



**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORTS
FOR THE REHABILITATION OF HIGHWAY
400 / COUNTY ROAD 23 OVERPASS,
TOWNSHIP OF COLDWATER, ONTARIO
G.W.P. 2190-10-02, SITE NO. 30/455-1/2
GEOCRES 31D-562**

McCormick Rankin Corporation

Project: TRANETOB20462AA
September 16, 2013

REPORT



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September 16, 2013

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

Dear Mr. Hui;

**RE: Preliminary Foundation Investigation and Design Reports
Rehabilitation of Highway 400 / County Road 23 Overpass, Township of Coldwater,
Ontario**

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", written over a set of horizontal blue lines.

Zuhtu Ozden, P.Eng.
Senior Principal



**PRELIMINARY FOUNDATION INVESTIGATION
REPORT FOR THE REHABILITATION OF
HIGHWAY 400 / COUNTY ROAD 23
OVERPASS, TOWNSHIP OF COLDWATER,
ONTARIO, G.W.P. 2190-10-02,
SITE NO. 30/455-1/2, GEOCRES 31D-562**

McCormick Rankin Corporation

TRANETOB20462AA
September 16, 2013

REPORT

CONTENTS

1	INTRODUCTION	1
2	SITE DESCRIPTION AND GEOLOGY	1
3	COMPILED SUBSURFACE CONDITIONS BASED ON PREVIOUS INVESTIGATIONS	2
4	CURRENT INVESTIGATION	2
4.1	Current Investigation Procedures	2
4.2	Subsurface Conditions	3
4.2.1	Asphalt	4
4.2.2	Embankment Fill	4
4.2.3	Lower Fill	4
4.2.4	Silty Sand to Sandy Silt Till	5
4.2.5	Granular (non-till) Overburden	6
4.2.6	Groundwater Conditions	6

Drawings

Drawing 1: Site Plan

Drawing 2: Borehole Location Plan

Drawing 3: Soil Strata

Appendices

Appendix A: Record of Borehole Sheets

Appendix B: Laboratory Test Results

Appendix C: Site Photographs

Appendix D: Explanation of Terms Used in Report

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
REHABILITATION OF HIGHWAY 400 / COUNTY ROAD 23 OVERPASS,
TOWNSHIP OF COLDWATER, ONTARIO,
G.W.P. 2190-10-02, SITE NO. 30/455-1/2**

1 INTRODUCTION

The existing bridges, which carry Highway 400 North Bound and South Bound lanes over County Road 23, will be rehabilitated at the site. Coffey was retained by McCormick Rankin Corporation (MRC) to carry out a desktop study using existing borehole information at the site for the proposed bridge rehabilitation, and a preliminary foundation investigation at the site (MTO Site No. 30-455-1/2) for the proposed bridge widening. It should be noted that the original project scope included rehabilitation and widening of the bridge. After completion of the field investigations, the scope was changed to rehabilitation only. However the investigation data provided in the following sections include that for widening for information.

The purpose of the current investigations was to obtain information about the subsurface conditions at the widening location (toward median) by means of boreholes, and to determine the engineering characteristics of the subsurface soils by means of field and laboratory tests.

The findings of the previous and current investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

The site is located at the intersection of Highway 400 and County Road 23 in Coldwater, Ontario. The topography at the site can be described as gently rolling.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984 edition, the project site is located within the Physiographic Region known as the Simcoe Uplands at the confluence with Carden Plain and Georgian Bay Fringe. The Simcoe Uplands comprises a series of broad curved ridges, separated by steep-sided, flat-floored valleys.

Previous investigations carried out in 1959 and 1974 by Geocon and in 1981 by MTO in the vicinity revealed the presence of a post-glacial clay plain underlain by sand and gravel, then granitic metamorphic gneiss bedrock. At or near the bridge site however the clay plain was not encountered and the site was found to be underlain by granular overburden. These deposits were found to vary from basically granular glacial tills to unstratified or stratified deposits of granular material, the difference being dependant on the part played by meltwater in their deposition. It is of interest to note that the 1959 Report mentions the presence of a gravel pit and an adjacent pond in the general area; pond elevation being at about 648 ft. (El. 197.5 m). However, the construction of structures and permanent drainage measures appear to have depressed the groundwater table.

3 COMPILED SUBSURFACE CONDITIONS BASED ON PREVIOUS INVESTIGATIONS

Based on the available information (GEOCRE 31D-218, 31D-294, GA drawings, MTO bridge inspection reports), it is assumed that Co. Rd 23 interchange overpass Highway 400 NBL and SBL may be the current northbound and southbound County Road 23/Highway 400 Overpasses. In 1974, GEOCON carried out two site investigations consisting of eight boreholes (i.e. Boreholes 1, 2, 3, 4, 5, and 6 for Site 30-455, NBL, and Boreholes 5 and 6 for Site 30-455B, SBL) for the existing bridge structures. Exact locations of the boreholes cannot be defined since borehole location plan were not available in MTO GEOCRE information system. Record of borehole sheets and laboratory testing results are included in Appendix A and B.

In general, the bridge sites are underlain by a dense to very dense granular soil deposit. Shallow veneer of topsoil and loose to compact granular soil (up to 1.8 m-6 feet) was found near the ground surface at the time of investigation. Cobbles and boulders were also noted within the overburden.

Groundwater condition at the time of investigation was noted close to the ground surface (north abutment location) and 6 to 8 feet (about 1.8 to 2.4 m) below the ground surface (south abutment location) at the NBL overpass location. Groundwater was encountered at 5 to 10 feet (about 1.5 to 3.0 m) below the grade at the time of investigation at the SBL overpass location.

4 CURRENT INVESTIGATION

4.1 Current Investigation Procedures

The fieldwork for the proposed bridge rehabilitation (focused on the widening option) was performed during the period of February 12-15 and March 4 (Borehole 1), 2013. The fieldwork consisted of drilling and sampling of eight boreholes (Boreholes 1 through 8). The following table summarizes the borehole locations and drilling depths.

Table 3.1
Borehole Locations and Drilling Depths

Borehole No.	Location	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
1	NBL	9.4	-
2	NBL	7.8	-
3	NBL	7.3	Yes
4	NBL	4.1	-
5	SBL	7.8	-
6	SBL	8.2	-
7	SBL	4.7	-
8	SBL	6.8	-

Marathon Drilling of Ottawa, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Geotechnical Engineer from Coffey. The boreholes were advanced using a CME 55 track mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. The boreholes were advanced using three different methods (i.e. continuous flight hollow-stem augers, wash boring in the overburden and coring) depending on the ground conditions.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey silts).

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in Borehole 3 to enable groundwater level monitoring in the borehole over a prolonged period of time without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures. The piezometer in Borehole 3 was not decommissioned as it will be useful during construction to monitor the groundwater table. It should however be decommissioned at that time.

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

The soil samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme consisting of natural moisture content tests and grain size analyses was performed on selected representative soil samples. The results of laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4.2 Subsurface Conditions

The subsurface conditions were explored by means of eight boreholes. Details of subsurface conditions encountered at each borehole location including the results of in-situ testing, groundwater observations in the open boreholes and laboratory test results, are presented on the Record of Borehole sheets in the Appendix B. Explanation of Terms Used in the Report is presented in Appendix D.

Four of the boreholes were advanced from the upper road (i.e. Highway 400) level at elevations ranging from 203.9 to 198.0 m and contacted embankment fill to depths of 6.0 to 7.4 m or to elevations ranging between 197.2 and 192.0 m.

The remaining four boreholes were advanced from the lower (Country Road 23) level, from El. 195.7 to 192.8 m and these boreholes contacted fill to depths 1.5 – 2.2 m below the ground surface or to El. 194.0 – 191.0 m. This is probably the result of construction activities that took place when the existing structures were first built.

Underlying the fill materials, the native soils were found to be granular in nature, ranging from glacial tills (silty sand to sandy silt, but typically silty sand till) to stratified and unstratified (deposited by glacial meltwaters) granular soils. The composition of these latter soils ranged from sandy silt/silty sand, sand with variable gravel content to basically cobbles and boulders within a sandy matrix. The tills and other granular deposits appeared to contain an unusually high cobble & boulder content, judging from the difficulties encountered in advancing the boreholes during augering and, in some instances, the necessity for coring.

It should be pointed out that because the fill is similar in nature to the underlying granular deposits, in many cases it was found that distinguishing the two from each other was difficult based solely upon split-spoon samples.

The Record of Borehole Sheets and sections indicate the subsurface conditions only at the borehole locations. Note that the material boundaries indicated on the logs are approximate and based on visual observations. These boundaries typically represent a transition from one material type to another and should not be regarded as an exact plane of geological change. It should be pointed out that the subsurface conditions may vary across this site.

The following paragraphs summarize the surface condition encountered in the boreholes.

4.2.1 Asphalt

Boreholes 1, 5 and 6 were drilled from the paved surface of Highway 400 and contacted 155 to 300 mm thick asphaltic concrete. Borehole 2 was also drilled from the paved surface of Highway 400 but from the top of the approach slab and contacted 110 mm of Asphaltic concrete, underlain by 450 mm thick concrete (i.e. approach slab).

4.2.2 Embankment Fill

Boreholes 1, 2, 5 and 6, which were advanced from the top of Highway 400 embankment, contacted embankment fill to depths ranging between 6.0 and 7.4 m below the ground surface or to El. 197.2 – 195.3 m at Boreholes 5 and 6 at SBL and El. 192.3 - 192.0 m at Boreholes 1 and 2 at the NBL.

The fill appears to have been derived from indigenous granular overburden and while its composition ranges from sandy silt to sand & gravel. In general, it consists of sand with some silt and gravel. The grain-size distribution of seven samples from the deposit is presented in an envelope form in Figure B-1 in Appendix B.

These show the following grain-size distribution

Gravel:	2 – 45 %
Sand:	48 – 91 %
Silt and Clay:	7 – 44 %

The embankment fill and the overlying pavement fill (immediately underlying the asphalt) are non-cohesive, granular soils.

The N-values recorded in the fill in Boreholes 1 and 2 (i.e. NBL embankment) range from 11 to 39 blows/0.3 m (except for gravelly layer where N value exceeded 50 blows/0.3 m). These results indicate that the fill received a reasonably good degree of compaction when it was first constructed. In Boreholes 5 and 6 (i.e. SBL boreholes), the recorded N-values show a greater degree of variation, ranging from 3 to 86 blows/0.3 m. From the recorded results it can be surmised that the lower half of the embankment in Borehole 5 received little or no compaction as evidenced by N-values 3-11 blows/0.3 m, while the upper zones were compacted to a reasonable degree of compaction. It also appears that in Borehole 6, the upper and lower zones were compacted but the middle half was irregularly compacted (i.e. contains loose zones).

4.2.3 Lower Fill

Boreholes 3, 4, 7 and 8 were advanced from County Road 23 (lower) level, near the toe of the embankment from El. 195.7 and 195.1 m (Boreholes 7 and 8, SBL) and 193.2 and 192.8 m (Boreholes 3 and 4, NBL) and encountered fill extending to depths ranging from 1.5 to 2.2 m below the ground surface or to El. 194.0

- 191.0 m. These fills appear to be derived from the indigenous granular overburden and as such it is difficult to distinguish them from the underlying native soils and the interface elevations shown on the Record of Borehole Sheets should be considered approximate only. These fills are believed to be related to the foundation construction of the existing structures at the site.

The grain-size distribution curves of three representative samples from the materials are given in Figure B-2 in Appendix B. These indicate the following grain-size distribution

Gravel:	8 – 20 %
Sand:	56 – 65 %
Silt and Clay:	19 – 30 %

These materials are considered to be non-cohesive, granular soils.

Standard Penetration tests performed in these previous construction backfill materials yielded N-values which ranged from 4 to 53 blows/0.3 m, indicating a very loose to very dense condition.

4.2.4 Silty Sand to Sandy Silt Till

As was mentioned before, the native overburden soils contacted in the boreholes are basically of two types, based on their depositional history; namely, non-cohesive glacial tills and granular interglacial or washed glacial soils. In terms of composition, the glacial tills consist of a heterogeneous mixture of sand and silt with some gravel, while the interglacial and washed glacial soils consist of stratified (interglacial) or relatively homogeneous (washed glacial) granular soils. In many cases these native deposits are interlayered.

The glacial till was contacted in Boreholes 1, 4, 5, 7 and 8 (i.e. it was not contacted in Boreholes 2, 3 and 6 within the depths drilled, but will likely be contacted throughout at greater depths during the detail investigation).

In Boreholes 1, 4, 5 and 8, the glacial till was contacted immediately underlying fill at elevations ranging from 195.3 to 191.7 m, while in Borehole 7, the deposit was encountered at El. 190.8 m, underlying a layer of sand & gravel. In most cases, the glacial till extended to the remaining depth of the boreholes at El. 194.4 - 189.0 m where refusal to further augering was experienced on boulders/cobbles. In Borehole 1 the till was found to be underlain by a sand deposit at El. 189.5 m. It should be pointed out that the interglacial deposits were frequently found to be interbedded with the till.

The grain-size distribution of seven samples from the till is given in Figure B-3 (Appendix B) in an envelope form. The results are also shown on the Record of Borehole Sheets and are as follows:

Gravel:	5 – 16 %
Sand:	36 – 79 %
Silt and Clay:	14 – 49 %

The presence of cobbles & boulders was inferred during the drilling operations.

The deposit is considered to be a basically non-cohesive (granular) type of soil.

N-values recorded in the deposit are generally in excess of 50 blows/0.3 m and this indicates a very dense relative density, except for a compact zone (N-value of 16 blows/0.3 m) in the upper zone of the till (immediately underlying the embankment fill) in Borehole 5. While some of the N-values may be on the

high side due to the presence of oversize gravel and cobbles/boulders. Nevertheless, the denseness condition of the till can be described as generally very dense.

4.2.5 Granular (non-till) Overburden

Granular overburden, ranging composition for sandy silt to cobbles/boulders in a sand matrix, was contacted in Boreholes 1, 2, 3, 6 and 7. In addition, layers of such 'interglacial' or 'washed till' origin soils were noted interbedded with the till deposits. In Boreholes 2, 3, 6 and 7, these granular overburden soils were contacted immediately underlying the fill deposits at El. 197.2 and 191.0 m. Boreholes 1, 2, 3 and 6 were terminated in these deposits at El. 195.7 - 185.6 m, upon encountering refusal on inferred boulders.

As mentioned, the grain-size of these deposits were found to be highly variable, ranging from silty sand/sandy silt; sand with some gravel-sand & gravel to cobbles & boulders in a sand matrix. On several occasions coring was resorted to advance the borehole in coarse zones containing cobbles/boulders.

The grain-size distributions of two typical samples are presented in Figures B-4 and B-5 (Appendix B). Figure B-4 represents a finer soil consisting of silty fine sand with traces of gravel (10 % gravel, 53 % sand and 37 % silt size particles), while in Figure B-5, a more typical soil is shown, consisting of sand with some gravel and traces of silt (11 % gravel, 80 % sand and 9 % silt size particles).


These deposits are considered to be non-cohesive, granular soil types. Standard Penetration tests performed in these deposits gave N-values generally ranging from 24 to in excess of 100 blows/0.3 m. Even though some of the recorded values may be too high due to the presence of over-sized particles which tend to distort the actual penetration values during testing nevertheless, the relative density of these soils can be considered to be dense to very dense, with some compact zones.

4.2.6 Groundwater Conditions

Water levels in the open boreholes were observed during drilling and upon completion of each borehole. The observations are given on the individual Record of Borehole Sheets in Appendix A.

In general the soil became wet, holes caved-in (wet cave), and the colour of the soil changed from brown to grey between El. 192.5 and 190.0 m.

For and on behalf of Coffey


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




Zuhtu Ozden, P.Eng.
Senior Principal



Drawings

METRIC

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

CONT No. -
WP: 2190-10-02



SHEET

HWY 400 / COUNTY ROAD 23 INTERCHANGE
OVERPASS BRIDGES REHABILITATION
BOREHOLE LOCATION PLAN

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole & Cone

No.	ELEVATION	STATION	OFFSET
BH1	198.026	30+524	0.88m Lt C/L
BH2	199.894	30+566	0.72m Lt C/L
BH3	192.837	30+537	8.2m Lt C/L
BH4	193.159	30+580	10.4m Lt C/L
BH5	202.269	30+498	2.80m Rt C/L
BH6	203.901	30+539	0.39m Rt C/L
BH7	195.087	30+498	10.6m Rt C/L
BH8	195.728	30+519	8.27m Rt C/L

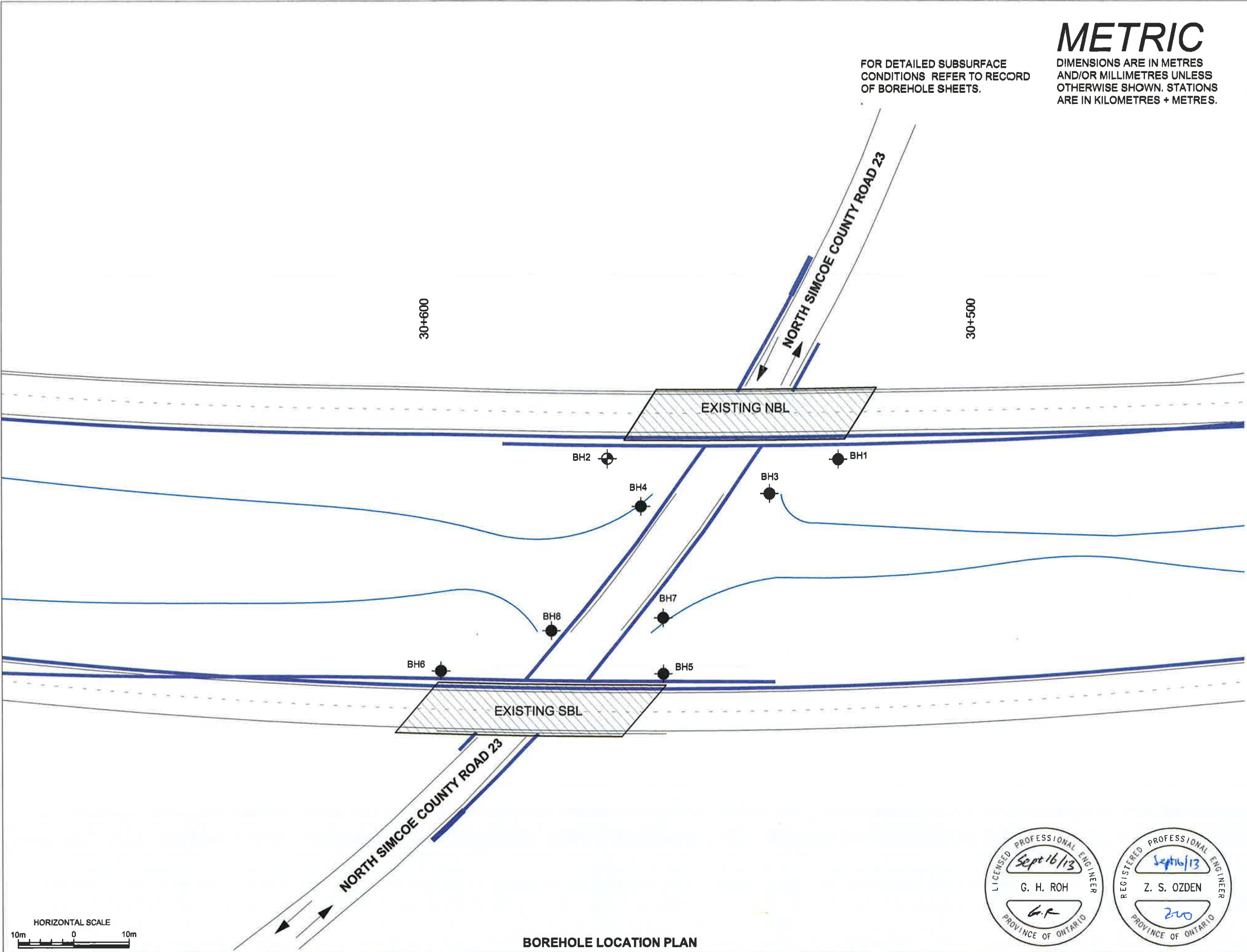
-NOTE-
The boundaries between soil strata have been established only
at borehole locations. Between boreholes the boundaries are
assumed from geological evidence.

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

REVISIONS	DATE	BY	DESCRIPTION



BOREHOLE LOCATION PLAN



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.
FOR DETAILED SUBSURFACE
CONDITIONS REFER TO RECORD
OF BOREHOLE SHEETS.

CONT No.

WP: 2190-10-02

HWY 400 NBL COUNTY ROAD 23
INTERCHANGE OVERPASS BRIDGE
REHABILITATION
BOREHOLE LOCATION PLAN
AND SOIL STRATA

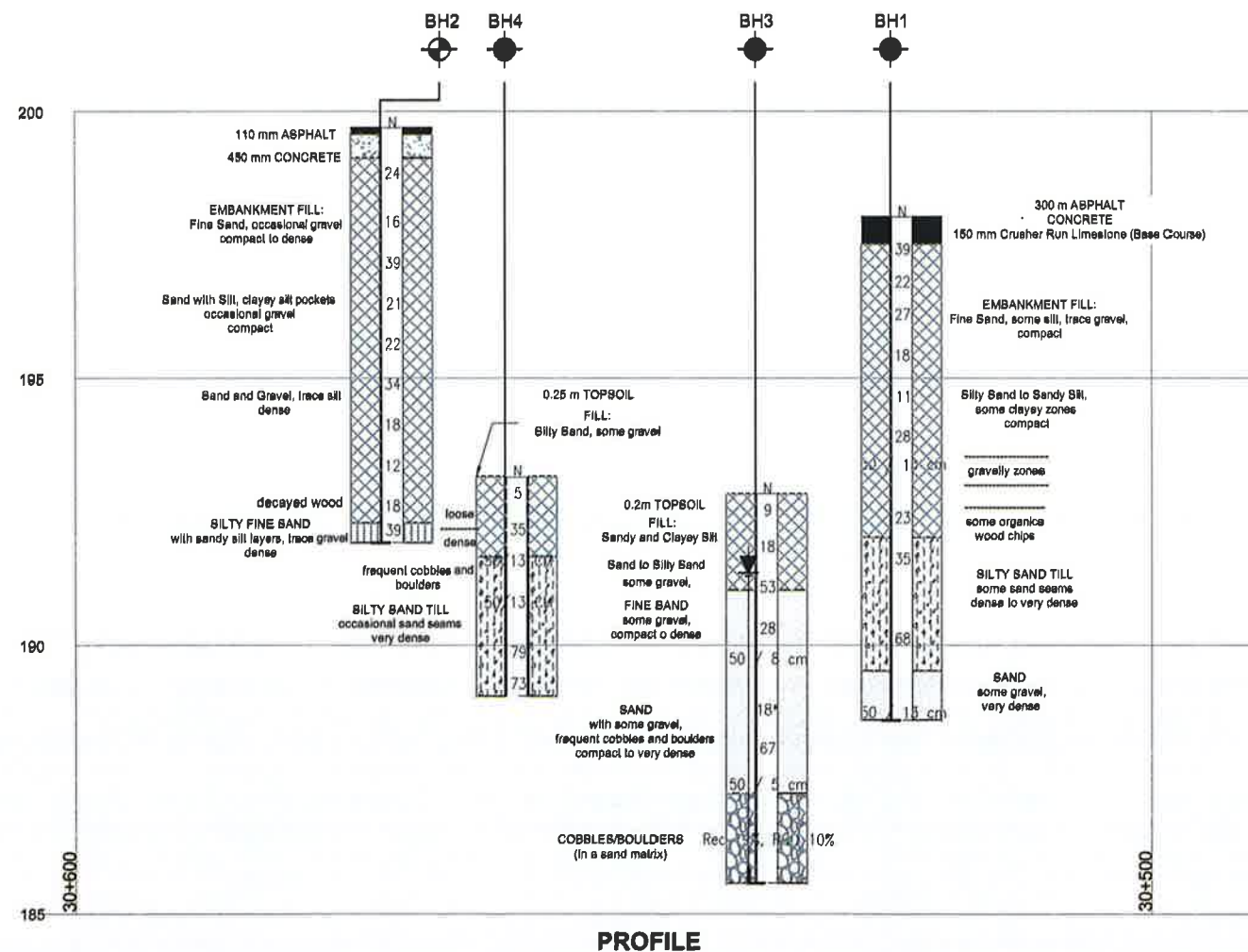
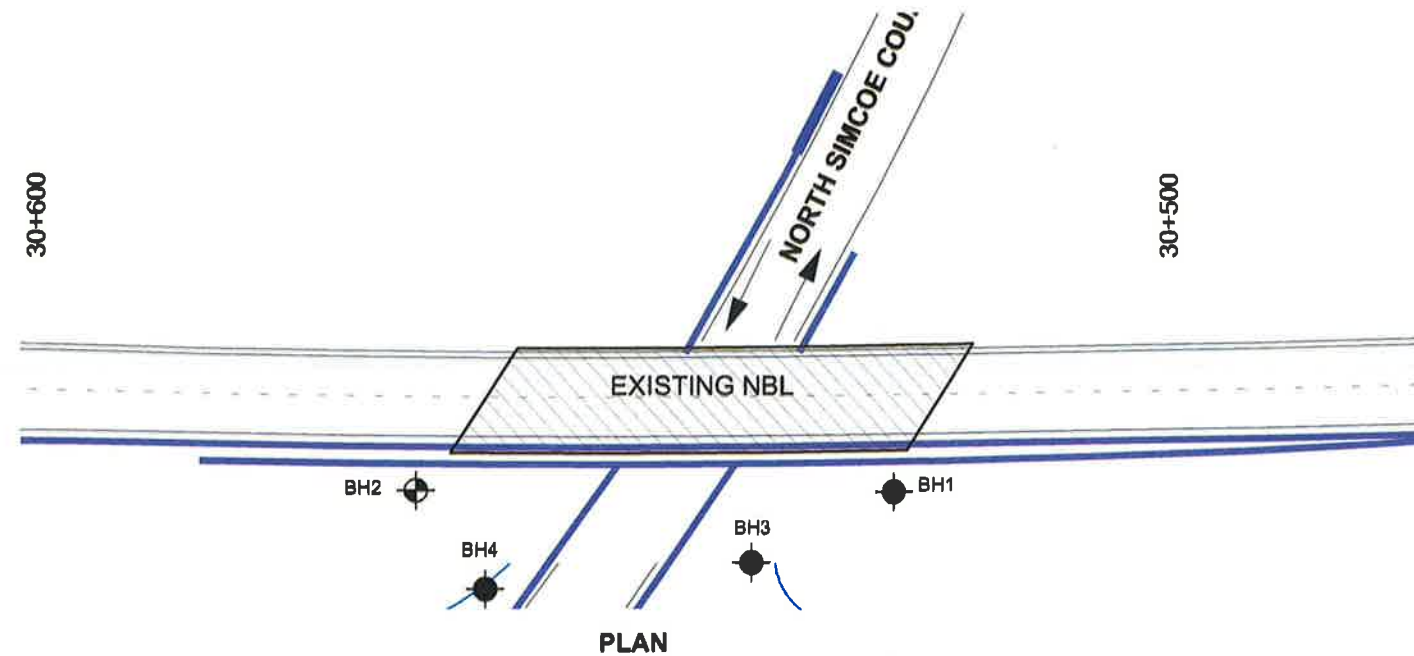


SHEET

coffey geotechnics
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KEY PLAN
N.T.S.



LEGEND

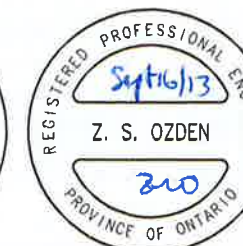
- Borehole
- Borehole & Cone
- N Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level in Piezometer
- Piezometer

No.	ELEVATION	STATION	OFFSET
BH1	198.026	30+524	0.88m Lt C/L
BH2	198.894	30+586	0.72m Lt C/L
BH3	192.837	30+537	8.2m Lt C/L
BH4	193.159	30+560	10.4m Lt C/L

-NOTE-

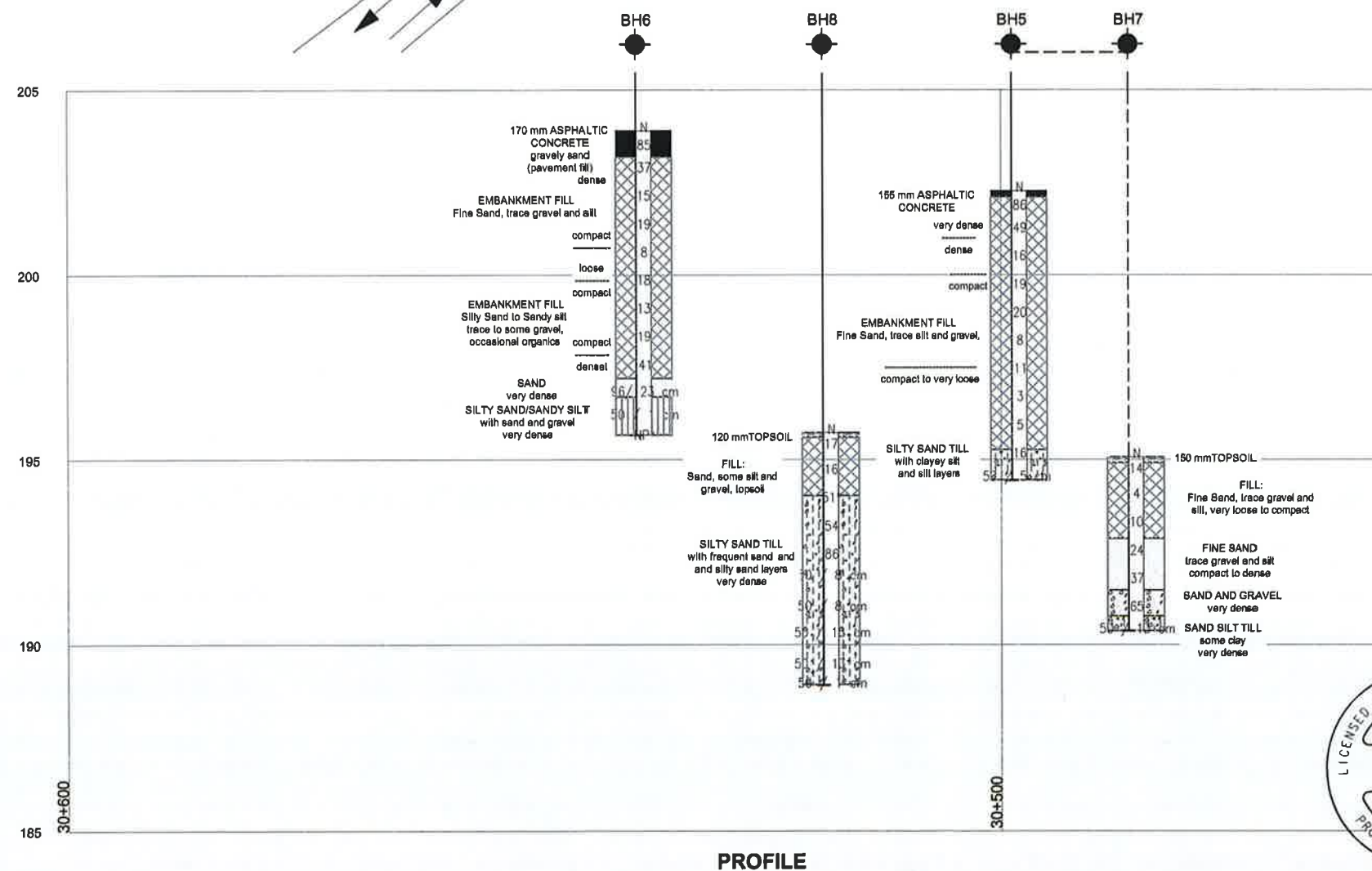
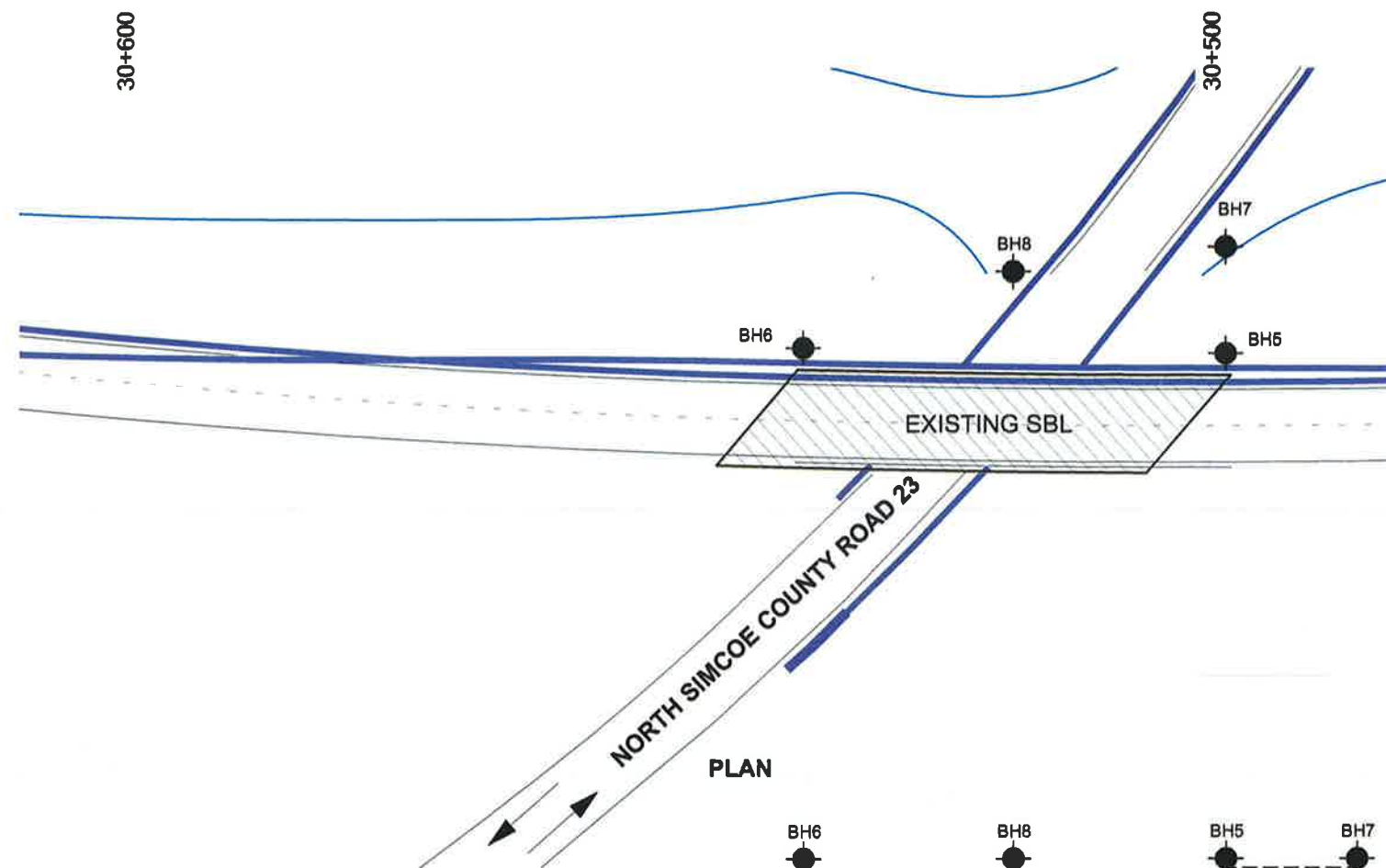
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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



REVISIONS	DATE	BY	DESCRIPTION

Geocres No - 31D-562			
TRANETOB20462AA			
SUBMD	CHECKED	DATE September, 2013	SITE
DRAWN	SBH	CHECKED G.R.	APPROVED ZO
DWG			2



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.
FOR DETAILED SUBSURFACE
CONDITIONS REFER TO RECORD
OF BOREHOLE SHEETS.

CONT No.

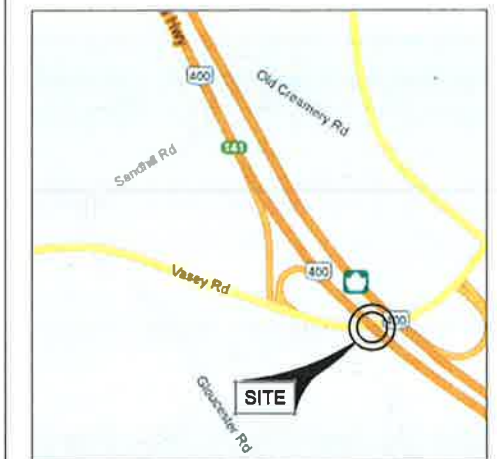
WP: 2190-10-02

HWY 400 SBL COUNTY ROAD 23
INTERCHANGE OVERPASS BRIDGE
REHABILITATION
BOREHOLE LOCATION PLAN
AND SOIL STRATA



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

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

No.	ELEVATION	STATION	OFFSET
BH5	202.269	30+498	2.80m Rt C/L
BH6	203.801	30+539	0.39m Rt C/L
BH7	195.067	30+498	10.65m Rt C/L
BH8	195.728	30+519	8.27m Rt C/L

-NOTE-

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No - 31D-582

TRANETOB20462AA				DIST
SUBMTD	CHECKED	DATE	September, 2013	SITE
DRAWN	SSH	CHECKED	G.R.	APPROVED
			ZO	DWG
				3



Appendix A

Record of Borehole Sheets

TRANETOB20462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

GWP G.W.P 2190-10-02 LOCATION (E 292099 807, N 4951764 734) ORIGINATED BY LG
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SSH
 DATUM Geodetic DATE 04/03/2013 04/03/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								
								20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	10 20 30	10 20 30	10 20 30		
								SHEAR STRENGTH (kPa)				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE								
								● POCKET PENETR × LAB VANE								
198.0	GROUND SURFACE						198									
0.0	300 mm ASPHALTIC CONCRETE															
197.5	150 mm Crusher Run Limestone (Base Course)		1	SS	39											
0.5			2	SS	22		197									
	EMBANKMENT FILL: Fine Sand, some silt, trace gravel some silt lenses brown, compact, damp		3	SS	27											
195.7			4	SS	18		196									
2.3			5	SS	11		195									
	EMBANKMENT FILL: Silty Sand to Sandy Silt, some clayey zones brown, compact, moist		6	SS	28		194									
			7	SS	50 / 13 cm		193									
			8	SS	23		192									
192.0			9	SS	35		191									
6.0			10	SS	68		190									
			11	SS	50 / 13 cm		189									
189.5																
6.5																
188.6																
9.4																
	End of Borehole Augers grinding, refusal to augering @ 9.4 m *Wet Cave @ 6.0 m upon completion, not stabilized															

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

TRANETOB20462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 3										1 OF 1		METRIC					
GWP <u>G.W.P 2190-10-02</u>		LOCATION <u>(E 292066 267, N 4951768 376)</u>				ORIGINATED BY <u>LG</u>											
DIST <u> </u> HWY <u>400</u>		BOREHOLE TYPE <u>Hollow Stem Auger, NQ Rock Coring</u>				COMPILED BY <u>SSH</u>											
DATUM <u>Geodetic</u>		DATE <u>14/02/2013</u> <u>14/02/2013</u>				CHECKED BY <u>ZO</u>											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL
192.8 0.0	GROUND SURFACE																
192.1 0.7	0.2 m TOPSOIL brown, loose, moist FILL: Sandy and Clayey Silt		1	SS	9												
	FILL: Sand to Silty Sand, some gravel brown, compact to dense, moist		2	SS	18												
191.0 1.8	FINE SAND some gravel, some silty zones (possible fill) brown, compact to very dense, wet		3	SS	53												
			4	SS	28												
190.0 2.8			5	SS	50/82												
	SAND with some gravel frequent cobbles and boulders brown, compact to very dense, wet		6	SS	18*												
			7	SS	67												
187.2 5.8			8	SS	50/52												
	COBBLES/BOULDERS (in a sand matrix)		9	NQ Rec 19% RC RQD 10%													
185.6 7.3	End of Borehole Water level @ 1.5 m on completion, not stabilized																

+ 3, X 3: Numbers refer to
Sensitivity

20
15 10 5
10 (%) STRAIN AT FAILURE

TRANETOB20462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 4

1 OF 1

METRIC

GWP G.W.P. 2190-10-02 LOCATION (E 292067 243, N 4951782 08) ORIGINATED BY LG
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SSH
 DATUM Geodetic DATE 15/02/2013 15/02/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100				
193.2 0.0	GROUND SURFACE 0.25 m TOPSOIL		1	SS	5		193									
	loose															
	FILL: Silty Sand, some gravel brown, moist	dense	2	SS	35		192									8 62 (30)
191.7 1.5	frequent cobbles and boulders		3	SS	50/13 cm											
	SILTY SAND TILL occasional sand seams very dense, moist		4	SS	50/13 cm		191									5 59 28 8
	brown															Refusal on auger @ 2.6 m on a boulder relocate 0.6 m and re drill
	grey		5	SS	79		190									
189.0 4.1	End of Borehole Refusal to further augering (probable boulder) no free-standing water in open borehole upon completion, not stabilized		6	SS	73											

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

TRANETOB20462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 5										1 OF 1		METRIC			
GWP <u>G.W.P 2190-10-02</u>		LOCATION <u>(E 292049 921, N 4951756 508)</u>				ORIGINATED BY <u>LG</u>									
DIST <u>HWY 400</u>		BOREHOLE TYPE <u>Hollow Stem Auger</u>				COMPILED BY <u>SSH</u>									
DATUM <u>Geodetic</u>		DATE <u>12/02/2013</u>		DATE <u>12/02/2013</u>		CHECKED BY <u>ZO</u>									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR × LAB VANE							
202.3	GROUND SURFACE						20	40	60	80	100				
202.6	155 mm ASPHALTIC CONCRETE														
0.2			1	SS	86										
	very dense														
	dense		2	SS	49										
	compact														
			3	SS	16										
	EMBANKMENT FILL: Fine Sand, trace silt and gravel brown, damp														
			4	SS	19										
			5	SS	20										
	compact to very loose														
			6	SS	8										
			7	SS	11										
			8	SS	3										
			9	SS	5										
195.3	SILTY SAND TILL		10	SS	16										
7.0	compact														
194.4	very dense		11	SS	50 / 5 cm										
7.8	with clayey silt and silt layers gray / brown, moist														
	End of Borehole Sampler bouncing and auger refusal @ 7.8 m Borehole dry and open on completion, not stabilized														

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

TRANETO820462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 6

1 OF 1

METRIC

GWP G.W.P. 2190-10-02 LOCATION (E 292020.185, N 4951783 741) ORIGINATED BY LG
 DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SSH
 DATUM Geodetic DATE 12/02/2013 12/02/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						WATER CONTENT (%)
								20 40 60 80 100						
203.9 0.0	GROUND SURFACE													
203.2 0.7	170 mm ASPHALTIC CONCRETE gravelly sand, brown, (pavement fill)		1	SS	85									
			2	SS	37		203						9 76 (15)	
	EMBANKMENT FILL Fine Sand, trace gravel and silt brown, damp		3	SS	15		202							
			4	SS	19		201							
			5	SS	8		200							
			6	SS	18		199						13 63 (24)	
199.6 4.3	EMBANKMENT FILL Silty Sand to Sandy Silt trace to some gravel, occasional organics greyish brown, moist		7	SS	13		198							
			8	SS	19		197							
			9	SS	41		196							
197.2 6.7	SAND brown, very dense, moist		10	SS	96/ 23 cm									
196.7 7.2	SILTY SAND / SANDY SILT with sand and gravel layers brown, very dense, moist		11	SS	50/ 1 cm									
195.7 8.2	End of Borehole sampler bouncing and auger refusal @ 8.2 m no free standing water observed on completion in open borehole, not stabilized		12	SS	NP								*NP, no penetration	

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

TRANETO20462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 7

1 of 1

METRIC

GWP G.W.P 2190-10-02 LOCATION (E 292056 714, N 4951764 171) ORIGINATED BY LG
DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SSH
DATUM Geodetic DATE 14/02/2013 14/02/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
195.1	GROUND SURFACE												
194.8	150 mm TOPSOIL		1	SS	14		195						
0.2													
	FILL: Fine Sand, trace gravel and silt brown, very loose to compact, moist		2	SS	4		194						
			3	SS	10								
192.9	FINE SAND trace gravel and silt brown, compact to dense		4	SS	24		193						
2.2			5	SS	37		192						
191.5	SAND AND GRAVEL brown, very dense, wet		6	SS	65		191						
3.6			7	SS	50/13 cm								
190.8	SANDY SILT TILL some clay brown, very dense, wet												
4.3													
190.4													
4.7	End of Borehole Auger refusal at 4.7 m wet cave at 3.6 m upon completion, not stabilized												

+³ . ×³: Numbers refer to
Sensitivity

20
15 10 6
(%) STRAIN AT FAILURE

TRANETOB20462AA: Highway 400, County Rd 23 Overpass, Coldwater

RECORD OF BOREHOLE No 8

1 OF 1

METRIC

GWP G.W.P 2190-10-02 LOCATION (E 292040.036, N 4951775.862) ORIGINATED BY LG
DIST HWY 400 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SSH
DATUM Geodetic DATE 15/02/2013 15/02/2013 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)									WATER CONTENT (%)		
								○ UNCONFINED 20 40 60 80 100	+ FIELD VANE	● POCKET PENETR 20 40 60 80 100	× LAB VANE								
195.7 196.6 0.1	GROUND SURFACE 120 mm TOPSOIL brown, moist to damp		1	SS	17														
	FILL: Sand, some silt and gravel, topsoil brown, compact, moist		2	SS	16														
194.0 1.7			3	SS	51**												16 65 (19) refusal @ 2.1 m (probable boulder). Relocate and re-auger borehole ***N- value may not be reliable due to large gravel		
			4	SS	54														
	SILTY SAND TILL with frequent sand and silty sand layers very dense, wet		5	SS	86														
			6	SS	50 / 8 cm														
			7	SS	50 / 8 cm												16 63 (21)		
			8	SS	50 / 13 cm														
			9	SS	50 / 13 cm												14 67 11 8		
188.9 6.8	End of Borehole sampler bouncing and auger refusal @ 6.8 m. *water level @ 5.5 m (not stabilized)		10	SS	50 / 13 cm														

+³, X³; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

NBL

W.P. 906-66-09

GEOCON

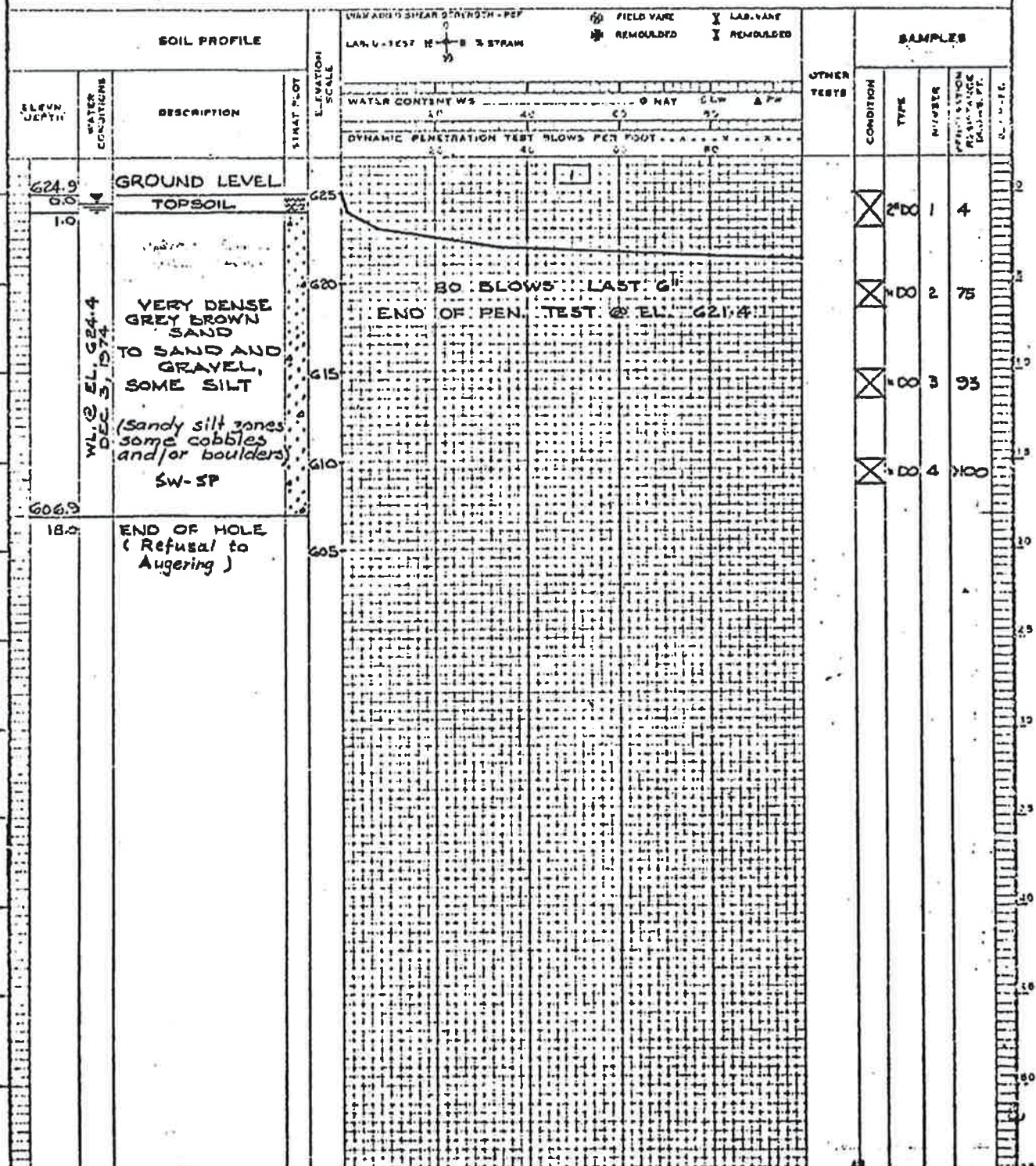
OFFICE REPORT ON SOIL EXPLORATION

BORING DATE DEC. 3, 1974 REPORT DATE DEC. 9, 1974 DATUM GEODETIC CARING
 SAMPLER HAMMER WT 140 LBS DRCP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY) COMPILED BY N.L. CHECKED BY T.H.

SAMPLE CONDITION
☒ DISTURBED
☐ FAIR
☐ GOOD
☐ BEST

SAMPLE TYPES
 AS - AUGER SAMPLE
 ST - SLOTTED TUBE
 WS - WANTED SAMPLE
 DO - DRIVE-FOOT VALVE
 DF - DRIVE-FOOT VALVE
 CS - CHUCK SAMPLE
 FS - FOIL SAMPLE
 SO - SLEEVE OPEN
 SF - SLEEVE-FOOT VALVE
 TO - THIN WALL VALVE OPEN
 RC - ROCK CORE

ABBREVIATIONS
 V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 CC - TRIAXIAL CONSOLIDATED UNDRAINED
 CU - TRIAXIAL UNCONFINED
 S - TRIAXIAL DRAINED
 T - TEST UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



W.P. 906-66-09

GEOCON

8

OFFICE REPORT ON SOIL EXPLORATION

BORING # 2 DATUM GEODETIC CASING BY 1
 BORING DATE NOV. 27, 1974 REPORT DATE DEC. 2, 1974 COMPILED BY AEL CHECKED BY 1/70
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

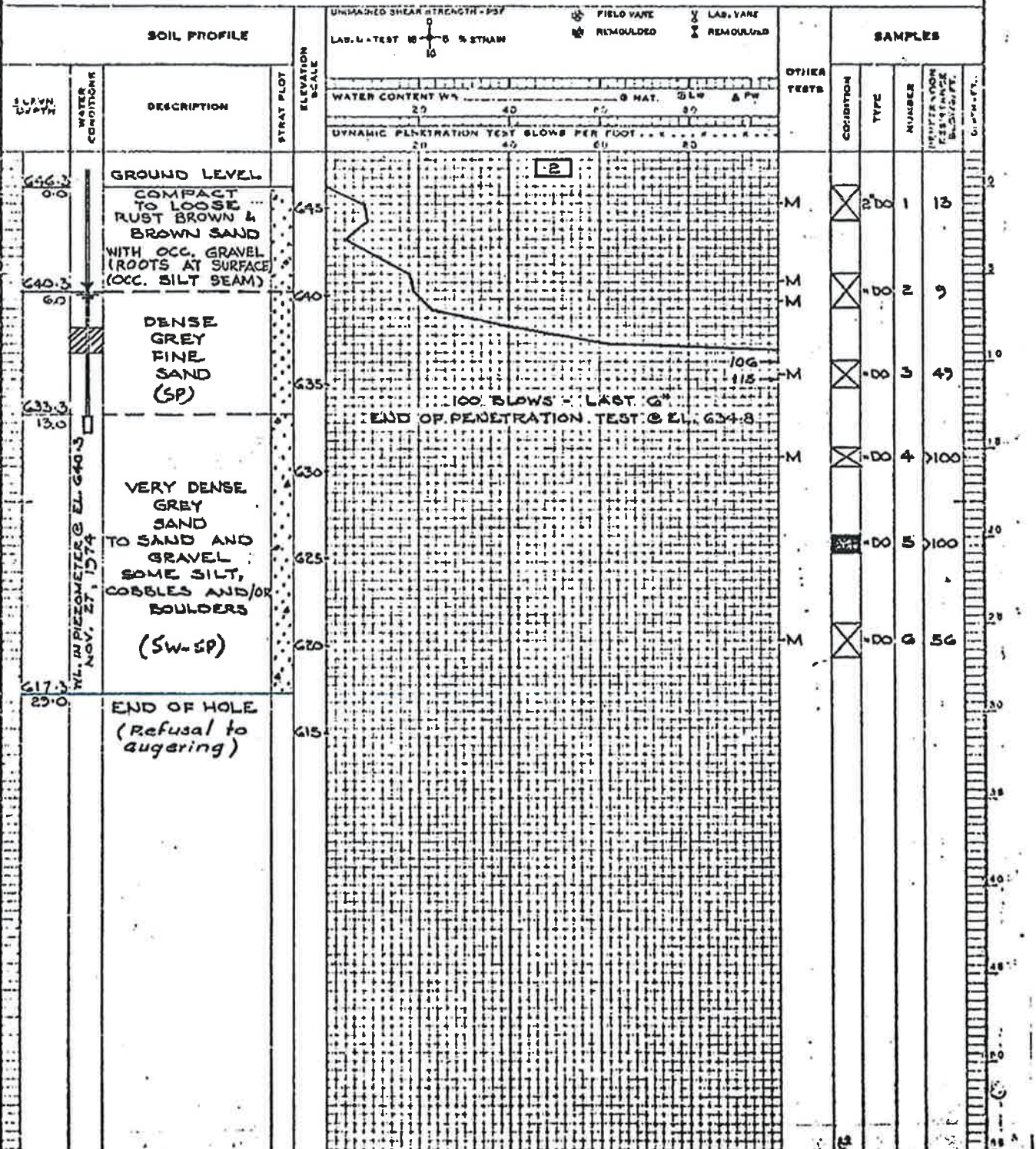
☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOSE

SAMPLER TYPES

A.S. AUGER SAMPLE
 S.T. SLOTTED TUBE
 W.S. WASHED SAMPLE
 D.O. DRIVE-OPEN
 D.P. DRIVE-FOOT VALVE
 C.S. CHUNK SAMPLE
 F.C. FOIL SAMPLE
 S.O. SLEEVE-OPEN
 S.F. SLEEVE-FOOT VALVE
 T.O. THIN WALLED OPEN
 R.C. ROCK CORE

ABBREVIATIONS

V. IN-SITU VANE TEST
 M. MECHANICAL ANALYSIS
 U. UNCONFINED COMPRESSION
 CC. TRIAXIAL CONSOLIDATED UN-DRAINED
 O. TRIAXIAL UN-DRAINED
 S. TRIAXIAL DRAINED
 W.T. UNIT WEIGHT
 N. PERMEABILITY
 C. CONSOLIDATION
 WL. WATER LEVEL IN CASING
 WT. WATER TABLE IN SOIL



NBL

11

W.P. 906-66-09

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

BORING NO. 5 DATUM GEODETIC CASING 3/4"
 REPORT DATE DEC. 2, 1974 COMPILED BY AEL CHECKED BY 3/4"
 SAMPLER HEIGHT WT. .140 LBS. DR. 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

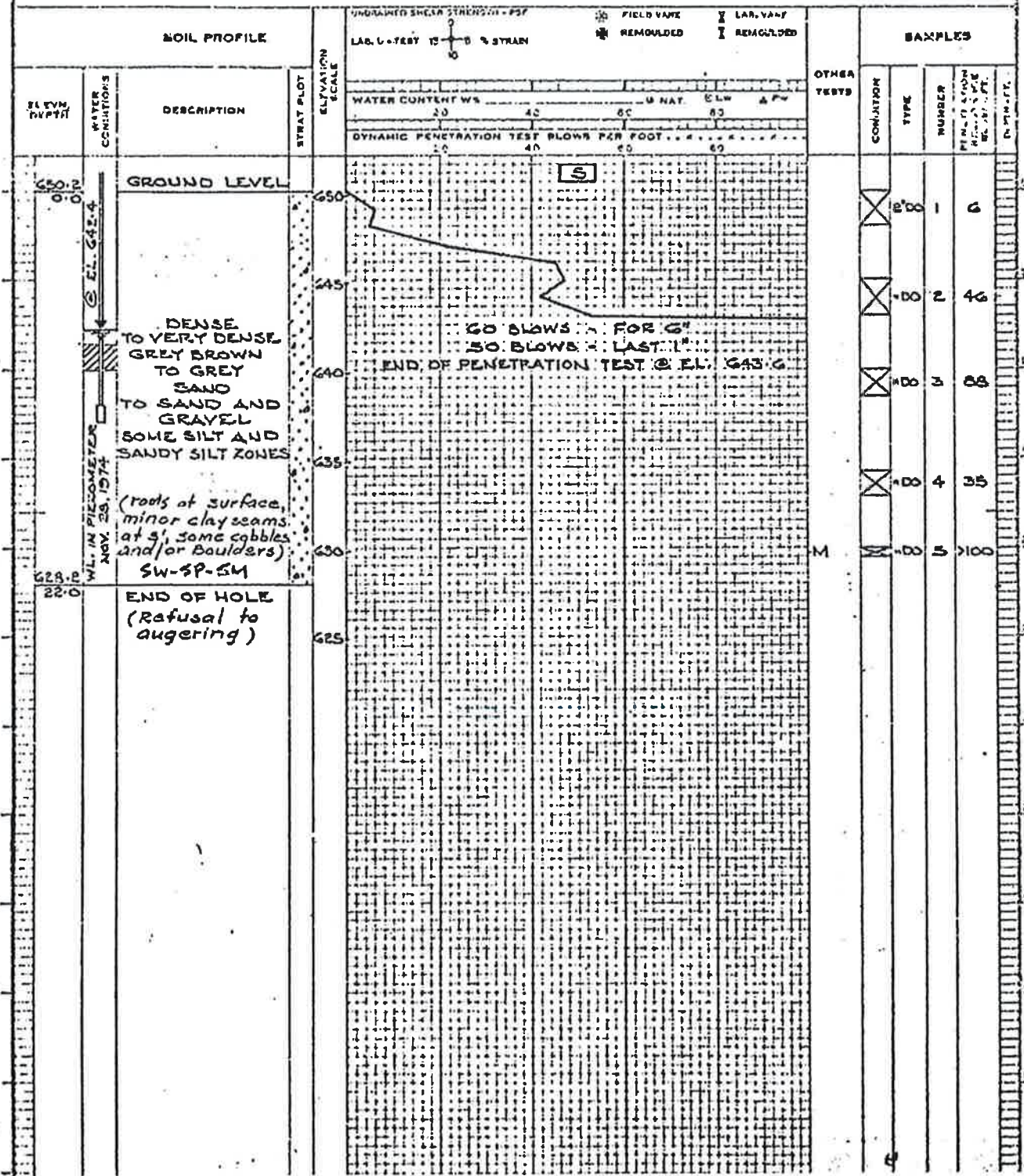
UNSTURBED
 FAIR
 GOOD
 EXCELLENT

SAMPLE TYPES

AS - AUGER SAMPLE
 ST - SLIGHTLY TUBED
 WS - WASHED SAMPLE
 DO - DRIVE-OPEN
 DF - DRIVE FOOT VALVE
 CS - CHUCK SAMPLE
 FS - SOIL SAMPLE
 SO - SLEEVE-OPEN
 EF - SLEEVE FOOT VALVE
 TO - THIN WALLED OPEN
 RC - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 GC - TRIAXIAL CONSOLIDATED UNDRAINED
 D - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINED
 W - WET UNIT WEIGHT
 H - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



NBL

W.P. 906-66-09

GEOCON

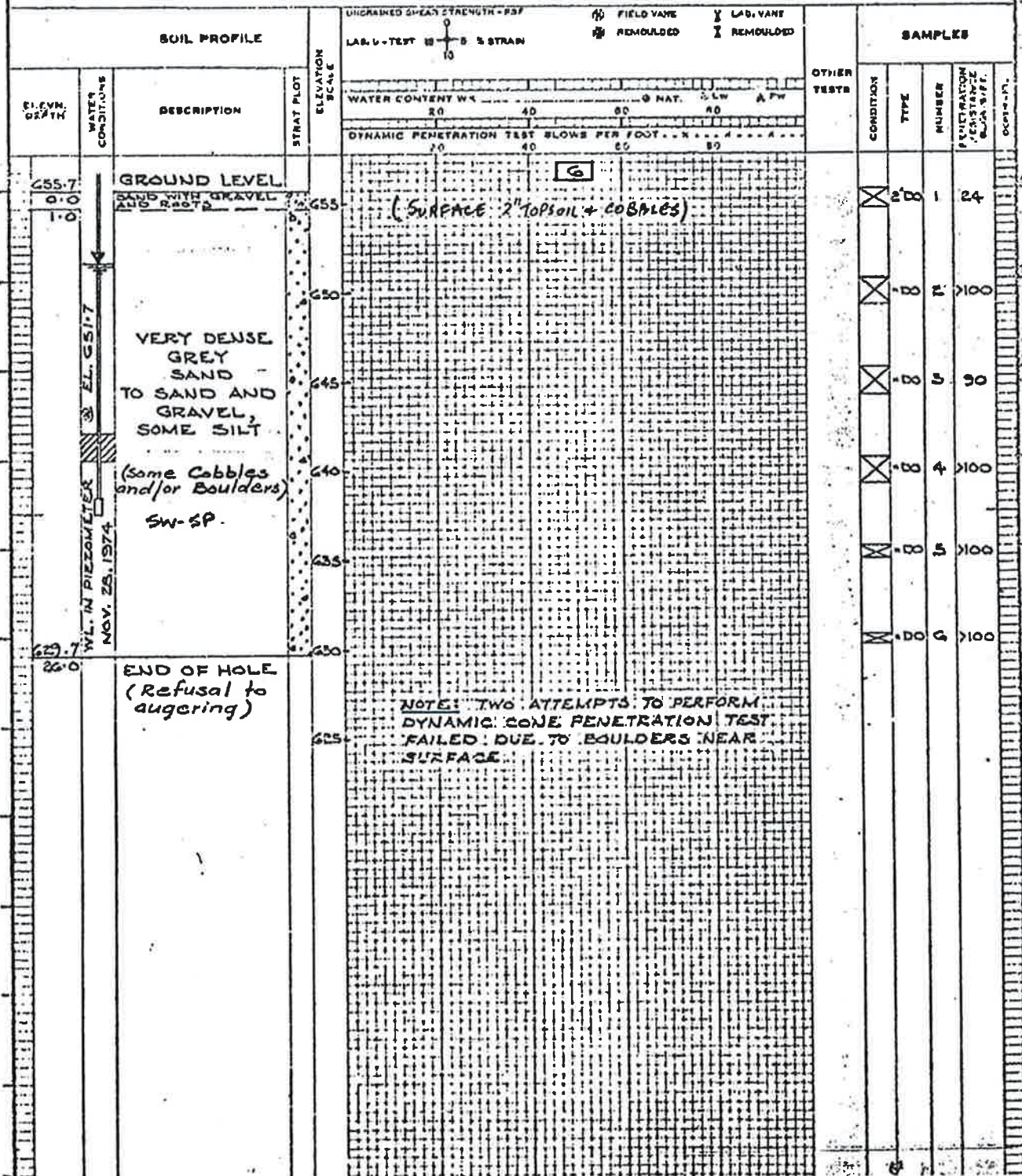
OFFICE REPORT ON SOIL EXPLORATION

BORING # 6 DAYUM GEODETIC CASING ---
 BORING DATE NOV. 23, 1974 REPORT DATE DEC. 2, 1974 COMPILED BY AEI CHECKED BY ---
 SAMPLER HAMMER WT. 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4500 IN. LBS. ENERGY)

SAMPLE CONDITION
☒ DISTURBED
☐ FINE
☐ GOOD
☐ LOST

SAMPLE TYPES
 AS - AUGER SAMPLE
 ST - SLOTTED TUBE
 WS - WATHEID SAMPLE
 DO - DRIVE OPEN
 DF - DRIVE FOOT VALVE
 CS - CHURN SAMPLE
 FS - FOIL SAMPLE
 SO - SLEEVE OPEN
 SF - SLEEVE FOOT VALVE
 TO - THIN WALLED OPEN
 RC - ROCK CORE

ABBREVIATIONS
 V - IN SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 OC - TRIAXIAL CONSOLIDATED UNDRAINED
 O - TRIAXIAL UNDRAINED
 D - TRIAXIAL DRAINED
 Y - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOTCON

OFFICE REPORT ON SOIL EXPLORATION

WP 906-66-14 BORING # G DATE NOV. 23, 1974 REPORT DATE DEC. 2, 1974 DATUM GEODETIC CARING
 SAMP. HAMMER WT 140 LBS DROP 30 INCHES PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS ENERGY

SAMPLE CONDITION		SAMPLE TYPES		ABBREVIATIONS	
SI - UNDISTURBED	AS - AUGER SAMPLE	FS - SOIL SAMPLE	U - IN SITU VALUE	W - WET UNIT WEIGHT	
SI - UNDISTURBED	ST - SPLITTED TUBE	SP - SLEEVE FOOT VALVE	U - UNCONFINED COMPRESSION	P - PERMEABILITY	
SI - UNDISTURBED	WS - WASHED SAMPLE	TO - THIN WALL OPEN	U - UNCONFINED COMPRESSION	C - CONSOLIDATION	
SI - UNDISTURBED	DC - DRIVE FOOT VALVE	RC - ROCK CORE	U - UNCONFINED COMPRESSION	WL - WATER LEVEL IN CASING	
SI - UNDISTURBED	CS - CHURN SAMPLE		U - UNCONFINED COMPRESSION	WT - WATER TABLE IN SOIL	

SOIL PROFILE		UNPAVED - STRENGTH - PCF		FIELD VANE		LAB. VANE		OTHER TESTS		SAMPLES	
DEPTH	DESCRIPTION	LAB. TEST	FIELD TEST	REMOVED	REMOVED	REMOVED	REMOVED	REMOVED	REMOVED	REMOVED	REMOVED
655.7	GROUND LEVEL										
655.0	SAND WITH GRAVEL AND ROOTS										
650.0	(SURFACE 2" TOPSOIL + COBBLES)										
645.0	VERY DENSE GREY SAND TO SAND AND GRAVEL, SOME SILT										
640.0	(Some Cobbles and/or Boulders)										
635.0	SW-SP										
630.0											
625.0	END OF HOLE (Refusal to augering)										

NOTE: TWO ATTEMPTS TO PERFORM DYNAMIC CONE PENETRATION TEST FAILED DUE TO BOULDERS NEAR SURFACE.

GEOCON

SBL

OFFICE REPORT ON SOIL EXPLORATION

WP 906-66-14

BORING DATA NOV. 28, 1974

REPORT DATE DEC. 2, 1974

CASONIC

COMPILED BY AEL

CHECKED BY

BATH, LB. MAX. HIT: 140

LBS. DROP: 30

INCHES

PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY

SAMPLE CONDITION

UNDISTURBED
 NO. 10
 NO. 10

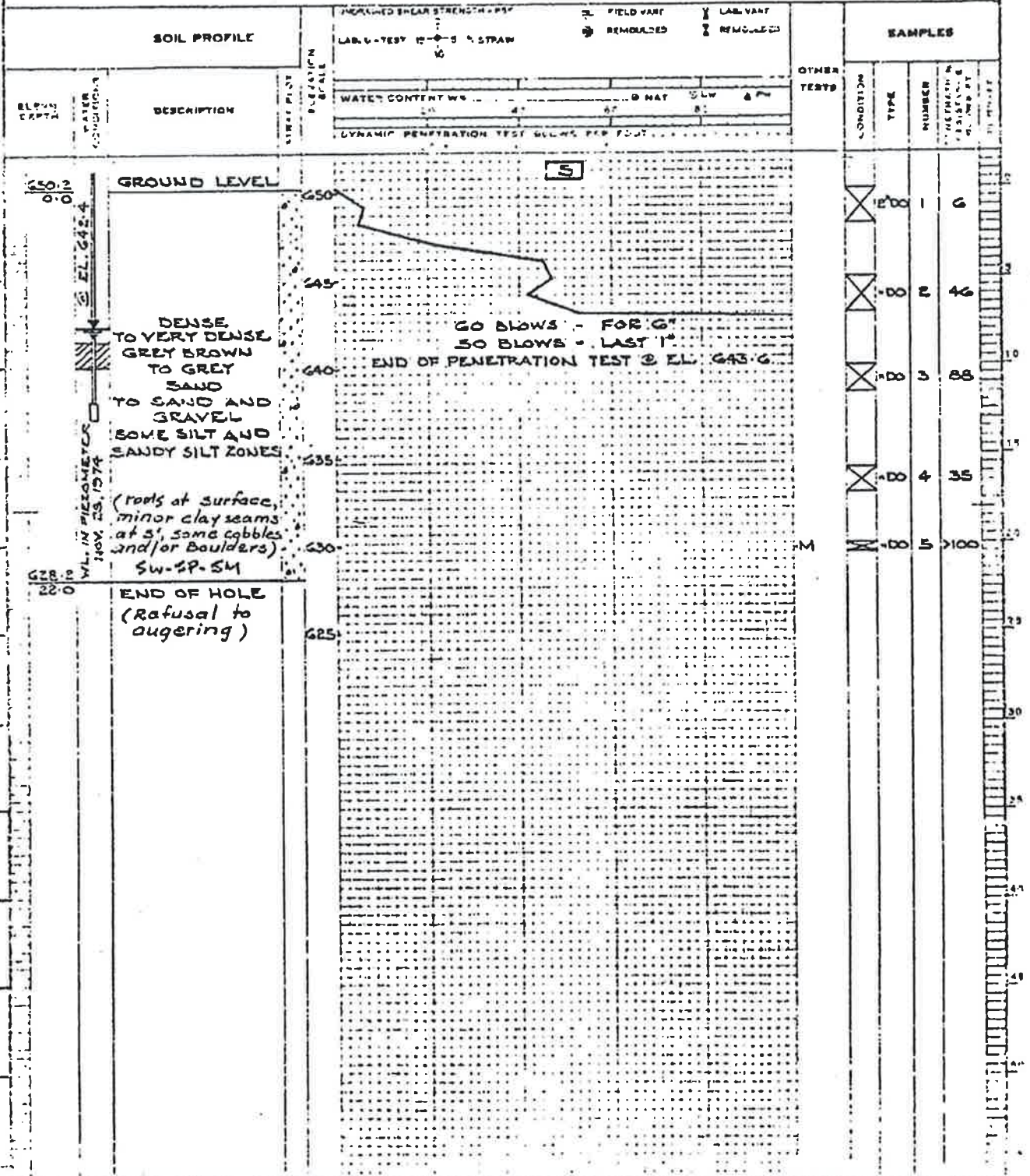
SAMPLE TYPES

AS - AUGER SAMPLE
 ST - SPLIT TUBE
 WS - WASHED SAMPLE
 OD - DRIVE OPEN
 OF - DRIVE, PILOT VALVE
 CS - CHURN SAMPLE

FS - FOIL SAMPLE
 SO - SLEEVE OPEN
 ST - SLEEVE, FOOT VALVE
 TO - THIN WALLED OPEN
 RC - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 OC - TRIAXIAL CONSOLIDATED UNDRAINED
 O - TRIAXIAL UNDRAINED
 S - TRIAXIAL DRAINED
 W - WET UNIT WEIGHT
 E - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

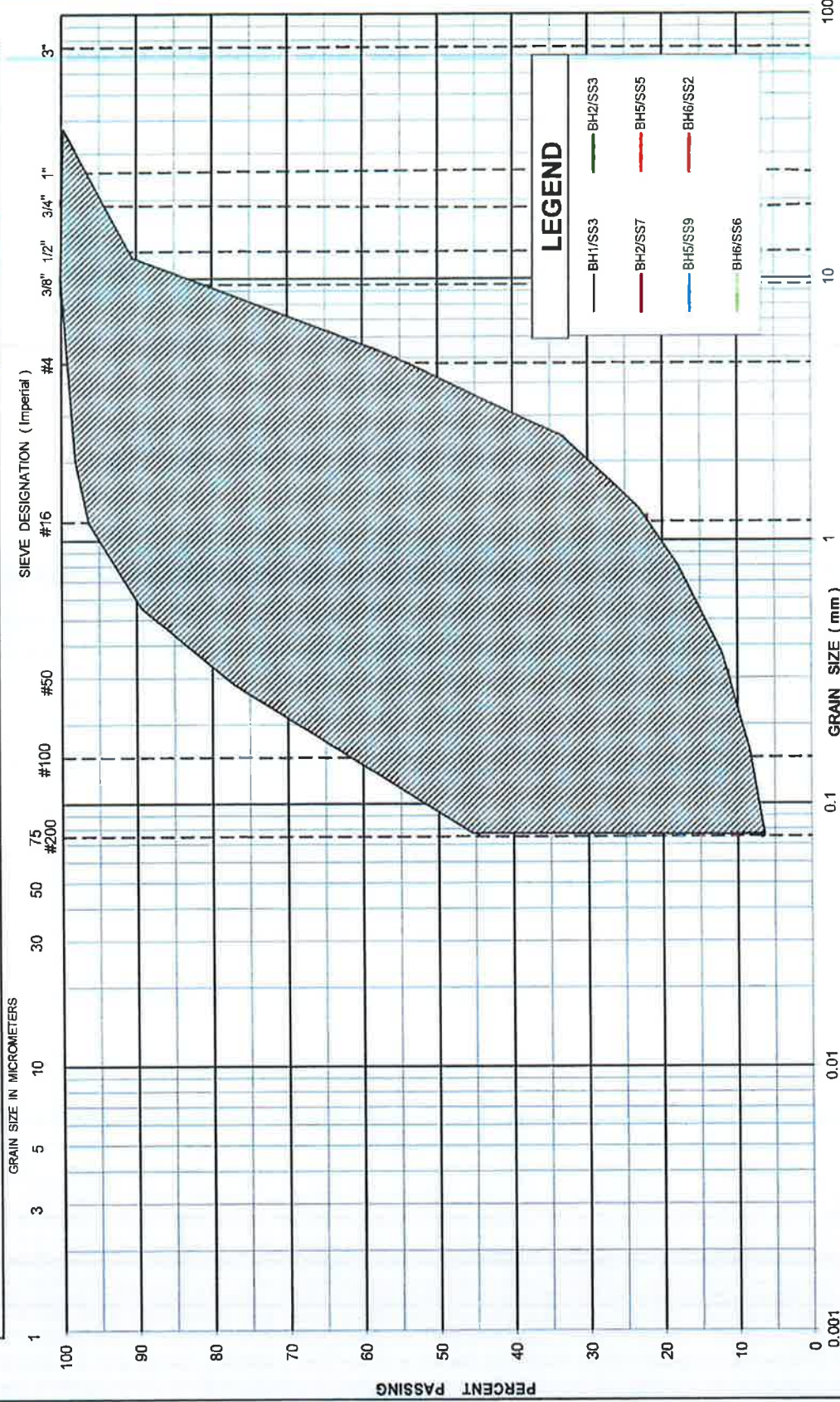


Appendix B

Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS		Fine	Medium	Coarse	Fine	Coarse	Coarse

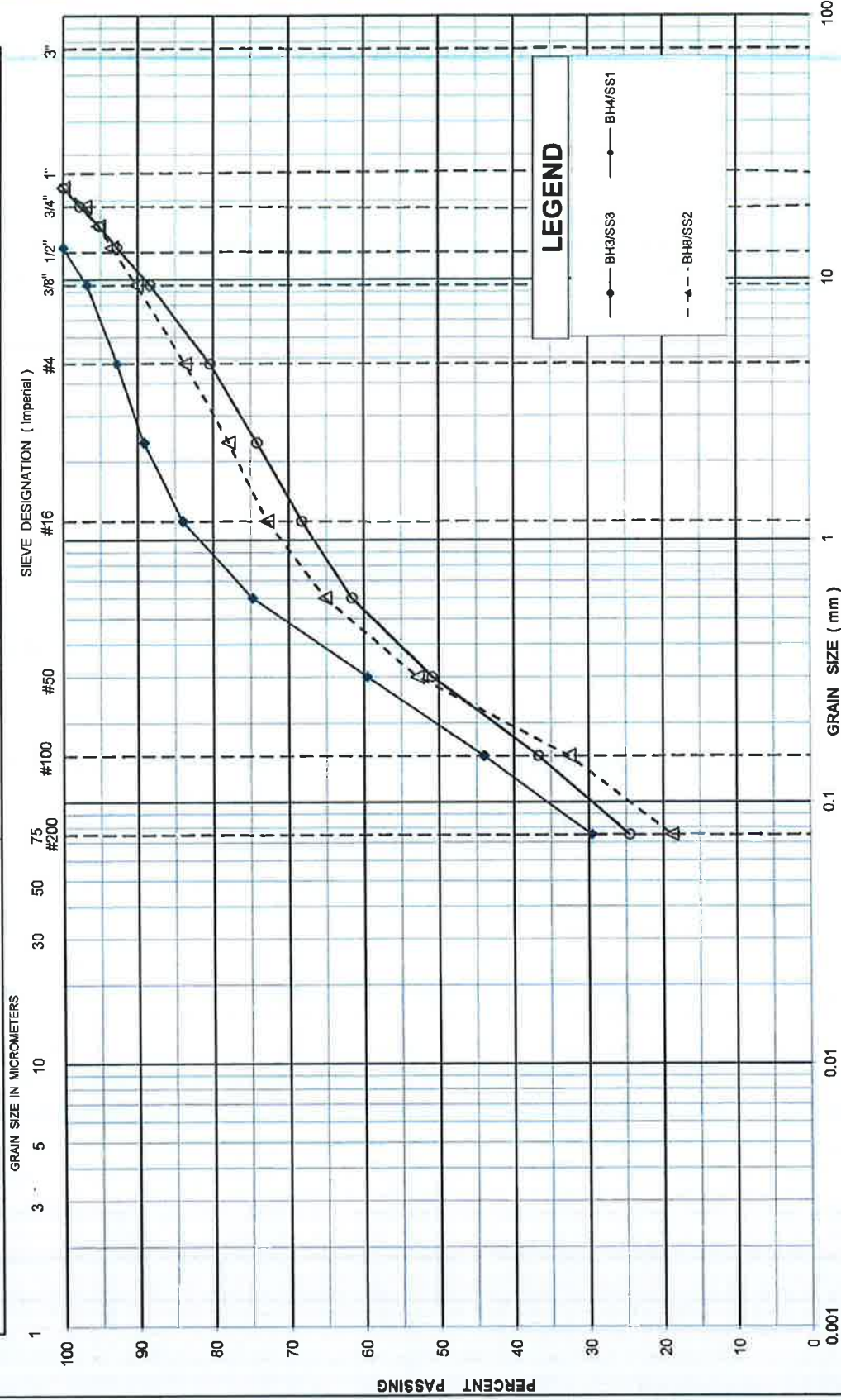


Figure: B-2

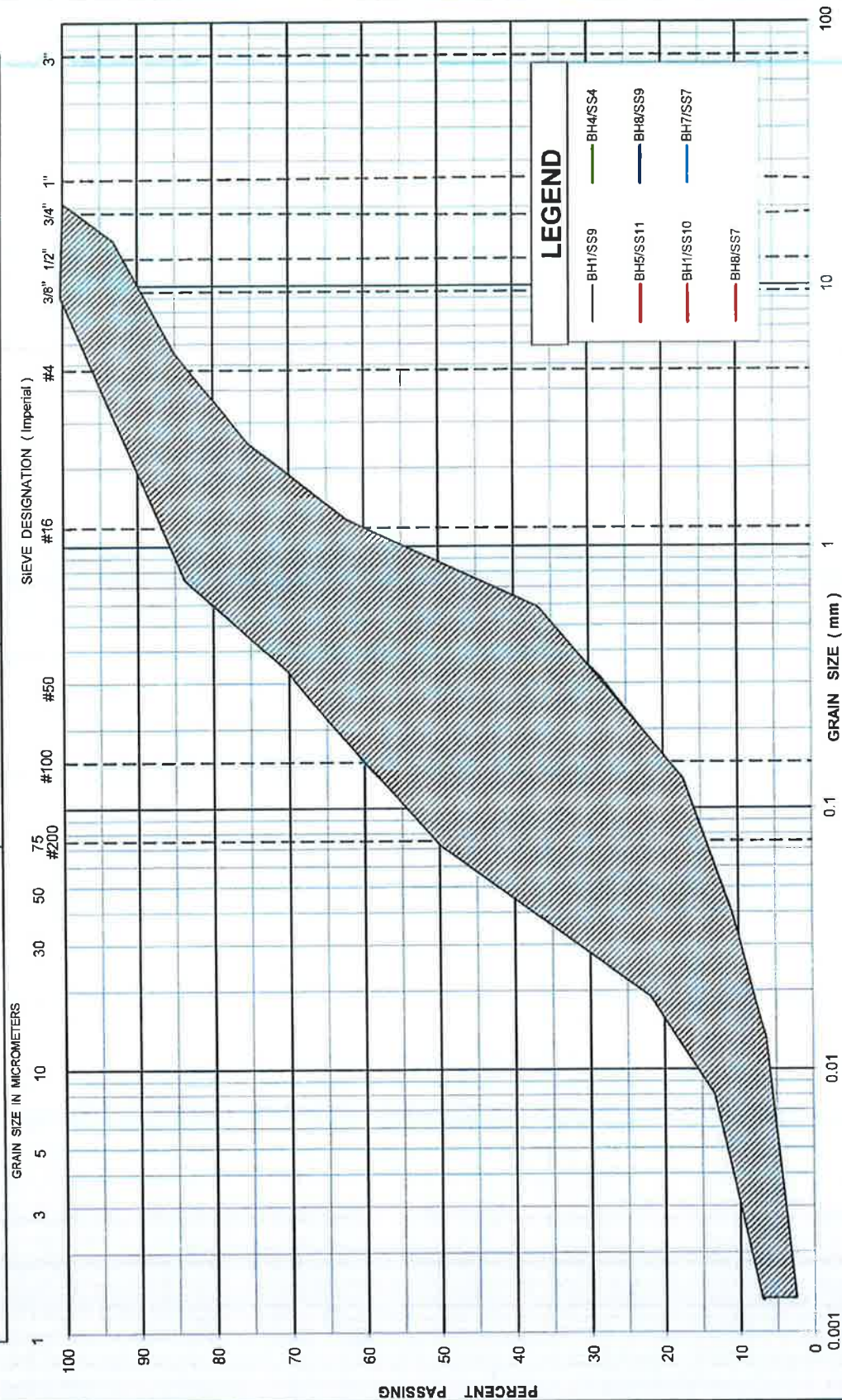
GRAIN SIZE DISTRIBUTION

FILL: Sand, some silt & gravel

PROJECT #: TRANETOB20462AA
DATE: MARCH, 2013

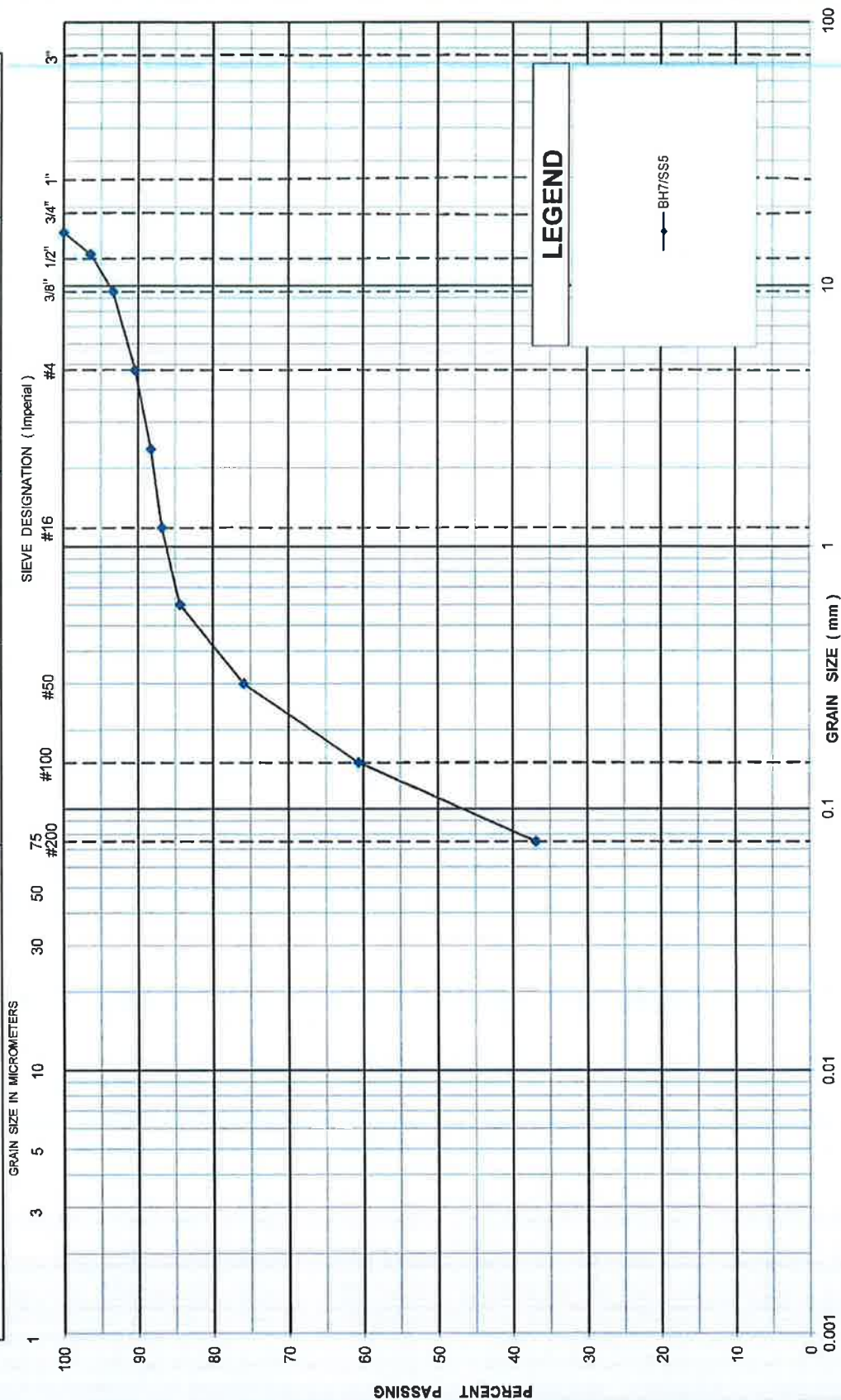
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
			Fine	Medium	Coarse	Fine	Coarse	

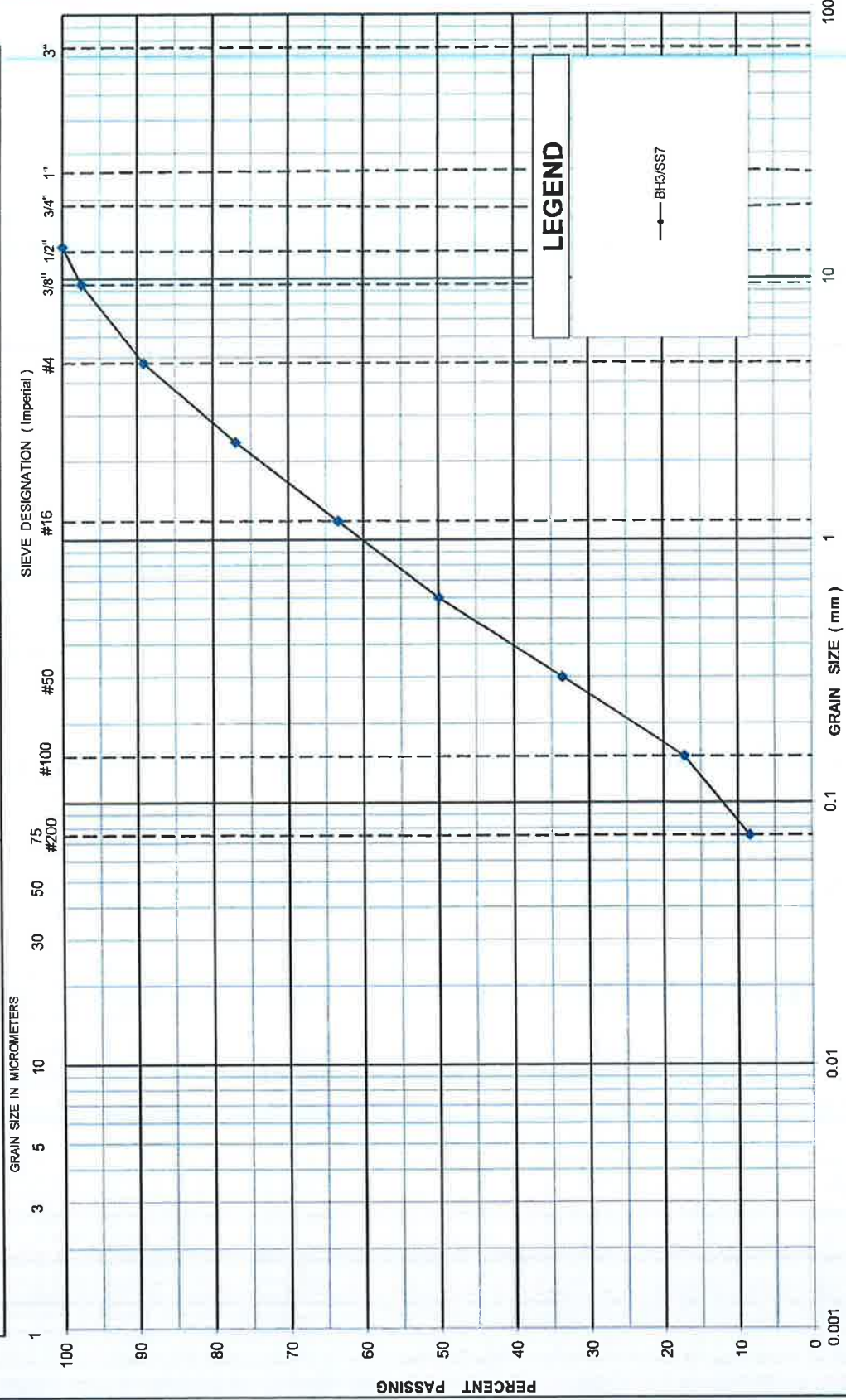
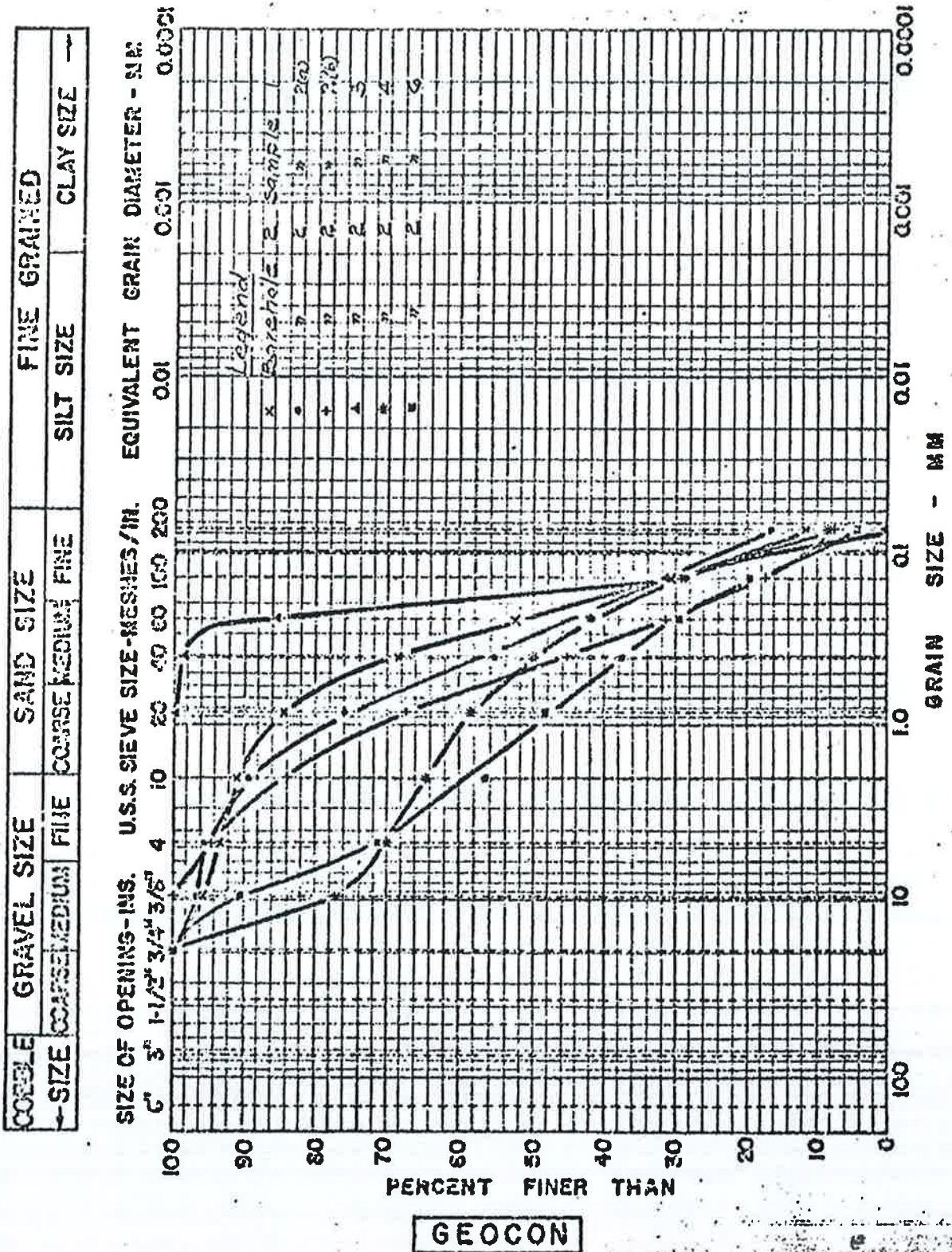


Figure : B-5
 PROJECT # : TRANETOB20462AA
 DATE : MARCH, 2013

GRAIN SIZE DISTRIBUTION
 SAND, some gravel

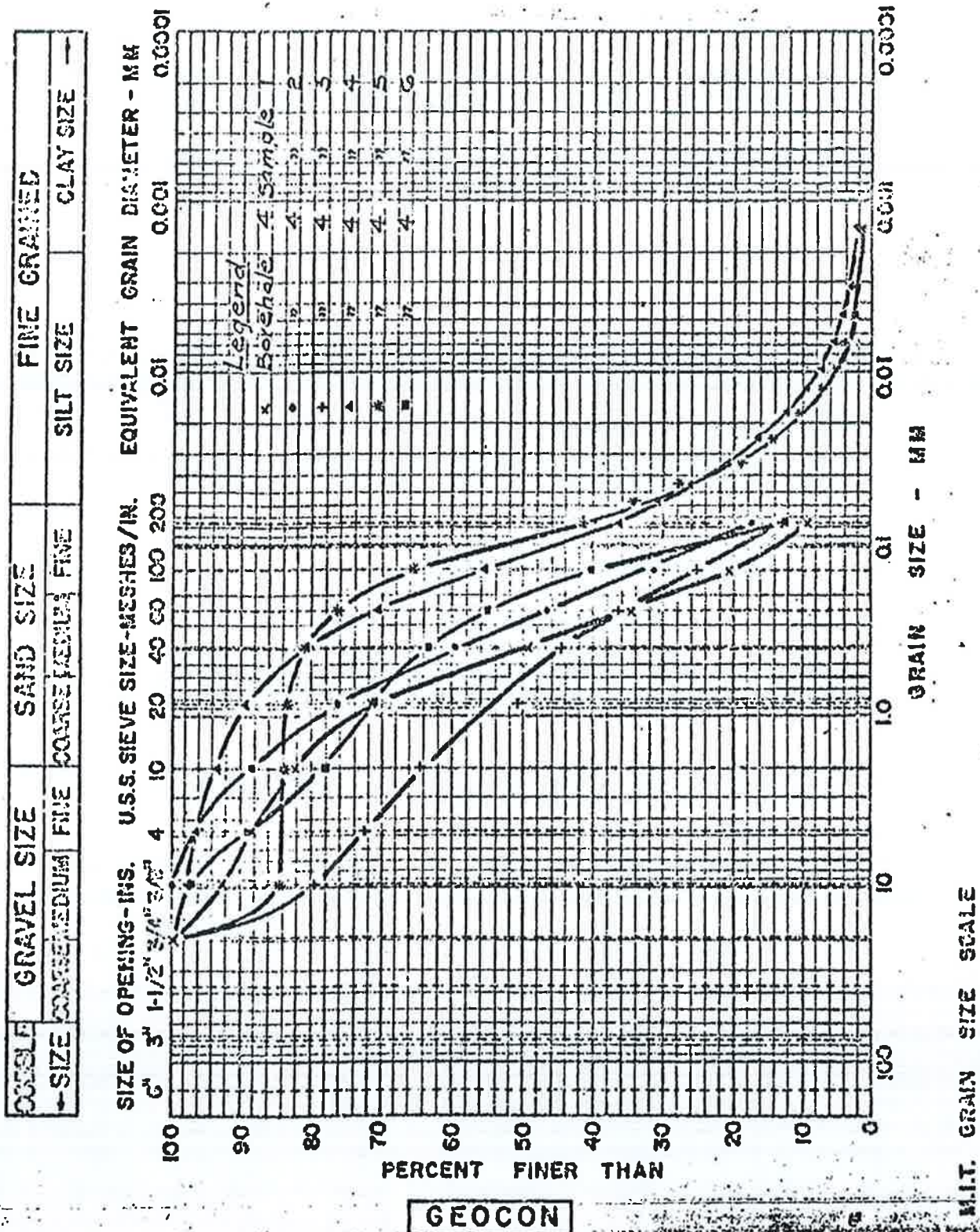
GRAIN SIZE DISTRIBUTION

FIGURE 1
PROJECT T9864



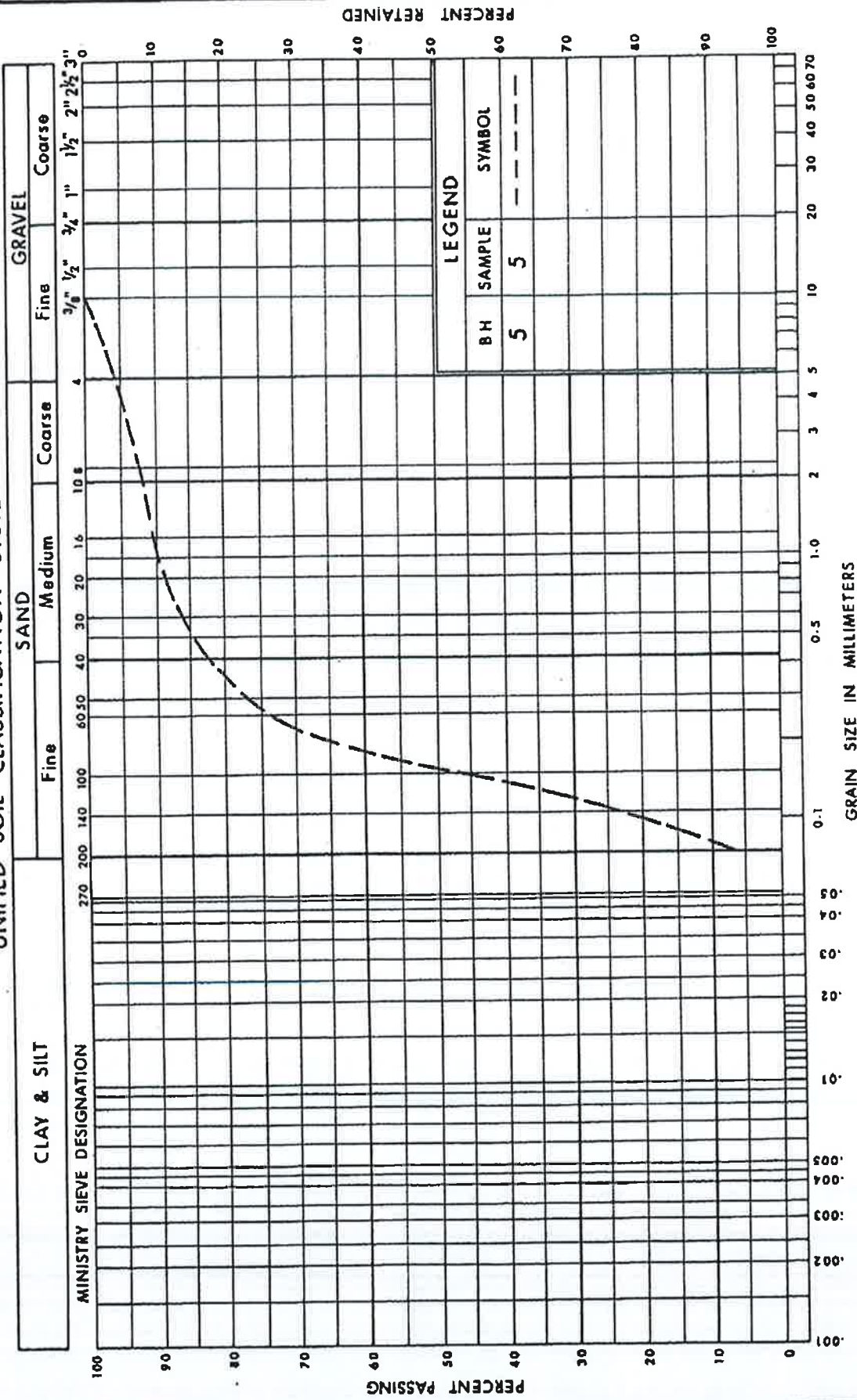
GRAIN SIZE DISTRIBUTION

FIGURE 2
PROJECT T9864



Oct 75, FF-S-22

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
SAND & GRAVEL

FIG No 1

WP 906-66-14

Appendix C

Site Photographs



Photograph 1. NBL Bridge



Photograph 2. SBL Bridge

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:

ROD (%)	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_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ_i	°	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_c	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = c_u / τ_c

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_a	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_P)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_P) / I_p$	l	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ²	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						



**PRELIMINARY FOUNDATION DESIGN
REPORT FOR THE REHABILITATION OF
HIGHWAY 400 / COUNTY ROAD 23
OVERPASS, TOWNSHIP OF
COLDWATER, ONTARIO
G.W.P. 2190-10-02, SITE NO. 30/455-1/2
GEOCRES 31D-562**

McCormick Rankin Corporation

TRANETOB20462AA
September 16, 2013

REPORT

CONTENTS

5	DISCUSSION AND RECOMMENDATIONS	7
5.1	General	7
5.2	Existing Bridge Foundations	8
5.3	Bridge Foundations for Widening	8
5.3.1	Spread Footing Foundations on Native Soil	9
5.3.2	Caisson Foundations	10
5.3.3	Driven Steel H-Piles	11
5.3.4	Micropiles	11
5.3.5	Continuous Flight Auger (CFA) Piles	12
5.3.6	Recommended Foundation Options for Widening	13
5.4	Earth Pressures	13
5.4.1	Seismic Design Data	14
5.4.1.1	Site Coefficient	14
5.4.1.2	Seismic Zone and Zonal Acceleration Ratio (A)	14
5.4.1.3	Single span bridges	14
5.4.1.4	Liquefaction Potential	14
5.5	Approach Embankments	15
5.6	Construction Comments	16
5.7	Frost Protection	17
6	CLOSURE	17

Appendices

Appendix E: GA Drawings

Appendix F: Advantages, Disadvantages, Costs and Risks/Consequences of Foundation Alternatives
(for widening only)

Appendix G: List of OPSS, OPSD, SP and Non-standard Specifications

Appendix H: Limitations of Report

**PRELIMINARY FOUNDATION DESIGN REPORT
REHABILITATION OF HIGHWAY 400 / COUNTY ROAD 23 OVERPASS,
TOWNSHIP OF COLDWATER, ONTARIO
G.W.P. 2190-10-02, SITE NO. 30/455-1/2**

5 DISCUSSION AND RECOMMENDATIONS

5.1 General

We understand that the existing NBL and SBL Highway 400 Bridges over the County Road 23 (i.e. Highway 400 overpass structures) will be rehabilitated at the site. It should be noted that the original project scope included rehabilitation and widening of the bridge. After completion of the field investigations, the scope was changed to rehabilitation only. The foundation design recommendations provided in the subsequent sections are based on the previous preferred rehabilitation option with bridge widening.

The existing bridges are single span structures with span lengths of 27.4 m for the NBL and 28.0 m for the SBL, at a skew with the County Road 23 alignment.

It is our understanding that the existing bridges, which were built in mid 1970's and early 1980's, are supported on spread footing foundations.

Previous site investigation for the existing bridge structures were carried out in 1974 by GEOCON and subsurface conditions mainly consisting of dense to very dense granular soil with up to 1.8 m (6 feet) of loose to compact granular soil near the ground surface was noted at the site. Groundwater at the time of previous investigation was recorded near ground surface to about 3 m (10 feet) below the grade at the time of investigation.

The current preliminary foundation investigation for the previously proposed widening option, consisting of eight boreholes, revealed the presence of granular fills, underlain by granular overburden consisting of silty sand to sandy silt glacial tills and granular interglacial and glacial wash deposits, ranging in composition from silty sand/sandy silt, gravelly sand to cobbles and boulders in a sand matrix. The groundwater table at the time of the investigation was inferred to be between El. 192.5 and 190.0 m*.

*the cave-in condition recorded at El. 198.3 m in Borehole 2 is due to embankment fill collapsing in the open borehole and does not reflect groundwater conditions.

5.2 Existing Bridge Foundations

As mentioned before, it is our understanding that the existing bridge is supported on spread footing foundations. Based on the original GA drawings (see Appendix E), founding elevations of existing bridge support elements are as follow;

Table 5.2.1
Existing Footing Elevations

Bridge Location	Foundation Element	Founding Elevation (m)	Remark
NBL	South West	189.9 (191.1*)	Bridge foundation top elevations were presented on GA Drawing. Typical 1.2 m (4 feet) thick concrete was assumed to estimate founding elevations
	South East	190.8 (192.0*)	
	North West	192.0 (193.2*)	
	North East	191.1 (192.3*)	
SBL	South	194.6	Founding elevation is underside of 0.6 m (2 feet) compacted Granular 'A' material
	North	195.7	

Note: *bridge foundation top elevation

The use of existing spread footing foundations will be the most favourable and economical choice for the bridge rehabilitation, if loading condition is similar to the existing bridge.

We understand that the existing bridge foundations have been designed for a net safe bearing pressure of 3.0 tons per sq.ft. Similar design resistance of 300 kPa at SLS and a value equal to 500 kPa at ULS can be utilized for the rehabilitation assuming that the existing footings were placed on undisturbed native soils under proper geotechnical inspection and constructed with good construction practices.

The geotechnical resistance quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the bearing resistance should be reduced in accordance with Clause 6.7.3 and Clause 6.7.4, CHBDC CAN/CSA-S6-06.

Based on the MRC provided information, above mentioned geotechnical resistances are found sufficient to support the proposed bridge rehabilitation. It is also our understanding that semi-integral abutment will be used for the proposed bridge rehabilitation and shallow foundation is a feasible foundation option for semi-integral abutment.

Sliding resistance can be provided by utilizing the sliding resistance between the concrete poured for the foundations and the native undisturbed subgrade. For the evaluation of the sliding resistance of the foundation (Clause 6.7.5, CHBDC CAN/CSA-S6-06), the ultimate friction between the bottom of the concrete foundation and underlying dense to very dense native undisturbed soils/compacted granular 'A' pad may be taken as 30° (i.e. friction factor of 0.5) assuming that the existing footings were placed on undisturbed native soils/compacted granular 'A' pad, under proper geotechnical inspection and constructed with good construction practices.

5.3 Bridge Foundations for Widening

Based on the available borehole data, the more logical choice is to use spread footing foundations for the widening, matching the existing bridge foundations.

Deep foundation options were also considered but are unlikely be suitable/economical for this project. A summary of various foundation options is given in Appendix F, and some are also discussed in the following sections.

5.3.1 Spread Footing Foundations on Native Soil

The bridge widening can be supported on spread footing foundations placed on undisturbed dense to very dense native soils. The depth to the sufficiently competent soils from the existing ground surface at the borehole location is given in the following table.

We understand that the existing bridge foundations have been designed for a net safe bearing pressure of 3.0 tons per sq.ft. Similar design resistance of 300 kPa at SLS and a value equal to 500 kPa at ULS can be utilized for the widening.

Table 5.4.1.1
Spread Footing Foundations on Native Soil

Location	Borehole No./Elevation (m)	Recommended Highest Founding Level Below Existing Ground At the Borehole Location (m)	Elevation (m)	Recommended Factored Bearing Resistance at ULS* (kPa)	Recommended Factored Bearing Resistance at SLS (kPa)	Subgrade Soil
North Bound Lanes	1 198.0	6.4	191.6	500	300	Silty Sand Till
	3 192.8	2.8	190.0**	500	300	Sand, some Gravel
	2 199.7	7.6	192.1	500	300	Silty Fine Sand
	4 193.2	1.6	191.6***	500	300	Silty Sand Till
South Bound Lanes	5 202.3	7.6	194.7	500	300	Silty Sand Till
	7 195.1	3.0	192.1	500	300	Fine Sand
	6 203.9	7.0	196.9	500	300	Sand
	8 195.7	2.0	193.7	500	300	Silty Sand Till

* Assuming a minimum footing width of 2 m

** The underside footing elevation can be changed to El. 191.3 m, if it can be verified during construction that the soil is dense native material

*** The underside of footing elevation can be changed to E. 192.3 m, if it can be verified during construction that the soil is dense native material

Where the underside footing elevations given in the above table are lower than the existing footing elevations, the following approach can be taken, if desired. The unsuitable (i.e. fill) soil can be removed to the native, suitable soil subgrade level under the supervision and evaluation of a geotechnical engineer familiar with this report and replaced with mass concrete to prevent extensive shoring and/or undermining of the existing footings. However, this may need to be carried out in narrow section (say 2 m wide) perpendicular to the existing footings and the excavations filled with mass concrete promptly to prevent cave-ins of the side of the excavation and also to prevent undermining of the existing footing.

The factored bearing resistance at ULS given in the above table incorporates a resistance factor of 0.5, as per Canadian Highway Bridge Design Code (CHBDC) S6-06. The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm, respectively. This can be achieved provided that the native dense to very dense founding subgrade is undisturbed during the construction. It should also be pointed out that the total settlements quoted here may translate into differential settlements between the existing and the widening portion. As some of

the quoted settlements will take place during construction (due to the presence of granular subgrade soils), this may not be detrimental. This aspect should however be taken into consideration in the design and in the construction sequencing.

The geotechnical resistance quoted above are for concentric, vertical loads only. In the case of eccentric or inclined loading, the bearing resistance should be reduced in accordance with Clause 6.7.3 and Clause 6.7.4, CHBDC CAN/CSA-S6-06.

As was mentioned before it is difficult to distinguish the native soils from the overlying fills. For this reason, careful inspection of the bearing surface will be required during the construction to ensure that the footings are placed on native, undisturbed dense to very dense soil. As well, the detailed foundation investigation should be conducted with this aspect in mind.

Allowance should be made to place a 120 mm thick mud mat (i.e. skim coat of concrete) in all footing excavation as soon as possible after excavation, inspection, evaluation and approval of bearing surface by qualified geotechnical personnel. This should be done within four hours of excavating.

For frost protection, the foundations should have a permanent earth cover of at least 1.5 m or equivalent artificial insulation.

As will be discussed in more detail later, some dewatering will likely be required to ensure that footings can be constructed in the dry.

Sliding resistance can be provided by utilizing the sliding resistance between the concrete poured for the foundations and the dense to very dense native till or interglacial and other granular subgrade soils. For the evaluation of the sliding resistance of the foundation (Clause 6.7.5, CHBDC CAN/CSA-S6-06), the ultimate friction between the bottom of the concrete foundation and underlying dense to very dense soils may be taken as 30° (i.e. friction factor of 0.5).

5.3.2 Caisson Foundations

Augured and cast-in-place concrete foundations (drilled caissons) can be considered.

The presence of frequent cobbles & boulders as well as installation of the caissons below the groundwater table can be expected to cause problems. Therefore, use of caissons is not considered a practical method. However, the following paragraphs are presented for the sake of completeness.

For caissons socketed at least 2.0 m into the very dense granular overburden material, geotechnical design resistance values of 1200 kPa at SLS and 1900 kPa at ULS can be assigned. These design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 and 1.8 m diameter) provided the minimum caisson length is 4.0 m below the bottom of the pile cap and also the adjacent finished ground. However, the use of relatively smaller caisson diameter sizes (i.e. between 0.76 and 1.35 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.9 m diameter caisson will have a base area of $r^2 \times \pi = (0.9/2)^2 \times 3.1416 = 0.64 \text{ m}^2$. When designed for a SLS value of 1200 kPa, the caisson would be capable of carrying an axial load of $0.64 \text{ m}^2 \times 1200 \text{ kN/m}^2 = 770 \text{ kN}$ at SLS. Similarly if a 1.2 m diameter caisson is used, then the caisson resistance at SLS would be $(1.2/2)^2 \times 3.1416 \times 1200 \text{ kN/m}^2 = 1360 \text{ kN/caisson}$.

As an example, at Borehole 8 location, the anticipated caisson bottom elevation would be about 191.8 m, with the requirement of minimum 2 m penetration into the very dense soil.

Higher resistances are available for caisson foundations extended deeper but this is not recommended as it would lead to greater difficulties during installation due to the presence of boulders as well as more extensive dewatering.

As the caissons will likely extend below the groundwater, base instability may present a problem and thus, dewatering will likely be required. The dewatering will need to be designed and conducted in a manner so as not to cause detrimental settlements of the existing spread footing foundations.

As caisson foundations are not well-suited for supporting the bridge widening, they will not be discussed any further in this preliminary report; however, further details can be provided on request.

5.3.3 Driven Steel H-Piles

Due to the presence of very dense zones along with the presence of frequent cobbles and boulders, the piles can be expected to reach refusal at rather variable depths and in some cases they may be very short (i.e. refusal encountered at high elevations). The subsurface conditions are therefore considered not to be favorable for the use of driven steel H-piles.

For the reasons cited (i.e. short pile lengths and the presence of cobbles/boulders and very dense zones in the overburden soils), the use of other types of driven piles, including timber piles, concrete piles and steel tube piles, is not recommended.

5.3.4 Micropiles

Another alternative which may be considered is the use of micropiles to support the structure.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in most soil and rock types and ground conditions. 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. Due to the small pile diameter (typically 160 to 260 mm), any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

Axial resistances of up to about 750 kN/micropile are available (at ULS) depending on the diameter and penetration into the very dense overburden. The lateral resistances would also depend on the diameter and penetration length into the very dense soil.

As mentioned before, the use of micropiles will likely be less economical than spread footing foundations, due to the fact that the installation requires a more specialized installer for the micropiles. However, it is advantageous if low overhead is a necessity and/or due to the interference of new foundation support with the existing foundations, and also due to the fact that they can be expected to reduce the shoring effort. As was mentioned before, geotechnical resistances will also depend on such factors as diameter, method of installation, micropile lengths, etc. Typically, the geotechnical resistance is calculated by multiplying the circumferential area (i.e. circumference x length) by bond strength. For preliminary estimating purposes, the bond strength between the micropile and the very dense soil can be taken as 250 to 300 kPa.

5.3.5 Continuous Flight Auger (CFA) Piles

CFA piles are a type of drilled foundation in which the pile is drilled to the final depth in one continuous process using continuous flight augers. As the auger is withdrawn from the hole, concrete or a sand/cement grout is placed by pumping the concrete/grout mix through the hollow center of the auger pipe to the base of the auger. Simultaneous pumping of the grout or concrete and withdrawing of the auger provides continuous support of the hole. Reinforcement for steel-reinforced CFA piles is placed into the hole filled with fluid concrete/grout immediately after withdrawal of the auger. CFA piles are typically installed with diameters ranging from 0.3 to 0.9 m (12 to 36 inches), but to our knowledge, locally available diameters are 0.5 to 0.6 m (20 to 24 inches) and installed lengths of up to about 24 m are locally available, although longer piles have occasionally been used. This maximum CFA pile length should be discussed with local contractors, if you wish to use CFA option. The steel reinforcement is often limited to the upper 10 to 15 m of the pile for ease of installation and also due to the fact that in many cases, relatively low bending stresses are transferred below these depths. In some cases, full-length reinforcement is used, as is most common with drilled shaft foundations. CFA piles can be constructed as single piles (similar to drilled shafts), for example, for noise wall or light pole foundations. For bridges or other large structural foundations, CFA piles are most commonly installed as part of a pile group in a manner similar to that of driven pile foundations. Similar to driven piles, the top of a group of CFA piles is terminated with a cap. Typical minimum center-to-center spacing is 3 to 5 pile diameters, preferably 5.

CFA piles differ from conventional drilled shaft or bored piles, and exhibit both advantages and disadvantages over conventional drilled shafts. The main difference is that the use of casing or slurry to temporarily support the hole is avoided. Drilling the hole in one continuous process is faster than drilling a shaft excavation, an operation that requires lowering the drilling bit multiple times to complete the excavation. In contrast, the torque requirement to install the continuous auger is high compared with a conventional drilled shaft of similar diameter; therefore, the diameter and length of CFA piles are generally less than drilled shafts, as well as limiting the depths. The use of continuous augers for installation also limits CFA piles to hard soil or very weak rock profiles, while drilled shafts are often socketed into rock or other very hard bearing materials. Because CFA piles are drilled and cast-in-place rather than being driven, as are driven piles, noise and vibration due to pile installation are reduced. CFA piles also eliminate splices and cutoffs. Soil heave due to driving can be eliminated when non-displacement CFA piles are used. Hydrostatic uplift conditions at the bottom of the borehole can be counter-balanced with concrete or a sand/cement grout. A disadvantage of CFA piles compared to driven piles is that the available QA methods to verify the structural integrity and pile bearing capacity for CFA pile are less reliable than those for driven piles. Another disadvantage of CFA piles is that CFA piles generate soil spoils that require collection and disposal. Handling of spoils can be a significant issue when the soils are contaminated or if limited room is available on site for the handling of material. The presence of cobbles and boulders will likely cause problems during their installation at this site.

In our opinion, CFA piles are unlikely to be economical for this project and their use is not recommended. We will however be pleased to discuss this issue further, upon request.

5.3.6 Recommended Foundation Options for Widening

We understand that the existing bridge structures are founded on spread footing foundations. Based on this and considering the subsurface conditions as revealed by the boreholes, advanced for previous and current investigations, it is our opinion that the most favourable foundation option is to use spread footing foundations for both rehabilitation and widening (if required).

For widening, disadvantages of this option are the requirement for extensive shoring that will be required to facilitate the construction and possible dewatering requirements. While in essence the use of micropiles is considerably more expensive in comparison with normal spread footing foundations, in this instance it may merit looking into, as it would reduce shoring costs and virtually eliminate the need for dewatering.

5.4 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, as given in Appendix G.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drains pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with CHBDC S6-06. For design purposes, the following 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/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$ $K_b = 0.35$

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

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$ $K_b = 0.41$

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

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

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

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

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

should be used in accordance with Canadian Highway Bridge Design Code (CHBDC S6-06). The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC.

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

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

5.4.1 Seismic Design Data

5.4.1.1 Site Coefficient

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-06). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S , for the site is 1.0.

5.4.1.2 Seismic Zone and Zonal Acceleration Ratio (A)

Table A3.1.1 of the CHBDC provides a zonal Acceleration Ratio (A) of 0.05 and Velocity Related Seismic Zone (Z_v) of 1 for Orillia. 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 $A=0.05$.

5.4.1.3 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.

5.4.1.4 Liquefaction Potential

The founding soils are considered not liquefiable.

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.05 g, a factor of safety of greater than 1.0 against liquefaction is obtained for magnitude 7.5 earthquake events under the approach embankment.

5.5 Approach Embankments

It is our understanding that no embankment widening is required for bridge rehabilitation. Approach embankment construction recommendations may not be required for rehabilitation.

Following approach embankment construction recommendations are for the previously proposed widening option, and these are presented for the sake of completeness of the report.

Based on the previously proposed widening option, the existing embankments can be widened by about 5 m along SBL and 3 m along NBL, on the inside (i.e. towards the median side) to accommodate the extra lane in each direction. It is further understood that the existing embankments are approximately 7 m high above the County Road 23 level and that the grade of Highway 400 level will remain essentially the same.

Based on the available borehole data, foundation failures are not anticipated for approach embankments of up to about 7 m in height, constructed with normal 2H:1V side slopes or flatter, provided that all organic, soft/very loose or otherwise unsuitable materials will be removed as per MTO standards, prior to placing the embankment fills.

After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a heavy compactor. If necessary, the groundwater table should be lowered to at least 0.6 m below the subgrade level, before any proof rolling and the application of significant compaction effort. The dewatering can be achieved by gravity drainage and pumping from strategically placed sumps and, if necessary, ditches.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized, 2H:1V side slope can be used for the construction of approach fills. It is recommended that in as much as possible the materials used for the widening consist of granular soils, similar to the existing embankment fill. On-site excavated materials, free of organics, can be used for this purpose, provided that proper moisture adjustments can be made. Proper erosion control measures should be implemented by seed and cover (OPSS 804) or sodding (OPSS 803).

The existing embankment side slopes should be properly benched as per MTO standard (OPSS 208.10) where the new embankment fills are to abut into the existing.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Selected Subgrade Materials-OPSS1010). Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirement of OPSS 501 and OPSS 206. Construction should be in accordance with OPSS 206. Quality assurance should be provided as per OPSS 501.

Based on the findings of the boreholes, the anticipated settlements under the stresses generated by the approximately 3 to 5 m widening should not exceed 25 mm. This should necessitate neither surcharging nor preloading. The settlement can have a detrimental effect on the existing pavement, but this too should be acceptable for a flexible pavement. These aspects can be further looked into during the detail investigation stage.

5.6 Construction Comments

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

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

The boreholes show that the excavation can be expected to extend through some fill materials underlain by granular soils ranging from silty sand/sandy silt; sand with some gravel; gravel/cobbles/boulders in a sand matrix to silty sand/sandy silt till. These soils can be classified as follows:

Embankment Fill	Type 3 soil above groundwater table
Lower Fill	Type 3 soil above groundwater table Type 4 soil below groundwater table
Silty Sand/Sandy Silt Till (dense to v.dense)	Type 2 soil above groundwater table Type 4 soil below groundwater table
Silty Sand/Sandy Silt Till (compact)	Type 3 soil above groundwater table Type 4 soil below groundwater table
Interglacial and other granular native soils (dense to v.dense)	Type 3 soil above groundwater table Type 4 soil below groundwater table
Interglacial and other granular native soils (compact)	Type 4 soil

Any excavation immediately adjacent to the existing foundations should proceed in a manner so as not to disturb the foundation soils of the existing footings. For bridge widening option, where excavation for the new foundation needs to extend deeper than the existing foundation, unsuitable soil (i.e. fill) can be removed to the native, suitable soil subgrade level under the supervision and evaluation of a geotechnical engineer familiar with this report and replaced with mass concrete to prevent extensive shoring and/or undermining of the existing footings. However, this may need to be carried out in narrow section (say 2 m wide) perpendicular to the existing footings and the excavations filled with mass concrete promptly to prevent cave-ins of the side of the excavation and also to prevent undermining of the existing footing. If required, shoring may be considered to achieve this.

Depending on the groundwater level at the time of construction and the depth of excavations, dewatering may be required to preserve the bearing resistance of the soil and to facilitate construction. It may be possible to dewater the site by up to 0.6 m by pumping from strategically placed shallow wells. These must be properly filtered (to prevent the removal of soil fines while pumping). For deeper drawdown, filtered deep wells and/or well points will likely be required. The presence of oversized gravel, cobbles and boulders can be expected to render the installation of deep wells and especially well points rather difficult. In addition any dewatering should be designed and implemented in a manner so as not to cause detrimental settlements of the existing structures.

Temporary support will be required to construct the foundations of the widening. This will likely consist of shoring. In Ontario, shoring is typically in the form of soldier piles and lagging. In this instance tie backs may also be required. The soldier piles can be expected to extend through the embankment fills into the dense to very dense overburden soils. Again, the presence of cobbles and boulders may present problems, including the installation of the tie backs (if used).

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. In this case, the required performance level is considered 1B or 2, but probably 2, which can be decided at the time of detailed investigation. The shoring system should be designed by a Professional Engineer, experienced in this type of work. As mentioned before all shoring should be in accordance with OPSS 539.

Table 5.7.1
Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Granular Embankment Fills	0.33	0.50	3.0	20.5
Lower (granular) Fills	0.33	0.50	3.0	21.0
Silty Sand / Sandy Silt Till (dense to v.dense)	0.28	0.44	3.5	22.0
Silty Sand / Sandy Silt Till (compact)	0.32	0.49	2.9	21.0
Interglacial and other granular native soils (dense to v.dense)	0.29	0.45	3.4	21.5
Interglacial and other granular native soils (compact)	0.30	0.46	3.2	20.5

5.7 Frost Protection

Design frost protection depth for the general area is 1.5 m. Therefore, a permanent soil cover of 1.5 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

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

For and on behalf of Coffey

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

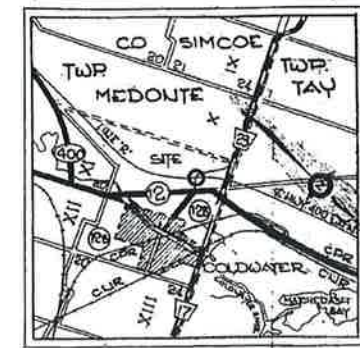


Zuhtu Ozden, P.Eng.
Senior Principal



Appendix E

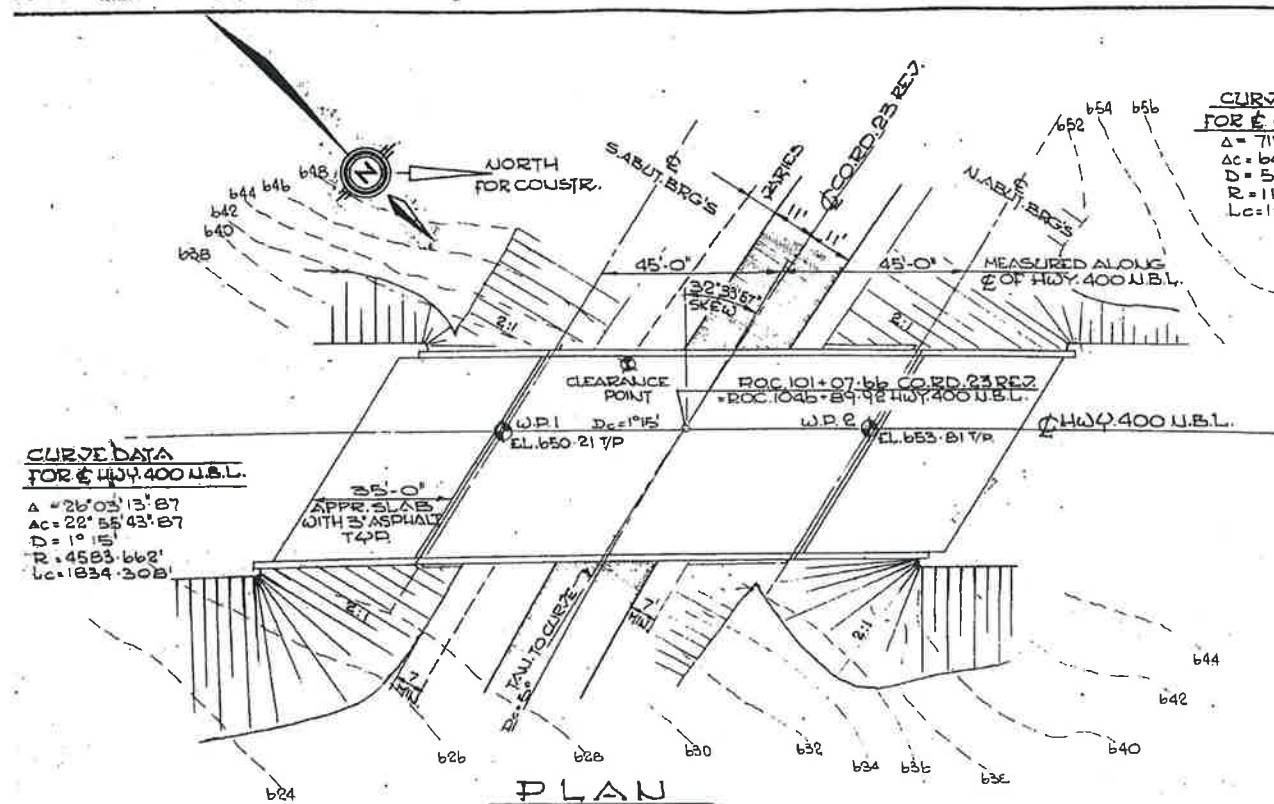
GA Drawings



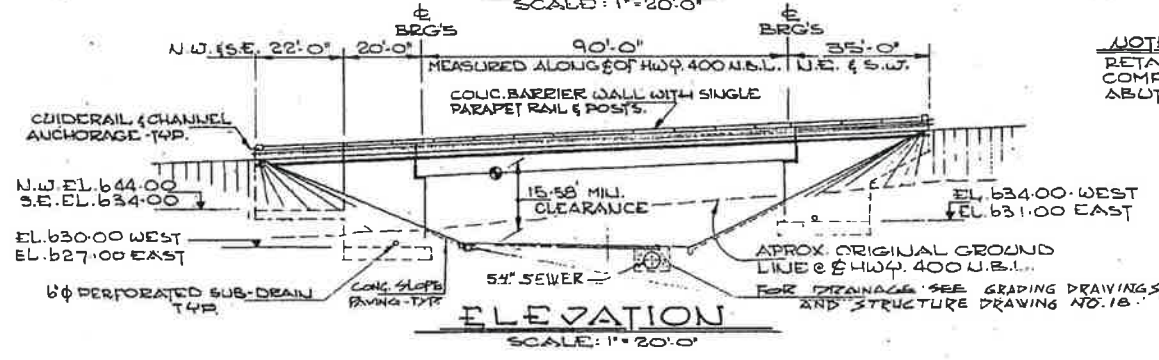
KEY PLAN
SCALE: 1/4" = 1 MILE

**CURVE DATA
FOR C.O.R.D. 23**
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Δc = 64° 02' 36.95"
D = 5° 00'
R = 1145.916
Lc = 1220.872'

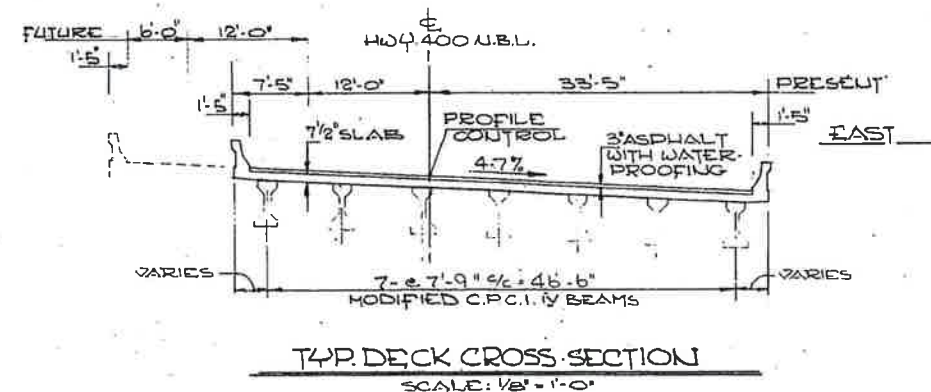
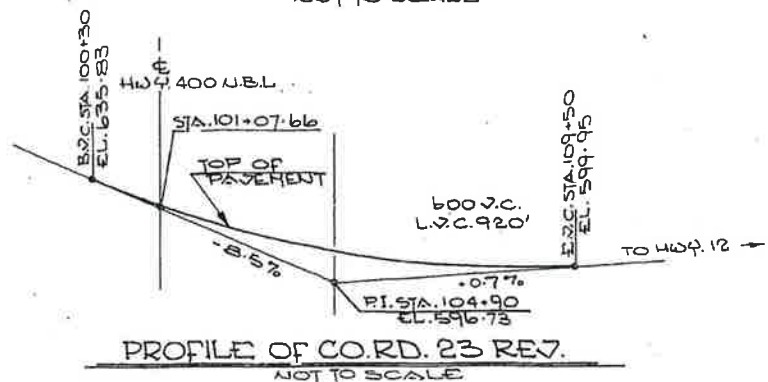
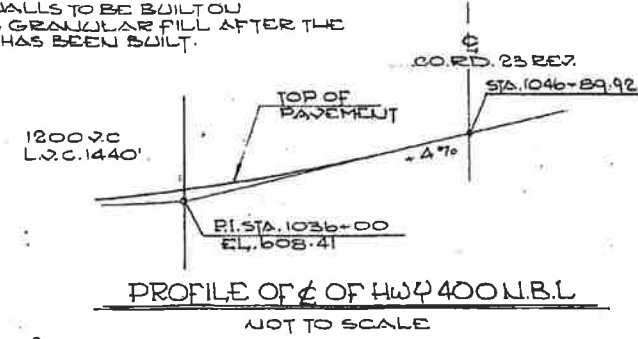
**CURVE DATA
FOR H.W.Y. 400 N.B.L.**
Δ = 26° 03' 13.87"
Δc = 22° 55' 43.87"
D = 1° 15'
R = 4583.662'
Lc = 1834.308'



PLAN
SCALE: 1" = 20.0'



NOTE:
RETAINING WALLS TO BE BUILT ON COMPACTED GRANULAR FILL AFTER THE ABUTMENT HAS BEEN BUILT.



- LIST OF DRAWINGS**
- 30-455-1 GENERAL LAYOUT
 - 2 BORE HOLE LOCATION & SOIL STRATA
 - 3 FOOTING LAYOUT
 - 4 FOOTING REINFORCEMENT
 - 5 SOUTH ABUTMENT
 - 6 NORTH ABUTMENT
 - 7 RETAINING WALLS & DETAILS OF FILL MATERIAL
 - 8 PRESTRESSED GIRDERS
 - 9 DECK AND SCREED ELEVATION
 - 10 CONCRETE BARRIER WALL (2'-8" HIGH)
 - 11 STEEL PARAPET RAILING (SINGLE TUBE)
 - 12 3/8" APPROACH SLAB (FOR BARRIER WALL)
 - 13 DETAILS OF CONC. SLOPE PAVING
 - 14 STANDARD DETAILS I
 - 15 STANDARD DETAILS II
 - 16 BRIDGE ELECTRICAL DETAILS - TYPE 12
 - 17 AS CONSTRUCTED ELEV. & DIM.
 - 18 54" STORM SEWER SHORING

- GENERAL NOTES**
- CLASS OF CONCRETE**
DECK SLAB, BARRIER WALL, APPR. SLABS - 4000 PSI
PRESTRESSED CONC. GIRDERS - 5000 PSI
REMAINDER - 3000 PSI
- CLEAR COVER ON REIN. STEEL**
FOOTINGS, ABUTMENTS - 3"; DECK, BOTT. 1" TOP 2"
DIAPHRAGMS - 1 1/2"; APPR. SLABS - 2"
BARRIER WALLS - 1 1/2"
- CONSTRUCTION NOTES**
- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARING SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF 1/8" ±
 - NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED.

- CONCRETE QUANTITIES**
- CONCRETE QUANTITIES ARE LISTED BELOW FOR THE APPROPRIATE CONCRETE LUMP SUM TENDER ITEMS:
- 1. CONCRETE IN ABUTMENTS, WING WALLS AND RETAINING WALLS. - 453 CY
 - 2. CONCRETE IN DECK AND DIAPHRAGMS. - 148 CY
 - 3. CONCRETE IN BARRIER WALLS. - 29 CY
 - 4. CONCRETE IN APPROACH SLABS. - 141 CY
 - 5. CONCRETE IN SLOPE PAVING. - 40 CY

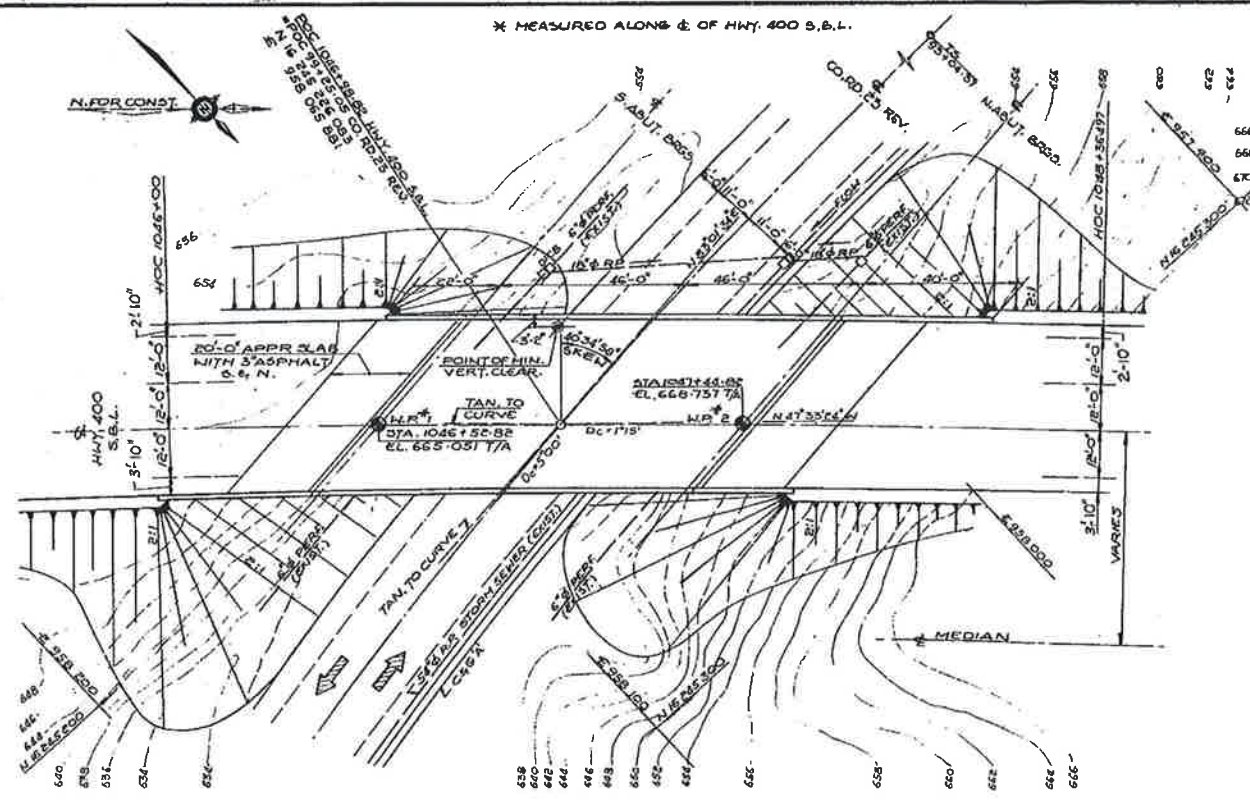
REVISIONS	
DATE	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS ONTARIO	
CO. RD. 23 INTERCHANGE OVERPASS 3.7 MILES NORTH OF EXISTING H.W.Y. 400	
KING'S HIGHWAY No. 400 EXTENSION N.B.L. DIST. No. 5	
CO. SIMCOE	
TWP. MEDONTE LOT 23 CON. 11	
GENERAL LAYOUT	
APPROVED: <i>[Signature]</i>	CONTRACT No. 77-25
DESIGN: G.A.D. CHECK: D.L.G.	W.P. No. 906-bb-09
DRAWING: E.A. CHECK: G.A.	SITE No. 30-455 SHEET 1
DATE: JULIE/75	LOADING: 1520.44

TWP# R52-433-1-F



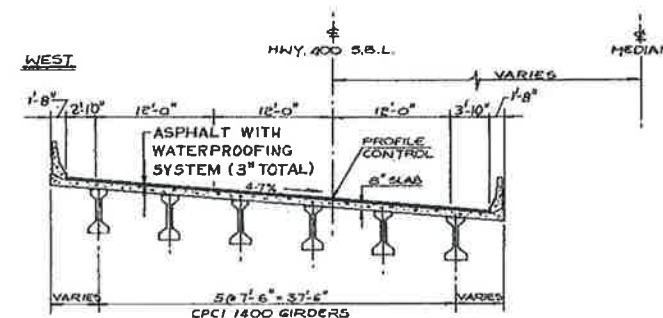
FOR REDUCED PLAN
USE SCALE BELOW
10 11 12 13
3 INCHES ON ORIGINAL PLAN



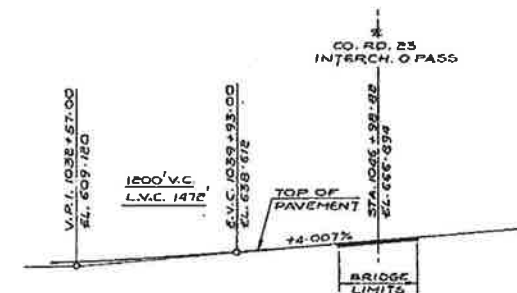
PLAN
1"=20'-0"

HWY. 400 S.B.L.
CURVE DATA
A = 26°03'14"
AL = 22°55'43.97"
D = 1°15'
R = 4583.662'
LC = 1834.204'

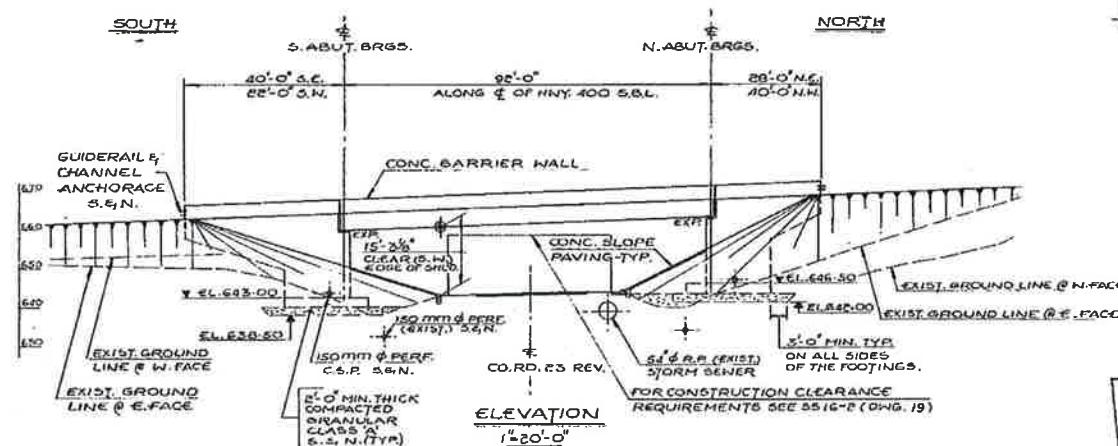
CO. RD. 23 REV.
CURVE DATA
A = 71°52'37"
AL = 64°02'36.05"
D = 5°00'
R = 1145.92'
LC = 1250.872'



TYP. DECK CROSS-SECTION
1/8" = 1'-0"



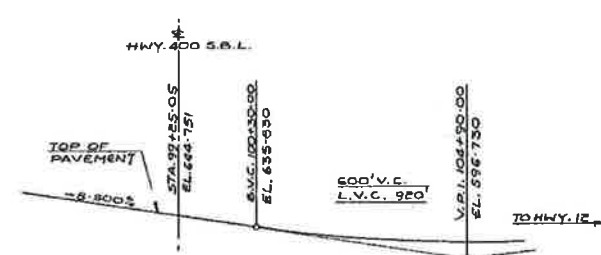
PROFILE OF C/L OF HWY. 400 S.B.L.
N.T.S.



ELEVATION
1"=20'-0"

DEPRESSION ADJACENT TO FRONT
FACE OF ABUTMENT FOOTING TO BE
FILLED WITH COMPACTED GRANULAR
CLASS "A" TO UNDERSIDE OF FOOTING.

DETAILS OF COMPACTED GRANULAR CLASS "A" REQUIREMENTS
(TYPICAL FOR N.A.S. ABUTMENTS)
N.T.S.



PROFILE OF CO. RD. 23 REV.
N.T.S.

* MIN. GRANULAR REQUIREMENTS TO BE
MEASURED PERPENDICULAR FROM THE
EDGE OF FOOTING AND ON ALL SIDES
OF THE FOOTING. (TYP. N.A.S. ABUT. FTGS)

B.M. 660.17
TOP OF BOLT IN TOP OF
CONC. HAND RAIL OF BRIDGE
1915 LT. 1047+85 C/L N.B.L.

DRAWING NOT TO BE SCALED
1" = 3' INCHES ON ORIGINAL PLAN

DIST. 5	CONT No 83-53	SHEET 109
WP No 906-66-14		
CO. RD. 23 INTERCH. O'PASS 4.8 KM SOUTH OF HWY. 12 INTERCH.	GENERAL ARRANGEMENT	

NOTES

CLASS OF CONCRETE
PRESTRESSED GIRDERS — 40 MPa
ABUTMENTS, WINGWALLS,
DECK SLAB, DIAPHRAGMS
& BARRIER WALLS — 30 MPa
REMAINDER — 20 MPa

REINFORCING STEEL
ALL STEEL GRADE 400
BAR MARK WITH SUFFIX 'C' DENOTES
COATED BAR.

CLEAR COVER TO REINFORCING STEEL
FOOTINGS — 4" ± 1"
ABUTMENTS & WINGWALLS
FRONT FACE — 3" ± 3/4"
BACK FACE — 2 3/4" ± 3/4"
DECK, TOP — 2 3/4" ± 3/4"
BOTTOM — 1 1/2" ± 3/8"
REMAINDER UNLESS OTHERWISE
NOTED — 2 3/4" ± 3/4"

CONSTRUCTION NOTES
THE CONTRACTOR IS RESPONSIBLE FOR
FINISHING THE BEARING SEATS DEAD LEVEL
TO THE SPECIFIED ELEVATION WITH A
TOLERANCE OF ± 1/8".

- LIST OF DRAWINGS**
- 30-4558-1 GENERAL ARRANGEMENT.
 - 2 BOREHOLE LOCATION & SOIL STRATA.
 - 3 FOOTING LAYOUT.
 - 4 FOOTING REINFORCEMENT.
 - 5 SOUTH ABUTMENT.
 - 6 NORTH ABUTMENT.
 - 7 NORTH & SOUTH ABUT. WINGWALLS.
 - 8 PRESTRESSED GIRDERS & BEARINGS.
 - 9 DECK DETAILS & SCREED ELEVATIONS.
 - 10 BARRIER WALL - WEST.
 - 11 BARRIER WALL - EAST.
 - 12 20 FT. APPROACH SLAB.
 - 13 DETAILS OF CONC. SLOPE PAVING.
 - 14 AS CONSTRUCTED ELEV. & DIM.
 - 15 BRIDGE DATA & SITE NUMBER DATA.
 - 16 BRIDGE ELECTRICAL DETAILS TYPE IV
 - 17 STANDARD DETAILS I
 - 18 STANDARD DETAILS II
 - 19 STANDARD DETAILS III
 - 20 STANDARD DETAILS IV
 - 21 QUANTITIES - STRUCTURE
 - 22 QUANTITIES - STRUCTURE



REVISION	DATE	BY	DESCRIPTION
1	01/04	K.Z.	DESIGN K.Z. CHECK H.N. LOADING CHANGES DATE 01/04
2	01/04	H.N.	DRAWING H.N. CHECK K.Z. SITE No 30-4558-090

Appendix F

**Advantages, Disadvantages, Costs and Risks/Consequences of
Foundation Alternatives (for widening only)**

Table F-1

Foundation Options for Widening of Highway 400 and County Road 23 Overpass

Foundation Type	Advantage/Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Will match existing abutment foundations and are least costly, but will necessitate a relatively deep excavation extending possibly below the groundwater table, immediately adjacent to the existing abutment foundations. Will require shoring and some dewatering.	Deformation of the existing foundations, due to adjacent excavation must be prevented during construction. Will require dewatering	Relatively economical but shoring and dewatering costs should be included when estimating costs	Recommended alternative for the prevailing subsurface conditions.
Spread Footings on Compacted Granular 'A' Pad	Not suitable for abutment footings due to the fact that the existing structures are founded on normal spread footings.	Engineered fill using Granular 'A' type soil properly compacted to not less than 100% of the Standard Proctor Maximum Density in not more than 0.3 m thick lifts, using resistance values of up to 300 kPa for SLS and a value equal to 500 kPa for ULS, for wing wall support could be suitable, depending on various factors such as geometry, type of material, space restriction for compaction, etc.	May be an economical alternative for wing wall foundations.	Not recommended for abutment footings but can be considered for wing wall footings.
Expanded Base (Frankie -type) Concrete Piles	Not well suited for the prevailing overburden conditions.	Extreme vibrations which may cause damage to the existing bridge structure.	Expensive	Not recommended.
Auger Press Concrete Piles	Not very suitable for the prevailing subsurface conditions, due to the presence of boulders.	May not provide adequate lateral resistance. Boulders will increase costs.	Expensive	Not recommended based on economics and reliability.
Timber Piles	Prone to damage during driving due to boulders and very dense zones in the till. Will not provide adequate axial resistance.	Damaged piles may go undetected. The piles may be too short.	Economical	Not recommended along a major highway based on reliability.
Driven Concrete Piles	High displacement piles, not suitable for the subsurface conditions at the site.	Can be damaged during driving due to the presence of cobbles and boulders.	Expensive	Not recommended based on cost and reliability.

Foundation Type	Advantage/Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Steel H-piles	Low displacement piles are well suited for the glacial till deposit underlying the site. However the piles will be shorter than normally accepted industry standards and as such relatively low axial and uplift resistances will be available. Problems may arise due to the presence of cobbles and boulders in the till and other granular overburden deposits. Minimizes dewatering and shoring.	The piles will be short and may even be extremely short if boulders are encountered during their driving.	Moderate	Not recommended based on reliability and due to the fact they do not match existing foundations.
Steel Tube Piles	Higher displacement piles in comparison with Steel H-piles; vulnerable to damage due to the presence of cobbles and boulders and very dense zones in the overburden. Less suitable than H-piles. Minimizes dewatering and shoring.	Considered unsuitable for the prevailing subsurface conditions.	Moderate	Not recommended based on reliability.
Drilled and Cast-in-place Concrete Piles (Drilled Caissons)	Minimizes shoring requirements and vibrations. Dewatering will be required, provides suitable resistances. The presence of boulders will present problems during construction.	Some problems may arise during the construction due to hydrostatic uplift and the presence of cobbles and boulders. Will require dewatering.	Moderate to Expensive	A possible option but dewatering and dealing with boulders should be carefully considered.
Micropile Foundations	Minimizes vibrations, dewatering and shoring requirements.	Problems may arise during the construction due to the presence of cobbles and boulders. But these can be overcome.	Expensive due to less competitive pricing.	A feasible option but probably more expensive than spread footing foundations.

Appendix G

List of OPSS, OPSD and Non-standard Specifications

OPSDs

OPSD 208.01 Benching of Earth Slopes

OPSD 3101.150 Walls, Abutment, Backfill Minimum Granular Requirement

OPSSs

OPSS 206 - Construction Specification for Grading

OPSS 212 - Construction Specification for Borrow

OPSS 501 - Construction Specification for Compacting

OPSS 539 - Construction Specification for Temporary Protection Systems

OPSS 803 - Construction Specification for Sodding

OPSS 804 - Construction Specification for Seed and Cover

OPSS 902 – Construction Specification for Excavating and Backfilling-Structures

OPSS 1010 - Material Specification for Aggregates-Base, Subbase, Select Subgrade, and Backfill Material

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.