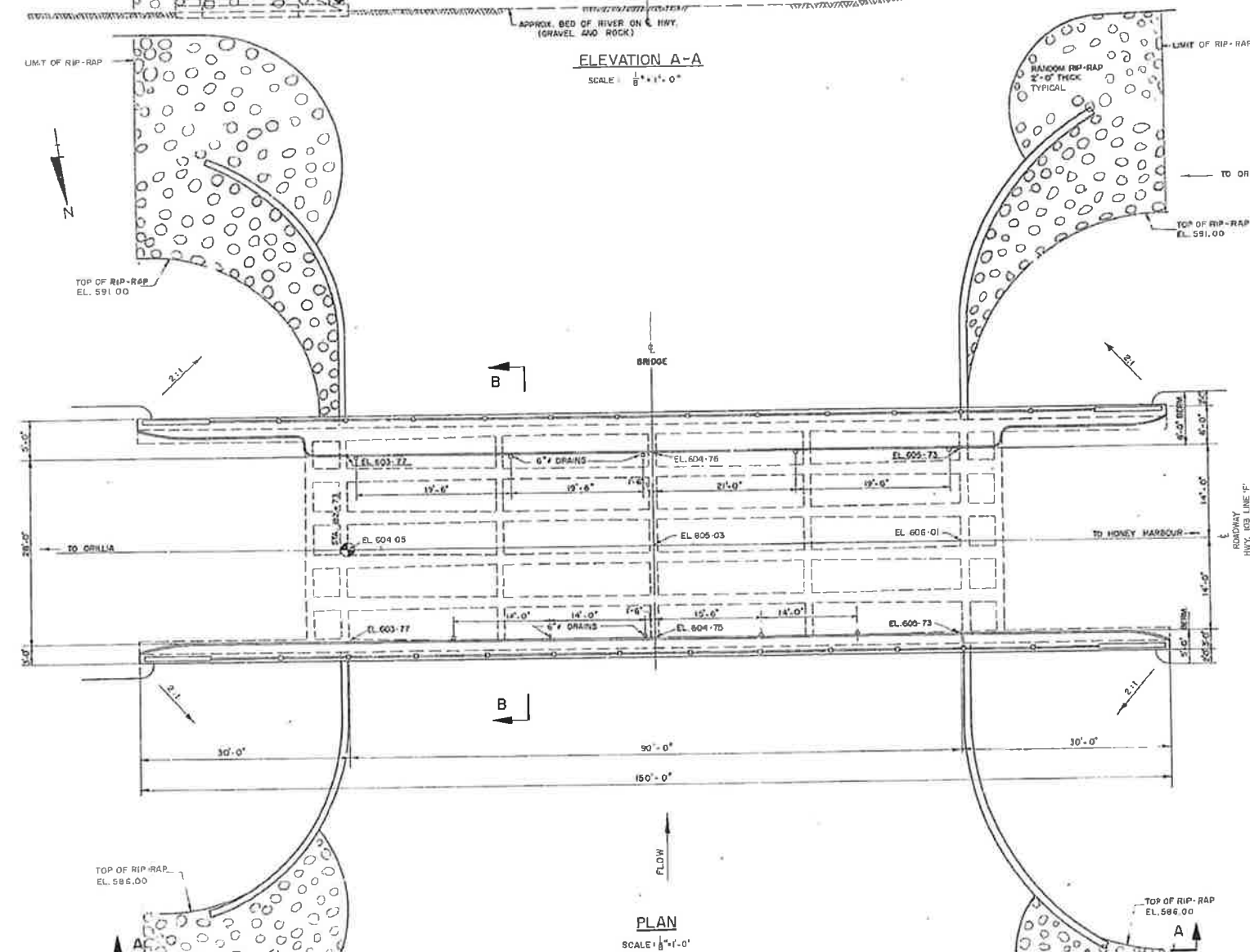


Appendix E

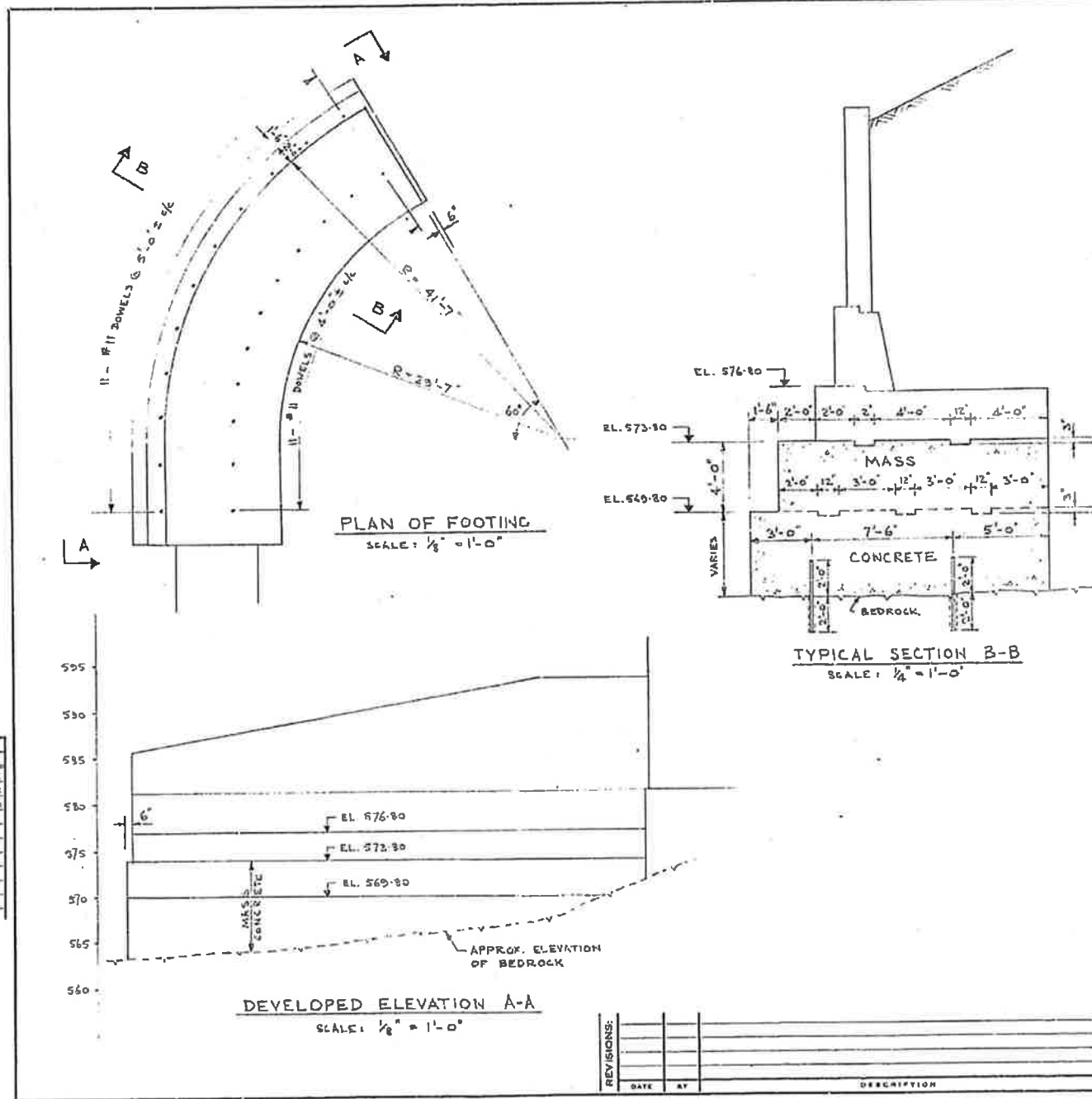
GA Drawings



- ### NOTES
- TO CONTRACT ENGINEER: CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMPLETED UNTIL INSPECTIONS TO FIX CONTROL POINTS HAVE BEEN DIRECTED AND CHECKED BY THE DISTRICT ENGINEER.
 - TO GENERAL CONTRACTOR: STRUCTURE TO BE BUILT IN ACCORDANCE WITH THE 1992 REVISED WATERSHED TEST AND THE SPECIAL PROVISIONS EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER.
 - MASS CONCRETE SHALL HAVE A MINIMUM ULTIMATE COMPRESSIVE STRENGTH OF 1500 p.s.i. AT 28 DAYS, ALL OTHER CONCRETE 3000 p.s.i.
 - ADMITTIVES: ADD 1% SPOZZOLITH "S" PER BAG OF CEMENT IN REINFORCED CONCRETE MIX.
 - REINFORCING STEEL SHALL BE OF HARD GRADE.
 - MAXIMUM SIZE OF AGGREGATE 3" FOR ALL CONCRETE.
 - ELEVATIONS ON DECK ARE TO TOP OF ASPHALT.

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THIS PRINT SUPERSEDES ALL PRINTS BEARING A PREVIOUS LETTER



- NOTES:
- THIS DRAWING TO BE READ IN CONJUNCTION WITH DRAWINGS D 3784-2 & 12
 - ELEVATION OF FOOTING REMAINS AS SHOWN ON DRAWING D 3784-2
 - THIS DRAWING SUPERSEDES DWGS D 3784-2 & 12 FOR ONLY THOSE DETAILS SPECIFICALLY SHOWN.
 - CONSTRUCTION JOINT AT EL 569-80 IS OPTIONAL.
 - DOWELS TO BE #11 BARS 4'-0" MIN. LENGTH, TO BE GROUTED INTO BEDROCK (22 REQD)

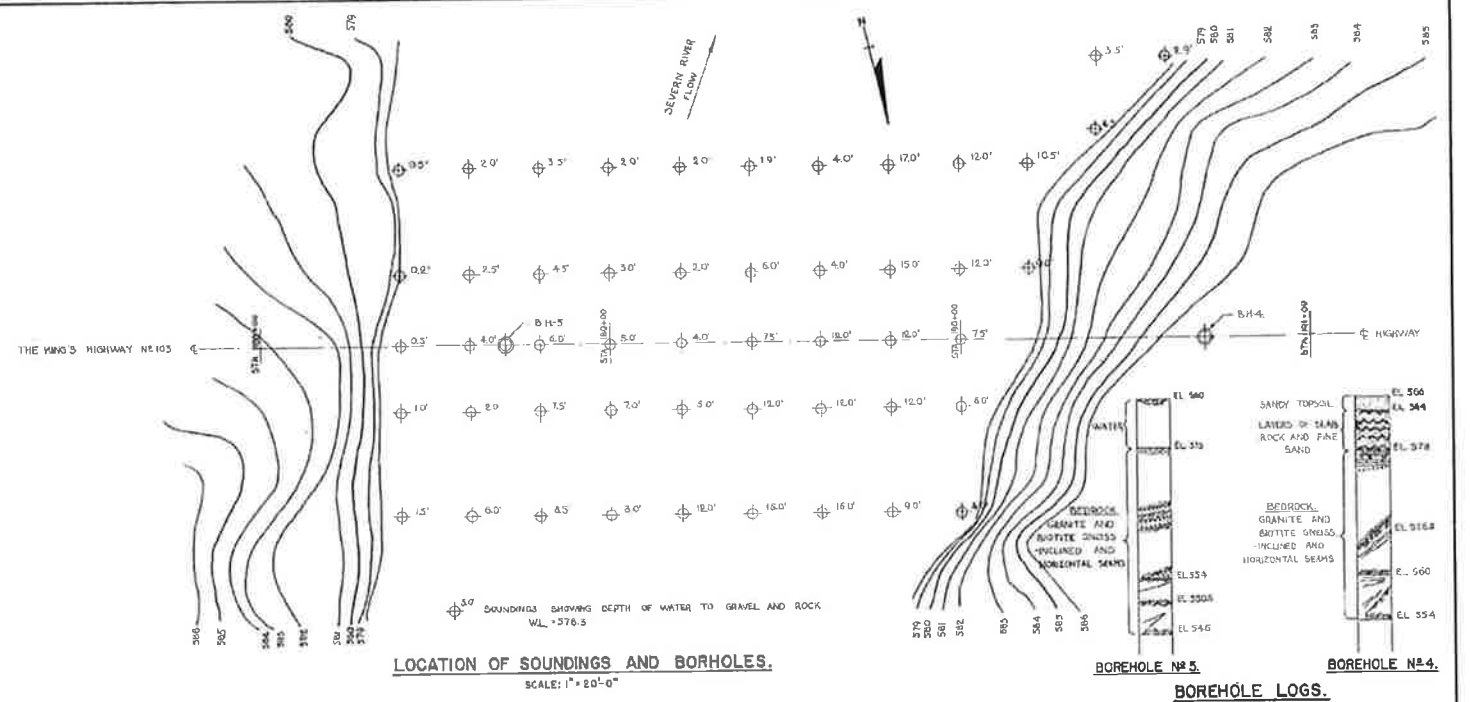
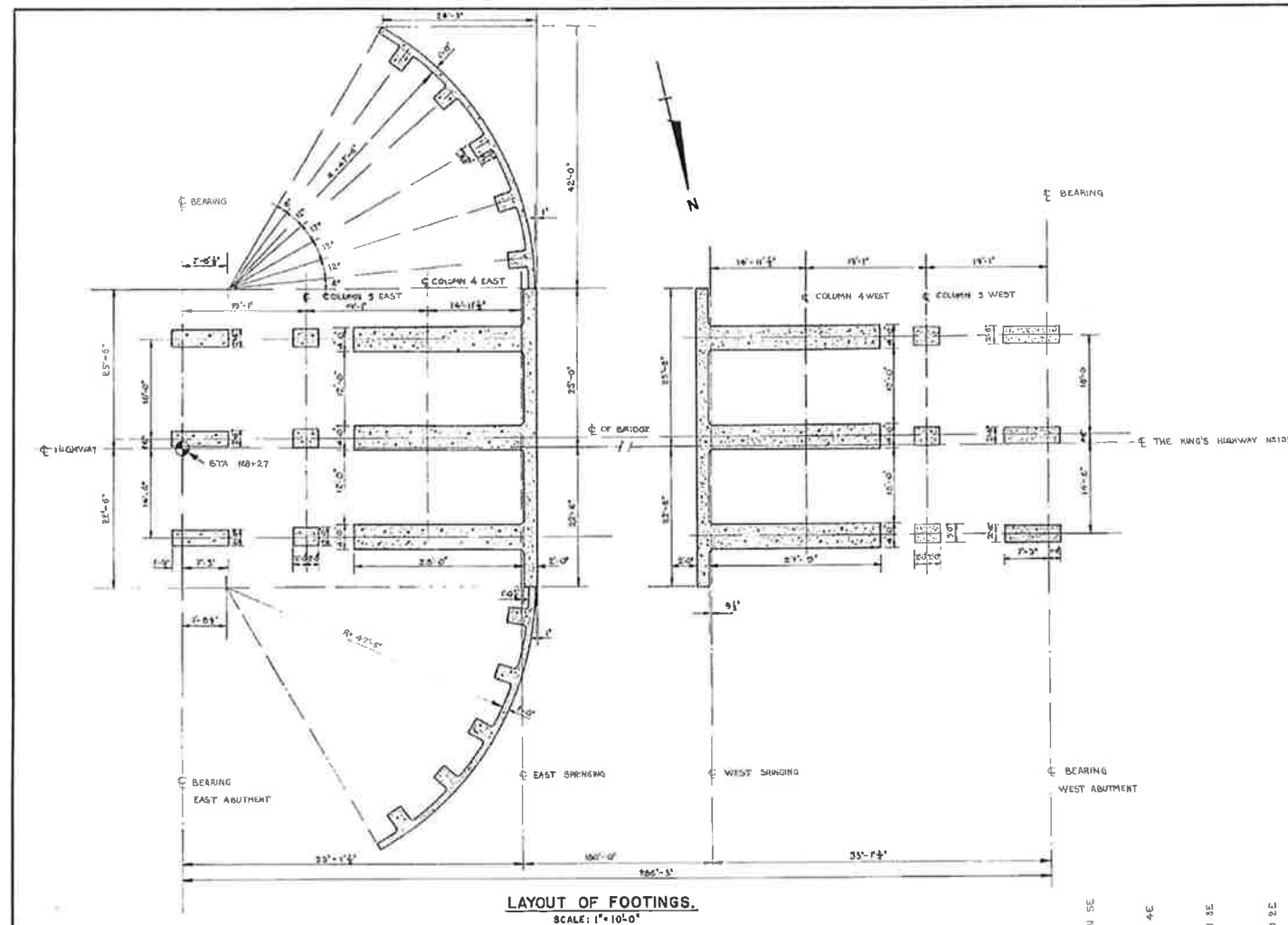
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T.O. LAZARIDES AND ASSOCIATES LTD. CONSULTING ENGINEERS 205 DAVENPORT RD. TORONTO			
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CO. SIMCOE	DIST. No. 11		
TWP. TAY	LOT 18	CON. XII	
FOR REVISED FOUNDATION FOR SOUTH WEST RETAINING WALL			
APPROVED			
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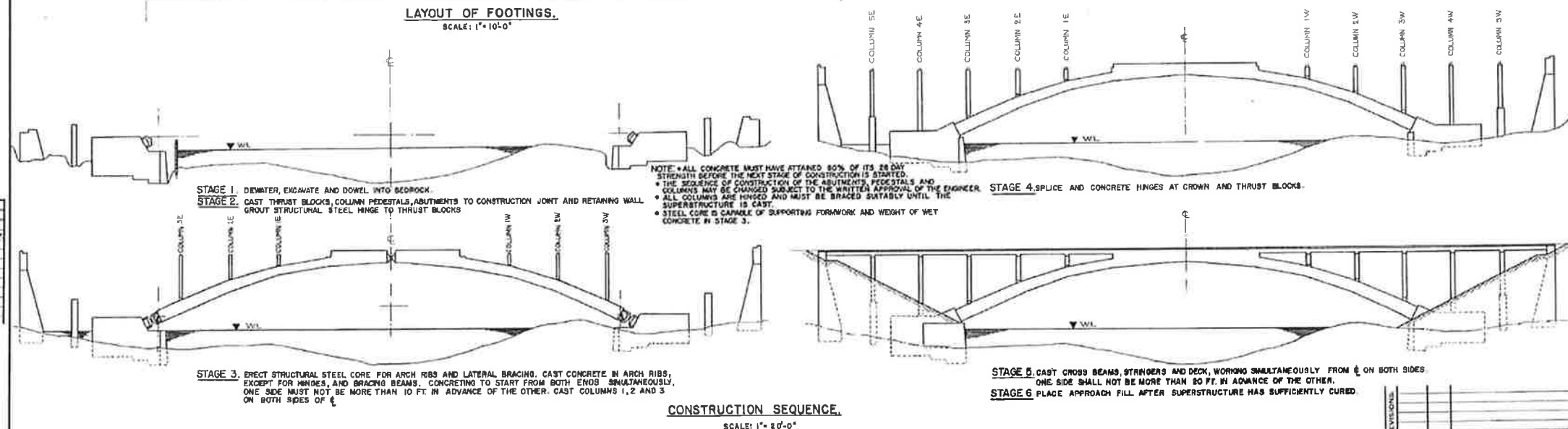
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88	"	1.15.66
89	"	2.15.66
90	"	3.15.66
91	"	4.15.66
92	"	5.15.66
93	"	6.15.66
94	"	7.15.66
95	"	8.15.66
96	"	9.15.66
97	"	10.15.66
98	"	11.15.66
99	"	12.15.66
100	"	1.15.67

REVISIONS:	DATE	BY	DESCRIPTION

Twp # 456.86 13.1



- NOTES:**
- THIS DRAWING TO BE READ IN CONJUNCTION WITH DWG. 03783-3, 4 & 5.
 - SOUNDINGS ARE REPRODUCED FROM D.H.D. PLAN E-317-1.
 - DRAWING IN BEDROCK OR CONCRETE WORK ON THIS STRUCTURE MUST NOT START BEFORE CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE ENGINEER.
 - SOUNDINGS AND ROCK ELEVATIONS ARE NOT GUARANTEED BY D.H.D.



VP56A-56

T.O. LAZARIDES, LOUNT AND PARTNERS
CONSULTING ENGINEERS

HEAD OFFICE: 208 DAVENPORT RD. TORONTO

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

SEVERN RIVER BRIDGE
BOAT CHANNEL

THE KING'S HIGHWAY No. 103 (T.G.H.) DIST. No. 11

DISTR. OF MURDOCH LOT 27 CON. II

LAYOUT OF FOOTINGS, BORHOLES AND CONSTRUCTION SEQUENCE

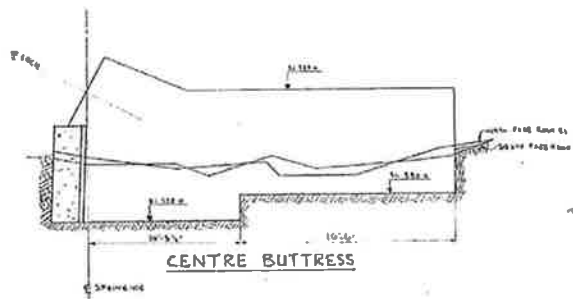
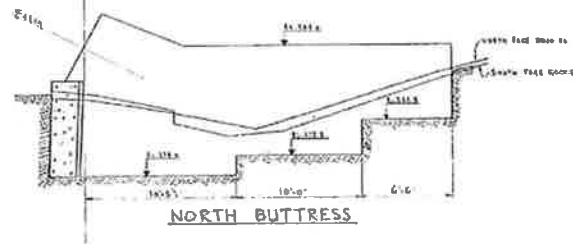
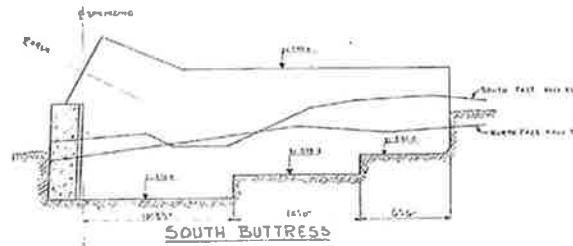
APPROVED: [Signature] BRIDGE ENGINEER

DATE: JANUARY 20, 1957

REVISIONS: [Table with columns for REVISION, DATE, BY, CHECKED, DATE, BY]

THIS PRINT SUPERSEDES ALL PRINTS BEARING A PREVIOUS LETTER

Drawing E-4



NOTES:
 1. ELEVATION OF ROAD TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 2. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 3. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 4. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 5. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 6. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 7. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 8. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 9. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.
 10. ELEVATION OF BUTTRESS TO BE SHOWN BY DASH LINE BEHIND THE BUTTRESS.

10154-56

T.O. LAZARIDES AND ASSOCIATES LIMITED
 CONSULTING ENGINEERS
 208 DAVENPORT ROAD TORONTO
 DEPARTMENT OF HIGHWAYS, ONTARIO
 TORONTO OFFICE - TORONTO

**SEVERN RIVER BRIDGE
 ROAT CHANNEL**

FOR THE ONTARIO HIGHWAYS DEPARTMENT
 DIST. NO. 10154-56
 DATE OF PREPARED 1957
 DATE OF REVIEW 1957

REVISION OF WEST THRUST BLOCK ROCK EXCAVATION

APPROVED
 C.M. F.

DESIGN	DATE	BY	CHECKED	DATE	BY
10154-56	1957	C.M. F.			

DATE, MARKED 10.1.1958

10154-56-12A

SEVERN RIVER BOAT CHANNEL BRIDGE
HIGHWAY 69 - SOUTHBOUND LANES
GENERAL ARRANGEMENT

SHEET
15192

<u>1. CLASS OF CONCRETE</u>	
ALL CONCRETE	30 MPa
<u>2. CLEAR COVER TO REINFORCING STEEL</u>	
FOOTINGS	50
Pier columns and cap beams	100±25
ABUTMENTS AND MARGINALS ³	80±20
FRONT FACE	80±10
BACK FACE	75±10
DECK SLAB	
TOP SURFACES	70±20
BOTTOM SURFACES	45±10
REINFORCER	70±20
	UNLESS
	OTHERWISE
	NOTED

4. CONSTRUCTION NOTES:-
IF THE ACTUAL BEARING HEIGHTS ARE DIFFERENT FROM THE ASSUMED HEIGHTS GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL ADJUST THE BEARING SEAT ELEVATIONS AND THE REINFORCED STEEL TO SUIT THE ACTUAL HEIGHTS.

PRE-ANCHOR PILE HOLES TO BEDROCK LEVEL, BACKFILL HOLE WITH M70 CLASS 4 AGGREGATE PRIOR TO DRIVING PILES.

ROCK SURFACES IN OVER-EXCAVATED AREAS SHALL BE SUBJECT TO APPROVAL BY THE ENGINEER.

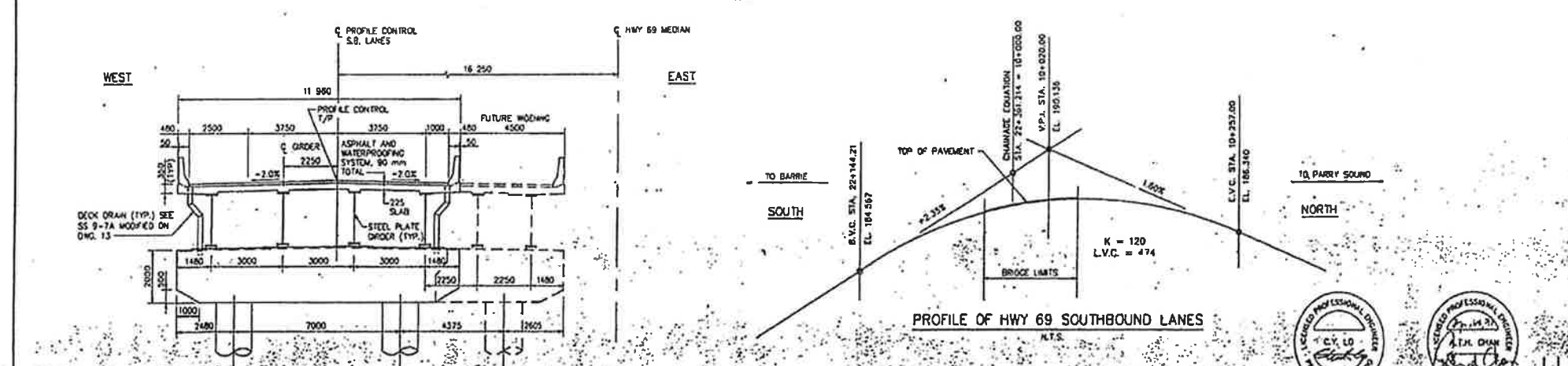
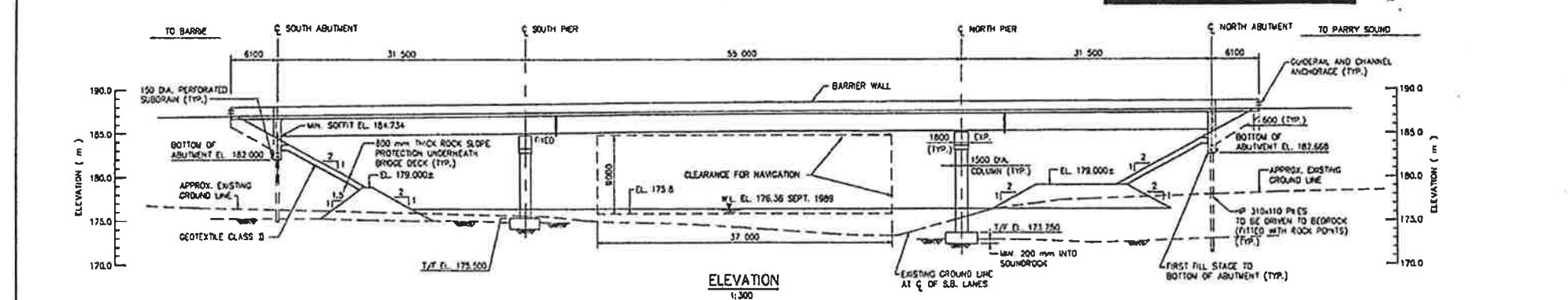
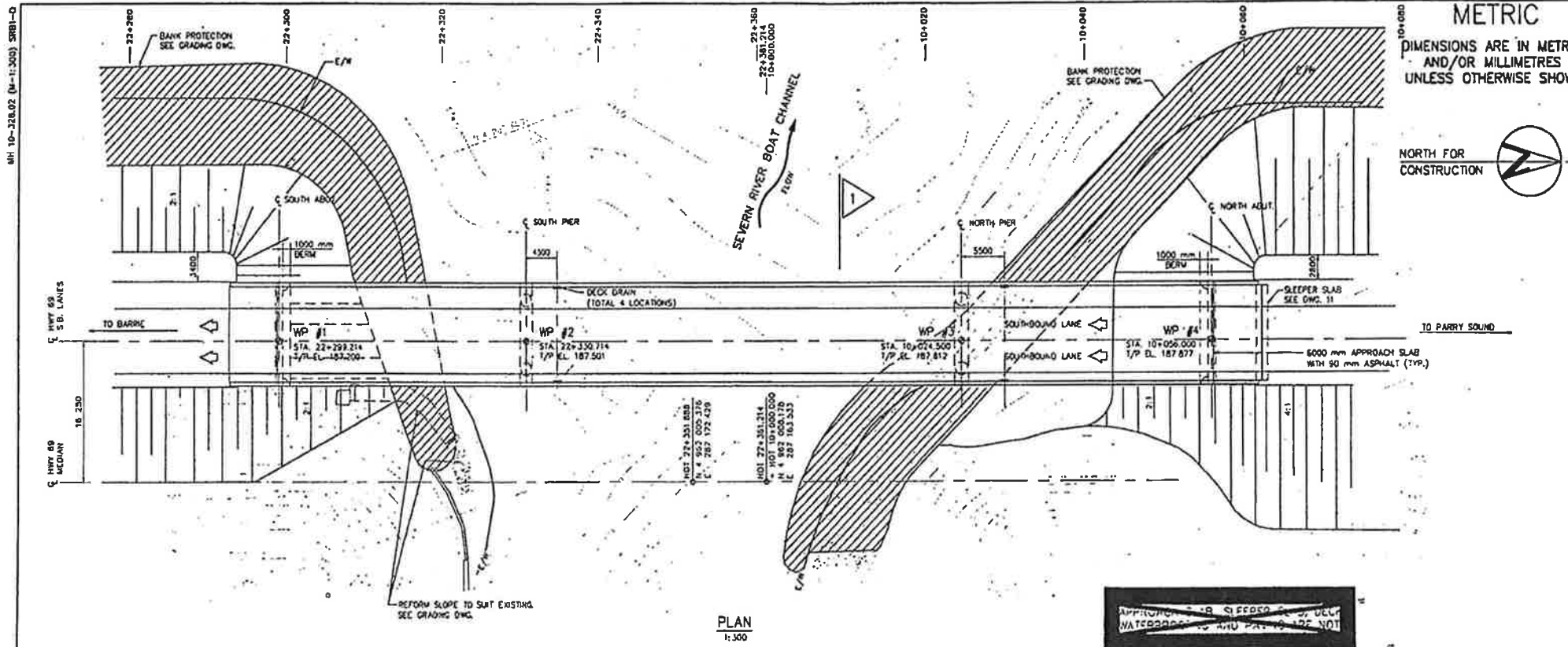
5. CONSTRUCTION SEQUENCE:

ROCK BERMS AS SPECIFIED ON GRADING DRAWINGS SHALL BE IN PLACE BEFORE ABUTMENT CONSTRUCTION.

1. GENERAL ARRANGEMENT
2. BOREHOLE LOCATIONS AND SOIL STRATA
3. FOOTING LAYOUT AND DETAILS
4. ABUTMENTS AND RETAINING WALLS
5. PIERS AND BEARING DETAILS
6. DECK LAYOUT AND SCORED ELEVATIONS
7. DECK REINFORCING DETAILS
8. STRUCTURAL STEEL - I
9. STRUCTURAL STEEL - II
10. BARRIER WALLS
11. 6000 mm APPROACH SLAB
12. AS CONSTRUCTED ELEVATION AND DIMENSION
13. STANDARD DETAILS
14. QUANTITIES STRUCTURE - I
15. QUANTITIES STRUCTURE - II

T/F	TOP OF FOOTING
T/C	TOP OF CONCRETE
T/P	TOP OF PAVEMENT
WP	WORKING POINT
E/W	EDGE OF WATER
W.L.	WATER LEVEL

00-3503 MAXIMUM GRANULAR BACKFILL REQUIREMENTS
(MODIFIED SEE OMC 13)



B.M. 187.080
C.C. on N.W. end of C.W. of Bridge
10.4 R/L 10+035.3, TWP GEORGIAN BAY

1
1:100
(NORTH PIER AS SHOWN, SOUTH PIER SIMILAR)

DRAWING NOT TO BE SCALED
100 mm GH ORIGINAL DRAWING

REVISIONS
DESIGN
DRAW

DATE		BY		DESCRIPTION			
CH	RL	CHK.	CL	CODE	0-800-83	LOAD CLASS A	DATE FEB/91
CH	HT	CHK.	ATC	SITE	42-825	STRUCT.	SCHEME DWG. 1

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

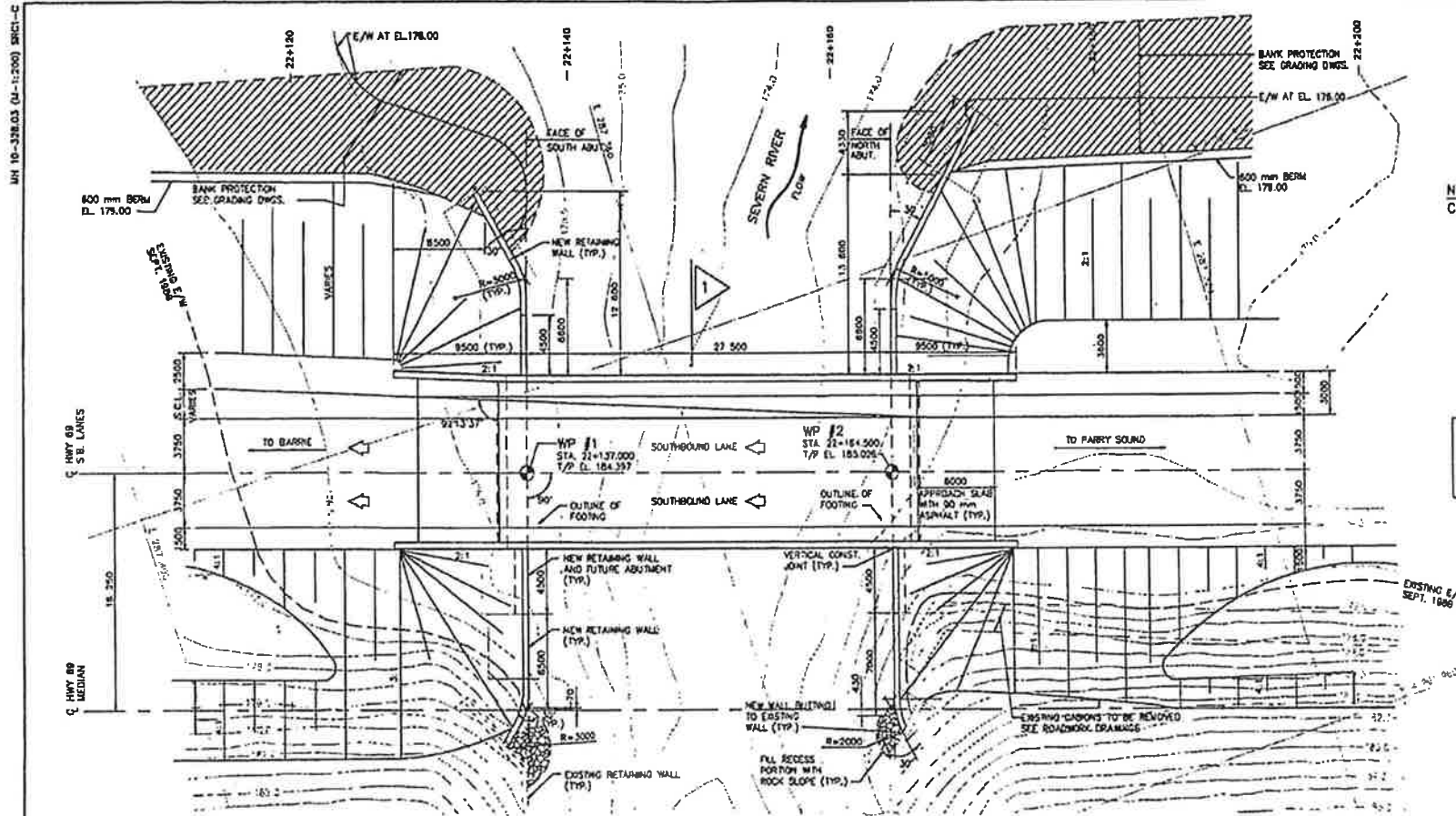
- T/T - TOP OF FOOTING
T/C - TOP OF CONCRETE
T/P - TOP OF PAVEMENT
WP - WORKING POINT
E/W - EDGE OF WATER
S.C.L. - SPEED CHANGE LANE
H.W.L. - HIGH WATER LEVEL
C.J. - CONSTRUCTION JOINT

APPLICABLE STANDARD DRAWINGS

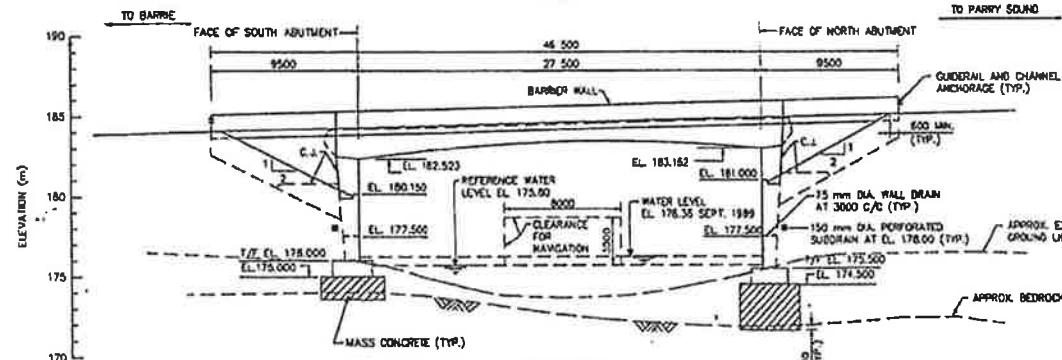
- DO-3501 MINIMUM GRANULAR BACKFILL REQUIREMENTS
DO-3504 RETAINING WALL BACKFILL REQUIREMENTS



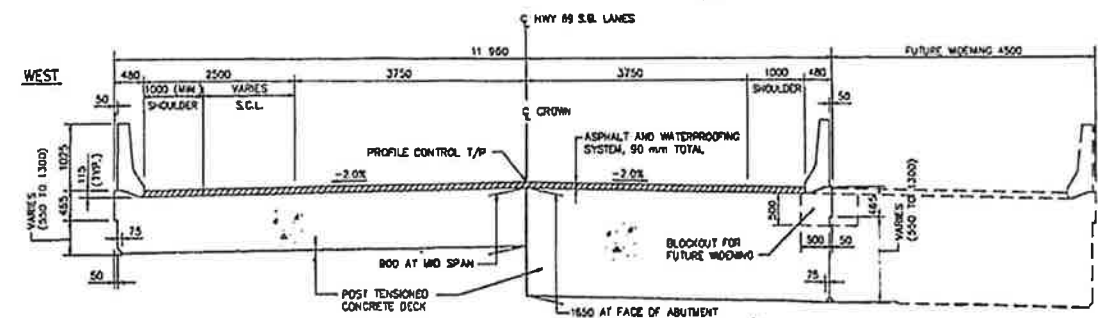
DATE	BY	DESCRIPTION
DESIGN	CL	CHK. ATC CODE 0400-83 LOAD CLASS A1 DATE DEC/90
DRAWN	MT	CHK. ATC SITE 42-563 STRUCT. SCHEME DWG. 1



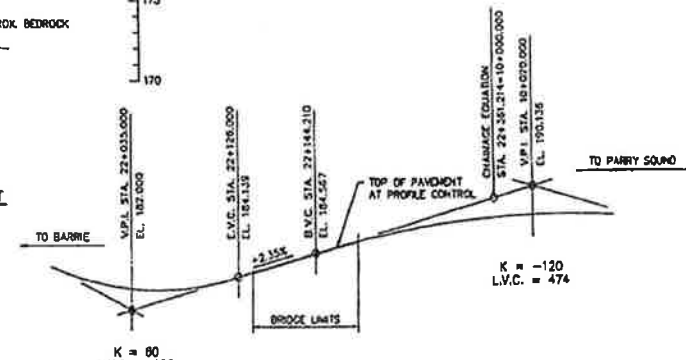
PLAN
1:200



ELEVATION
1:200



BM 184.008
Cut Cross D.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 F

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

Table F-1

Foundation Options for Northbound Severn River Bridge Replacement

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	- Will require extensive excavations and dewatering		Moderate to High	-Merits consideration for abutments. -Recommended for piers (if any)
Driven steel H-piles foundations	-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles -No dewatering is required -Pre-augering to bedrock surface may be required -Feasible for integral abutment design	-Vibration -Presence of rock fill -Extent of existing foundations	Moderate	-If rock fill was used for existing embankment construction, this option is not feasible -Feasible if piles will be driven beyond the existing shallow foundations and mass concrete and if rock fill was not used to build the embankments -Pre-augering may be required
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-Less vibrations and noise created than driven piles -Socketing into sound bedrock may be difficult -Feasible for semi-integral abutment	-Hole stability should be maintained -Presence of rock fill -Extent of existing foundations	More expensive than driven piles	-If rock fill was used for existing embankment construction, this option is not feasible -Feasible but less suitable for the prevailing subsurface conditions in comparison with steel H-piles
Micropiles	-Suitable for advancing through rock fill and through existing mass concrete and reinforced concrete footings - Equipment easier to operate in difficult access conditions		Expensive	-There is some evidence that the existing embankments were constructed using rock fill. In that case, the use of micropiles may present an attractive alternative

Table F-2

Foundation Options for Northbound Severn River Boat Channel Bridge Replacement

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	- Will require extensive excavations and dewatering at abutment locations		Moderate to High (for abutments) Low (for piers)	-May not be economical for abutments if deep excavations are required -Favourable option for piers
Driven steel H-piles foundations	-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles -No dewatering is required -Pre-augering to bedrock surface may be required -Feasible for Integral abutment	-Vibration -Possible presence of rock fill -Extent of existing foundations	Moderate	-Only applicable for abutments (not for piers) -If rock fill was used for existing embankment construction, this option is not feasible -Feasible if piles will be driven beyond the existing shallow foundations and thrust blocks -Pre-augering may be required
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-Less vibrations and noise created than driven piles -Socketing into sound bedrock may be difficult -Feasible for semi-integral abutment	-Presence of rock fill -Extent of existing foundations	More expensive than driven piles	-Only applicable for abutments (not for piers) -If rock fill was used for existing embankment construction, this option is not feasible -Feasible option but less suitable for the prevailing soil condition in comparison with steel H-piles
Micropiles	-Equipment easier to operate in difficult access conditions -Can drill through rock fill or existing concrete including reinforcing bars in the concrete		Expensive	-Unlikely to be economical unless the existing embankment is constructed from rock fill or drilling through existing concrete is required

Table F-3

Foundation Options for Southbound Severn River Boat Channel Bridge Widening

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Shallow foundations (on bedrock or on mass concrete placed on bedrock)	-For pier foundation, lower price than other options		Low (for piers)	-Not feasible for abutments -Favourable option for piers
Driven steel H-piles foundations	-Low displacement piles and as such more suitable than other types of driven piles such as precast concrete or steel tube piles -No dewatering is required -Pre-augering to bedrock surface may be required -Feasible for Integral abutment	-Vibration	Moderate	-Only applicable for abutments (not for piers) -Favourable option -Pre-augering may be required
Drilled and cast-in-place Concrete piles (drilled caissons) foundations	-Less vibrations and noise created than driven piles -Socketing into sound bedrock may be difficult -Feasible for semi-integral abutment	-Hole stability should be maintained	More expensive than driven piles	-Only applicable for abutments (not a cost effective option for piers) -Feasible option but less suitable for the prevailing soil condition in comparison with steel H-piles
Micropiles	-Can be installed more easily in case of low overhead and/or difficult access		More expensive than the other options	-The additional expense is not warranted for piers, but can be considered for abutments access/low overhead conditions present problems

Table F-4

Foundation Options for Southbound 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 other options -Dewatering and unwatering may be required -May require extensive shoring 		Low	<ul style="list-style-type: none"> -Feasible -Temporary support system may be required
Driven steel H-piles 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"> -Unlikely to be feasible
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"> -Should only 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 		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 G

List of OPSS, OPSD and Non-standard Specifications

OPSDs

OPSD 208.01 Benching of Earth Slopes

OPSD 3000.201 Foundation Piles Steel HP 310 Oslo Point

OPSD 3101.150 Walls, Abutment, Backfill Minimum Granular Requirement

OPSD 3101.200 Walls, Abutment, Backfill Rock

OPSSs

OPSS 511 Construction Specification for Rip-Rap, Rock Protection and Granular Sheting

OPSS 539 - Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures

OPSS 903 – Construction Specification for Deep Foundations

NSSPs

NSSP – Vibration Monitoring

Special Provision

Vibration Monitoring

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

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

The Contractor shall take readings on the first pile in each pile group (i.e. at each corner of the abutment), starting with the pile furthest away from the existing structure. As a minimum, the readings should be taken and recorded during the first 3 m of driving and during seating of the pile onto the competent materials.

The results shall be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specification. The results shall be submitted to the Contract Administrator prior to continuing with the remaining piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results. Field records of blasting should be submitted as well.

If the results are acceptable, the Contractor may continue with the remaining blasting/piles with readings taken during driving of each pile. Subsequent vibration readings should be taken for each pile during the seating on the competent materials. The results of the subsequent piles should be certified by the Quality Verification Engineer as being accurate and meeting the requirements of the specifications. 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 driving/blasting procedures until the vibrations on the existing and newly built structure are within acceptable levels. The above process must be repeated for each pile/set of blasting.

Appendix H

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.