

**FOUNDATION INVESTIGATION AND DESIGN REPORT
STURGEON RIVER BRIDGE REPLACEMENT
HIGHWAY 11
GERALDTON COMMUNITY
DISTRICT OF THUNDER BAY, TOWNSHIP OF COLTER, ONTARIO**

G.W.P. 6056-10-00, Site No. 48E-8

Geocres Number: 42H-13

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed replacement of the existing bridge structure which carries Highway 11 over Sturgeon River located approximately 18 km east of Jellicoe and 30 km west of Geraldton, in the District of Thunder Bay, Township of Colter, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

In the preparation of this report and in addition to the boreholes drilled under the current assignment, reference has been made to information on subsurface conditions contained in a previous foundation report. The title of this report is listed as follows:

- Foundation Investigation and Design Report for Sturgeon River Bridge (18 km east of Jellicoe), W.P. 335-85-01, Site 48E-8, Hwy. 11, District 19, Thunder Bay, Contract No. 87-202, Geocres No. 42E-3, prepared by Engineering Materials Office, Foundation Design Section, dated June 27, 1986. (Reference 1).

2 SITE DESCRIPTION

The Sturgeon River Bridge is located on Highway 11 approximately 18 km east of Jellicoe and 30 km west of Geraldton, Ontario in the District of Thunder Bay, Township of Colter.

At present, the highway crosses the Sturgeon River on a three-span structure supported on two piers and two abutments. The span lengths are 12 m, 17 m and 12 m. The total length of the bridge is 41.0 m and the width is 10.9 m.

At this site, the Sturgeon River flows to the north. The river channel is approximately 25 m to 30 m wide and 1.0 m to 3.0 m deep at the bridge. The river is relatively fast-flowing at this site. Rock fill erosion protection is visible above the river level throughout the lower parts of the approach embankments.

The area surrounding the bridge site is gently to moderately rolling. The areas to east and west of the site are generally heavily treed. Camp 51 Road is located approximately 90 m east of the bridge. Rock outcrops are visible near the site.

Photographs in Appendix F show the general nature of the site.

The site is located on a morainal landform, which consist of thin till deposits over bedrock. The site lies within the physiographic region known as the Wabigoon Subprovince of the Superior Province of the Canadian Shield. The region is characterized by metasedimentary rocks. Locally, the bedrock is mantled by sand and silt till deposit. Rock knobs are associated with this terrain.

3 SITE INVESTIGATION AND FIELD TESTING

A site investigation and field testing for this project were carried out on July 26 and 27 and August 3 and 5, 2011 and consisted of drilling and sampling six boreholes (numbered STR-01 to STR-03 and STR-08 to STR-10).

Locations of Boreholes STR-01 to STR-03 and STR-08 to STR-10 were selected based on the bridge design information available at the time of the investigation, including the possibility of retaining the existing abutments. However, after the field investigation was completed, the design concept was finalized to a full bridge replacement. For a bridge span of 35.0 m, the drilled boreholes were not positioned at the foundation units of the new design. A second investigation, consisting of four boreholes (numbered STR-04 to STR-07) was conducted from September 26 to 28, 2012 to identify soil conditions and establish the bedrock elevation at the revised abutment locations.

All the boreholes were drilled through the highway embankment or the bridge deck in the area of the existing and proposed west and east approaches and abutments.

Boreholes STR-01 and STR-10 were drilled through the existing west and east approaches, respectively, and were both terminated at 4.9 m and 6.4 m (elevations 337.2 and 335.8).

Boreholes STR-02 to STR-05 were drilled near the existing and proposed west abutments to depths ranging from 4.1 m to 8.7 m (elevations 333.4 to 338.0).

Boreholes STR-06 to STR-09 were drilled at the location of the existing and proposed east abutments and were terminated at depths ranging from 6.8 m to 12.7 m (elevations 329.5 to 335.4).

Bedrock was proved in Boreholes STR-03 to STR-08 by NQ size diamond coring. Boreholes STR-03 to STR-08 were advanced 2.8 m to 4.4 m into bedrock.

A Dynamic Cone Penetration Tests (DCPTs) was conducted adjacent to Borehole STR-09 to supplement the data collected from the borehole. The DCPT was terminated at 7.1 m depth (elevation 335.1), upon refusal on probable bedrock.

Records of Boreholes 1 to 4 drilled during the previous investigation (Reference 1) and their respective laboratory test results are enclosed in Appendix C.

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

The drilling was carried out from the highway grade using both CME 75 and CME 55 truck-mounted drill rigs. Hollow stem augers, NW casing and tricone were used to advance the boreholes through the overburden deposits and NQ coring methods were used to advance boreholes through the bedrock. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Two standpipe piezometers consisting of 19 mm PVC pipe with slotted screen and enclosed in filter sand were installed at this site to permit longer term groundwater level monitoring. The boreholes were backfilled with bentonite holeplug in general accordance with O.Reg. 903 upon completion. The location and completion details of the piezometer and boreholes are presented in Table 3.1.

Table 3.1 – Borehole Abandonment Details

Location	Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
West Approach	STR-01	None installed	Borehole backfilled with bentonite holeplug to 0.9 m, sand and gravel to 0.6 m, concrete to 0.15 m then asphalt to surface.
West Abutment	STR-02	None installed	Borehole backfilled with bentonite holeplug to 0.9 m, sand and gravel to 0.6 m, concrete to 0.15 m, then asphalt to surface.
	STR-03	8.5/333.6	Sand from 8.5 m to 6.7 m, bentonite holeplug from 6.7 m to 2.0 m, sand from 2.0 m to 0.4 m, then asphalt to surface.
	STR-04	None installed	Boreholes backfilled with bentonite holeplug to ground surface. Bridge deck backfilled with 0.28 m of concrete, then asphalt to highway surface.
	STR-05	None installed	
East Abutment	STR-06	None installed	
	STR-07	None installed	
	STR-08	11.0 / 331.2	Sand from 11.0 m to 9.1 m, bentonite holeplug from 9.1 m to 1.8 m, sand from 1.8 m to 0.4 m, then asphalt to surface.
	STR-09	None installed	Borehole backfilled with bentonite holeplug to 1.5 m, sand and gravel from 1.5 m to 0.45 m, concrete to 0.15 m, then asphalt to surface.
East Approach	STR-10	None installed	Borehole backfilled with bentonite holeplug to 0.9 m, sand and gravel from 0.9 m to 0.15 m, asphalt to surface.

Piezometers have been decommissioned in accordance with O. Reg. 903 in September 2012.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are summarized on the Record of Borehole sheets in Appendix A and shown on the figures contained in Appendix B.

Point load tests were carried out on selected samples of intact bedrock upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Results of point load tests on the rock core samples are included in Appendix B and on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

In general terms, the stratigraphy encountered at this site consists of pavement structure overlying granular embankment fill. A layer of sandy silt with organics was encountered below the fill at the west approach. On the east side of the river, native sand and silt till was contacted below the granular fill. A layer of cobbles and boulders was contacted below the granular fill at the west abutment and below the sand and silt till at the east abutment. Grey metasedimentary bedrock as well as auger refusal on probable bedrock were encountered below the fill, the native sand and silt till and layer of cobbles and boulders at depths ranging from 2.7 m to 9.4 m (elevations 332.8 to 338.0).

More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

Pavement structure was encountered in all the boreholes, which were drilled through the existing Highway 11 roadway at this site. Boreholes STR-01 and STR-10 revealed a 150-mm thick layer of asphalt overlying granular fill.

Boreholes STR-02 to STR-09, drilled through the approach slab, revealed 75 mm to 150 mm of asphalt over 225 mm to 275 mm of concrete. Granular fill was encountered below the concrete.

5.2 Sand and Sand and Gravel Fill

Two distinctive layers of road base and embankment fill were encountered below the asphalt pavement in the boreholes:

- Brown to dark brown sand and gravel containing trace to some silt and clay, occasional cobbles and occasional organics.
- Brown sand containing trace gravel to gravelly, some silt and clay and occasional organics.

A layer of organics, approximately 200 mm thick, was encountered within the fill at 0.9 m depth (elevation 341.3) in Borehole STR-10.

The thickness of the granular fill ranged from 1.9 m to 3.9 m.

The depth to the base of the granular fill ranged from 1.9 m to 4.3 m (elevations 337.2 to 338.6).

SPT 'N' values recorded in the sand/sand and gravel fill ranged from 4 to 25 blows for 0.3 m penetration, indicating a loose to compact relative density. Higher SPT 'N' values of 33 to 54 blows per 0.3 m of penetration and 50 blows per 0.1 m of penetration, indicating a dense to very dense relative density, were measured in Boreholes STR-04, STR-05, STR-06, STR-08 and STR-10 near elevations 338.0 to 339.0, which may indicate the presence of cobbles within the fill. In 1986, two test pits excavated by MTO revealed numerous cobbles and boulders within the sand fills (see Reference 1). It must be recognized that embankment fills are heterogeneous and may contain obstructions such as cobbles, boulders and rockfill.

The moisture content of samples of the sand/sand and gravel fill generally ranged from 3% to 26%. A moisture content of 37% was measured in Borehole STR-09 near elevation 339.0. A moisture content of 164% was measured in the organic layer encountered in Borehole STR-10.

Grain size distribution curves for sand fill and sand and gravel fill samples are presented on the Record of Borehole sheets and on Figures B1 and B2 of Appendix B..

The results of the laboratory tests are summarized as follows:

Soil Particles	Percentage (%) Sand to Gravelly Sand Fill	Percentage (%) Sand and Gravel Fill
Gravel	1 to 26	44
Sand	60 to 88	41
Silt and Clay	11 to 19	15

5.3 Sandy silt with organics

A layer of dark brown to black sandy silt with organics was encountered below the sand fill at 3.5 m depth (elevation 338.6) in Borehole STR-01, which was drilled at the west approach.

The thickness of the sandy silt with organics was 1.4 m.

The depth to the base of this layer was 4.9 m (elevation 337.2). Borehole STR-01 was terminated, below this layer, at 4.9 m depth, upon auger refusal on probable bedrock or boulder.

An SPT 'N' value of 13 blows for 0.3 m penetration, indicating a compact relative density was recorded in the sandy silt with organics.

The moisture content of samples of the sandy silt with organics ranged from 11% to 31%.

5.4 Sand and Silt Till

Native brown to grey sand and silt till containing trace to some clay, trace gravel and occasional cobbles was contacted in Boreholes STR-06 to STR-10, drilled near the east abutment and east approach. Cobbles and boulders were encountered below the sand and silt till near elevation 334.3 in Borehole STR-07.

The thickness of the sand and silt till ranged from 2.3 m to 3.9 m.

In Boreholes STR-06 to STR-08, the depths to the base of the sand and silt till layer ranged from 5.1 m to 8.2 m (elevations 333.9 to 334.3). Boreholes STR-09 and STR-10 were terminated upon refusal on probable bedrock or boulders, encountered below the sand and silt till, at 6.8 m and 6.4 m depth (elevations 335.4 and 335.8).

SPT 'N' values recorded in the sand and silt till layer ranged from 6 to 61 blows per 0.3 m of penetration, indicating loose to very dense relative density. Higher SPT 'N' values of 50 to 100 blows per 0.075 m of penetration, indicating a very dense relative density were measured near the base of the sand and silt till layer in Boreholes STR-06, STR-08 and STR-10, and in Borehole STR-07 where cobbles and boulders were encountered.

The moisture contents of the sand and silt till samples ranged from 7% to 21%.

Three samples of the sand and silt till were selected for gradation analysis and the results are summarized below. These results are also presented on the Record of Borehole sheets in Appendix A and on Figure B3 of Appendix B.

Soil Particles	Percentage (%)
Gravel	0 to 8
Sand	37 to 38
Silt	35 to 45
Clay	18 to 19

Glacial tills inherently contain cobbles and boulders.

5.5 Cobbles and Boulders

A layer of cobbles and boulders with some sand was contacted below the approach embankment fill at 3.9 m, 2.1 m and 2.8 m depth (elevations 338.2, 337.9 and 337.2) in Boreholes STR-03, STR-04 and STR-05 drilled at the west abutment. The thickness of this layer varies from 0.2 m to 1.7 m. A 1.5-m thick layer of cobbles and boulders was encountered at 5.1 m depth (elevation 334.3) in Borehole STR-07, drilled at the east

abutment. The layers of cobbles and boulders were encountered immediately above the bedrock. Coring through the cobbles and boulders was required to advance these boreholes.

In Boreholes STR-03, STR-04 and STR-05, drilled at the west abutment, the depths to the base of the cobbles and boulders were 5.6 m, 2.7 m and 3.0 m (elevations 336.5, 337.3 and 337.0), respectively. At the east abutment, in Borehole STR-07, the depth to the base of the cobbles and boulders was 6.6 m (elevation 332.8).

An SPT 'N' value of 100 blows for 0.2 m of penetration was recorded in this layer in Borehole STR-03.

Based on depths to refusal and elevations of top of bedrock encountered in adjacent boreholes, it is possible that a layer of cobbles and boulders exists immediately above the bedrock in Boreholes STR-02, STR-09 and STR-10.

5.6 Bedrock and Auger Refusal

The overburden soils described above are underlain by metasedimentary bedrock. The bedrock was grey with occasional white bands. Occasional mechanical breaks and horizontal joints were noted throughout the bedrock cores.

Bedrock was proved by coring in Boreholes STR-03 to STR-08. Boreholes STR-01, STR-02, STR-09 and STR-10 were terminated upon auger refusal on probable bedrock or boulders. Table 5.1 summarizes depths and elevations to the top of bedrock and auger refusal from the present investigation and the 1986 investigation (Boreholes 1 to 4).

Table 5.1 – Depths and Elevations of Top of Bedrock and Auger Refusal on Probable Bedrock or Boulders

Foundation Element	Borehole	Top of Bedrock/Auger Refusal on probable bedrock or boulders	
		Depth (m)	Elevation (m)
West Approach	STR-01	4.9	337.2
	3	5.2	336.8
West abutment	STR-02	4.1	338.0
	STR-03	5.6*	336.5*
	STR-04	2.7*	337.3*
	STR-05	3.0*	337.0*
	4	4.7	337.4
East abutment	STR-06	5.4*	334.3*
	STR-07	6.6*	332.8*
	STR-08	8.2*	333.9*
	STR-09	6.8	335.4
	2	6.4	335.7
East Approach	STR-10	6.4	335.8
	1	6.6	335.5

*Bedrock proved by coring

Core recovery in the bedrock was 100%. The RQD values ranged from 0% to 94%, indicating very poor to excellent rock quality. An RQD of 40% was measured in Borehole STR-04, Run 1, indicating a poor rock quality. RQD values of 0% were noted in Boreholes STR-08 Run 1, STR-06 Runs 1 to 4 and STR-07 Run 1 and 4. The rock quality of the upper 3.0 m of rock at the east abutment appears to be of very poor quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 8. FI greater than 10 was noted in Borehole STR-06.

The estimated unconfined compressive strength of the rock cores (average per Run) generally ranged from 77 MPa to 255 MPa, indicating a strong to very strong rock. Low unconfined compressive strengths ranging from 26 MPa to 38 MPa, indicating a weak to medium strong rock, were estimated in Boreholes STR-03 and STR-08. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results are presented in Appendix B.

5.7 Water Levels

Water levels were monitored in the open boreholes during and upon completion of drilling. Two standpipe piezometers were installed in Boreholes STR-03 and STR-08 to

monitor water levels after completion of drilling. The water levels measured in the piezometer and open boreholes are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Foundation Unit	Borehole	Date	Water Level (m)		Comments
			Depth	Elevation	
West Approach	STR-01	July 27, 2011	3.7	338.4	Open borehole
West Abutment	STR-02	July 27, 2011	3.4	338.7	Open borehole
	STR-03	August 5, 2011	3.9	338.2	Open borehole
		November 28, 2011	4.6	337.5	In piezometer
East Abutment	STR-06	September 27, 2012	1.9	337.8	Open borehole
	STR-08	November 28, 2011	4.1	338.0	In piezometer
	STR-09	July 26, 2011	5.9	336.3	Open borehole

Piezometric readings indicate that the water level is near elevations 337.5 to 338.0.

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

Preliminary GA drawing indicates that water level in the Sturgeon River was near Elevation 338.1 in April, 2011.

5.8 Data from previous investigation, 1986 (Reference 1)

Boreholes 1 to 4 were drilled at this site during the previous investigation conducted in 1986. Data from the 1986 investigation revealed that the subsurface soils conditions are consistent with the data obtained by Thurber. The 1986 boreholes were advanced through the embankment fill and into the underlying native materials. The subsurface conditions encountered were as follows:

- A 100-mm thick layer of asphalt was encountered surficially in the boreholes.
- Below the asphalt, sand fill containing silt, gravel and numerous cobbles and boulders was contacted. Augering through the fill was difficult due to the presence of boulders. The thickness of the fill varied from 2.7 m to 2.9 m. Two test pits completed on either side of the existing structure revealed the presence of numerous cobbles and boulders. The boulders were up to 0.5 m in diameter. Boulders and cobbles were also surficially observed on the river banks.
- A 0.6-m thick layer of organics was encountered below the fill in Borehole 2, drilled at the east abutment.

- Native compact to very dense sand containing some silt, trace gravel and organics was contacted below the fill in Boreholes 1, 3 and 4 and below the organics in Borehole 2.
- Very stiff to hard glacial till consisting of, heterogeneous mixture of gravel, sand, silt and clay was encountered below the sand layer in Boreholes 1 and 2, drilled at the east abutment and east approach. The thickness of this layer was less than 3.0 m.
- Boreholes were terminated upon refusal to auger on probable bedrock, at depths ranging from 4.7 m to 6.6 m.
- Groundwater level was noted at elevation 338.3 in 1986.

6 MISCELLANEOUS

Borehole locations for the first stage of the investigation were selected and established in the field by Thurber Engineering Ltd. MMM Group Limited surveyed the borehole locations and provided the co-ordinates and the ground surface elevations.

For the second investigation, MRC provided plan drawings to obtain the co-ordinates and the ground surface elevations for the boreholes.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario and TBT Engineering from Thunder Bay, Ontario supplied truck mounted CME 75 and CME 55 drill rigs and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations in the field were supervised on a full time basis by Mr. Jason Mei, Mr. Stephane Loranger, C.E.T. and Mr. Mark Farrant, P. Eng. of Thurber Engineering Ltd.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng.

Interpretation of the data and preparation of the report was carried out by Ms. R. Palomeque Reyna, P.Eng. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for design of a new bridge to replace the existing bridge located at the crossing of Highway 11 over the Sturgeon River, approximately 18 km east of Jellicoe and 30 km west of Geraldton, in the District of Thunder Bay, Township of Colter, Ontario.

The Sturgeon River bridge was constructed in 1941 and underwent rehabilitation in 1987. At present, Highway 11 crosses the Sturgeon River on a three-span structure supported on two abutments and two piers. The piers are supported on footings and the abutments on timber piles and steel H-piles (310 x 110). The HP 310x110 piles were installed at the abutments during the 1987 rehabilitation program. Construction of new wingwalls and new approach slab, placement of new granular backfill and installation of new bearings, among other repair works, were also part of the 1987 rehabilitation program.

The length and width of the bridge are 41.0 m and 10.9 m, respectively. The span lengths are 12 m, 17 m and 12 m. The highway grade is near elevation 342.1. The existing embankments are approximately 4.0 m high.

Based on the preliminary General Arrangement (GA) drawing provided by MRC, a single-span structure supported on two abutments is proposed. GA drawings show that the abutments are proposed to be founded on 610-mm diameter steel pipe piles filled with concrete with a sheet pile wall driven just behind the pipe piles. Precast pre-stressed box girders will then be spanning the abutment pile caps to support the deck finishing. The total length of the structure will be 35.0 m. The proposed structure will be approximately 11.4 m wide. The existing structure will be

replaced maintaining the same alignment for the new structure. It has been indicated by MRC that the highway grade will be raised approximately 300 mm.

It is understood that the existing structure, including the piers, will be removed. The existing timber piles and the H-piles at the abutments will be left in place. The GA shows that the new abutments will be built 2.7 m to 3.5 m in front of the existing abutments.

The discussion and recommendations presented in this report are based on the information provided by MRC and on the factual data obtained in the course of the investigations.

8 STRUCTURE FOUNDATIONS

In general terms, the stratigraphy encountered at this site during the present investigation consists of pavement structure over loose to compact sand and sand and gravel fill. The fill contained occasional cobbles and organics. The thickness of the fill ranged from 1.9 m to 3.9 m. A 1.4-m thick layer of sandy silt with organics was encountered below the fill at the west approach. At the east abutment, native loose to very dense sand and silt till was contacted below the fill. A layer of cobbles and boulders was contacted at the west abutment below the fill and below the sand and silt till in Borehole STR-07 drilled on the north side of the east abutment. The layer of cobbles and boulders was 0.2 m to 1.7 m thick on the west abutment and 1.5 m thick on the east abutment. The cobbles and boulders layer was encountered immediately above the bedrock. Metasedimentary bedrock as well as auger refusal on probable bedrock or cobbles/boulders was encountered below the fill, native soils and cobbles and boulders. The bedrock was contacted and proved in six boreholes at the east and west abutments at 2.7 m to 8.2 m depth (elevations 332.8 to 337.3). The upper 2.0 to 3.0 m of bedrock at the east abutment appears to be of poor quality.

The 1986 investigation (Reference 1), indicated that augering through the fill was difficult due to the presence of numerous cobbles and boulders. The size of the boulders was up to 0.5 m in diameter. These obstructions in the form of cobbles and boulders and a distinct layer of cobbles and boulders at the west and east abutments immediately above the bedrock will impact the installation of driven pipe piles, H-piles, augered caissons and sheet piles.

Piezometric readings indicate that the water level is near elevations 337.5 to 338.0. Preliminary GA drawing indicates that water level in the Sturgeon River was near Elevation 338.1 in April, 2012. The 1986 investigation indicates that water level was found at elevation 338.3.

Based on existing site conditions, initial consideration was given to the following foundation types:

- Spread footings on native soils or on bedrock
- Augered Caissons (drilled shafts)
- Drilled-in pipe piles socketed into bedrock
- Driven steel H-piles founded on bedrock or upon refusal on cobbles and boulders

A comparison of the foundation alternatives based on advantages and disadvantages of each one is included in Appendix D.

8.1 Spread Footings on Native Soils or Bedrock

Consideration was given to supporting the structure on spread footings founded on native soils or bedrock, however these options are not recommended due to the following reasons:

- Footings on native soils - Excavation of the fill, sandy silt with organics and loose sand and silt till will be required to place footings on the native competent soils. This excavation will be approximately 2.0 m to 4.0 m deep and will extend in cohesionless soils below the ground water level. Such an excavation will require temporary shoring and dewatering so as not to destabilize the base and sides of the excavation.
- Footing on bedrock - Founding the footings on bedrock will also require deep excavations requiring temporary shoring and dewatering.

For both options, installation of the temporary shoring (sheet pile cofferdam) will be difficult due to the presence of cobbles and boulders in the embankment fill, in the native sand and silt till and above the bedrock.

The footings will require scour and erosion protection and footing excavations are likely to have environmental impact on the river.

In light of the above factors, these spread footings options were not further developed.

8.2 Socketted Drilled Shafts/Caissons

Caissons socketted into bedrock may be employed to support the structural loads at this site.

Bedrock was encountered and proved by coring at elevations given in Table 8.1.

Table 8.1 - Depths and Elevations of Top of Bedrock at Abutments

Foundation Element	Borehole	Top of Bedrock	
		Depth below existing ground surface (m)	Founding Elevation
West abutment	STR-04	2.7*	337.3*
	STR-05	3.0*	337.0*
East abutment	STR-06	5.4*	334.3*
	STR-07	6.6*	332.8*

*Bedrock proved by coring

The vertical geotechnical resistance computed for 0.76 m, 0.91 m, 1.2 m and 1.5 m diameter sockets for various socket depths below the bedrock surface (defined in Table 8.1) are presented in Table 8.2.

Table 8.2 – Recommended Resistance Values for Caisson Design

Caisson Diameter (m)	Socket Length below bedrock surface (m)	Factored Geotechnical Resistance at ULS (kN)
0.76	1.5	2,500
	2.0	3,250
	3.0	5,000
0.91	1.5	3,000
	2.0	4,000
	3.0	6,000
1.2	1.5	3,500
	2.0	5,000
	3.0	7,500
1.5	1.5	4,500
	2.0	6,500
	3.0	9,500

The SLS condition will not govern for caissons socketted into the rock.

The selection of a suitable socket depth will be governed by axial loads, lateral load and maximum shear and moment demand on each caisson. The depth of rock socket should not be less than 1.5 m and the axial load, shear and moment demands may require a deeper depth of rock socket.

The assessment of rock socket depth allows for the presence of some weathered and broken up rock just below the bedrock surface. Since the elevation of the bedrock surface is variable across the site and there is evidence of cobbles and boulders immediately above the bedrock at the west abutment, it is critical to determine in the field during inspection of caisson installation that the entire depth of socket is formed in sound bedrock and not partly in cobbles and boulders and partly in bedrock. This issue is addressed in an NSSP included in Appendix E.

8.2.1 Caisson Socket Lateral Resistance

The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by:

$$P_p = 6 \cdot c \cdot D \cdot L$$

Where

c = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

L = Depth of socket in rock, m

D = Socket diameter, m

8.2.2 Caisson Socket Installation

Caisson installation must be in accordance with OPSS 903.

Caisson installation at this site will require excavation through cohesionless sandy silt, a layer of cobbles and boulders and sand and silt till below the groundwater table and construction of sockets in the underlying bedrock. The installation of caissons at this site must consider the following issues:

- Installation method must prevent collapse of caisson sidewall and washing of cohesionless soils into the rock socket.
- The caisson installation equipment must be able to get through cobbles, boulders and large fragments of rock in the soils above the bedrock.
- The strength and hardness of the bedrock at this site must be considered when selecting equipment to excavate the rock socket. Blasting to facilitate rock removal is not permitted.

Selection of the methods and equipment employed to address the above issues is the responsibility of the Contractor. The contract documents must contain a statement to alert bidders of the above facts. The wording for an NSSP addressing this issue is included in Appendix E. This NSSP must be included in the tender documents.

8.3 Drilled-in pipe pile socketed into bedrock

The GA drawing indicates that the proposed foundation alternative to support the bridge consists of 610-mm diameter drilled-in steel pipe piles socketed into bedrock and filled with concrete.

The subsurface conditions reported in the current investigation as well as the 1986 investigation indicate presence of cobbles and boulders in the embankment fill and 0.2 m to 0.6 m thick layer of cobbles and boulders above the bedrock surface at west abutment (See Boreholes STR-04 and STR-05). In Borehole STR-07, drilled at the east abutment, cobbles and boulders were encountered below the sand and silt till near elevation 334.3.

Driving pipe piles will be difficult and it is recommended that pipe piles be drilled in and not driven in these soil conditions. The pipe piles should be socketed into bedrock.

For drilled in pipe piles socketed into bedrock, the vertical geotechnical resistance estimated for 304 mm, 455 mm and 610 mm diameter pipe piles for a socket depth of 1.5 m below the bedrock surface are presented in Table 8.3.

Table 8.3 – Recommended Resistance Values for Drilled in Pipe Piles

Pipe pile diameter (mm)	Socket length below bedrock surface (m)	Factored Geotechnical Resistance at ULS (kN) of each pipe pile filled with concrete
304	1.5	1,400
455	1.5	2,600
610	1.5	4,000

The factored Geotechnical Resistance at ULS has been reduced to account for the following factors:

- The tip of the pipe pile will not be in direct contact with the bedrock. The area of contact between the bedrock and the teeth of the cutting shoe at the tip of a pipe pile will be less than the full contact area between a shoeless pipe pile and the bedrock.
- Cleaning the rock socket from debris and water will be difficult.

The above resistances are for pipe wall thickness of 12.5 mm, steel yield strength of 245 MPa and concrete strength of 30 MPa. The depth of the socket will be governed by the axial and lateral resistance requirement, base fixity requirement and shear and moment demand for each pile. The structural resistance of the pipe pile must be checked by the structural engineer.

8.3.1 Pipe Pile Installation

Installation of pipe pile must follow the specification of OPSS 903.

The method of installation of the pipe piles is the responsibility of the Contractor. One option for installing the pipe piles is to drill them in by a Rotary Duplex Drilling Method. Contractor's drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or large rock fragments in the overburden soils. Care must be exercised while drilling the socket within the bedrock; the drilling methodology must be capable of excavating the bedrock to the specified socket

dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate rock removal is not permitted.

The drilling method must also maintain sidewall stability of the drilled hole and allow cleaning of the socket without cohesionless soils running into the socket.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile. It is recommended that the annular space between the pipe pile and socket wall be grouted to the bedrock surface to achieve fixity.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

A NSSP addressing these issues is included in Appendix E. This NSSP must be included in the tender documents.

8.3.2 Lateral Resistance for Pipe Pile Sockets

The ultimate passive force that can be mobilized by the embedded portion of a pipe pile socket within rock is constant with depth and is given by the lateral resistance formula presented in Section 8.2.1 of this report.

8.4 Steel H-piles driven to refusal on bedrock or cobbles and boulders

Consideration was given to supporting the abutments on driven steel H-piles founded on bedrock or upon refusal in cobbles and boulders above the bedrock.

In general, it is anticipated that the piles will encounter refusal at the bedrock surface. However, in Boreholes STR-04 and STR-05, drilled at the west abutment and in Borehole STR-07 drilled at the east abutment, a layer of cobbles and boulders was contacted immediately above the bedrock. Coring through the layer of cobbles and boulders was required to extend the boreholes to bedrock.

Although it is anticipated that driven H-piles may reach bedrock at the abutments, the resistance values have been reduced to account for the possibility that some piles may encounter refusal in the cobbles and boulders layer above the bedrock.

The anticipated pile tip elevations, soil conditions, vertical, factored geotechnical resistance at Ultimate Limit States (ULS_f) and geotechnical resistance at Serviceability Limit States (SLS) for H-piles driven to refusal on cobbles and boulders are presented in Table 8.4.

Table 8.4 – Estimated Pile Tip Elevation and Recommended Pile Resistance Values

Foundation	Borehole	Soil Conditions at Pile Tip	Estimated Pile Tip Elevation	Pile Section HP 310 x 110	
				Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS
West abutment	STR-04	Cobbles and boulders	337.8	1,600	1,400
	STR-05		337.1		
East abutment	STR-06	Bedrock	334.3		
	STR-07	Cobbles and boulders	334.2		

The length of driven H-piles, particularly at the west abutment will be very short and may not provide sufficient lateral stability or fixity.

The factored structural resistance of the piles at ULS must be checked by the structural designer as per Section 6.8.8 of the CHBDC.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any fills through which the piles will be driven.

8.4.1 Pile Tips

The tips of all H-piles should be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

8.4.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

In general The Contract Documents should contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders within the highway embankment and immediately above the bedrock.
- The possibility of some driven piles meeting refusal on a large boulder. When boulders are encountered within a shallow depth, pre-drilling through the boulders may be required to advance and install the piles to the required pile tip elevation. Rock coring and rock breaking equipment may be required in addition to augering equipment. A NSSP addressing these issues is included in Appendix E.

We understand that the proposed bridge design may require that the deviation at the top of the pile be limited to 12 mm. Use of a driving template or other means may be required to achieve the specified maximum deviation.

8.4.3 Piles driven to refusal on cobbles and boulders

For piles driven to refusal on cobbles and boulders, pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are within 1.0 m of the bearing stratum. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile”. “R” must have a minimum value of twice the design load at ULS.

8.4.4 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

8.4.5 Lateral Resistance for H-piles

For cohesionless soils, the lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \cdot z / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \quad (\text{kPa})$$

where z = depth of embedment of pile in metres

D = pile width/diameter in metres

n_h = value from Table 8.5

γ = unit weight (Table 8.5)

K_p = passive earth pressure coefficient (Table 8.5)

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width/diameter (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \cdot L \cdot D$. This represents the ultimate load at which the pile

fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in a HP 310x110 pile driven to refusal on bedrock or in the layer of cobbles and boulders be limited to no more than 110 kN at ULS and 40 kN at SLS.

Table 8.5 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	K_p	Unit Weight (kN/m ³)	Soil Conditions
West Abutment	OGI to 338.0	2,500	3.3	21	Sand/sand and gravel loose to compact (FILL)
	338.0 to 337.0	7,000	3.7	11*	Cobbles and boulders
East Abutment	OGI to 337.2	2,500	3.3	21	Sand/sand and gravel loose to compact (FILL)
	337.2 to 334.3	4,000	3.3	11*	Sand and silt till, loose to very dense
	334.3 to 332.8	7,000	3.7	11*	Cobbles and boulders

*Buoyant unit weight below the water table.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.5 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions drilled-in pipe piles socketed into bedrock and filled with concrete is considered the suitable foundation type for supporting the bridge at this site.

8.6 Depth of Frost Protection

The design depth of frost penetration at this site is 2.6 m.

9 SHEET PILE WALLS

Steel sheet pile walls are proposed adjacent to the H-pile foundations at each abutment. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill. Alignment of the sheet pile walls should be carefully selected so that during installation of the sheet piles, they do not encounter the timber and steel piles below the existing abutment.

Numerous cobbles and boulders were encountered in the embankment fill during the 1986 investigation, by MTO (Reference 1). In Boreholes STR-03 to STR-05, drilled at and behind the west abutment, a distinct layer of cobbles and boulders was encountered just above the bedrock, approximately 2.1 m to 3.9 m below ground surface. Cobbles and boulders were also encountered below the sand and silt till at 5.1 m depth in Borehole STR-07 drilled at the east abutment. The presence of cobbles and boulders in the highway embankment fill, and immediately above the bedrock at the west and east abutments will potentially have an adverse impact on the installation of sheet piles. At some locations the sheet piles may not be able to penetrate the cobbles and boulders and refusal may be encountered at varying depths. The Contractor shall be prepared to remove, pre-auger, drill through and/or penetrate these obstructions and extend the sheet piles to the design depth.

The available depth of sheet pile embedment, particularly at the west abutment, may be quite limited. The design of sheet piles should consider the potential issue of limited depth of embedment of the sheet piles from a lateral stability point of view. Use of EPS or light weight fill may be considered as backfill behind the sheet pile wall at the west abutment.

An NSSP for sheet pile installation is included in Appendix E.

Sheet piles should be provided with sheet pile tip protector to minimize any tip damage and to provide additional lateral toe resistance. Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The native soils in front of the sheet pile should be protected from river erosion so that the sheet piles do not lose lateral support.

Backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A or Granular B Type II or III material. All granular material should meet the specifications of OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to

be used adjacent to retaining structures should be restricted in accordance with OPSS 501. These recommendations also apply for backfilling of a conventional abutment wall.

Earth pressures acting on the sheet pile walls or conventional abutment wall may be assumed to be triangular and to be governed by the characteristics of the backfill and the existing native soils. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$P_h = K(\gamma h + q)$$

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the sheet pile wall or abutment wall are dependent on the material used as backfill. Typical values are shown in Table 9.1.

Table 9.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$, $\gamma = 22.8 \text{ kN/m}^3$		Existing sand/sand and gravel Fill, Native sand and silt till and OPSS Granular B Type I or Type III $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in Table 9.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or 1.7 m for Granular A or Granular B Type II and Type III.

10 RETAINED SOIL SYSTEMS

Retained Soil System (RSS) could be considered as an alternative to the sheet pile wall to provide resistance to lateral earth pressures from the approach fill.

In general, RSS walls used in conjunction with the new abutments must be “High Performance” and “High Appearance”. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

It is important that the RSS walls be founded on soil capable of supporting the imposed loading and limiting settlements under the RSS wall to acceptable magnitudes. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. It is critical that the total and differential settlement of the RSS walls due to compression of the foundation soils and embankment fill be limited to values acceptable to the wall supplier. It is understood that RSS walls are able to tolerate up to 50 mm of total settlement and a differential settlement of less than 15 mm over a 1.5 m wide panel. These values must be confirmed with the RSS wall supplier.

The RSS mass must be constructed in the dry and the excavation must be unwatered as necessary to achieve the dry conditions. GA drawing indicates that water level in the river is at elevation 338.1. Piezometric readings indicate that the water level is near elevations 337.5 to 338.0. Therefore, in order to avoid excavation of cohesionless soils below the water level in the river, RSS wall base or the granular pad below the RSS wall should be founded at elevation 338.2.

Taking account of the groundwater level at this site, the RSS wall would be founded on native sand or existing embankment sand fill at elevation 338.2 as presented in Table 10.1.

Table 10.1 –Elevation at Underside of Wall Base

Location		Borehole	RSS founded level	Soil conditions
West abutment	North	STR-03	338.2	Very dense cobbles and boulders, sand
		STR-05		Very dense sand fill
	South	STR-02		Compact sand fill
		STR-04		Very dense sand fill with cobbles and boulders
East abutment	North	STR-07	338.2	Loose to compact sand fill
		STR-09		Compact sand fill
	South	STR-06		Dense sand fill
		STR-08		Very dense sand and gravel fill

Since the RSS wall base will be close to the river level, and the native soils are erodable, it is critical that the RSS wall base be protected from any erosion and undermining by the river.

As the founding level (elevation 338.2) consists of existing fill with variable density and cobbles and boulders at one borehole location, it is recommended that an engineered fill pad, 0.5 m thick, be placed along the base of the proposed RSS walls. Engineered fill placed under the RSS mass to achieve the design founding level must consist of OPSS Granular “A” compacted to 100% of its SPMDD at a moisture content within 2% of optimum.

At one location, along the north side of the east abutment, the sand fill is in loose to compact state at the proposed founding level. Therefore, the entire subgrade area under the RSS wall should be uniformly densified using a plate tamper. An NSSP for subgrade densification is included in Appendix E.

The subgrade must be competent and free of organics, soft or deleterious soils.

A RSS wall founded on the engineered fill should be designed for a factored bearing resistance of 300 kPa at ULS and a bearing resistance of 200 kPa at SLS.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2006 Section 6.7.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. For an RSS block founded on native cohesionless soils, coefficient of

sliding friction of 0.45 may be used. Sliding resistance along the base of the wall on engineered granular fill may be estimated using an ultimate friction coefficient of 0.55.

The global stability of the RSS systems must be reviewed after the details of the design are confirmed.

11 EXCAVATION AND GROUNDWATER CONTROL

If any earth excavation is required, it must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 soils below the water table.

The excavation must be carried out in accordance with OPSS 902.

Piezometric readings indicate that the water level is near elevations 337.5 to 338.0. Preliminary GA drawing indicates that water level in the Sturgeon River was near Elevation 338.1 in April, 2012. The 1986 investigation indicates that water level was found at elevation 338.3.

Based on the preliminary GA for the bridge structure and the use of pile foundations, it is not expected that work at the abutments will require excavation below the river/groundwater level.

It is recommended that excavation for removal of existing structures be maintained above the water level in the river. Any excavation below the groundwater level/river level without prior dewatering is not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work.

In general, the design of the dewatering system should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility.

12 APPROACH EMBANKMENTS

Based on site observations and GA drawing provided by MRC, it was estimated that the existing approach embankments are approximately 4.0 m high with forward slopes near inclinations of about 2.6H:1V. The foundation soils governing stability of the approach embankments consist generally of native loose to very dense sand and silt till and cobbles and boulders overlying bedrock.

Communication with MRC indicates that the existing Highway 11 grade will be raised approximately 300 mm.

It is understood that additional fill will be required within the sheet pile enclosure. This new fill is expected to have a maximum thickness of about 2.0 m and extend for a length of up to 6.0 m behind the new sheet pile abutment wall. The sides of the new approach fill will be contained by sheet pile walls installed along each edge of the road.

Any embankment construction and widening should be carried out in accordance with OPSS 206. Prior to placement of new fill, existing sloped embankment surfaces should be appropriately benched as per OPSD 208.010, after stripping of vegetation/organics, soft soils or otherwise unsuitable materials.

Comments regarding stability of embankment slopes and settlement of the foundations soils are provided in the following sections.

12.1 Slope stability

The global, internal and surficial stability of the approach embankment fills depends on the slope geometry and also to a large degree on the material used to construct the embankment.

The existing embankments bearing on the foundation soils at this site appear to be performing satisfactorily under the existing conditions. Placement of approximately 300 mm of new fill to raise the road grade is expected to have minimal impact on the stability of the embankments.

The additional approach fill to be placed behind the new abutment will be supported within a sheet pile enclosure.

The GA indicates that the sheet piles are proposed to be driven to bedrock and therefore no stability issues are anticipated.

12.2 Settlement

The placement of approximately 2.0 m of new fill behind the sheet pile abutments and approximately 300 mm of granular fill to raise the existing highway grade will induce immediate (elastic) settlement in the existing non-cohesive fill and native soils.

Based on these analyses, the anticipated immediate settlement at the bridge approaches is in the order of 20 mm. Due to the non-cohesive nature of the foundation soils, this settlement will be immediate and essentially complete when construction of the bridge is completed.

13 EROSION PROTECTION

Rock protection should be provided over all surfaces that may be in contact with the river flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

14 ROADWAY PROTECTION

The bridge construction will be done in stages in order to keep at least one highway lane operational. Roadway protection will be required to facilitate staging of removals and support the existing Highway 11.

Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile and timber lagging walls is one option to provide temporary support to the roadway.

It may be difficult to install the sheet piles through the layer of fill containing cobbles/boulders.

The ultimate choice of a roadway protection system should be left to the Contractor.

The following parameters apply for design of the temporary shoring system:

γ	=	21 kN/m ³	(bulk unit weight)
γ_w	=	11 kN/m ³	(submerged unit weight under groundwater table)
K_a	=	0.31	(Active pressure coefficient for: road embankment fill and native sand and silt till)
K_p	=	3.3	(Passive pressure coefficient for: road embankment fill and native sand and silt till)
h_w	=	338.1	(Ground water level)

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall and these factors must be considered when designing the shoring system.

Temporary groundwater and surface water control measures may be required during construction.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0

- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ $\gamma = 22.8 \text{ kN/m}^3$	Existing sand/sand and gravel Fill, Native sand and silt till and OPSS Granular B Type I $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32
Passive (K_{PE})	3.7	3.2
At Rest (K_{OE})**	0.45	0.50

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method for cohesionless soils.

Using the method, it is estimated that under the existing conditions the foundation soils at the abutments are not prone to liquefaction.

16 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- Excavation of rock sockets for caisson and drilled-in pipe piles at this site must consider the issues of the sloughing of cohesionless soils and cobbles and boulders into the socket and hardness of bedrock. NSSP's addressing these issues are attached. The Contractor must be alerted to these NSSP's.

- The potential variability of the length of driven H-piles. Bedrock and auger refusal on probable bedrock or cobbles and boulders were contacted at elevations ranging from 337.0 to 337.3 at the west abutment and 332.8 to 334.3 at the east abutment.
- The embankment fill contains cobbles and boulders which may impact both H-pile and sheet pile installation. Also, a layer of cobbles and boulders was contacted above the bedrock. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the sheet piles to the design depth.
- If construction of RSS wall is selected, the RSS base should be maintained above groundwater level. The loose native soils and fill at the subgrade level, below the RSS wall bases, must be densified using a plate tamper to improve the subgrade conditions.

17 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. R. Palomeque Reyna and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Rocio Palomeque Reyna, P.Eng., M.Eng.
Geotechnical Engineer



Murray R. Anderson, P.Eng., M.Eng.
Senior Foundations Engineer



Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal



Appendix A

Record of Borehole Sheets (Present investigation)

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT ⁽¹⁾ 'N' VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

- 1) Laboratory Triaxial Testing
- 2) Field Insitu Vane Testing
- 3) Laboratory Vane Testing
- 4) SPT value
- 5) Pocket Penetrometer



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT "N" VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$


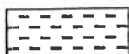



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
Fresh (FR)	No visible signs of weathering.		
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.		CLAYSTONE
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.		SILTSTONE
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.		SANDSTONE
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.		COAL
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.		Bedrock (general)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
		Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
		Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS	
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

RECORD OF BOREHOLE No STR-01

1 OF 1

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 736.8 E 282 728.1 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.07.27 - 2011.07.27 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) w P w w L				
342.1								20 40 60 80 100								
0.0	ASPHALT: (150mm)						342									
0.2	SAND and GRAVEL, trace silt Compact Brown Moist (FILL)		1	AS												
			1	SS	19		341									
340.6																
1.4	SAND, some gravel to gravelly, some silt and clay Compact to Loose Brown Moist (FILL) Occasional organics		2	SS	17		340									
			3	SS	7											
			4	SS	5		339									
338.6																
3.5	Sandy SILT, with organics, trace gravel, occasional cobbles Compact Dark Brown to Black Moist		5	SS	13		338									
			6	SS	100/											
337.2																
4.9	END OF BOREHOLE AT 4.8m UPON AUGER REFUSAL ON PROBABLE BOULDER OR BEDROCK. BOREHOLE OPEN TO 4.8m AND WATER LEVEL AT 3.7m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.9m, SAND AND GRAVEL TO 0.6m, CONCRETE TO 0.15m, THEN ASPHALT TO SURFACE.				0.125											

RECORD OF BOREHOLE No STR-02

1 OF 1

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 737.3 E 282 735.3 ORIGINATED BY JM
HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2011.07.27 - 2011.07.27 CHECKED BY MRA

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		
342.1														
0.0	ASPHALT: (150mm)						342							
0.2	CONCRETE: (225mm)													
341.7														
0.4	SAND and GRAVEL Compact Brown Moist (FILL)		1	AS										
			1	SS	16		341							
340.6														
1.5	SAND, some gravel to gravelly, some silt and clay Compact Brown Moist (FILL)		2	SS	16		340							
			3	SS	16									
	Occasional organics		4	SS	25		339							26 60 14 (SH+CL)
338.0														
4.1	END OF BOREHOLE AT 4.1m UPON AUGER REFUSAL ON PROBABLE BOULDER OR BEDROCK. BOREHOLE OPEN TO 4.1m AND WATER LEVEL AT 3.4m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.3m, SAND AND GRAVEL TO 0.6m, CONCRETE TO 0.15m, THEN ASPHALT TO SURFACE.													

+ 3 . X 3 : Numbers refer to 20
Sensitivity 15 0.5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No STR-03

1 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 743.3 E 282 732.5 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.05 - 2011.08.05 CHECKED BY MRA

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		
342.1														
0.0	ASPHALT: (150mm)						342							
0.2	CONCRETE: (250mm)													
341.7														
0.4	SAND and GRAVEL Loose Brown Moist (FILL)		1	SS	6		341							
			2	SS	7		340							
			3	SS	6		339							
			4	SS	5									
338.2														
3.9	COBBLES and BOULDERS, some sand Very Dense Grey Wet Cored through cobbles and boulders at 4.6m		5	SS	100/ 0-200		338							
							337							
336.5														
5.6	BEDROCK, metasedimentary, slightly to fresh weathered, grey with white bands, occasional sub-vertical and horizontal breaks Coring started at 5.6m Quartz seam (50mm) at 6.7m		1	RUN			336							
							335							
			2	RUN			334							
333.4														
8.7	END OF BOREHOLE AT 8.7m. WATER OBSERVED AT 3.9m DURING DRILLING. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS:													

Continued Next Page

+ 3 . X 3 Numbers refer to
Sensitivity 20
15 0.5
10 (%) STRAIN AT FAILURE

METRIC

[illegible]

+ 3, x 3: Numbers refer to Sensitivity

METRIC

[illegible]

ONTMT4S 1197.GPJ 10/19/12

+ 3, × 3: Numbers refer to Sensitivity

METRIC

[illegible]

RECORD OF BOREHOLE No STR-06

1 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 741.8 E 282 776.1 ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE Solid Stem Augers/NW Casing/Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.09.26 - 2012.09.27 CHECKED BY MEF

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 w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	
342.2	ASPHALT: (75mm)											
0.0												
0.1												
341.8	CONCRETE: (275mm)						342					
0.4	Air gap between underside of bridge deck and ground surface											
							341					
							340					
339.7	Rockfill and cobbles at surface											
2.5	SAND and GRAVEL Compact (FILL)		1	SS	12		339					
338.5	Layer of topsoil (50mm)											
3.7	SAND, some gravel, trace to some silt and clay Dense Brown Moist (FILL)		4	SS	33		338					21 60 19 (SI+CL)
337.8	SAND and SILT, trace to some gravel, trace clay Very Dense Brown Wet (TILL) Compact Grey		2	SS	40							
4.4			3	SS	14		337					
							336					
							335					
			5	SS	50/ 0.125							
334.3	Coring started at 7.9m											
7.9	BEDROCK, metasedimentary, grey with white bands		1	RUN			334					FI >10 RUN #1 TCR=78% SCR=0% RQD=0%
	Highly broken		2	RUN								>10 RUN #2 TCR=82% SCR=0% RQD=0%
	Sand and silt seam at 9.4m		3	RUN			333					>10 RUN #3 TCR=100%

Continued Next Page

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

RECORD OF BOREHOLE No STR-06

2 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 741.8 E 282 776.1 ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE Solid Stem Augers/NW Casing/Coring COMPILED BY AN
 DATUM Geodetic DATE 2012 09 26 - 2012 09 27 CHECKED BY MEF

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								WATER CONTENT (%)	
	Continued From Previous Page							20	40	60	80	100					
331.4	BEDROCK, metasedimentary, grey with white bands Sand and silt seam at 10.1m Highly broken		4	RUN			332										SCR=86% RQD=0%
10.8	END OF BOREHOLE AT 10.8m. WATER OBSERVED AT 1.9m BELOW GROUND SURFACE DURING DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO GROUND SURFACE, BRIDGE DECK BACKFILLED WITH CONCRETE TO 0.28m, THEN ASPHALT TO HIGHWAY SURFACE.																RUN #4 TCR=100% SCR=48% RQD=0%

RECORD OF BOREHOLE No STR-07

1 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 747.8 E 282 775.6 ORIGINATED BY MEF
HWY 11 BOREHOLE TYPE Tricone/NW Casing/Coring COMPILED BY AN
DATUM Geodetic DATE 2012.09.28 - 2012.09.28 CHECKED BY MEF

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 W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
342.2												
0.0	ASPHALT: (100mm)											
0.1												
341.8	CONCRETE: (275mm)											
0.4	Air gap between underside of bridge deck and ground surface											
339.4	Rockfill and cobbles at surface											
2.8	SAND and GRAVEL, trace silt Compact to Loose Grey (FILL)		3	SS	11							
			1	SS	10							
337.2			2	SS	5							
5.0	SAND and SILT, trace gravel, some clay Dense Grey Wet (TILL)											
			4	SS	41							
	Some cobbles and boulders (spoon bouncing, no recovery)		5	SS	50/							
334.3					0.050							
7.9	Coring through COBBLES (25mm to 125mm in diameter) in a sand and silt matrix		1	RUN								
332.8												
9.4	BEDROCK, metasedimentary, greenish grey with white bands, vertical and subvertical breaks		2	RUN								

Continued Next Page

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

RECORD OF BOREHOLE No STR-07

2 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 747.8 E 282 775.6 ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE Tricone/NW Casing/Coring COMPILED BY AN
 DATUM Geodetic DATE 2012.09.28 - 2012.09.28 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)					
	Continued From Previous Page							20 40 60 80 100							
	BEDROCK , metasedimentary, greenish grey with white bands, vertical and subvertical breaks		3	RUN			332	○ UNCONFINED + FIELD VANE						>10	RUN #3 TCR=87% SCR=60% RQD=40% UCS=45MPa (Average)
								● QUICK TRIAXIAL × LAB VANE						>10	RUN #4 TCR=95% SCR=45% RQD=10% UCS=52MPa (Average)
	Rubble zone: 75mm at 11.4m 100mm at 11.5m		4	RUN			331							>10	
														>10	
			5	RUN			330							>10	RUN #5 TCR=89% SCR=18% RQD=0%
329.5														>10	
12.7	END OF BOREHOLE AT 12.7m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO GROUND SURFACE, BRIDGE DECK BACKFILLED WITH CONCRETE TO 0.3m, THEN ASPHALT TO HIGHWAY SURFACE.														

RECORD OF BOREHOLE No STR-08

1 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 742.4 E 282 782.3 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.03 - 2011.08.05 CHECKED BY MRA

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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
342.1	ASPHALT: (150mm)						342							
0.0														
0.2	CONCRETE: (250mm)													
341.7														
0.4	SAND and GRAVEL, some silt and clay Compact Brown Moist (FILL)		1	SS	18		341							
			2	SS	19		340							
			3	SS	6		339							
	Very Dense Dark Brown Moist to Wet		4	SS	54		338							44 41 15 (SI+CL)
337.8														
4.3	SAND and SILT, some clay, trace gravel Loose Brown Moist (TILL)		5	SS	6		337							
							336							
	Occasional cobbles		6	SS	61		335							
	Very Dense		7	SS	100/0.075		334							
333.9														
8.2	BEDROCK, metasedimentary, moderately to slightly weathered, grey with white bands, sub-vertical breaks Coring started at 8.2m Sub-vertical joints at 8.3m, 8.4m Vertical joint (75mm) at 8.2m Horizontal joints (50mm) at 9.4m		1	RUN			333							RUN #1 TCR=100% SCR=100% RQD=0% UCS=83MPa (Average)
			2	RUN										RUN #2 TCR=100% SCR=79% RQD=58% UCS=63MPa (Average)

Continued Next Page

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

RECORD OF BOREHOLE No STR-08

2 OF 2

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 742.4 E 282 782.3 ORIGINATED BY SLL
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.08.03 - 2011.08.05 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
	Continued From Previous Page							20 40 60 80 100		W _P W W _L				
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100		20 40 60				

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No STR-10

1 OF 1

METRIC

W.P. 6056-10-00 LOCATION Sturgeon River Bridge N 5 506 748.8 E 282 787.9 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2011.07.26 - 2011.07.26 CHECKED BY MRA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE		WATER CONTENT (%) w _P w w _L				
342.2							20 40 60 80 100							
0.0	ASPHALT: (150mm)													
0.2	SAND and GRAVEL, trace silt Loose Brown Moist (FILL) Layer of organics at 0.9m (200mm)		1	AS		342								
			1	SS	9	341								
			2	SS	8	340								
			3	SS	7	339								
			4	SS	53	338								
338.1	Occasional cobbles, occasional organics Dense													
4.1	SAND and SILT, some clay, trace gravel Compact Brown to Grey Moist (TILL)		5	SS	22	337								
			6	SS	100/ 0.075	336								
335.8	Grey													
6.4	END OF BOREHOLE AT 6.4m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 6.4m AND DRY. BOREHOLE BACKFILLED WITH HOLEPLUG TO 0.9m, SAND AND GRAVEL TO 0.15m, THEN ASPHALT TO SURFACE.													

+³ . X³ : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

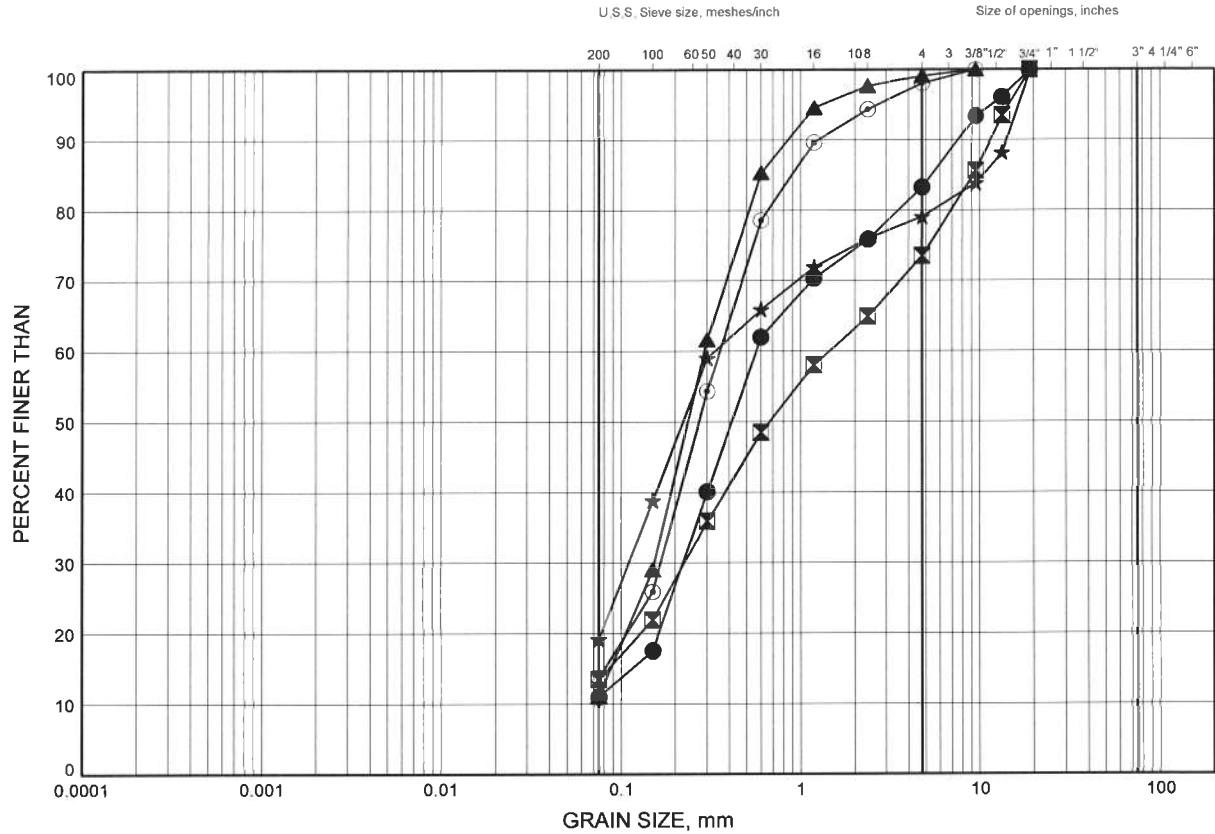
Appendix B

Laboratory Test Results (Present investigation)

NWR 32 Rehabs GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND to GRAVELLY SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	STR-01	1.83	340.27
⊠	STR-02	2.59	339.51
▲	STR-04	4.01	338.09
★	STR-06	3.96	338.24
⊙	STR-09	2.59	339.61

Date: October 2012

W.P.#: 6056-10-00



Prep'd: AN

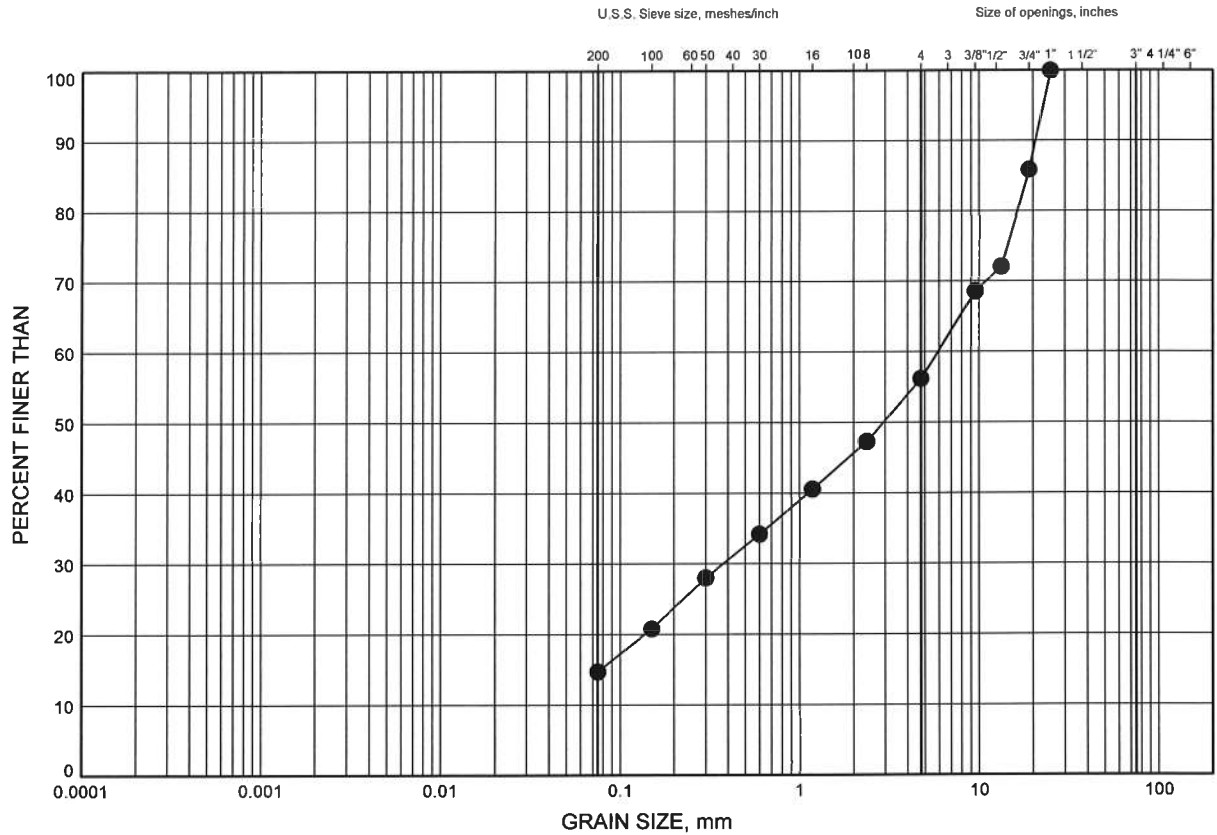
Chkd.: RPR

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	STR-08	3.35	338.74

GRAIN SIZE DISTRIBUTION - THURBER 1197.GPJ 7/24/12

Date July 2012
W.P.# 6056-10-00



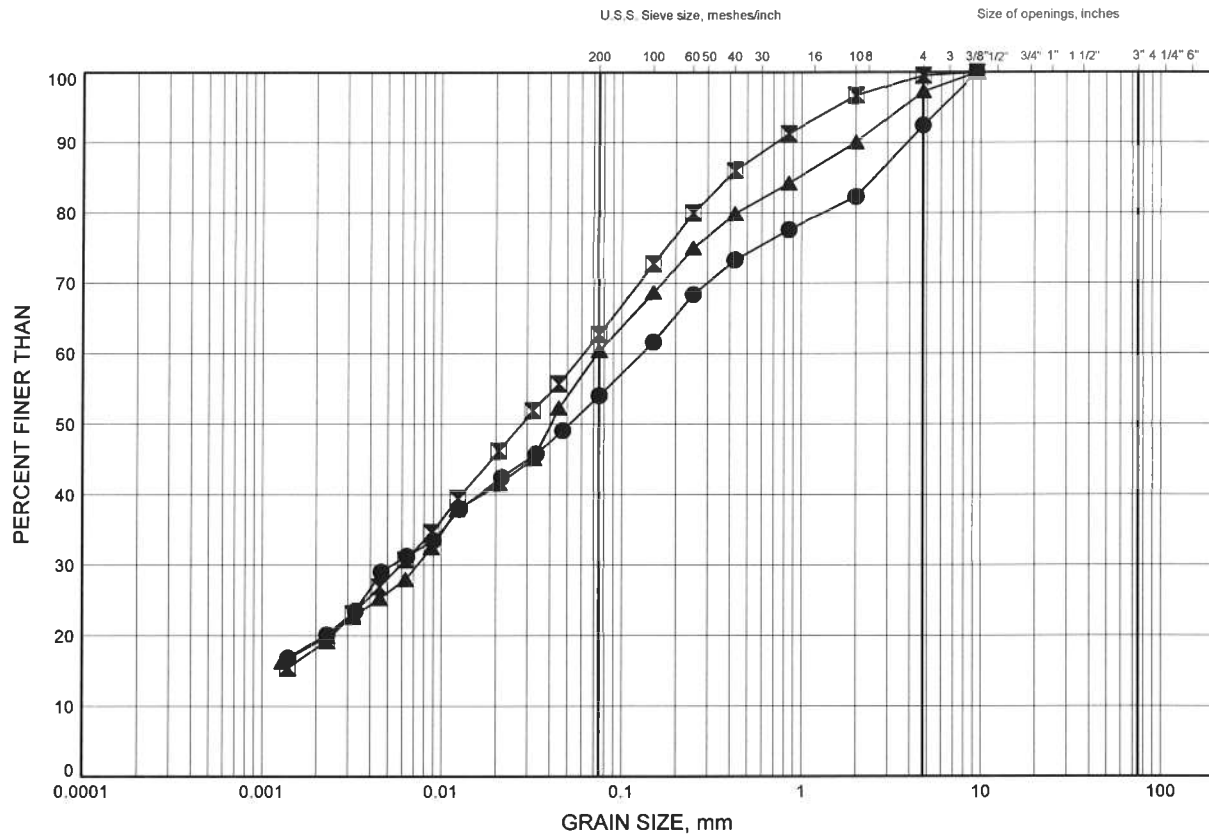
Prep'd AN
Chkd. RPR

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND & SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	STR-07	6.40	335.80
■	STR-09	4.88	337.32
▲	STR-10	4.88	337.32

Date: October 2012
W.P.#: 6056-10-00



Prep'd: AN
Chkd.: RPR



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POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : GENIVAR
Date Drilled : 8/5/2011
Project Name : Sturgeon River Bridge Date Tested : 9/8/2011
Core Size : NQ BH No : STR-03 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	5.9	A	4.9	47.3	55.4	37.9	Metasedimentary	Medium Strong
2	1	6.5	D	4.3	47.2	55.4	44.9	Metasedimentary	Medium Strong
3	1	7.2	D	16.3	47.2	60.5	171.1	Metasedimentary	Very Strong
4	1	7.4	A	31.3	47.2	62.3	219.5	Metasedimentary	Very Strong
5	2	7.8	D	20.7	47.2	64.9	217.7	Metasedimentary	Very Strong
6	2	8.1	A	36.5	47.1	54.4	284.7	Metasedimentary	Extremely Strong
7	2	8.5	D	12.9	47.2	65.4	135.2	Metasedimentary	Very Strong
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : GENIVAR
Date Drilled : 9/28/2012
Project Name : Sturgeon River Bridge Date Tested : 11/1/2012
Core Size : NQ BH No : STR-04 Tester : RK

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	4.9	D	6.9	46.8	115.0	73.4	Metasedimentary	Strong
2	1	5.3	D	17.9	47.5	77.4	186.8	Metasedimentary	Very Strong
3	1	5.6	D	17.4	44.6	95.3	199.6	Metasedimentary	Very Strong
4	2	6.5	D	20.5	48.0	89.1	209.5	Metasedimentary	Very Strong
5	2	6.9	D	17.3	48.4	86.4	174.6	Metasedimentary	Very Strong
6	2	7.3	D	17.0	47.6	100.0	175.9	Metasedimentary	Very Strong
7	2	7.8	D	11.1	48.3	78.3	112.1	Metasedimentary	Very Strong
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- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.



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POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : GENIVAR
Date Drilled : 9/28/2012
Project Name : Sturgeon River Bridge Date Tested : 11/1/2012
Core Size : NQ BH No : STR-05 Tester : RK

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	5.5	D	21.0	47.4	94.9	218.7	Metasedimentary	Very Strong
2	2	6.7	D	11.1	47.5	102.0	114.9	Metasedimentary	Very Strong
3	2	7.0	D	17.5	47.4	99.1	182.4	Metasedimentary	Very Strong
4	2	7.1	D	24.9	48.0	98.9	255.0	Metasedimentary	Extremely Strong
5	2	7.8	D	20.9	48.2	76.9	212.2	Metasedimentary	Very Strong
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- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.



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POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : GENIVAR
Date Drilled : 9/28/2012
Project Name : Sturgeon River Bridge Date Tested : 11/1/2012
Core Size : NQ BH No : STR-07 Tester : RK

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	3	10.2	D	10.8	46.8	93.4	115.1	Metasedimentary	Very Strong
2	4	11.9	D	14.3	46.9	99.1	151.8	Metasedimentary	Very Strong
3									
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- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No : 19-1351-197 Client : GENIVAR
Date Drilled : July 26 2011
Project Name : Sturgeon River Bridge Date Tested : 9/8/2011
Core Size : NQ BH No : STR-08 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	8.4	A	7.3	47.2	33.5	83.2	Metasedimentary	Strong
2	2	8.6	A	4.5	47.2	51.6	36.6	Metasedimentary	Medium Strong
3	2	8.8	D	12.0	47.2	60.1	126.0	Metasedimentary	Very Strong
4	2	9.5	A	3.5	47.2	56.5	26.2	Metasedimentary	Medium Strong
5	3	10.4	A	13.6	47.3	57.1	102.4	Metasedimentary	Very Strong
6	3	10.5	D	12.7	47.3	61.0	133.3	Metasedimentary	Very Strong
7	3	10.9	A	11.5	47.2	54.3	90.2	Metasedimentary	Strong
8	3	12.0	D	8.0	47.2	62.7	83.5	Metasedimentary	Strong
9	3	12.5	A	16.6	47.2	51.1	135.9	Metasedimentary	Very Strong
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11									
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have $0.7 \times D$ on either side of test point.

Appendix C

**Record of Borehole Sheets and Laboratory Results
(previous investigation)**

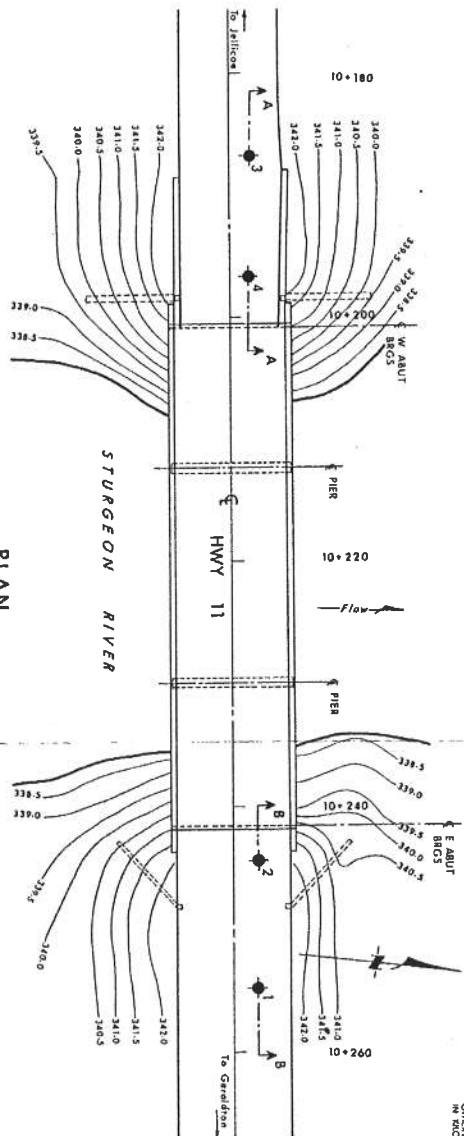
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DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS UNLESS
OTHERWISE SHOWN. STATISTICAL
IN KILOMETERS - METERS.

METRIC

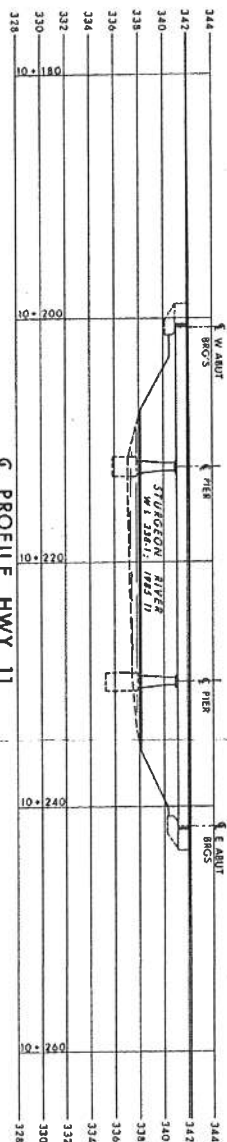
AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATICS IN KILOMETRES - METRES.

CONT No
WP No 335-85-01
STURGEON RIVER BRIDGE #4
REHABILITATION
BORE HOLE LOCATIONS & SOIL STRAT

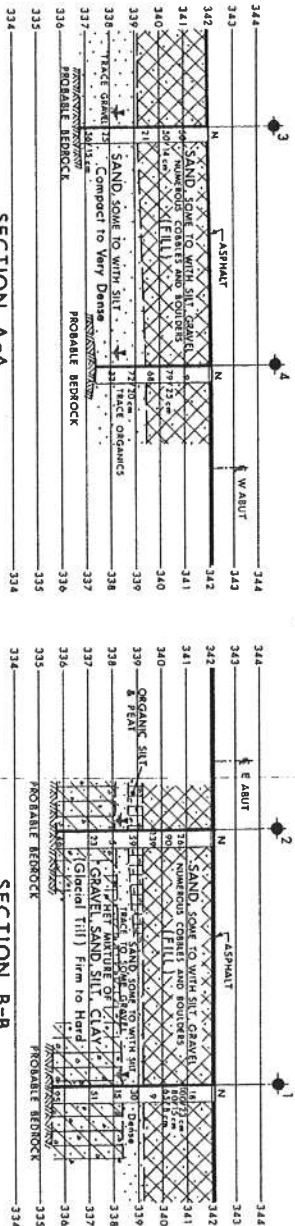
SHEET



PLAN
SCALE
4m 2 0 4m

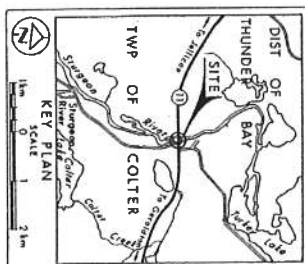


SCALE
PROFILE HWY 11



SECTION A-A

SECTION B-B



LEGEND

◆	Bore Hole
◆	Dynamic Cone Penetration Test (Cone)
◆	Bore Hole & Cone
N	Blow/0.3m (Std Pen Test), 4.75 J/blow
CONE	Blow/0.3m (60° Cone, 475 J/blow)
✚	WL at time of investigation 1986 Oct

No	ELEVATION	STATION	OFFSET
1	342.1	10+254.8	2.0 m lt
2	342.1	10+244.4	2.0 m lt
3	342.0	10+186.8	1.6 m lt
4	342.1	10+196.6	1.5 m lt

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REV.	DATE	BY	DESCRIPTION

Geocras No 42E-3

HWY No 11	DIST 10
SUMMARY CHECKED	DATE 1986 06 17
DATE 1986 06 17	SITE 48 E-8
DRIVEN BY CHECKED	OWG 3358301-A

RECORD OF BOREHOLE No 1

METRIC

W P 335-85-01 LOCATION STA. 10 + 234.8; 0/a 2.0 m Lt. E Hwy. 11 ORIGINATED BY LP
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY LP
 DATUM Geodetic DATE 86 04 22 CHECKED BY *LP*

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					
342.1 0.0	Pavement Surface Asphalt															GR SA SI CL
	Fill															
	Sand, Some to with Silt, Gravel		1	SS	18											14 53 30 3
	Numerous Cobbles and Boulders		2	SS	100	23 cm										
			3	SS	80	15 cm										
			4	SS	65	8 cm										
339.2			5	SS	9											28 47 23 2
2.9	Sand with Silt, Some Gravel		6	SS	30											23 44 30 3
338.4	Dense															18 29 40 13
3.7	Heterogeneous Mixture Gravel, Sand, Silt, Clay (Glacial Till)		7	SS	15											3 34 47 16
	Very Stiff to Hard		8	SS	51											
335.5			9	SS	95											9 35 43 13
6.6	End of Borehole															
	Refusal to Auger, Probable Bedrock															

OFFICE REPORT ON SOIL EXPLORATION

+3, x³: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W P 335-85-01 LOCATION STA. 10 + 244.4; 0/s 2.0 m Lt. 4 Hwy. 11 ORIGINATED BY LP
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY LP
 DATUM Geodetic DATE 86 04 22 CHECKED BY *LP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _p	W	W _L		
342.1 0.0	Pavement Surface Asphalt						342						
	<u>Fill</u> Sand, Some to with Silt, Gravel						341						
	Numerous Cobbles and Boulders		1	SS	26		340						18 45 35 2
			2	SS	90								
339.2			3	SS	139								
2.9	Organics						339						
338.6	Silt and Peat		4	SS	59								
3.5	Sand Some Silt, Trace Gravel												
338.1													
4.0	Heterogeneous Mixture of Gravel, Sand, Silt, Clay (Glacial Till)		5	SS	5		338						7 36 48 9
	Firm to Hard		6	SS	23		337						
335.7			7	SS	46		336						6 35 45 14
6.4	End of Borehole Refusal to Auger, Probable Bedrock												

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15 10 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 3

METRIC

WP 335-85-01 LOCATION STA. 10 + 186.8; 0/8 1.6 m Lt. 4 Hwy. 11 ORIGINATED BY LP
DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY LP
DATUM Geodetic DATE 86 04 23 CHECKED BY *SP*

[illegible]

+3, x5: Numbers refer to Sensitivity

15 ϕ 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

METRIC

W P 335-85-01 LOCATION STA. 10 + 196.6; 0/a 1.5 m Lt. 4 Hwy. 11 ORIGINATED BY LP
 DIST 19 HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY LP
 DATUM Geodetic DATE 86 04 23 CHECKED BY GP

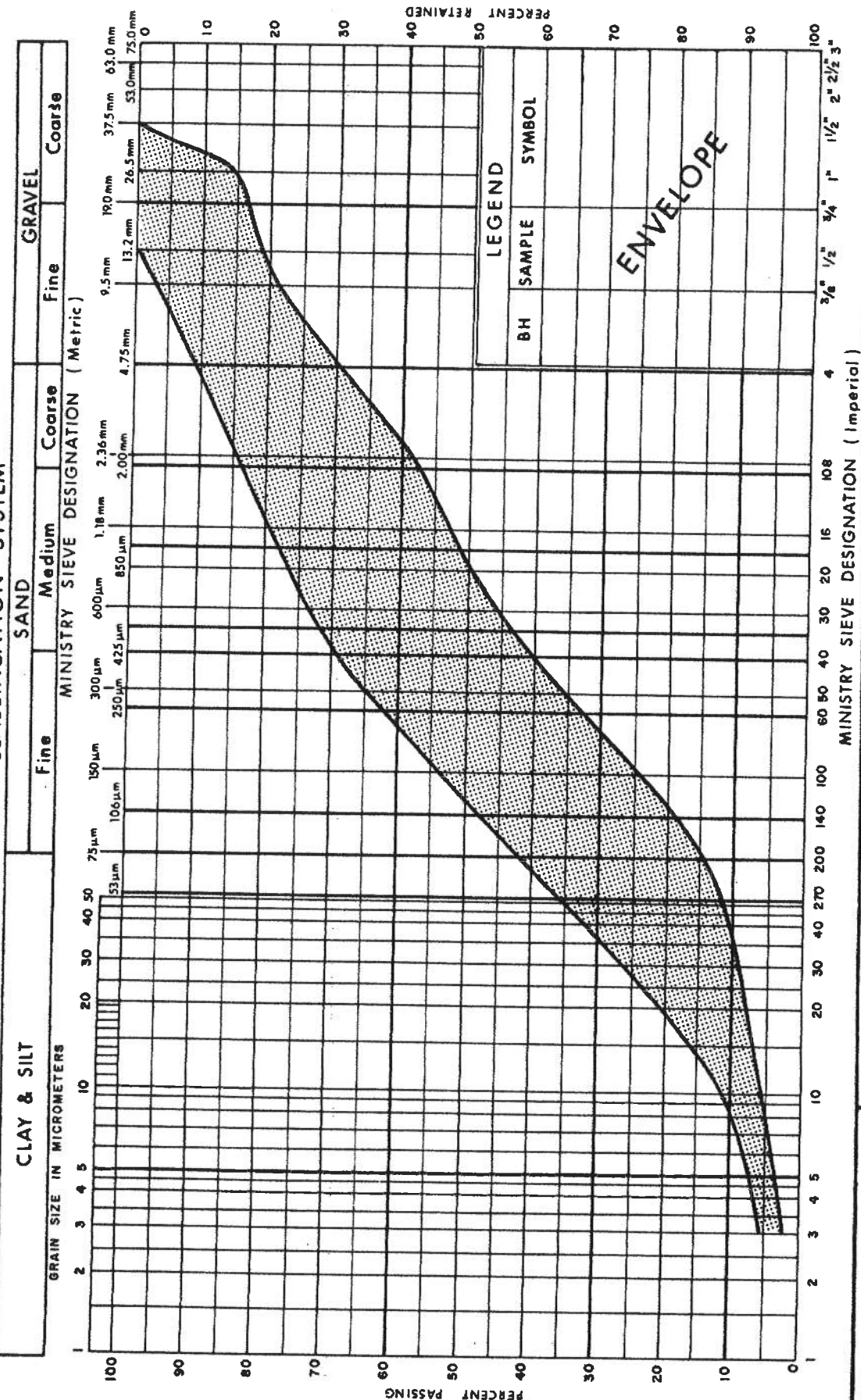
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100				
342.1	Pavement Surface														
0.0	Asphalt					342									
	Fill														
	Sand, Some to with Silt, Gravel		1	SS	9	341								10	50 38 2
	Numerous Cobbles and Boulders		2	SS	79	340									
339.4			3	SS	68	339								33	50 15 2
2.7	Sand, with Silt Trace Organics		4	SS	72	338									
			5	SS	32										
337.4	Dense to Very Dense		6	SS	*										
4.7	End of Borehole														
	* Spoon Bouncing Probable Bedrock														
	** Spoon Bouncing on Boulder														

OFFICE REPORT ON SOIL EXPLORATION

*3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

UNIFIED SOIL CLASSIFICATION SYSTEM

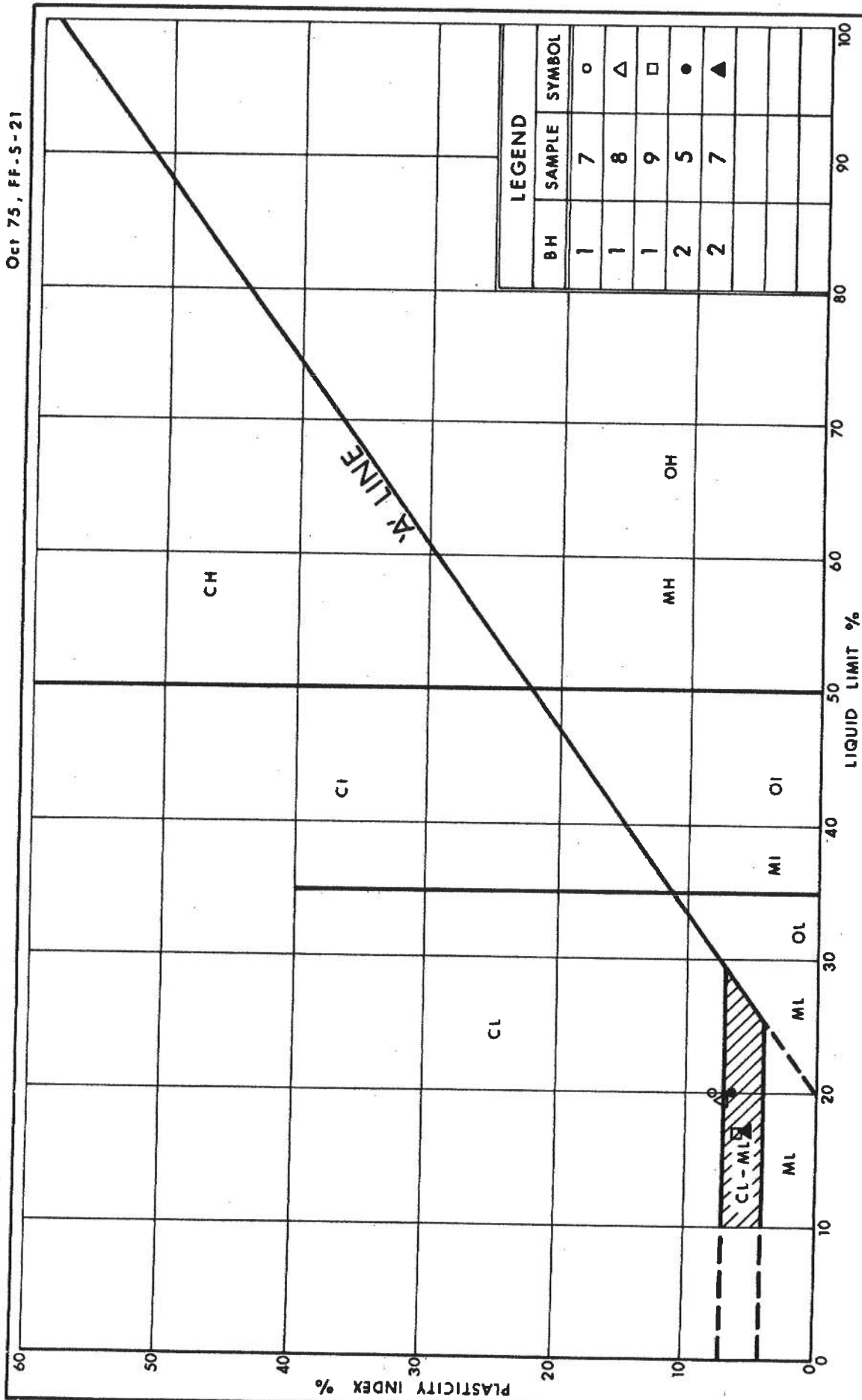
Ministry of
Transportation and
Communications

GRAIN SIZE DISTRIBUTION
SAND, SOME TO WITH SILT, GRAVEL (FILL)
NUMEROUS COBBLES AND BOULDERS

FIG No 1

WP 335-85-01

Oct 75, FF-S-21



PLASTICITY CHART
HET MIXTURE OF
GRAVEL, SAND, SILT, CLAY (Glacial Till)

FIG No 3

W P 335-85-01

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

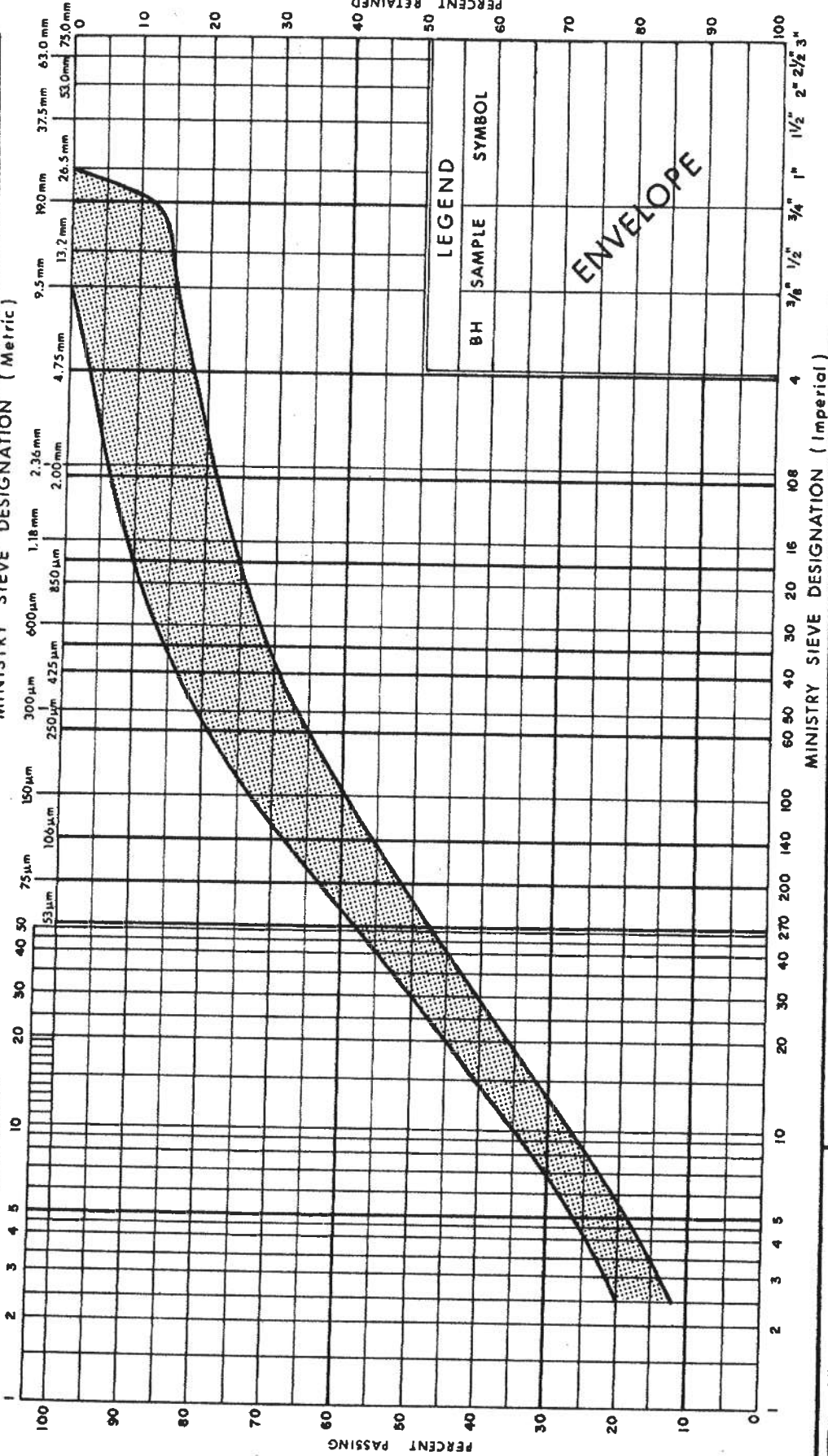
GRAVEL

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

Coarse

Fine



LEGEND

BH SAMPLE SYMBOL

ENVELOPE

Ministry of
Transportation and
Communications



FIG No 4

W P 335-85-01

GRAIN SIZE DISTRIBUTION
HET MIXTURE OF
GRAVEL, SAND, SILT, CLAY (Glacial Till)

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footings on Native Soil	Augered Caissons (drilled shafts) socketed into bedrock	Drilled in Pipe Piles socketed into bedrock	Driven Piles to Bedrock
<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Relatively deep excavation in cohesionless soils extending below the groundwater level is required. This will necessitate prior dewatering. ii. It will be difficult to install temporary shoring through cobbles and boulders. iii. Foundations close to river flow would be at risk due to scour and erosion. iv. Environmental impact on the river during excavation. <p>NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for caissons socketed into bedrock ii. Construction of caissons could continue in freezing weather. iii. Subexcavation of fill not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting rock sockets. iv. Installation through cobbles and boulders will be difficult. <p>FEASIBLE BUT NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for units socketed into bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Possibly higher unit cost compared to other foundation options such as footings. ii. Difficulties in obtaining seal below the liner to pour concrete in dry conditions. iii. Specialized installation. iv. Potential difficulty in cleaning and inspecting rock socket. <p>RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> ii. High geotechnical resistance available by driving piles to achieve resistance on the bedrock. iii. Installation of piles could continue in freezing weather. iv. May require less volume of excavation than footings. v. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Very short pile length particularly at the west abutment may not provide sufficient fixity or lateral stability. iii. Pile lengths required to achieve design resistance may vary. iv. Pre-augering might be required due to the presence of cobbles and boulders within and under the fill, especially at the west abutment. <p>NOT RECOMMENDED</p>

Appendix E

List of SPs and OPSS, and Suggested Text for Selected NSSP

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 903
- OPSS 902
- OPSS 804
- OPSD 208.010
- OPSD 3101.150
- OPSS 539
- Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.

2. Suggested Text for NSSP on “Construction of Caissons”

Caisson installation shall be in accordance with OPSS 903 and the following.

Caisson installation at this site will require excavation through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability in the caisson excavation and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- Caisson installation may encounter cobbles, boulders and/or large rock fragments in the soils overlying the bedrock. The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating such obstructions.
- The bedrock consists of strong to very strong metasedimentary rock. The strength and hardness of this rock must be taken into account when selecting equipment to advance the caisson into rock. Equipment supplied to construct the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of caisson above the bedrock surface will not be considered part of the specified length of rock socket.

3. Suggested Text for NSSP on “Construction of Drilled-in pipe piles”

Drilled-in pipe pile installation shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require excavation through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability during installation of the pipe piles and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- Drilled-in pipe pile installation may encounter cobbles, boulders and/or large rock fragments in the soils overlying the bedrock. The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating such obstructions.
- The bedrock consists of strong to very strong metasedimentary rock. The strength and hardness of this rock must be taken into account when selecting equipment to advance the drilled-in pipe pile into rock. Equipment supplied to construct or drill the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of drilled-in pipe pile above the bedrock surface will not be considered part of the specified length of rock socket.
- Pipe piles shall be placed centred into the holes, bearing directly on the sound rock at the bottom of the hole. Pipe piles shall be stabilized in place by temporary supports.
- The annular space between the rock socket wall and the pipe pile shall be filled with 30 MPa concrete to the top of the bedrock surface at the location. Concrete may be tremied into the pipe pile. The plumbness and alignment of the pipe pile shall be maintained during concreting

4. Suggested text for a NSSP on Steel Sheet Pile and Driven H-pile installation

The existing embankment fill contains cobbles and boulders and a layer of cobbles and boulders was also noted above the bedrock surface at the west abutment.

These cobbles and boulders may impede the driving of sheet piles as well as H-piles and at some locations the sheet piles or H-piles may not be able to penetrate the cobbles and boulders and reach the design depth of installation and refusal may be encountered at varying depths.

The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the sheet piles to the design depth.

The sheet piles and H-piles should be provided with pile tip protectors to minimize tip damage.

If the piles meet refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving. The QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

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5. Suggested text for a NSSP on Subgrade Densification

The entire subgrade area for the RSS walls shall be uniformly densified using a plate tamper.

Appendix F

Site Photographs



Photograph 1– Highway 11 and Sturgeon River Bridge crossing



Photograph 2– Existing Highway 11 embankment



Photograph 3–Sturgeon River Bridge



Photograph 4–Sturgeon River Bridge



Photographs 5 and 6 –Sturgeon River south and north sides of the Highway 11

Appendix G

Drawing titled “Borehole Locations and Soil Strata”

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No 6056-10-00

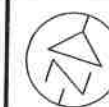
HIGHWAY 11
STURGEON RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA



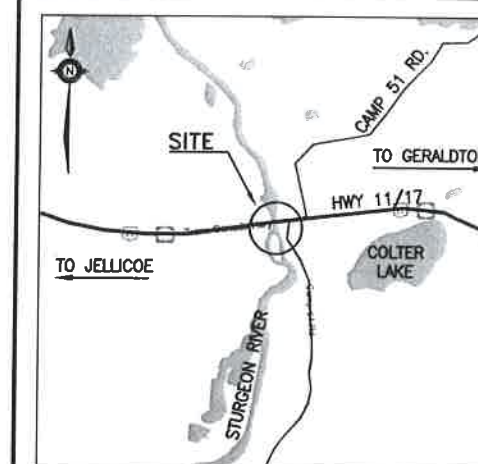
McCORMICK RANKIN
A member of the MWH GROUP



THURBER ENGINEERING LTD



SHEET



KEYPLAN

LEGEND

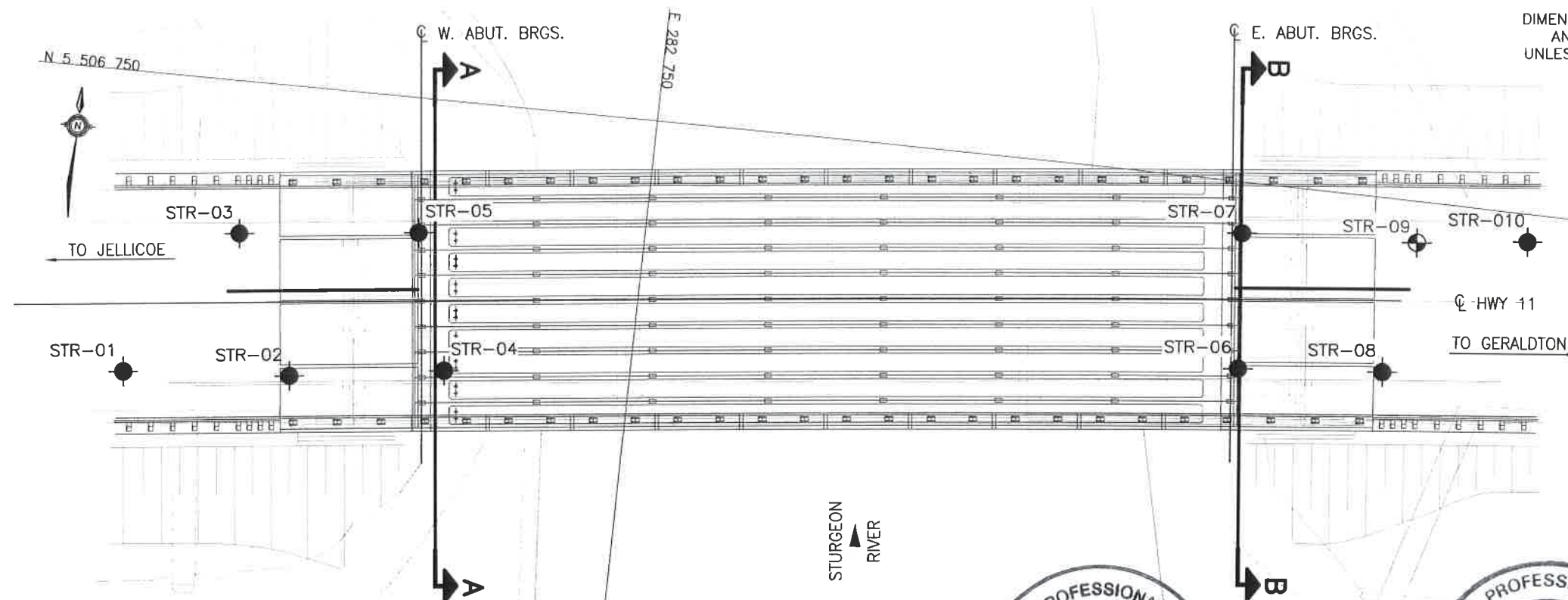
●	Borehole
⊙	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
HA	Head Artesian Water
P	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
STR-01	342.1	5 506 736.8	282 728.1
STR-02	342.1	5 506 737.3	282 735.3
STR-03	342.1	5 506 743.3	282 732.5
STR-04	342.1	5 506 738.2	282 742.0
STR-05	342.1	5 506 744.1	282 740.2
STR-06	342.2	5 506 741.8	282 776.1
STR-07	342.2	5 506 747.8	282 775.6
STR-08	342.1	5 506 742.4	282 782.3
STR-09	342.2	5 506 748.2	282 783.1
STR-10	342.2	5 506 748.8	282 787.9

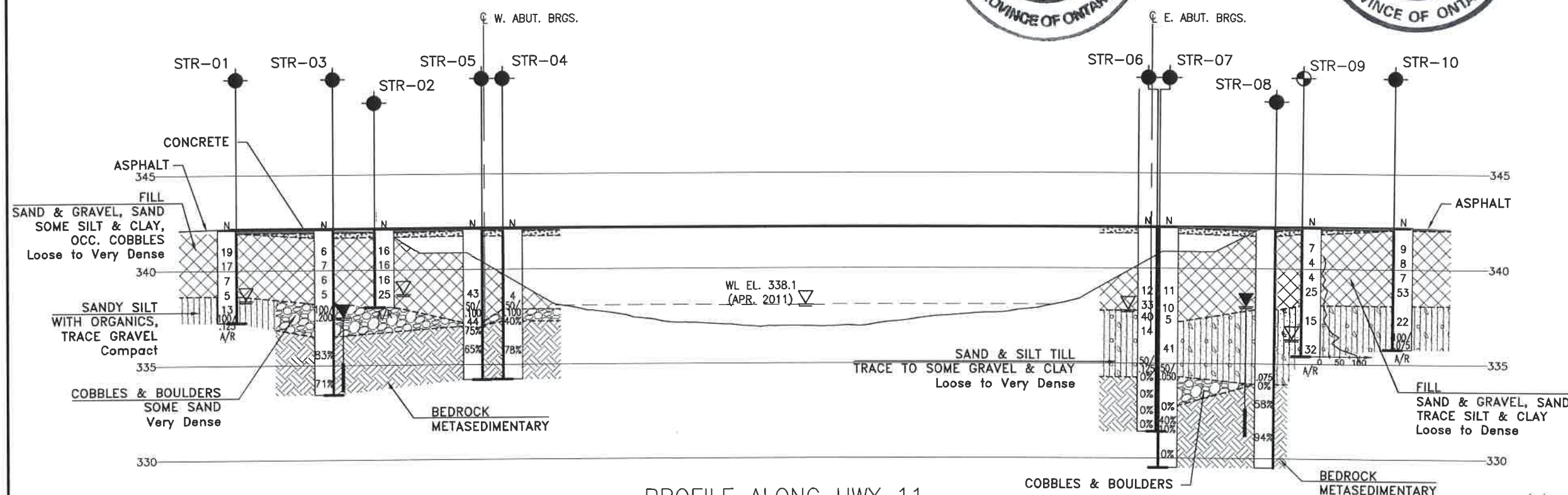
-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRIS No. 42H-13



PLAN



PROFILE ALONG HWY 11



DATE	BY	DESCRIPTION
DESIGN	RPR	CHK RPR
DRAWN	AN	CHK PKC
		SITE 48C-46
		STRUCT
		DWG 1

CONT No
WP No 6056-10-00

HIGHWAY 11
STURGEON RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

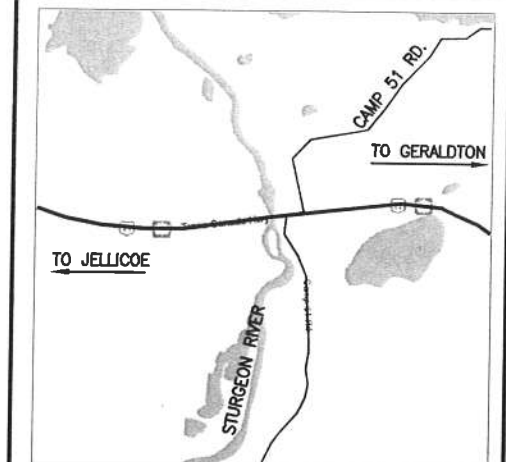
SHEET 1



McCORMICK RANKIN
A member of **MMM GROUP**








THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

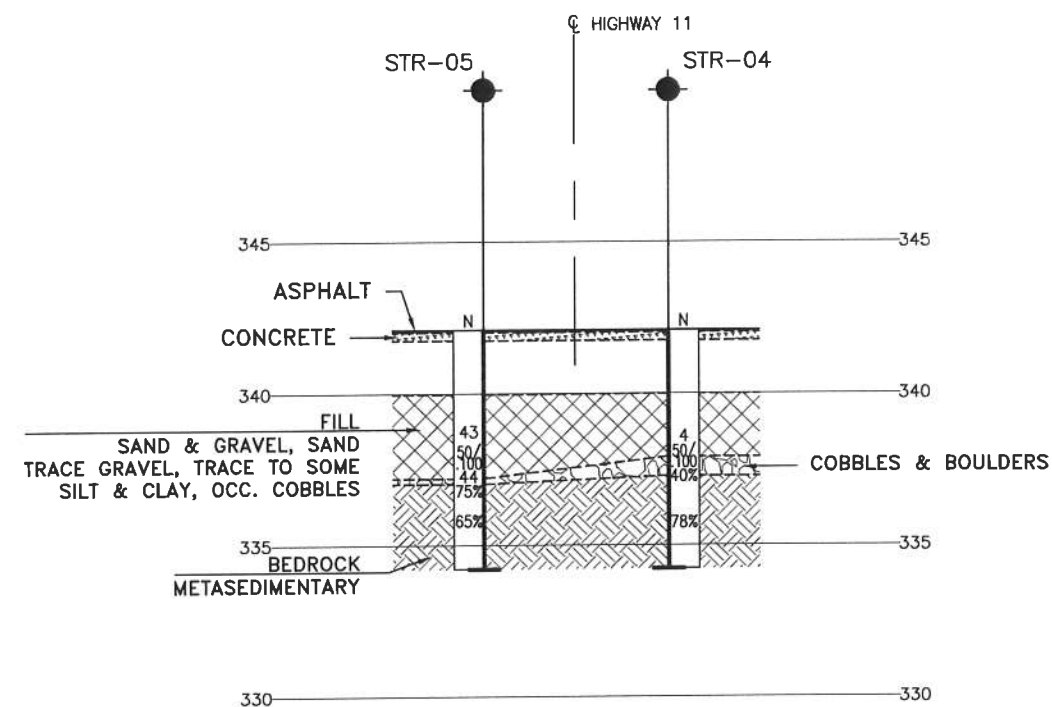
- | | |
|---|---------------------------------------|
|  | Borehole |
|  | Borehole and Cone |
| N | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE | Blows /0.3m (60° Cone, 475J/blow) |
| PH | Pressure, Hydraulic |
|  | Water Level |
|  | Head Artesian Water |
|  | Piezometer |
| 90% | Rock Quality Designation (RQD) |
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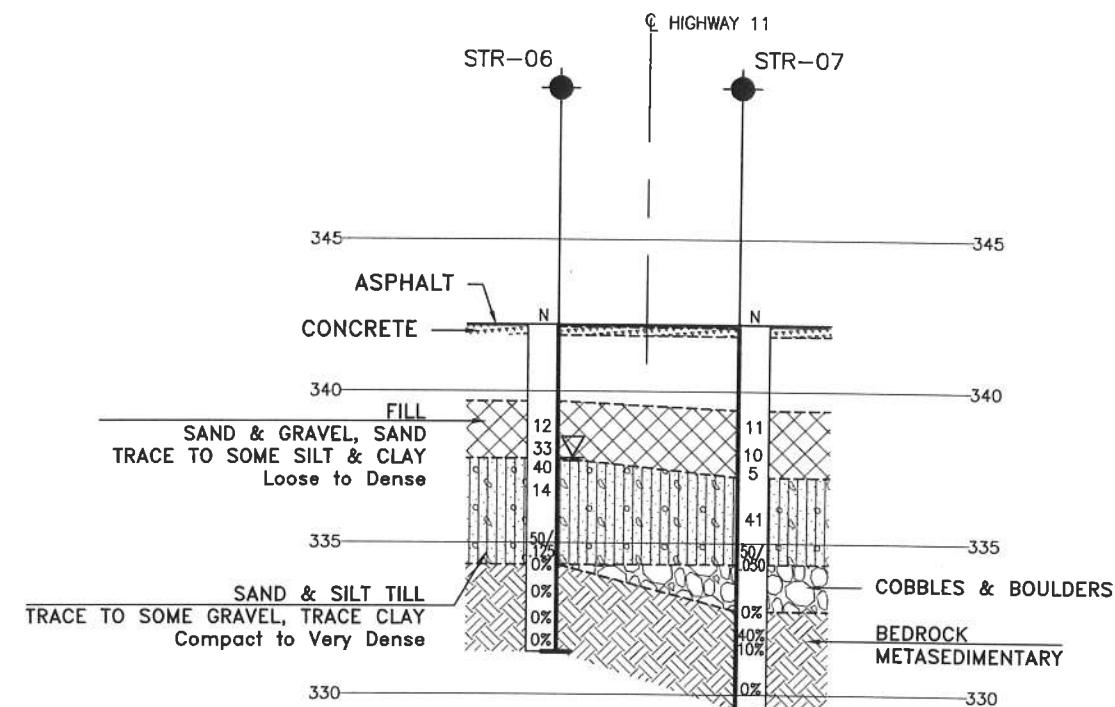
-NOTES-

- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 42H-13



SECTION ALONG A-A



SECTION ALONG B-B

[illegible]