

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
JACKFISH RIVER BRIDGE REPLACEMENT  
HIGHWAY 17, CORRIGAL AND PATIENCE TOWNSHIPS  
THUNDER BAY UNORGANIZED DISTRICT  
G.W.P. 465-00-00, STRUCTURE NO. 48C-8**

**Geocres Number: 52H-18**

**Report to  
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**PART 1: FACTUAL INFORMATION**

**1 INTRODUCTION**

This report presents the factual findings obtained from a geotechnical investigation conducted at the location of an existing bridge carrying Highway 17 over Jackfish River in Corrigan and Patience Townships, Ontario.

The original purpose of the investigation was to explore the subsurface conditions pertinent to the design of roadway protection at the existing bridge abutments and strengthening of foundation of the existing pier, in anticipation of superstructure replacement. As the design progressed, it became apparent that a full structure replacement would be a better option which necessitated additional borehole geotechnical information, at the abutments. Based on discussions with MTO, the development of the subsurface model has been based on the following foundation report previously prepared for the existing bridge:

- Foundation Investigation Jackfish River Crossing, T.C.H. No. 17 near Nipigon, Ontario, W.P. 936-58, prepared by Trow Soderman and Associates, dated October 20, 1958. (Reference 1).

On the basis of the data obtained during the current and previous investigations, this report provides a borehole location plan, borehole logs, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions.

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.

## **2 SITE DESCRIPTION**

The site is located on Highway 17 approximately 14.2 km east of Nipigon, Ontario. The bridge is approximately 3.4 km east of the unnamed road leading to Fire Hall Lake, north of Highway 17, in Corrigal and Patience Townships, Thunder Bay Unorganized District, Ontario.

At the bridge location, Highway 17 is a two-lane paved roadway. The existing Jackfish River bridge is a two-span structure supported on two abutments and a central pier. The total length of the bridge is 52.3 m and the width is 11.0 m

The section of Highway 17 at the bridge site has a north-south orientation, however a construction north has been assumed for this site, with Nipigon to the west and Rossport to the east. All direction indications in this report are relative to the construction north. At the bridge site, Jackfish River flows from north to south and the existing river channel is approximately 35 m wide. The lands immediately surrounding the bridge site consist of undeveloped forested areas.

Photographs included in Appendix H show the general nature of the bridge site.

The site lies within the Canadian Shield, characterized by low, rounded hills of Pre-Cambrian bedrock mantled by varying thicknesses of overburden. At this site, the overburden consists primarily of fluvial deposits of silty sand to sand and gravel. Silt and clay deposits were also encountered at this site. Granite bedrock was encountered at depth during the previous investigation conducted at this site.

## **3 SITE INVESTIGATION AND FIELD TESTING**

The present site investigation and field testing for this project were carried out between February 9 and 20, 2011. A total of six sampled boreholes (numbered JFR-01 to JFR-06) were drilled to depths ranging from 5.5 m to 17.4 m (elevations 170.9 to 179.3). Dynamic Cone Penetration Tests (DCPTs) were also performed from the bottom of each borehole, except in Borehole JFR-04 where the DCPT was conducted adjacent to the borehole from a depth of 3.1 m. The DCPTs extended to depths ranging from 7.9 m to 19.2 m (elevations 169.1 to 177.8).

Boreholes JFR-01 and JFR-05 were drilled at the west and east approaches, respectively, while Borehole JFR-02 was drilled near the west abutment and Borehole JFR-04 was drilled near the east abutment. Boreholes JFR-03 and JFR-06 were drilled through the tremie concrete forming the existing pier. Foundation core samples were obtained from the tremie concrete at the pier.

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing in Appendix I. The coordinates and elevations of the boreholes are listed on the drawings and are presented on the individual Record of Borehole Sheets in Appendix A. The co-ordinates and ground surface elevations of the boreholes were obtained from plan drawings provided by MRC.

Eight boreholes (1 to 5, 7, 8, and 10) and 2 DCPTs (6 and 9) were previously drilled at this site during the investigation conducted in 1958 (Reference 1). The results of this previous investigation have been incorporated into this report. The logs for these boreholes and DCPTs are included in Appendix D. Boreholes 1, 2, 5, and 7 were drilled near the west abutment, while Boreholes 3, 4, 8, and 10 were drilled near the east abutment. The two DCPTs (6 and 9) were conducted in the river near the west and east river banks, respectively.

The approximate locations of the boreholes drilled during the 1958 investigation are shown on the Borehole Locations and Soil Strata Drawing included in Appendix I. The locations of these boreholes are approximate since borehole coordinates were not included in the 1958 investigation report.

Prior to commencement of drilling, utility clearances were obtained for all borehole locations.

Hollow stem augers were used to advance Boreholes JFR-01, JRF-02, JFR-04, and JFR-05. Portable, light-weight coring equipment was used to advance Boreholes JFR-03 and JFR-06 through the concrete pier foundation and then a portable tripod set-up was used to advance the boreholes below the tremie concrete. Soil samples were obtained at selected intervals using a 50 mm diameter split spoon sampler in conjunction with Standard Penetration Testing (SPT).

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes, visually examined the recovered samples, and prepared the samples for transport back to Thurber's laboratory for further examination and testing.

Groundwater conditions were observed in the open boreholes throughout the drilling operations. The boreholes were backfilled with a mixture of bentonite and auger cuttings in general accordance with O.Reg. 903 upon completion.

#### **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) and Atterberg Limits testing, where appropriate. The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures contained in Appendix B.

The concrete cores obtained from the pier foundation were tested for compressive strength. The laboratory test results of the concrete cores as well as photos of the concrete cores are included in Appendix C.

#### **5 DESCRIPTION OF SUBSURFACE CONDITIONS**

Reference is made to the Record of Borehole sheets included in Appendix A for details of the encountered soil stratigraphy. A stratigraphic profile and sections are presented on the Borehole Locations and Soil Strata Drawings included in Appendix I. Overall descriptions of the



stratigraphy are 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.

Based on the six boreholes drilled during the current investigation as well as the eight boreholes drilled previously during the 1958 investigation, the stratigraphy encountered at this site generally consists of asphalt over sand fill overlying layers of native clayey silt to silty clay at the west abutment and layers of native silt, sand, and sand and gravel at the east abutment. A deposit of silty clay was also encountered locally at the east approach below the silt layer. The native soil layers are underlain by an extensive deposit of silty sand to sand. Below the silty sand/sand, a thin layer of sand and gravel was contacted overlying granite bedrock. A layer of organics was encountered surficially in five boreholes in 1958. More detailed descriptions of the individual strata are presented below.

### **5.1 Organics**

A layer of organics was encountered at surface in Boreholes 1 to 5 and 7 drilled during the 1958 investigation. The thickness of the organics ranged from 150 mm to 1.6 m.

### **5.2 Asphalt and Concrete**

Asphalt was encountered at surface in four of the boreholes drilled at this site during the current investigation (JFR-01, JFR-02, JFR-04, and JFR-05). The thickness of the asphalt ranged from 75 mm to 125 mm.

Two boreholes from the current investigation (Boreholes JFR-03 and JFR-06) were drilled through the tremie concrete surrounding the existing pier. The concrete was 4.9 m thick in both boreholes.

The concrete cores from Borehole JFR-03 were tested for compressive strength in accordance with CSA Standard A23.14C, "Obtaining and Testing Drilled Cores for Compressive Strength Testing" in a dry condition. Report of the laboratory test is included in Appendix C. Photographs of the concrete cores are presented in Appendix C.

### **5.3 Sand Fill**

Granular fill consisting of brown sand containing trace gravel to gravelly, some silt to silty and trace clay was encountered below the asphalt pavement in the four boreholes (JFR-01, JFR-02, JFR-04, and JFR-05) from the current investigation that were drilled through the highway embankment.

The thickness of the granular fill ranged from 2.9 m to 4.5 m, with the lower boundary of the granular fill encountered at depths of 3.0 m to 4.6 m (Elevation 186.1 to 183.7). The granular fill was typically thicker in the boreholes drilled on the east side of Jackfish River.

SPT 'N' values recorded in the sand fill ranged from 12 blows for 0.3 m penetration to 50 blows for 0.025 m penetration, indicating a compact to very dense relative density. The high SPT 'N' values in the upper part of the fill might be related to the frozen condition of the fill at the time of drilling.

Moisture contents of the granular fill typically ranged from 2% to 16%.

Three samples of the sand fill underwent laboratory grain size analysis testing, the results of which are summarized below. The results of these tests are also presented on the Record of Borehole sheets included in Appendix A and the grain size distribution curves for these three samples are presented on Figure B1, Appendix B.

Gravel %	3 to 20
Sand %	73 to 75
Silt & Clay %	7
Silt %	18
Clay %	6

#### 5.4 Silty Clay to Clayey Silt

Silty clay to clayey silt was encountered in Boreholes (JFR-01, JFR02, 1 and 2) drilled at the west abutment and west approach as well as in Borehole JFR-05 drilled at the east approach. The silty clay to clayey silt layer was encountered below a thin layer of organics in Boreholes 1 and 2 at 0.15 m depth, and below the sand fill in Boreholes JFR-01 and JFR-02 at 3.0 m and 3.1 m depth (elevations 186.1 and 185.9). In Borehole JFR-05, the silty clay was encountered at 8.7 m depth (elevation 179.6), below a layer of silt. The silty clay to clayey silt was brown to grey and contained trace sand and occasional silt seams and organics.

The thickness of the silty clay to clayey silt ranged from 4.4 m to 7.1 m, in Boreholes JFR-01, JFR-02, 1 and 2 with the lower boundary of the silty clay to clayey silt encountered at depths of 4.6 m to 10.2 m (Elevation 181.6 to 178.8). The silty clay layer in Borehole JFR-05, drilled at the east approach, was 8.4 m thick. The depth to the base of the clay in Borehole JFR-05 was 17.1 m (elevation 171.2).

SPT 'N' values recorded in the silty clay to clayey silt ranged from 0 to 22 blows for 0.3 m penetration, indicating a consistency varying from very soft to very stiff. In general, the silty clay to clayey silt had a soft to stiff consistency at the west abutment and approach (Boreholes 1, 2, JFR-01 and JFR-02) and a very soft consistency at the east approach (Borehole JFR-05).

Samples of the silty clay to clayey silt generally had moisture contents ranging from 20% to 30%. The samples from Borehole JFR-05 had higher moisture contents, ranging from 37% to 45%.

Four samples of the silty clay to clayey silt from the current investigation underwent laboratory grain size analysis testing. The results of these tests are presented on the Record of Borehole sheets included in Appendix A and the grain size distribution curves for these samples are plotted on Figure B2 of Appendix B. The results are summarized as follows:

Gravel %	0
Sand %	2 to 4
Silt %	57 to 78
Clay %	18 to 41

Three of these samples also underwent Atterberg Limits testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and the results of these tests are plotted on Figure B6.

Liquid Limit %	27 to 44
Plastic Limit %	16 to 22

These results indicate that the silty clay to clayey silt has a low to medium plasticity with group symbols of CL and CI.

### **5.5 Sand and Silty Sand**

Layers of brown to grey sand and silty sand were encountered in the boreholes at depths and elevations indicated in Table 5.1. The sand and silty sand layers contained trace to some gravel, trace clay and occasional gravelly zones. The sand is described as clean in places. A layer of sand with decayed wood was encountered directly below the sand fill in Boreholes JFR-04, below the tremie concrete surrounding the pier in Borehole JRF-03 and below the organics in Borehole 4. The layer of sand with decayed wood is also presented in Table 5.1.

**Table 5.1 – Depths and Elevations of Native Sand and Silty Sand**

Foundation Unit	Borehole	Depth below existing ground surface/riverbed (m)	Elevation (m)	Thickness (m)	Soil
West Abutment	1	4.6 to 27.1 (borehole termination depth)	181.6 to 159.1	22.5	Silty sand
	2	4.7 to 15.8 (borehole termination depth)	180.5 to 169.4	11.1	Silty sand
	5	1.3 to 22.3 <sup>(2)</sup>	178.9 to 157.9	21.0	Silty sand and sand
	7	1.6 to 13.8 <sup>(2)</sup>	180.6 to 168.4	12.2	Silty sand and sand
Pier	JFR-03	4.9 to 5.8	178.6 to 177.7	0.9	Silty sand <sup>(1)</sup>
	JFR-06	4.9 to 5.5 (borehole termination depth)	178.6 to 178.0	0.6	Silty sand
East Abutment	JFR-04	4.6 to 8.7	184.0 to 179.9	4.1	Sand <sup>(1)</sup>
	3	9.2 to 15.9 (borehole termination depth)	175.4 to 168.7	6.7	Silty sand
	4	1.5 to 3.7	183.5 to 181.4	2.1	Sand <sup>(1)</sup>
		9.4 to 25.6 (borehole termination depth)	175.6 to 159.4	16.2	Silty sand
	8	5.7 to 22.2 <sup>(2)</sup>	176.5 to 160.1	16.5	Silty sand
	10	5.2 to 13.7 <sup>(2)</sup>	176.8 to 168.3	8.5	Silty sand
East Approach	JFR-05	17.1 to 17.4 (borehole termination depth)	171.2 to 170.9	0.3	Sand

<sup>(1)</sup> With decayed wood fragments

<sup>(2)</sup> Depth from the riverbed

SPT ‘N’ values recorded during the current investigation and the 1958 investigation ranged from 0 to 60 blows for 0.3 m penetration indicating a relative density varying from very loose to very dense. In general, SPT ‘N’ values ranged from 10 to 35 blows for 0.3 m penetration, indicating a compact to dense relative density.

Moisture contents of the silty sand to sand samples from the current investigation ranged from 15% to 22%.

One sample from the current investigation of the silty sand to sand layer underwent laboratory grain size analysis testing. The results of this test are presented on the corresponding Record of Borehole sheets included in Appendix A and the grain size distribution curve for this sample is plotted on Figures B4 of Appendix B. The results of this test are summarized as follows:



	Silty Sand
Gravel %	12
Sand %	59
Silt and Clay %	29

### 5.6 Silt to Sandy Silt

Native silt to sandy silt was encountered below the clayey silt to silty clay at 8.7 m and 10.2 m depth (elevations 180.4 and 178.8) in Boreholes JFR-01 and JFR-02 and below the sand fill at 4.6 m (elevation 183.7) in Borehole JFR-05. A layer of sandy silt was also encountered in Borehole JFR-03 at 5.8 m depth (elevation 177.7). The silt to sandy silt was greenish grey to brown and contains trace clay and occasional sand layers.

The silt to sandy silt layer was fully penetrated in Borehole JFR-05 only and was found to be 4.1 m thick with the lower boundary of this layer encountered at a depth of 8.7 m (Elevation 179.6). Boreholes JFR-01 to JFR-03 were advanced 1.1 m to 4.1 m into the silt layer and terminated at depths ranging from 7.6 m to 14.3 m (Elevations 179.3 to 174.7). A layer of silt, 1.2 m thick, was also encountered below the silty sand layer in Borehole 4 with a lower boundary at Elevation 180.1 m.

Boreholes JFR-01 to JFR-03, drilled on the west side of the Jackfish River, revealed that the silt and sandy silt layers are in a compact to dense state, based on SPT 'N' values ranging from 12 to 66 blows per 0.3 m of penetration. SPT 'N' values measured in Borehole JFR-05 were 1 and 3 blows per 0.3 m of penetration, indicating a very loose relative density.

Moisture contents of the silt to sandy silt ranged from 16 % to 38%. In general, higher moisture contents were measured in silt/sandy silt samples collected from the east side of the river.

Three samples of the silt to sandy silt from the current investigation underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the corresponding Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are plotted on Figure B3, Appendix B.

Gravel %	0
Sand %	8 to 27
Silt %	69 to 83
Clay %	4 to 9

### 5.7 Upper Sand and Gravel

A layer of sand and gravel was encountered in the boreholes drilled near the east river bank and east abutment (Boreholes 3, 4, 8, and 10 from the 1958 investigation and Borehole JFR-04 from the current investigation). The sand and gravel layer was encountered at the

surface of the river bed in Boreholes 8 and 10, below a thin layer of organics in Borehole 3 and below sand layer in Boreholes JFR-04 and 4.

The sand and gravel is grey to red and fine to coarse grained with occasional silty zones. Pieces of decayed wood were encountered in the sand and gravel in Borehole 8.

In Borehole 3, wood in the form of logs was encountered in the sand and gravel layer.

In Boreholes 3, 4, 8, and 10 the thickness of the sand and gravel layer ranged from 4.6 m to 7.7 m, with the lower boundary of the sand and gravel encountered at depths of 5.2 m to 9.4 m (Elevation 176.8 to 175.4). The sand and gravel layer was not fully penetrated in Borehole JFR-04, which was terminated at a depth of 11.3 m (Elevation 177.3), 2.6 m into the sand and gravel layer.

SPT 'N' values recorded in the sand and gravel ranged from 2 to 43 blows for 0.3 m penetration, indicating a variable density ranging from very loose to dense. In general, SPT 'N' values were greater than 12 (compact) and increased with depth.

Moisture contents of the sand and gravel samples from the current investigation ranged from 8% to 9%.

One sample of the sand and gravel from the current investigation underwent laboratory grain size analysis testing. The results of this test are presented on the corresponding Record of Borehole sheets included in Appendix A and the grain size distribution curve for this sample is plotted on Figure B5 of Appendix B. The results of this test are summarized as follows:

Gravel %	38
Sand %	43
Silt & Clay %	19

## **5.8 Lower Sand and Gravel**

A thin layer of sand and gravel was encountered at depth in Boreholes 1, 4, 5, and 8. Occasional cobbles and boulders were noted within this layer.

This sand and gravel layer was fully penetrated in Boreholes 5 and 8 only and the layer is present just above the bedrock surface. In these two boreholes the sand and gravel layer was 0.8 m thick, with the lower boundary of the layer encountered at Elevations 157.2 and 159.3. Boreholes 1 and 4 were terminated at Elevations 159.0 and 159.4, respectively and penetrated 0.9 m to 2.3 m into the sand and gravel layer.

Coring techniques with an AX sized core barrel were used to advance the boreholes through the sand and gravel layer containing cobble and boulders.

## 5.9 Bedrock

Granite bedrock was proven by coring in two of the boreholes drilled during the 1958 investigation (Boreholes 5 and 8). Table 5.2 summarizes depths and elevations to the top of bedrock in these boreholes.

**Table 5.2 – Depths and Elevations of Top of Bedrock**

Foundation Unit	Borehole	Top of Bedrock	
		Depth (m) Below riverbed	Elevation (m)
West bank	5	23.1	157.2
East bank	8	23.0	159.3

Core recovery and RQD values for the granite bedrock were not recorded in the investigation report from 1958.

## 5.10 Water Levels

Water levels were observed upon completion of drilling operations in the open boreholes. The water levels measured in the open boreholes are summarized in Table 5.3.

**Table 5.3 – Water Level Measurements**

Borehole	Date	Water Level (m)		Comment
		Depth	Elevation	
JFR-02	February 12, 2011	8.6	180.4	Open borehole
JFR-04	February 10, 2011	7.2	181.4	Open borehole
1	Fall 1958	2.0	184.0	A few days after drilling
2	Fall 1958	1.3	183.9	A few days after drilling
3	Fall 1958	1.6	183.1	A few days after drilling
4	Fall 1958	1.5	183.5	A few days after drilling

Based on the General Arrangement drawing dated June 2012, the water level in Jackfish River was at Elevation 182.8 m in February 2011 and the high water level has been identified at Elevation 183.3.

The water level of the Jackfish River during the 1958 investigation was at elevation 183.1.

The investigation conducted in 1958 reported that when the sand and gravel layer located immediately above the bedrock was reached, water quickly rose in the casing, denoting artesian pressure in the coarse deposit. The maximum levels recorded in Boreholes 5 and 8 were at elevations 185.4 and 185.5, respectively. This represented a maximum artesian

head of 2.3 m to 2.4 m of water above the existing river level at the time. No artesian pressures were observed in the soils above this deep sand/gravel layer.

Fluctuations of the groundwater level and river level are to be expected and subject to seasonal conditions. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

## 6 MISCELLANEOUS

The borehole locations were established in the field by Thurber Engineering. 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. supplied a truck-mounted drill rig and conducted the drilling, sampling and in-situ testing operations for the boreholes located on the highway at this site. Ohlmann Geotechnical Services (OGS) supplied portable drilling equipment and conducted the drilling, sampling and in-situ testing operations for the two boreholes drilled through the tremie grout of the existing pier.

The concrete core testing was conducted by Davroc Testing Laboratories Inc.

The field program was supervised on a full time basis by Mr. Ryan Kromer, E.I.T of Thurber Engineering Ltd. Overall supervision of the field program was provided by Mr. Alastair E. Gorman, P.Eng. and Mr. Tony Harte, M.Sc.

Interpretation of the data and preparation of the report was carried out by Ms. Lindsey Blaine, E.I.T. 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 INTRODUCTION**

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical recommendations for the proposed replacement of the existing Jackfish River Bridge on Highway 17.

The existing Jackfish River Bridge is a two-span structure with a total length of 52.3 m and width of 11.0m supported on two abutments and a pier. The abutments are supported on 0.3 m diameter concrete filled steel tube piles having a minimum length of 9.1 m. The pier foundation consists of an approximate 2.9 m wide by 12.0 m long steel sheet pile enclosure infilled with tremie concrete above the river bottom. The sheet piles are approximately 7.6 m long and driven to elevation 176.2. The existing west and east embankments are approximately 3 m to 5 m high.

Based on the preliminary General Arrangement (GA) drawing provided by MRC, a single-span structure supported on two abutments is proposed. The abutments are proposed to be founded on driven steel H-piles with a sheet pile wall driven just behind the H-piles to contain the approach fill. The total length of the structure will be 43.5 m. The proposed structure will be approximately 12.75 m wide. The existing structure will be replaced maintaining the same alignment for the new structure, as well as the same highway grade.

It must be noted that the field investigation at this site was carried out for the original design of bridge rehabilitation and the investigation was designed to provide recommendations for roadway protection at the abutments and strengthening of pier foundation. The design was subsequently changed to a bridge replacement design (consisting of a single-span structure), after the field investigation was completed. Although a request was made for drilling additional boreholes to bedrock at the new abutment locations, MTO indicated that the foundation recommendations for the new bridge abutments be based on the available factual data obtained during the course of the present investigation and the 1958 foundation report prepared for MTO by Trow Soderman and

Associates. The plans and profiles used for preparation of this report were provided by McCormick Rankin Corporation.

## **8 STRUCTURE FOUNDATIONS**

The borehole information indicates that the existing roadway embankment fill consists of sand, trace gravel to gravelly, and is approximately 3.0 m to 3.1 m thick in the west approach and 4.6 m thick in the east approach. In general terms, the west approach embankment is underlain by very soft to very stiff cohesive deposits of silty clay to clayey silt, and the east embankment is underlain by very loose to compact cohesionless deposits of silt, sand, and sand and gravel. An 8.4 m thick layer of very soft silty clay was also encountered below the silt about 20 m east of the east abutment. These layers are underlain by an extensive deposit of typically compact to dense sand to silty sand with a thickness 16.2 to 22.7 m in boreholes where fully penetrated. A thin layer of sand and gravel with cobbles was encountered below the silty sand/sand, and granite bedrock was contacted at depths of 23.1 and 23.0 m (elevations 157.2 and 159.3) in two boreholes.

Water was observed in Boreholes JFR-02 and JFR-04 at 8.6 m and 7.2 m depth (elevations 180.4 and 181.4) upon completion of drilling. The groundwater level is expected to be at or slightly above the water level in Jackfish River, which is indicated on the preliminary General Arrangement drawing to have been at Elevation 182.8 in February 2011, with a high water level at Elevation 183.3.

Although not noted on the borehole logs, the 1958 report indicates that artesian pressures exist within the sand and gravel layer immediately above the bedrock, with an artesian head measured at elevation 185.4 to 185.5, some 2.2 to 2.3 m above the water level in the river at that time.

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

- Spread footings on native soils or engineered fill
- Augered Caissons (drilled shafts)
- Steel H-piles driven to refusal on bedrock
- Steel pipe piles to bedrock

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

The existing bridge abutments should be monitored for settlement during pile driving.

### **8.1 Spread Footings on Native Soils or Engineered Fill**

Consideration was given to supporting the structure on spread footings founded on native soils or engineered fill, however this option is not recommended due to the following factors:



- The loose/soft native soils immediately underlying the embankment fill at this site are considered unsuitable for support of spread footings.
- Excavation for construction of spread footings or engineered fill on the underlying competent soils would extend some 3 to 4 m below the river water level (to approximate elevation 179 to 180) and require installation of sheet pile cofferdams for dewatering and excavation stability purposes.
- The native soils are variable and differential settlements may occur if footings are placed on the native soils.
- Spread footings could be subject to erosion or undermining/scour during high river flows.
- Footing excavation may have adverse environmental impact on the adjacent river.

In light of the above factors, the spread footings option was not further developed

## **8.2 Augered Caissons (drilled shafts)**

Augered caisson foundations were also considered for supporting the structure at this site.

However, in order to reach competent bearing strata, the caissons would have to extend through a thick deposit of cohesionless sand/silty sand below the groundwater table, and be founded on bedrock contacted at 23.1 m and 23.0 m depth in two boreholes. Further, the bedrock is overlain by a layer of sand and gravel exhibiting artesian groundwater pressure. Sealing of the caisson liner into the founding stratum to exclude groundwater and prevent washing of cohesionless soils into the liner would be difficult in these conditions.

In view of the artesian pressures, cohesionless soils below the groundwater table, and the significant depth to bedrock, the use of augered caissons is not recommended at this site from a geotechnical perspective.

## **8.3 Driven Steel H-Piles**

Consideration was given to supporting the abutments on steel H-piles driven to refusal on bedrock or terminating into the layer of dense sand and gravel with cobbles and boulders contacted just above the bedrock.

In general, it is anticipated that the piles will encounter refusal at the bedrock surface. In the 1958 investigation, the top of bedrock was proven by coring in two boreholes (Boreholes 5 and 8) at elevation 157.2 and 159.3. However, in two additional boreholes (Boreholes 1 and 4), casing refusal was encountered at elevation 158.9 and 159.4 in the sand and gravel layer containing cobbles and boulders which overlies bedrock.

The new abutments will be situated in front of the existing abutments, between the borehole locations at which bedrock and casing refusal were contacted in the previous



investigation. On the basis of the existing borehole data, the estimated pile lengths and pile tip elevations for piles driven to bedrock or terminating into the overlying sand and gravel layer containing cobbles and boulders are as follows:

**Table 8.1 – Pile Lengths and Tip Elevations**

<b>Locations</b>	<b>Anticipated Pile Length (m)*</b>	<b>Anticipated Pile Tip Elevation</b>
West Abutment	28.4	157.6
East Abutment	27.8	158.2

\* Assuming a pile cut-off elevation of 186.0

It should be noted that the bedrock elevation was not established at the abutment locations and therefore the pile tip elevation may vary from the anticipated levels.

The actual pile tip elevations will be controlled as described in Section 8.3.2 Pile Installation.

In recognition of the fact that some piles may not reach bedrock and instead may terminate in the overlying sand and gravel layer containing cobbles and boulders, a reduced geotechnical resistance of 1,600 kN factored Ultimate Limit States (ULS<sub>f</sub>) and 1,400 kN Serviceability Limit States (SLS) is recommended for the HP 310x110 terminated in the sand and gravel layer just above the bedrock.

The existing bridge abutments should be monitored for settlement during pile driving.

### **8.3.1 Pile Tips**

For H-piles driven to bedrock or refusal on cobbles and boulders, the tips of all piles should be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

### **8.3.2 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 possibility of some driven piles meeting refusal on cobbles and boulders above the bedrock.

For piles driven into the layer of sand and gravel with 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 2.0 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.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerances, a driving template or other means may be required to achieve the specified maximum deviation.

### 8.3.3 Artesian Conditions

Artesian conditions were noted in the sand and gravel layer immediately above the bedrock in Boreholes 5 and 8, drilled during the investigation conducted in 1958. The maximum artesian head was 2.2 m to 2.3 m above the water level present in the river at the time.

Typically, artesian pressure has the potential to cause flow up the pile shaft. At this site however, the piles will extend through relatively permeable, cohesionless sand and silty sand. A clayey silt to silty clay cap exists in the west approach area. It is anticipated that any artesian pressures will dissipate within the thick sand deposit and not rise to the ground surface along the pile shaft. Furthermore, Boreholes 1, 4, JFR-02 and JFR-04 in the west and east abutment areas did not exhibit artesian conditions. Therefore the artesian pressure is not considered to be an issue.

### 8.3.4 Downdrag

Since no grade raise is anticipated at this site, downdrag on the piles is not considered to be an issue.

### 8.3.5 Lateral Resistance

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 in metres
	$n_h$	=	value from Table 8.2
	$\gamma$	=	unit weight (Table 8.2)
	$K_p$	=	passive earth pressure coefficient (Table 8.2)

For cohesive soils, the lateral resistance of the piles may be calculated as follows:

$$k_s = 67 \cdot S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 \cdot S_u \text{ (kPa) at and below a depth of } 3 \cdot D \text{ (m) reduced to zero at the ground surface}$$

where

$$D = \text{pile width in metres}$$

$$S_u = \text{undrained shear strength (kPa)}$$

**Table 8.2 – Parameters for Lateral Pile Resistance**

Location	Elevation	$n_h$ ( $\text{kN/m}^3$ )	$S_u$ kPa	$K_p$	Unit Weight ( $\text{kN/m}^3$ )	Soil Conditions
West abutment	185.9 to 178.8	-	30	2.7	9*	Silty clay to clayey silt, very stiff to very soft
	178.8 to 159.0	5,500	-	3.0	10.5*	Silty sand to sand, compact to dense
East abutment	184.0 to 179.9	3,000	-	3.0	10.5*	Sand with decayed wood fragments, compact
	179.9 to 175.6	5,000	-	3.0	10.5*	Sand and gravel, compact
	175.6 to 159.4	5,000	-	3.0	10.5*	Silty sand, compact to dense

\*Buoyant unit weight below the water table.

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,  $K = k_s \cdot L \cdot D$  ( $\text{kN/m}$ ), where  $k_s$  is the coefficient of horizontal subgrade reaction ( $\text{kN/m}^3$ ),  $D$  is the pile width (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 one pile be limited to no more than 120 kN at ULS and 50 kN at SLS.

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.

#### 8.4 Driven Steel Pipe Piles

A deep foundation alternative to support the bridge consists of steel pipe piles driven open-ended to refusal on bedrock or terminated in the layer of sand and gravel with cobbles and boulders contacted just above the bedrock then cleaned and filled with concrete.

It is anticipated that the pipe piles will encounter refusal on bedrock, however, some piles may terminate in the layer of sand and gravel containing cobbles and boulders, just above the bedrock.

For pipe piles driven to bedrock or terminating in the sand and gravel layer containing cobbles and boulders, just above the bedrock, the vertical geotechnical resistances recommended for selected pile sections are presented in Table 8.3. The reduced capacities provided in Table 8.3, account for piles terminating in the sand and gravel layer above the bedrock.

**Table 8.3 – Recommended Geotechnical Resistance for Pipe Piles**

Pipe Pile Section		Factored Geotechnical Resistance at ULS (kN)	Serviceability Limit States (SLS) (kN)
Diameter (mm)	Wall Thickness (mm)		
324	9.5	1,300	1,100
	12.7	1,400	1,200
508	12.7	2,500	2,100

The above resistances are for a steel yield strength of 240 MPa and concrete strength of 30 MPa. The structural resistance of the pipe pile must be checked by the structural engineer.

The anticipated pile length and tip elevations are as summarized in Table 8.1. The tips of all piles should be fitted with pipe pile tip protection from an approved manufacturer.

Pile installation should be in accordance with OPSS 903.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerances, a driving template or other means may be required to achieve the specified maximum deviation.

During installation, the pipe pile is likely to contain water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

Since no grade raise is anticipated at this site, downdrag on the piles is not considered to be an issue.

The lateral resistance of pipe piles may be computed as outlined in Section 8.3.4.

## **8.5 Recommended Foundation**

From a geotechnical perspective and based on the subsurface conditions, a deep foundation system comprising steel H-piles driven to bedrock or terminating in the overlying sand and gravel layer containing cobbles and boulders is the recommended foundation option for supporting the bridge at this site.

## **8.6 Frost Cover**

The design depth of frost penetration at this site is 2.3 m.

The GA drawing indicates that the pile cap will be exposed below the girder. In light of the significant depth of the pile foundation, frost jacking of the piles is not an issue at this site.

If the design concept changes to the use of a pile cap, frost protection should be provided which consists of a minimum of 2.3 m of soil cover.

# **9 SHEET PILE WALLS AND LATERAL EARTH PRESSURES**

Steel sheet pile walls will be driven 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.

Driving of the sheet piles through the existing approach fill, soft to very stiff clay and very loose to compact silt/sand, is considered feasible based on the borehole data. Any rip rap or rock protection must be removed from the area prior to driving of the sheet piles.

Backfill to the sheet pile walls should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150. All granular material should meet the specifications of Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Earth pressures acting on the sheet pile walls may be assumed to be triangular and to be governed by the characteristics of the abutment backfill and the underlying 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)

$\gamma$  = 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 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 $\phi = 35^\circ$ , $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$ , $\gamma = 21.2 \text{ kN/m}^3$	
	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) is 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.

It is understood that lightweight materials such as EPS or foamed concrete are being considered for use behind the sheet pile walls to reduce the lateral loads imposed on the walls and minimize any embankment settlement. Provided the back face of the lightweight material is inclined at 2H:1V or flatter, the underlying ground will be essentially free-standing and the lower unit weight of the lightweight material may be used to compute the lateral earth pressures on the wall. A 300 mm thick sand bedding layer and a minimum 1.0 m granular cover must be provided for this material.

## 10 APPROACH EMBANKMENTS

Based on site observations and the GA drawing, the existing approach embankments are typically 3 to 5 m in height above the adjacent grades. The forward slopes towards the river are approximately 7.5 m to 8.5 m high from road grade to river channel bottom, with inclinations of 2H:1V or flatter. The foundation soils governing stability of the approach embankments consist generally of native soft to very stiff clayey silt to silty clay over compact to dense silt/sand at the west approach and loose to compact sand/silt over compact sand and gravel at the east approach.

The design of the proposed bridge replacement calls for shortening of the overall bridge span and placement of additional fill between the new sheet pile wall abutment and the existing abutment. Road grade on Highway 17 will be maintained. The use of lightweight material (EPS or foamed concrete) in the backfill is being considered.

Comments regarding the stability of embankment slopes and new sheet pile abutment, as well as settlement of the foundations soils under the new fill loading are provided in the following sections.

If any new fill is placed against an existing embankment, the existing sloped embankment surfaces should be appropriately benched, as per OPSD 208.010, after stripping of vegetation/organics, soft soils or otherwise unsuitable materials. Embankment construction and widening should be carried in accordance with OPSS 206.

### 10.1 Slope Stability

The existing embankment side slopes appear to be performing satisfactorily under the existing conditions. No grade raise or widening is planned.

A stability analysis was conducted to assess the global stability of the new sheet pile wall abutment, for the case with full granular backfill and for a case incorporating EPS (unit weight of  $1.5 \text{ kN/m}^3$  and 3.0 m thick). The analyses were carried out using the Morgenstern-Price method of slope stability analysis.



The results of the analysis indicate that an adequate factor of safety of 1.5 is achieved for the sheet pile wall enclosure backfilled with granular material (long term/drained condition) provided the sheet piling is driven to at least elevation 176.8 at the west abutment and 179.0 at the east abutment. For backfill incorporating lightweight material, an adequate factor of safety of 1.5 is achieved provided the sheet piling is driven to at least elevation 177.3 at the west abutment and 181.7 at the east abutment.

The slope stability computation output is included in Appendix F.

The stability of the embankment was not checked under seismic loading as the zonal acceleration at this site is 0.0 g.

## 10.2 Settlement

The placement of new granular fill behind the sheet pile abutments will induce immediate (elastic) settlement in the existing non-cohesive fill and silt/sand layers as well as time dependent (consolidation) settlement in the silty clay/clayey silt at the west abutment.

The total immediate and consolidation settlements were assessed using elastic methods and one-dimensional consolidation theory. Based on these analyses, the anticipated immediate and consolidation settlements under the new fill loading at the bridge approaches, for the cases with full granular backfill and with incorporation of 3 m of EPS are as follows:

Location	Case	Elastic Settlement (mm)	Consolidation Settlement (mm)	Total Settlement (mm)
West Approach	Granular	20	30	50
	EPS	10	15	25
East Approach	Granular	25	-	25
	EPS	15	-	15

The consolidation settlement at the west embankment is expected to be completed within 4 to 6 months of completion of fill placement. Inspection of the roadway surface and padding of the asphalt at the approaches to re-establish grades as necessary should be implemented during and after construction.

At the east abutment, due to the noncohesive nature of the foundation soils, the elastic settlement will be immediate and essentially completed when construction of the fill is completed.

## 11 EROSION CONTROL

Erosion and scour protection should be provided along the lower parts of any slopes 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.

## **12 EXCAVATION AND GROUNDWATER CONTROL**

If temporary excavations are required at the abutments, they 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.

Water levels were observed upon completion of drilling in Boreholes JFR-02 and JFR-04 at 8.6 m and 7.2 m depth, (elevations 180.4 and 181.4), respectively. Based on the General Arrangement drawing dated March 2012, the water level in Jackfish River was at Elevation 182.8 m in February 2011 and the high water level has been identified at Elevation 183.3.

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

It is recommended that any excavation for removal of existing structures be maintained above the water level in the river. If required, any excavation below the groundwater level/river level without dewatering is not recommended since the inflow of groundwater will make 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.

## **13 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 17.

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

Conventional steel soldier pile and timber lagging walls or continuous sheet pile walls are two options to provide temporary support to the roadway during excavation. Timber lagging boards should be installed as soon as the soil face is exposed and properly prepared.

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



$\gamma$	=	21 kN/m <sup>3</sup>	bulk unit weight
$\gamma'$	=	11 kN/m <sup>3</sup>	submerged unit weight below groundwater table
$K_a$	=	0.33	active pressure coefficient for sand fill and native sand /silt
	=	0.38	active pressure coefficient for silty clay
$K_p$	=	3.0	passive pressure coefficient for sand fill and native sand /silt
	=	2.7	passive pressure coefficient for silty clay
$h_w$	=	183	elevation of groundwater table

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.

#### 14 SEISMIC CONSIDERATIONS

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 has been classified as Type II. Therefore, according to Table 4.4 of the CHBDC, respective Site Coefficients "S" (ground motion amplification factor) of 1.2 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 14.1 may be used:

**Table 14.1 – Earth Pressure Coefficients for Earthquake Loading**

Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ $\gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Type III $\phi = 32^\circ$ , $\gamma = 21.2 \text{ kN/m}^3$	Existing Sand fill, Native Sand/Silt/Sand and gravel $\phi = 30^\circ$ $\gamma = 20 \text{ kN/m}^3$	Native Silty Clay $\phi = 27^\circ$ $\gamma = 20 \text{ kN/m}^3$
Active ( $K_{AE}$ )*	0.28	0.32	0.34	0.39
Passive ( $K_{PE}$ )	3.7	3.2	2.9	2.7
At Rest ( $K_{OE}$ )**	0.45	0.50	0.53	0.57

\* 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.

The existing embankments are above the groundwater level and are not considered to be in danger of undergoing liquefaction. Some toe failure may occur but it is expected to be of limited nature and readily repairable.

## 15 CONSTRUCTION CONCERNS

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

- The bedrock elevation was not established at the abutment locations and therefore the pile tip elevations may vary from the anticipated levels.
- A sand and gravel layer with cobbles was encountered above the bedrock surface. Piles may encounter refusal in this layer above the bedrock surface.
- The existing bridge abutments should be monitored for settlement during pile driving.
- Any required excavation should be maintained above the water level in the river.

## 16 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.

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# 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 <sup>(1)</sup> '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

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


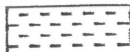



 Water Level  
 Shear Strength Determination by Pocket Penetrometer

- (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
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.






**Appendix A**  
**Record of Borehole Sheets**  
**(Current Investigation)**

RECORD OF BOREHOLE No JFR-01

1 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 504.9 E 225 976.5 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2011.02.12 - 2011.02.12 CHECKED BY TJH

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
								○ UNCONFINED		+ FIELD VANE		w <sub>P</sub> w      w <sub>L</sub>				
								● QUICK TRIAXIAL		× LAB VANE						
189.1						20	40	60	80	100	20	40	60			
0.0	ASPHALT: (75mm)  SAND, trace to some gravel, some silt Compact to very dense (frozen) Brown Moist (FILL)		1	AS												
0.1																
			1	SS	100/											
					0.150											
			2	SS	100/											
			2	SS	100/											
			3	SS	18											
	Layer of organics (50mm)															
186.1																
3.0	Silty CLAY, silt seams, trace sand Very Stiff Greenish Grey to Grey   Soft		4	SS	17											
					5	SS	3									
183.0																
6.1	Clayey SILT Firm Grey		6	SS	5											
181.9																
7.2	Silty CLAY, silt seams Very Stiff Grey		7	SS	22											
180.4																
8.7	Sandy SILT Dense Grey Moist to Wet		8	SS	39											
179.3																
9.8	End of sampling at 9.8m and start															

Continued Next Page

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



RECORD OF BOREHOLE No JFR-01

2 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 504.9 E 225 976.5 ORIGINATED BY RK  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
 DATUM Geodetic DATE 2011.02.12 - 2011.02.12 CHECKED BY TJH

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	20 40 60	WATER CONTENT (%)		GR SA SI CL
	Continued From Previous Page												
177.8	DCPT						179						
11.3	END OF BOREHOLE AND DCPT AT 11.3m. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO 6.0m, HOLEPLUG AND AUGER CUTTINGS TO 3.0m, AUGER CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.						178						

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+<sup>3</sup> x<sup>3</sup>: Numbers refer to Sensitivity

20  
15 10 5  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No JFR-02

1 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 522.0 E 225 982.6 ORIGINATED BY RK  
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/Casing COMPILED BY AN  
 DATUM Geodetic DATE 2011.02.12 - 2011.02.12 CHECKED BY TJH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
189.0 0.0 0.1	ASPHALT: (75mm)  SAND, trace to some gravel Very Dense (frozen) Brown Moist (FILL)		1	AS			189							
			1	SS	100/ 0.275		188							
			2	SS	70		187							
			3	SS	66		186							
186.3 2.7 185.9 3.1	Silty SAND Brown Moist (FILL) Layer of organics (50mm)  Silty CLAY, silt seams, trace sand Very Stiff to Very Soft Greenish Grey		4	SS	18		185							
			5	SS	4		184							
			6	SS	0		183							
			7	SS	4		182							
181.7 7.3	Clayey SILT, trace sand Firm to Very Stiff Grey Wet		8	SS	18		181							
							180							

Continued Next Page

+ <sup>3</sup> , X <sup>3</sup> : Numbers refer to  
Sensitivity

20  
15 5  
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No JFR-02

2 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 522.0 E 225 982.6 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Hollow Stem Augers/Casing COMPILED BY AN  
DATUM Geodetic DATE 2011.02.12 - 2011.02.12 CHECKED BY TJH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100					
	Continued From Previous Page												
178.8	Clayey SILT, trace sand					179							
10.2	SILT, some sand to sandy Compact Grey to Brown Wet		9	SS	12	178							
						177							
			10	SS	20	176							
						175							
174.7			11	SS	13	174							
14.3	End of sampling at 14.3m and start DCPT					173							
						172							
						171							
170.5													
18.5	END OF BOREHOLE AND DCPT AT 18.5m. WATER LEVEL AT 8.6m UPON COMPLETION. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO 7.6m, HOLEPLUG AND AUGER CUTTINGS TO 3.0m, AUGER CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.												

+<sup>3</sup> ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No JFR-03

1 OF 1

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 551.9 E 225 998.4 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Coring/Tripod - Wash Coring COMPILED BY MFA  
DATUM Geodetic DATE 2011.02.17 - 2011.02.17 CHECKED BY TJH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
								○ UNCONFINED      + FIELD VANE		● QUICK TRIAXIAL    x LAB VANE					
183.5							20	40	60	80	100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
0.0	CONCRETE (Cored through concrete surrounding the pier)		1	RUN								W <sub>P</sub>	W	W <sub>L</sub>	
			2	RUN											
			3	RUN											
			4	RUN											
			5	RUN											
178.6															
4.9	Silty <b>SAND</b> , some gravel, some clay, decayed wood fragments Compact Grey Wet		1	SS	10										12 59 29 (SI+CL)
177.7															
5.8	Sandy <b>SILT</b> , trace silt and clay, occasional sand layers Compact to Very Dense Grey Wet		2	SS	24										
			3	SS	29										
			4	SS	66										0 27 69 4
175.9															
7.6	End of sampling at 7.6m and start DCPT														
175.4															
8.1	END OF BOREHOLE AND DCPT AT 8.1m. BOREHOLE BACKFILLED WITH SILICA SAND TO 4.9m, THEN CEMENT TO SURFACE.														

+<sup>3</sup>, X<sup>3</sup>: Numbers refer to Sensitivity  
20  
15 10 5  
(%) STRAIN AT FAILURE

## METRIC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100		w <sub>p</sub> w w <sub>L</sub>				
								SHEAR STRENGTH kPa		WATER CONTENT (%)				
188.6								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	20 40 60 80 100	20 40 60			GR SA SI	

[illegible]

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity

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RECORD OF BOREHOLE No JFR-04

2 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 583.1 E 226 007.3 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2011.02.10 - 2011.02.10 CHECKED BY TJH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page													
177.3	SAND and GRAVEL, some silt and clay Compact Grey Wet		9	SS	22									
11.3	END OF BOREHOLE AT 11.3m. WATER LEVEL AT 7.2m UPON COMPLETION. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO 0.10m, THEN ASPHALT TO SURFACE.													
172.8														
15.8	End of DCPT at 15.8m.													

# RECORD OF BOREHOLE No JFR-05

1 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 603.8 E 226 009.3 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2011.02.13 - 2011.02.13 CHECKED BY TJH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
188.3														
0.0	ASPHALT: (75mm)													
0.1	SAND, some gravel to gravelly, trace silt and clay Very Dense to Compact (frozen) Brown Moist (FILL)		1	AS			188							
			1	SS	50/ 0.025		187							
			2	SS	100/ 0.150		186							
			3	SS	33		185							
			4	SS	18		184							
183.7							183							
4.6	Sandy SILT Loose Greenish Grey Moist		5	SS	9		182							
182.7							181							
5.6	SILT, trace clay, trace sand Very Loose Dark Brown to Grey Moist		6	SS	3		180							
							179							
179.6														
8.7	Silty CLAY, trace sand, occasional organics Very Soft Grey		8	SS	0									

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No JFR-05

2 OF 2

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 603.8 E 226 009.3 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN  
DATUM Geodetic DATE 2011.02.13 - 2011.02.13 CHECKED BY TJH

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
	Continued From Previous Page												
	Silty <b>CLAY</b> , trace sand Very Soft		9	SS	0								
			10	SS	0								
			11	SS	0								
			12	SS	0								
171.2			13	SS	0								
17.1	<b>SAND</b> , some clay												
170.9	Grey												
17.4	Wet												
	End of sampling at 17.3m and start DCPT.												
169.1													
19.2	END OF BOREHOLE AND DCPT AT 19.2m. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO 0.10m, THEN ASPHALT TO SURFACE.												

+<sup>3</sup>. ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No JFR-06

1 OF 1

METRIC

W.P. 465-00-00 LOCATION Jackfish River Bridge N 5 430 554.2 E 225 991.7 ORIGINATED BY RK  
HWY 17 BOREHOLE TYPE Coring/Tripod - Wash Boring COMPILED BY MFA  
DATUM Geodetic DATE 2011.02.20 - 2011.02.20 CHECKED BY TJH

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 <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W <sub>P</sub> W W <sub>L</sub>	20 40 60		
183.5 0.0	CONCRETE (Cored through concrete surrounding the pier)		1	RUN									
			2	RUN									
			3	RUN									
			4	RUN									
178.6 4.9	Silty SAND, some gravel to gravelly Dense Dark Brown Wet		1	SS	32								
178.0 5.5	End of sampling at 5.5m and start DCPT												
175.6 7.9	END OF BOREHOLE AND DCPT AT 7.9m. BOREHOLE BACKFILLED WITH HOLEPLUG TO 4.9m, THEN CONCRETE TO SURFACE.												

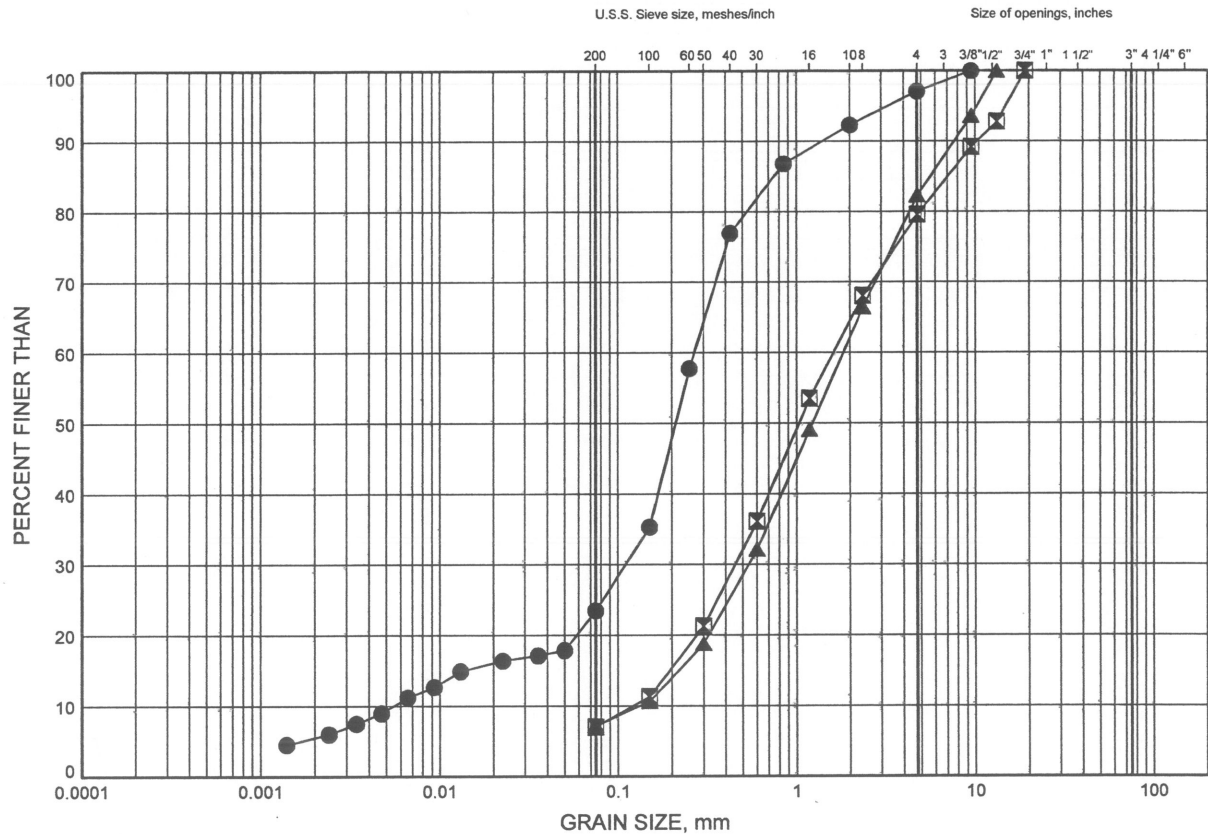
ONTMT4S 1197.GPJ 8/20/12

**Appendix B**  
**Laboratory Test Results**  
**(Current Investigation)**

# NWR 32 Rehabs GRAIN SIZE DISTRIBUTION

FIGURE B1

## SAND FILL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	JFR-01	2.59	186.51
■	JFR-04	2.59	186.01
▲	JFR-05	2.59	185.71



W.P.# .465-00-00.....  
Prepared By .MFA.....  
Checked By .MRA.....

## FIGURE B2

Figure 1 is a semi-logarithmic plot showing the relationship between grain size (mm) and percent finer than. The x-axis is logarithmic, ranging from 0.0001 mm to 100 mm, with corresponding U.S.S. Sieve size (meshes/inch) and Size of openings (inches) at the top. The y-axis is linear, ranging from 0 to 100 percent finer than. Five curves are plotted, each representing a different material, showing varying degrees of fineness. The curves generally show that as grain size increases, the percent finer than decreases, with some materials being significantly finer than others.

Grain Size (mm)	U.S.S. Sieve Size (meshes/inch)	Size of Openings (inches)	Curve 1 (Solid Circle) % Finer	Curve 2 (Solid Star) % Finer	Curve 3 (Open Square) % Finer	Curve 4 (Solid Triangle) % Finer	Curve 5 (Open Triangle) % Finer
0.0015	100	0.0039	35	25	24	14	-
0.0025	60	0.0075	42	35	31	19	-
0.0035	40	0.0106	53	43	37	24	14
0.005	30	0.0149	62	55	46	33	24
0.0075	20	0.0200	72	68	55	42	33
0.01	16	0.0250	81	77	63	53	42
0.015	10	0.0354	88	82	72	66	53
0.025	60	0.0500	92	86	78	77	66
0.035	40	0.0635	94	89	84	84	77
0.05	30	0.0850	96	91	87	87	84
0.075	20	0.1063	97	93	89	89	84
0.1	16	0.1250	98	94	90	90	84
0.15	10	0.1667	99	95	91	91	84
0.25	60	0.2500	100	96	92	92	84
0.425	40	0.4000	100	97	93	93	84
0.6	30	0.5000	100	98	94	94	84
1.0	16	0.7500	100	99	95	95	84
2.0	10	1.1875	100	100	96	96	84
4.75	4	2.0000	100	100	97	97	84
7.5	3	2.5000	100	100	98	98	84
10.0	3/8"	3.1250	100	100	99	99	84
12.5	1 1/2"	3.7500	100	100	100	100	84
15.0	1"	4.0000	100	100	100	100	84
17.5	3/4"	4.5000	100	100	100	100	84
20.0	3/4"	5.0000	100	100	100	100	84
25.0	3"	6.2500	100	100	100	100	84
30.0	4 1/4"	7.5000	100	100	100	100	84
35.0	4"	8.0000	100	100	100	100	84
40.0	3"	8.5000	100	100	100	100	84
45.0	3"	9.0000	100	100	100	100	84
50.0	3"	9.5000	100	100	100	100	84
55.0	3"	10.0000	100	100	100	100	84
60.0	3"	10.5000	100	100	100	100	84
65.0	3"	11.0000	100	100	100	100	84
70.0	3"	11.5000	100	100	100	100	84
75.0	3"	12.0000	100	100	100	100	84
80.0	3"	12.5000	100	100	100	100	84
85.0	3"	13.0000	100	100	100	100	84
90.0	3"	13.5000	100	100	100	100	84
95.0	3"	14.0000	100	100	100	100	84
100.0	3"	14.5000	100	100	100	100	84

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

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	JFR-01	7.92	181.18
☒	JFR-02	3.35	185.65
▲	JFR-02	9.45	179.55
★	JFR-05	12.50	175.80



W.P.# 465-00-00  
Prepared By MFA  
Checked By MRA

## FIGURE B3

Figure 1 is a semi-logarithmic graph showing the grain size distribution of a sample. The x-axis represents Grain Size in millimeters (mm) on a logarithmic scale, ranging from 0.0001 to 100. The y-axis represents Percent Finer Than, ranging from 0 to 100. The graph includes four data series, each represented by a different symbol and connected by lines. The top curve (triangles) shows the highest percentage of material finer than a given grain size, while the bottom curve (crosses) shows the lowest percentage. All curves exhibit a sharp drop in the percentage finer between approximately 0.075 mm and 0.425 mm, indicating a narrow range of grain sizes. The top curve reaches 100% finer at 0.425 mm, while the bottom curve reaches 100% finer at 0.850 mm.

Grain Size (mm)	Percent Finer Than (Triangles)	Percent Finer Than (Circles)	Percent Finer Than (Squares)	Percent Finer Than (Crosses)
0.0075	7	4	4	4
0.015	13	6	6	6
0.03	20	8	8	8
0.06	29	12	10	10
0.12	36	16	14	14
0.25	45	22	20	20
0.5	58	36	34	34
1.0	71	48	46	46
2.0	82	60	58	58
4.0	92	73	71	71
8.0	98	82	80	80
16.0	100	94	92	92
32.0	100	98	96	96
64.0	100	100	100	100

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

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	JFR-02	14.02	174.98
⊠	JFR-03	7.32	176.18
▲	JFR-05	6.40	181.90

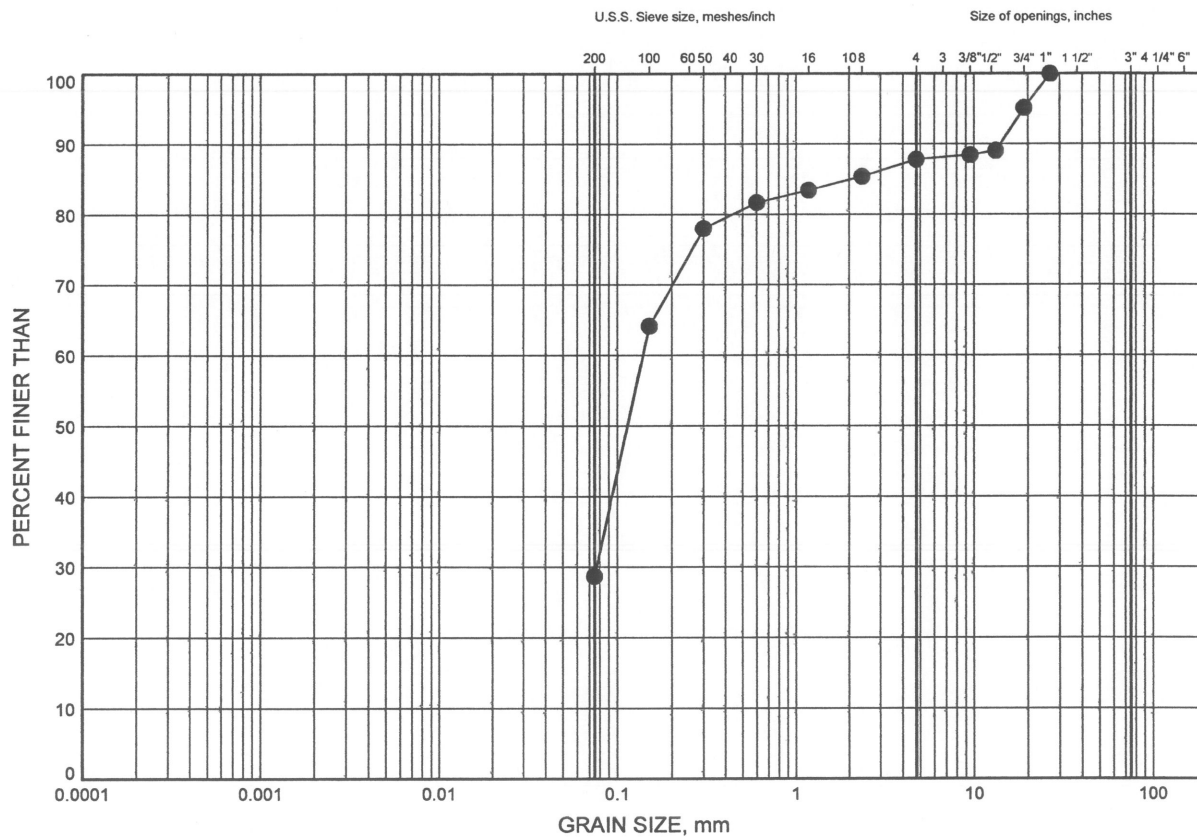


W.P.# 465-00-00  
Prepared By MFA  
Checked By MRA

# NWR 32 Rehabs GRAIN SIZE DISTRIBUTION

FIGURE B4

## SILTY SAND



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	JFR-03	5.33	178.17

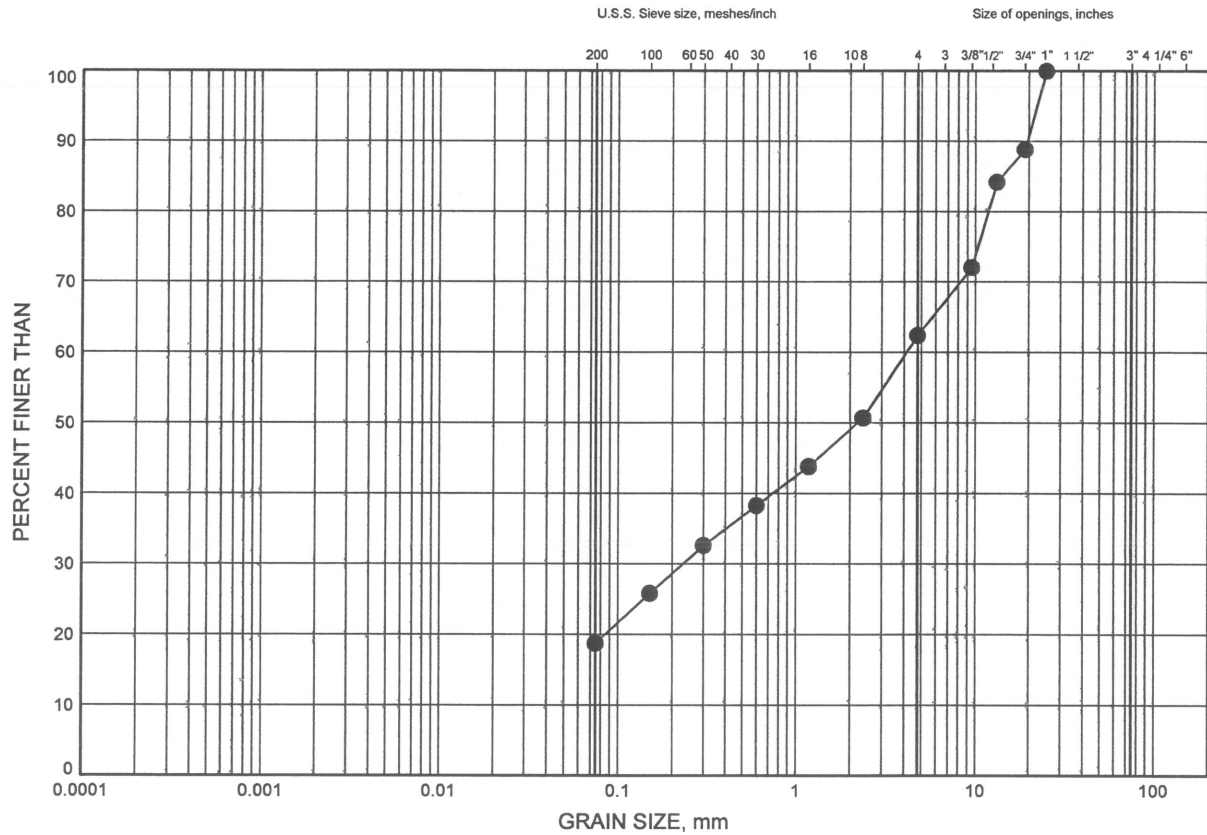


W.P.# 465-00-00  
Prepared By MFA  
Checked By MRA

# NWR 32 Rehabs GRAIN SIZE DISTRIBUTION

FIGURE B5

## SAND & GRAVEL



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

### LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	JFR-04	9.45	179.15

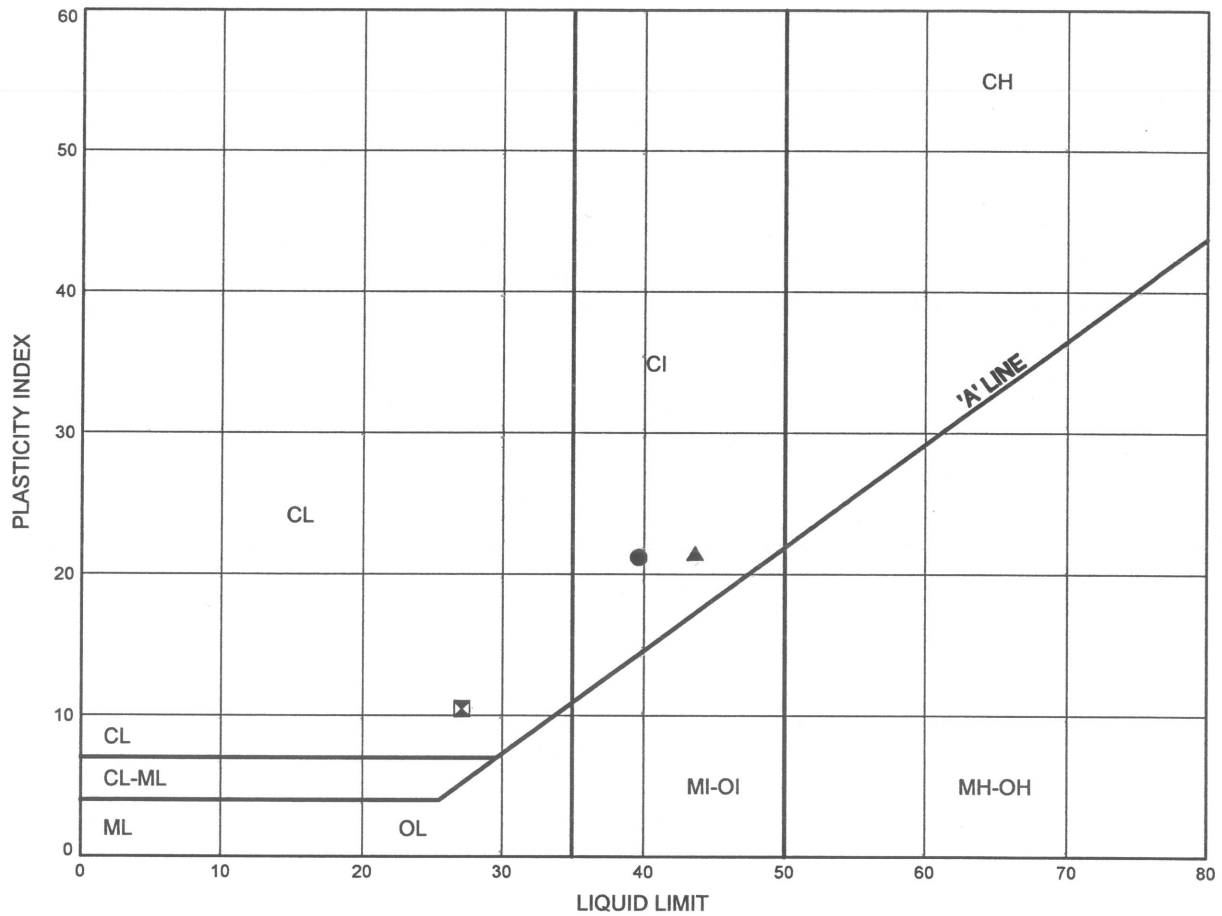


W.P.# 465-00-00  
Prepared By MFA  
Checked By MRA

NWR 32 Rehabs  
**ATTERBERG LIMITS TEST RESULTS**

FIGURE B6

**SILTY CLAY TO CLAYEY SILT**



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	JFR-02	3.35	185.65
⊠	JFR-02	9.45	179.55
▲	JFR-05	12.50	175.80

Date September 2011

Project 465-00-00



Prep'd MFA

Chkd. MRA



**Appendix C**  
**Laboratory Test Results of Concrete Cores and Core Photos**  
**(current investigation)**

**File: L11-0165AVS**

**Thurber Engineering Ltd.  
2010 Winston Park Dr., Suite 103  
Oakville, Ontario**

**November 14, 2011**

**Attn: Mr. Weiss Mehdawi, M.Eng., P.Eng.**

**Dear Sir;**

***Concrete Core Testing  
Project No.: 19-1351-197***

---

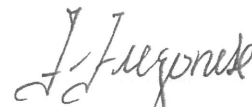
Further to receipt of four (4) 75 mm diameter nominal size concrete core samples in our laboratory on November 11, 2011, Davroc Testing Laboratories Inc., is pleased to report the results of our tests.

As per your request, each core was trimmed, end ground and tested for compressive strength in accordance with CSA Standard A23.2-14C, "Obtaining and Testing Drilled Cores for Compressive Strength Testing," in a dry condition.

The results of our core tests are shown on the attached Core Test Report form.

We trust this provides you with the information you require at this time. Should you require any further information, please do not hesitate to contact the undersigned.

**Your very truly,  
Davroc Testing Laboratories Inc.**



**Fabio Fregonese, C.E.T.  
Manager Materials Testing Services**



**Sal Fasullo, C.E.T.  
Vice President**

**SF/ff  
11-0165-12**

## CONCRETE CORE TEST REPORT

File No. L11-0165CC

Client Project Number: 13-1351-197

Davroc Sample No.:2421

Core No.	1	2	3	4
Location	Bore Hole JFR 03			
Nominal Size of Coarse Aggregate (mm)	20	20	20	20
Date Cast	Not Given	Not Given	Not Given	Not Given
Date Cored	Not Given	Not Given	Not Given	Not Given
Date Tested	November 11, 2011	November 11, 2011	November 11, 2011	November 11, 2011
Ground Height -(mm)	148.5	149.5	150.0	149.5
Average Diameter (mm)	75.0	75.0	75.0	75.0
Corrected Compressive Strength (MPa)	31.8	39.0	30.3	31.6
Mode of Failure	Satisfactory	Satisfactory	Satisfactory	Satisfactory
*Direction of Loading	Not Given	Not Given	Not Given	Not Given
** Moisture Condition at Time of Test	Dry	Dry	Dry	Dry
Concrete Compaction	Satisfactory	Satisfactory	Satisfactory	Satisfactory

Remarks: None

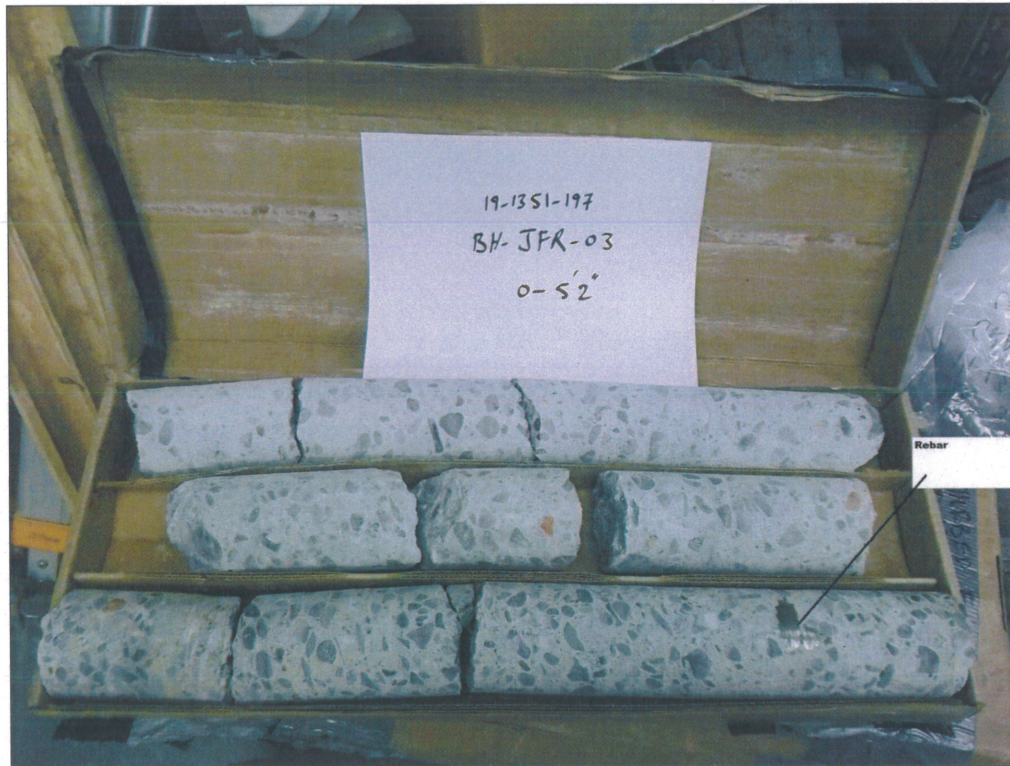
Date: November 11, 2011

Signed:

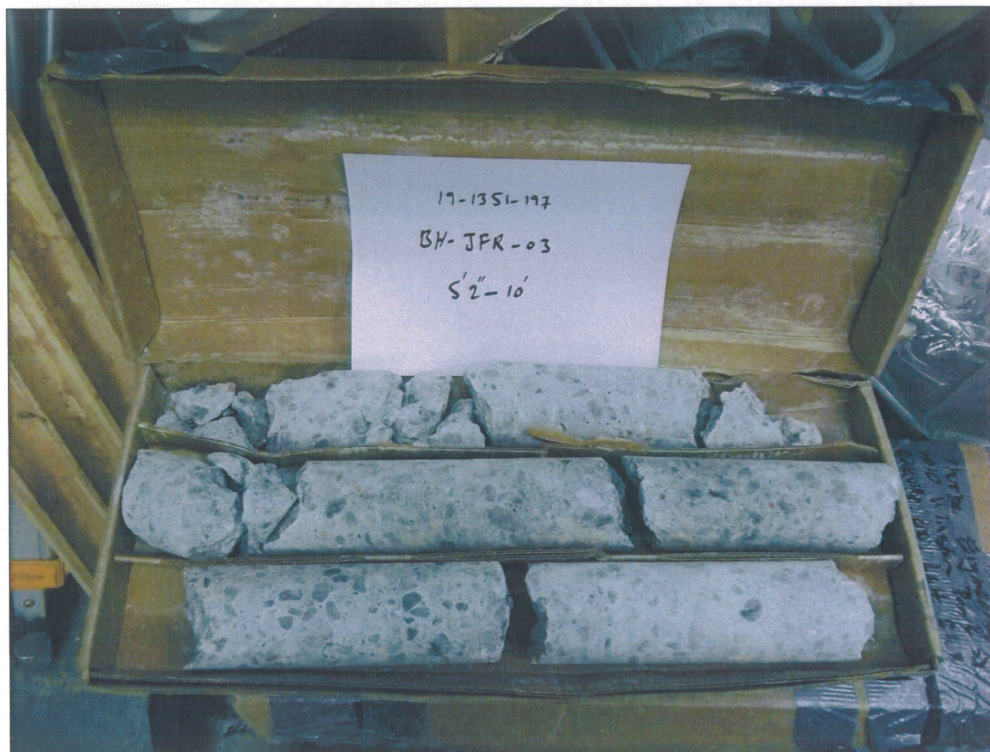
  
Sal Fasullo, C.E.T.

\*Relative to direction of compaction of concrete when placed.

\*\* Moisture conditioning as per clause 5.3.2 of the Test Method CSA A23.2-14C



**Photograph 1C – Concrete cores, Borehole JFR-03, 0.0 m to 1.6 m depth**



**Photograph 2C** – Concrete cores, Borehole JFR-03, 1.6 m to 3.0 m depth





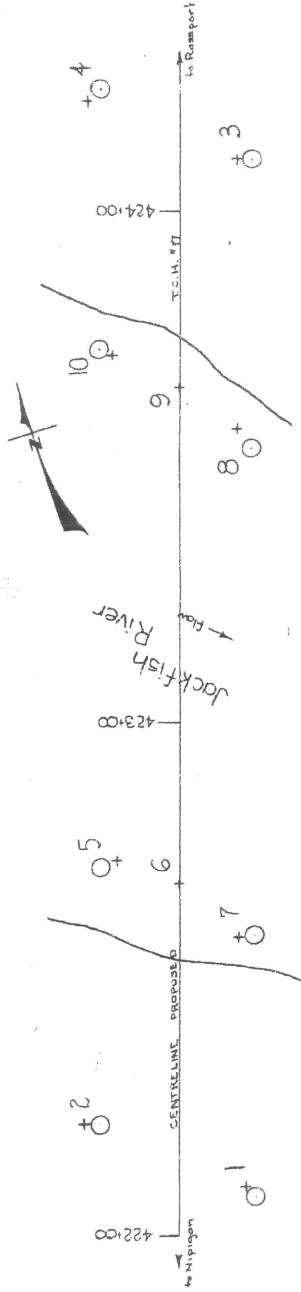
**Photograph 3C** – Concrete cores, Borehole JFR-03, 3.0 m to 4.2 m depth



**Photograph 4C** – Concrete cores, Borehole JFR-03, 4.2 m to 4.9 m depth

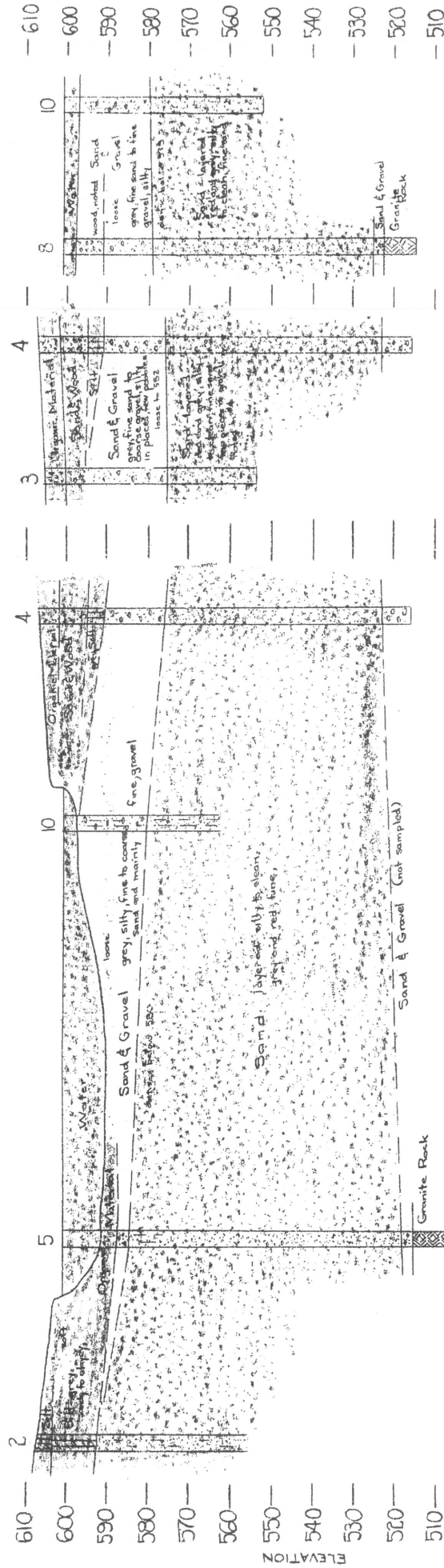
**Appendix D**  
**Record of Borehole Sheets**  
**(Previous Investigation)**

# JACKFISH RIVER



Borehole Location Plan SCALE 1"=20'

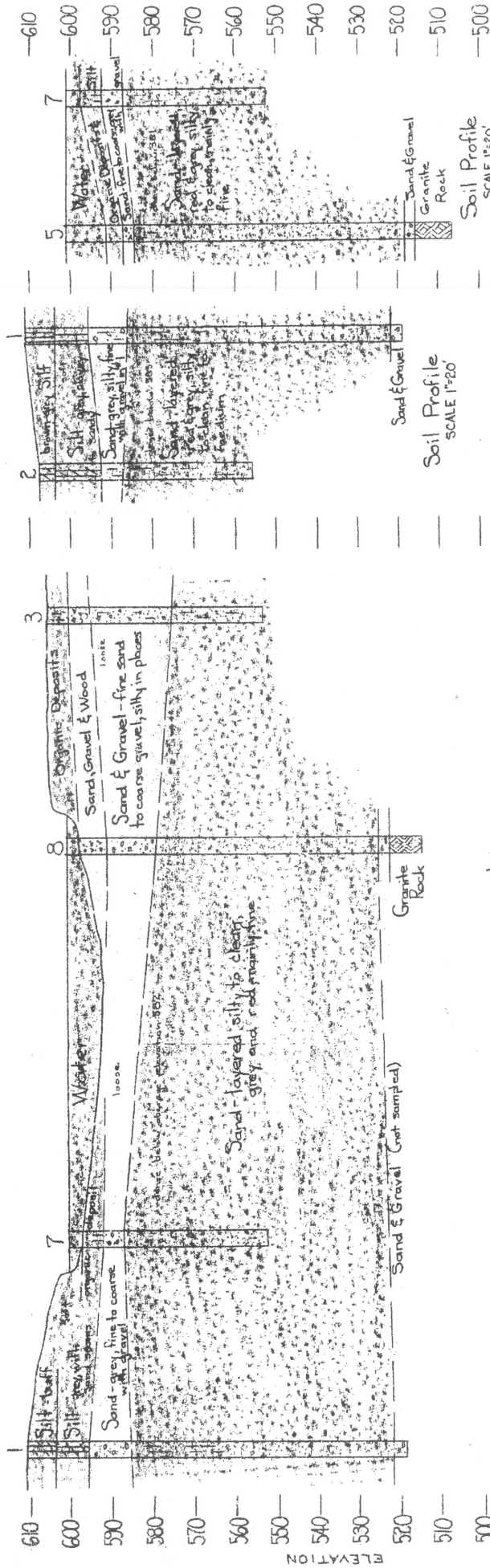
Legend Hole O Conn +



Projected Soils Profile between Boreholes 2, 5, 10 & 4 SCALE 1"=20'

Soil Profile SCALE 1"=20'

Soil Profile SCALE 1"=20'



Projected Soils Profile between Boreholes 1, 7, 8 & 3  
SCALE 1"=20'

# JACKFISH RIVER CROSSING



## TROW SODERMAN AND ASSOCIATES

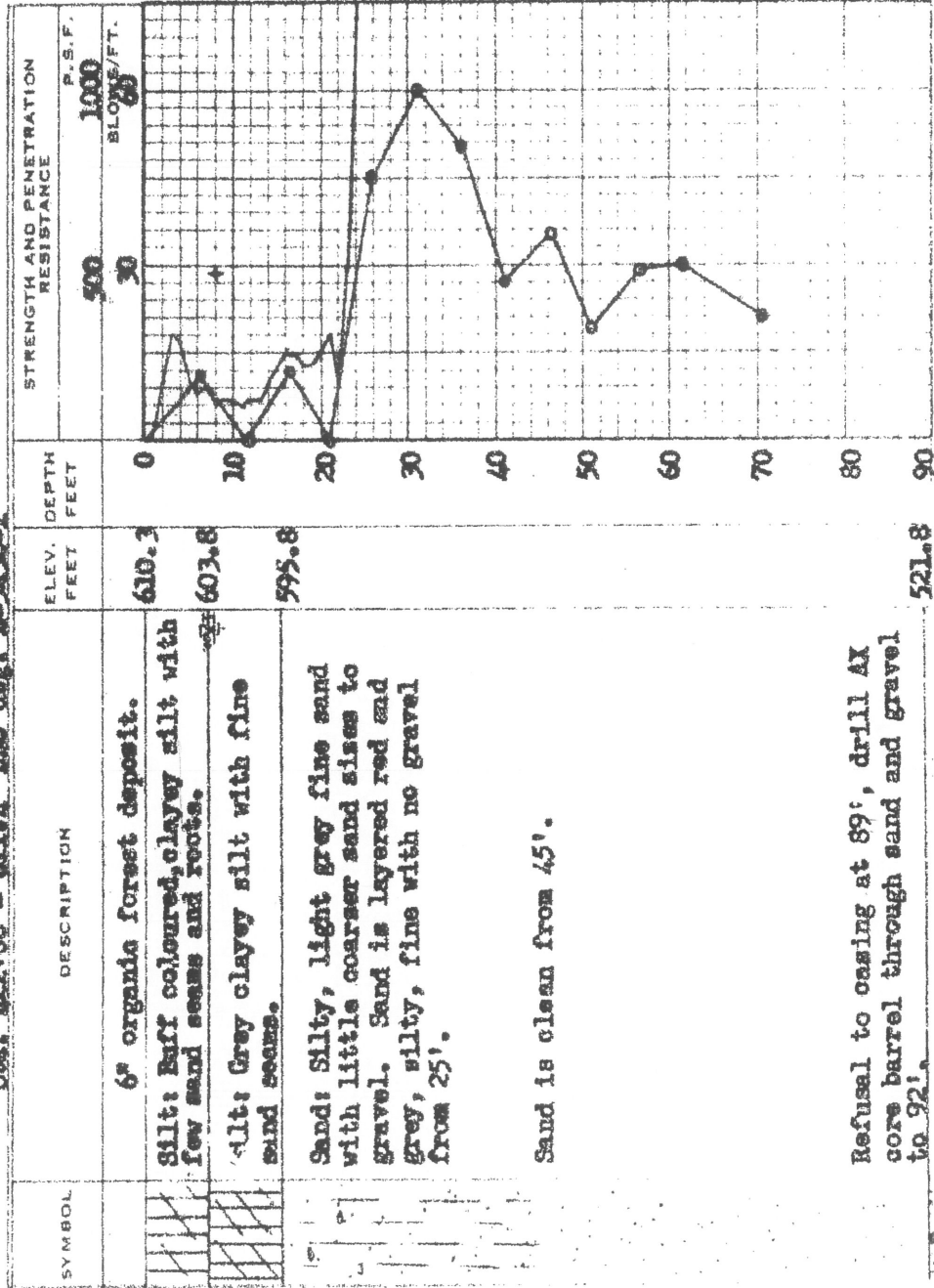
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

## LEGEND

2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
CASING

1/2 SHELBY  
UNCONFINED COMPRESSION (QU)  
VANE TEST (C) AND SENSITIVITY (S)  
NATURAL MOISTURE AND  
LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT

PROJECT Jackfish River Crossing, Hwy #17  
LOCATION T.C.H. near Nipigon, Ont.  
HOLE LOCATION See diag. No. 1  
HOLE ELEVATION AND DATUM 610.8  
Sta. 422+00 = 611.4 BEO diag. E-3432-1



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	
	SS11	
	SS12	
	SS13	

PROJECT NO. C108/1259

# TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

DRAWING NO. 4

## LEGEND

- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (QU)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT Jackfish River Crossing, Hwy #17,

LOCATION T.C.E. near Nipigon, Ont.

HOLE LOCATION See dug No. 1

HOLE ELEVATION AND DATUM 607.3

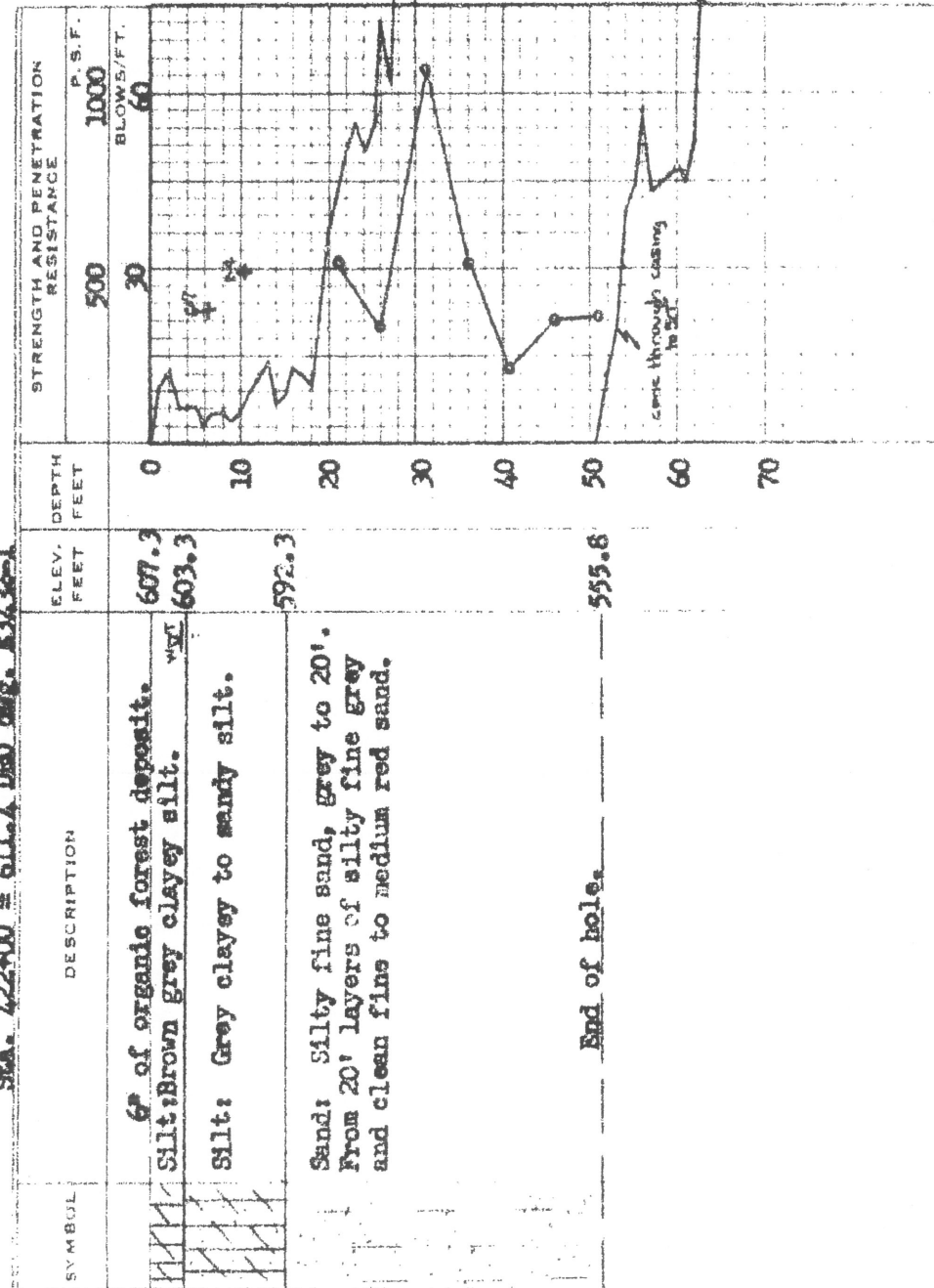
Sta. 422+00 = 611.4 DSD dug. E3432-1

BOREHOLE NO. 2

FIELD SUPERVISOR D.S.

DRILLER E.S.

PREP. D.S.



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	TV1	
	TV2	
	TV3	
	TV4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	
	SS11	

## TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

## LEGEND

- 2" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 CASING  
 2" SHELBY  
 1/2 UNCONFINED COMPRESSION (QU)  
 VANE TEST (C) AND SENSITIVITY (S)  
 NATURAL MOISTURE AND  
 LIQUIDITY INDEX  
 LIQUID LIMIT  
 PLASTIC LIMIT

PROJECT Jackfish River Crossing, Hwy #17,

LOCATION T.C.H. near Nipigon, Ont.

HOLE LOCATION See dwg No. 1

HOLE ELEVATION AND DATUM 605.6

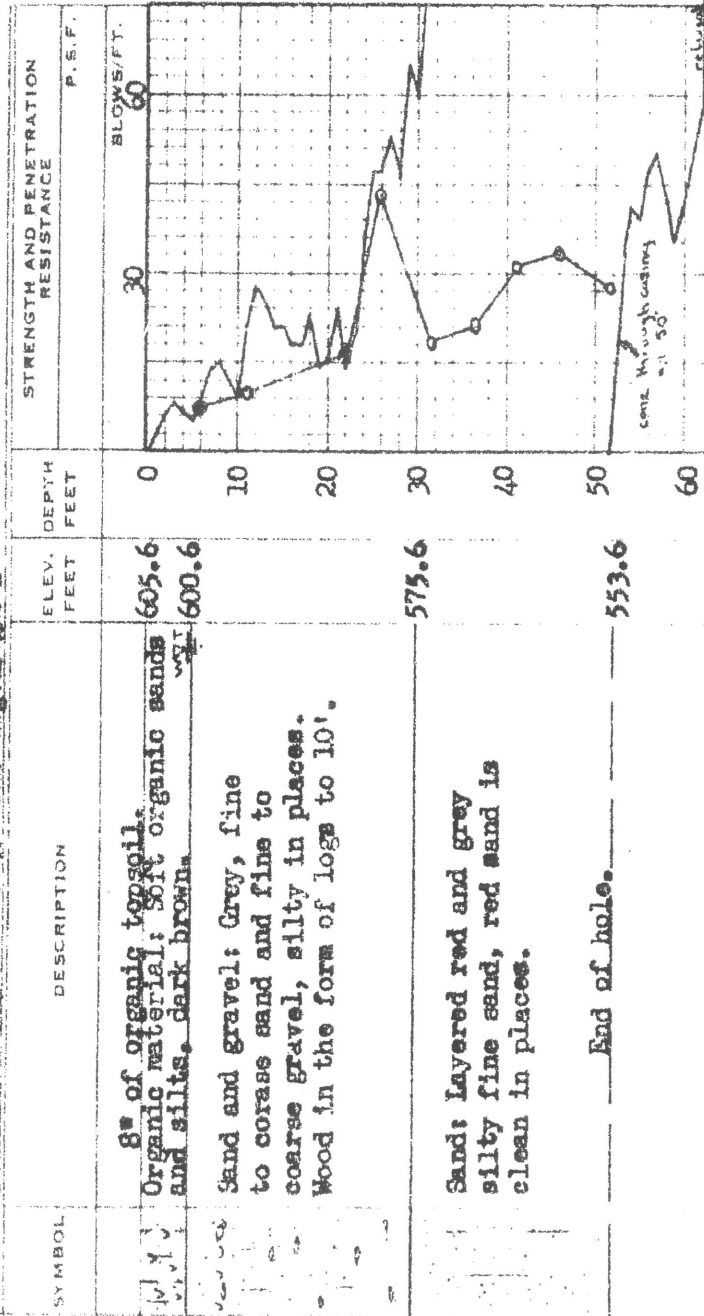
Sta. 424+00 = 604.8 DHO dwg. E3432-1

BOREHOLE NO. 3

FIELD SUPERVISOR D.S.

DRILLER E.S.

PREP. D.S.



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	

## TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Jackfish River Crossing Hwy #17,

LOCATION T.C.H. near Nipigon, Ont.

HOLE LOCATION See dwg. No. 1

HOLE ELEVATION AND DATUM 607.0

Sta. 424+00 = 604.8 HDD dwg. E3432-1

BOREHOLE NO. 4

FIELD SUPERVISOR D.S.

DRILLER E.S.

PREP. D.S.

## LEGEND

2" DIA. SPLIT TUBE

2" DIA. SHELBY TUBE

2" DIA. SPLIT TUBE

2" DIA. CONE

CASING

2" SHELBY

1/2 UNCONFINED COMPRESSION (QU)

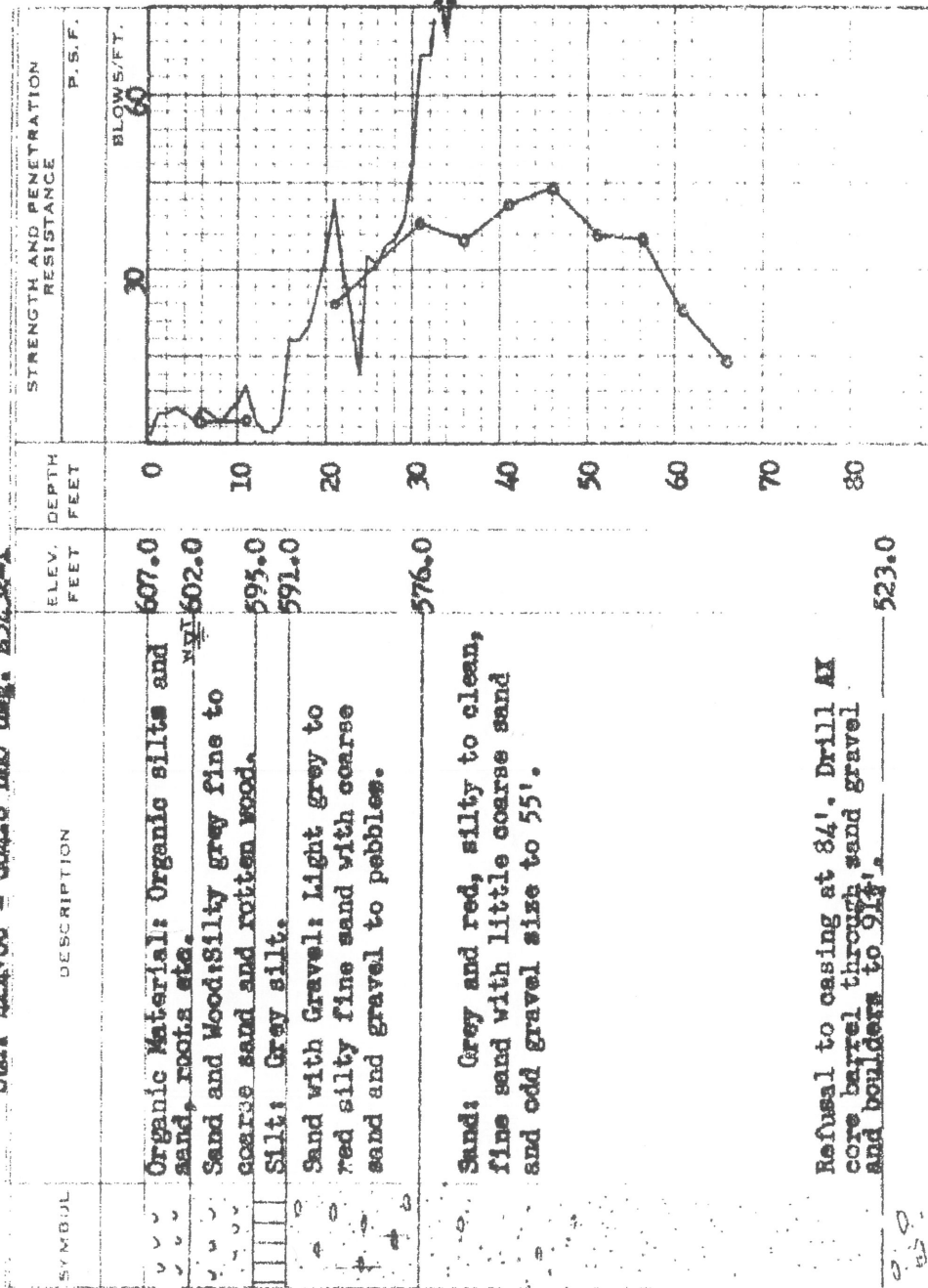
VANE TEST (C) AND SENSITIVITY (S)

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

PLASTIC LIMIT



DRAWING NO.

CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	TW3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	
	SS11	
	SS12	

PROJECT NO. CL08/J259

# TROW SODERMAN AND ASSOCIATES

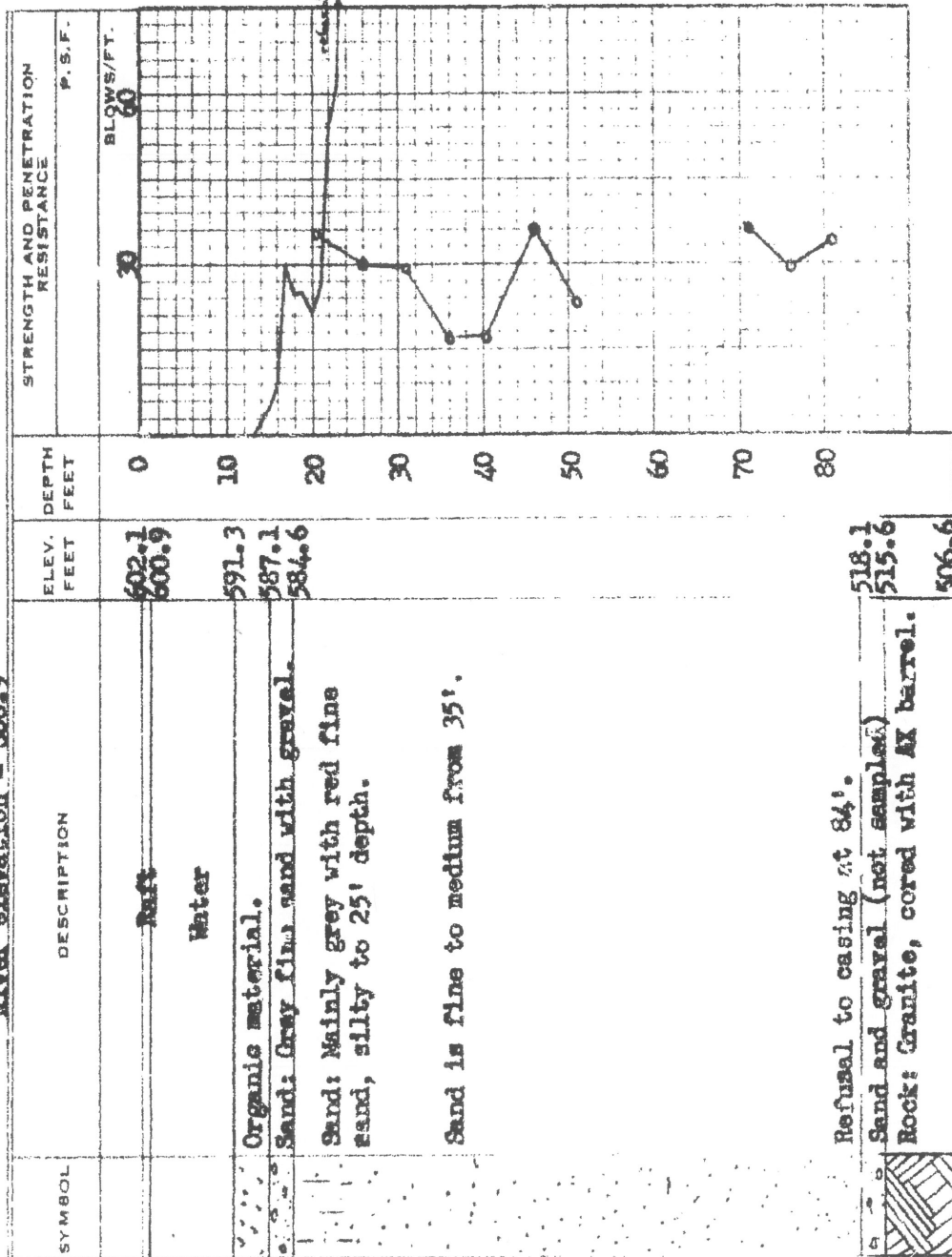
SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

DRAWING NO. 7

## LEGEND

- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (QU)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

PROJECT: Jackfish River Crossing Hwy #17,  
LOCATION: T.C.H. near Nipigon, Ont.  
HOLE LOCATION: See diag. No. 1  
HOLE ELEVATION AND DATUM: 602.1  
River elevation = 600.9



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	



PROJECT NO C108/J259

## TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

DRAWING NO.

9

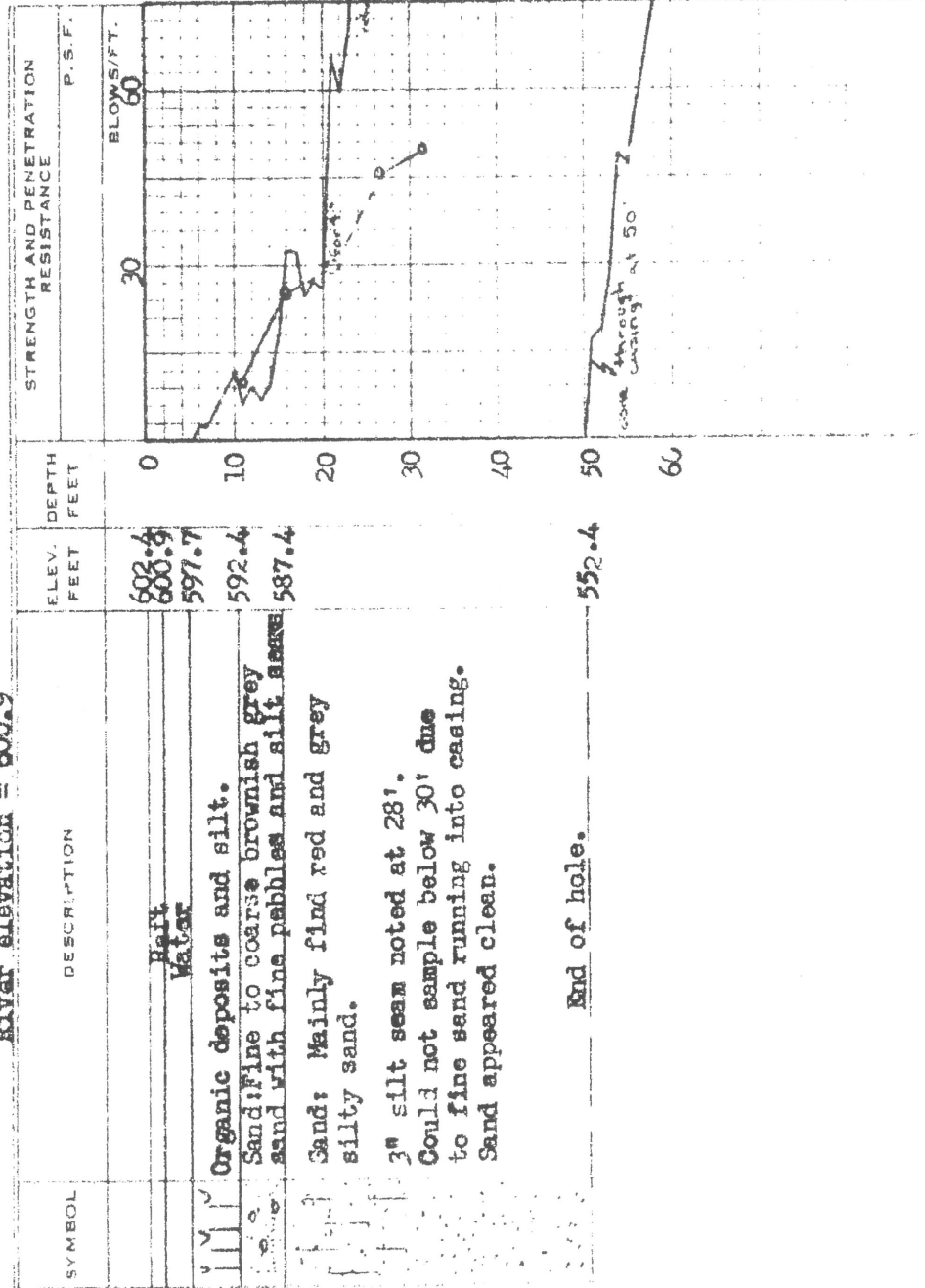
## LEGEND

2" DIA. SPLIT TUBE  
2" SHELBY TUBE  
2" SPLIT TUBE  
2" DIA. CONE  
CASING

2" SHELBY  
1/2 UNCONFINED COMPRESSION (OU)  
VANE TEST (C) AND SENSITIVITY (S)  
NATURAL MOISTURE AND  
LIQUIDITY INDEX  
LIQUID LIMIT  
PLASTIC LIMIT

PROJECT Jackfish River Crossing, Hwy #17  
LOCATION T.C.H. near Wipigon, Ont.  
HOLE LOCATION See dwg. No. 1  
HOLE ELEVATION AND DATUM 602.4  
River elevation = 600.9

BOREHOLE NO. 7  
FIELD SUPERVISOR D.S.  
DRILLER H.J.  
PREP. D.S.



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	



## TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

## LEGEND

- 2" DIA. SPLIT TUBE  
 2" SHELBY TUBE  
 2" SPLIT TUBE  
 2" DIA. CONE  
 CASING  
 2" SHELBY  
 1/2 UNCONFINED COMPRESSION (QU)  
 VANE TEST (C) AND SENSITIVITY (S)  
 NATURAL MOISTURE AND  
 LIQUIDITY INDEX  
 LIQUID LIMIT  
 PLASTIC LIMIT

PROJECT Jackfish River Crossing, Hwy #17

LOCATION T.C.H. near Wipigon, Ont.

HOLE LOCATION See Dwg. No. 1

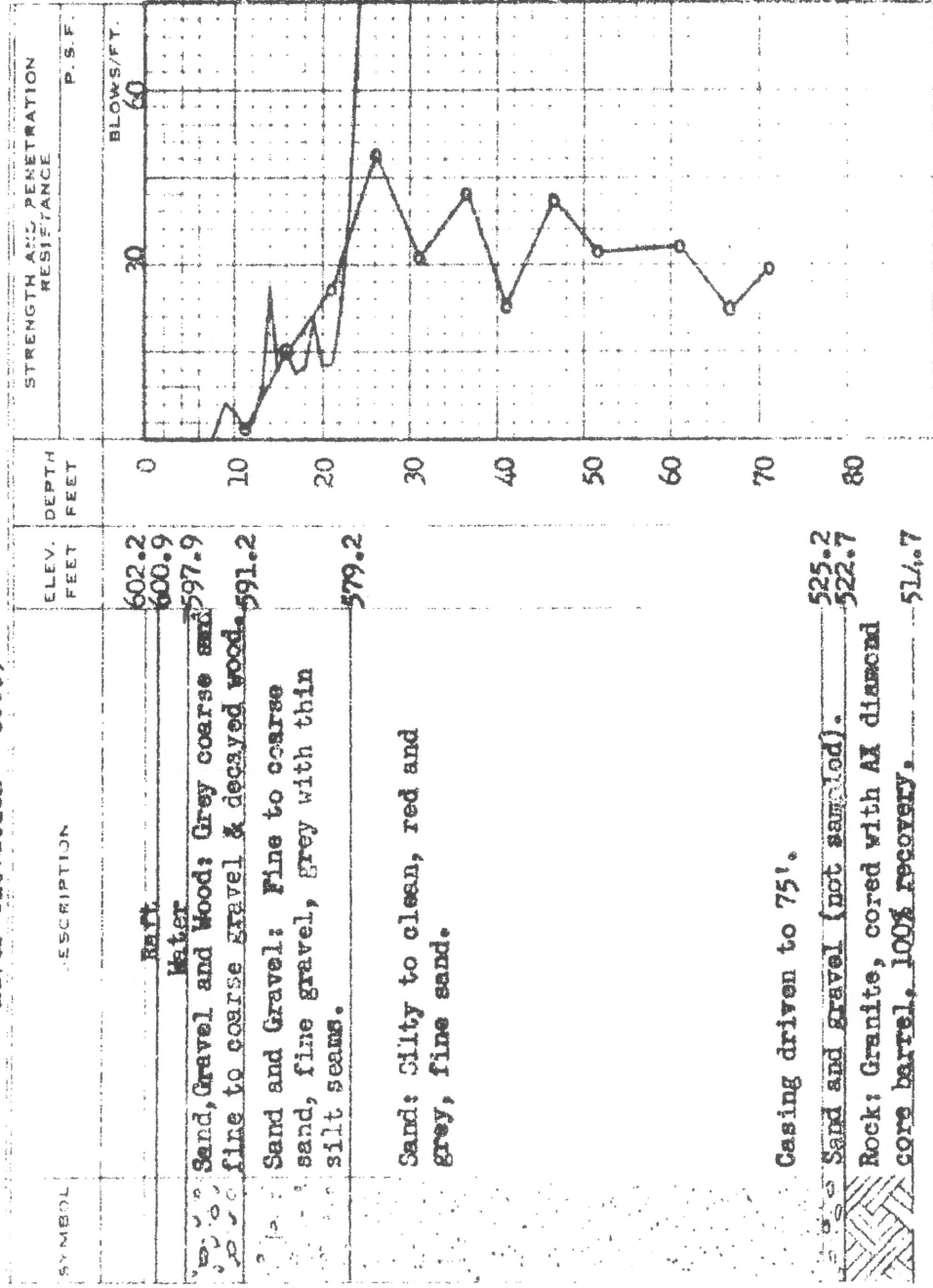
HOLE ELEVATION AND DATUM 602.2  
River elevation = 600.9

BOREHOLE NO. 8

FIELD SUPERVISOR D.S.

DRILLER H.J.

PREP. D.S.



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	
	SS8	
	SS9	
	SS10	
	SS11	
	SS12	



PROJECT NO C108/J259

TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

DRAWING NO. 11

LEGEND

- 2" DIA. SPLIT TUBE
- 2" SHELBY TUBE
- 2" SPLIT TUBE
- 2" DIA. CONE
- CASING
- 2" SHELBY
- 1/2 UNCONFINED COMPRESSION (QU)
- VANE TEST (C) AND SENSITIVITY (S)
- NATURAL MOISTURE AND LIQUIDITY INDEX
- LIQUID LIMIT
- PLASTIC LIMIT

BOREHOLE NO. 9  
FIELD SUPERVISOR D.C.  
DRILLER H.J.  
PREP. D.C.

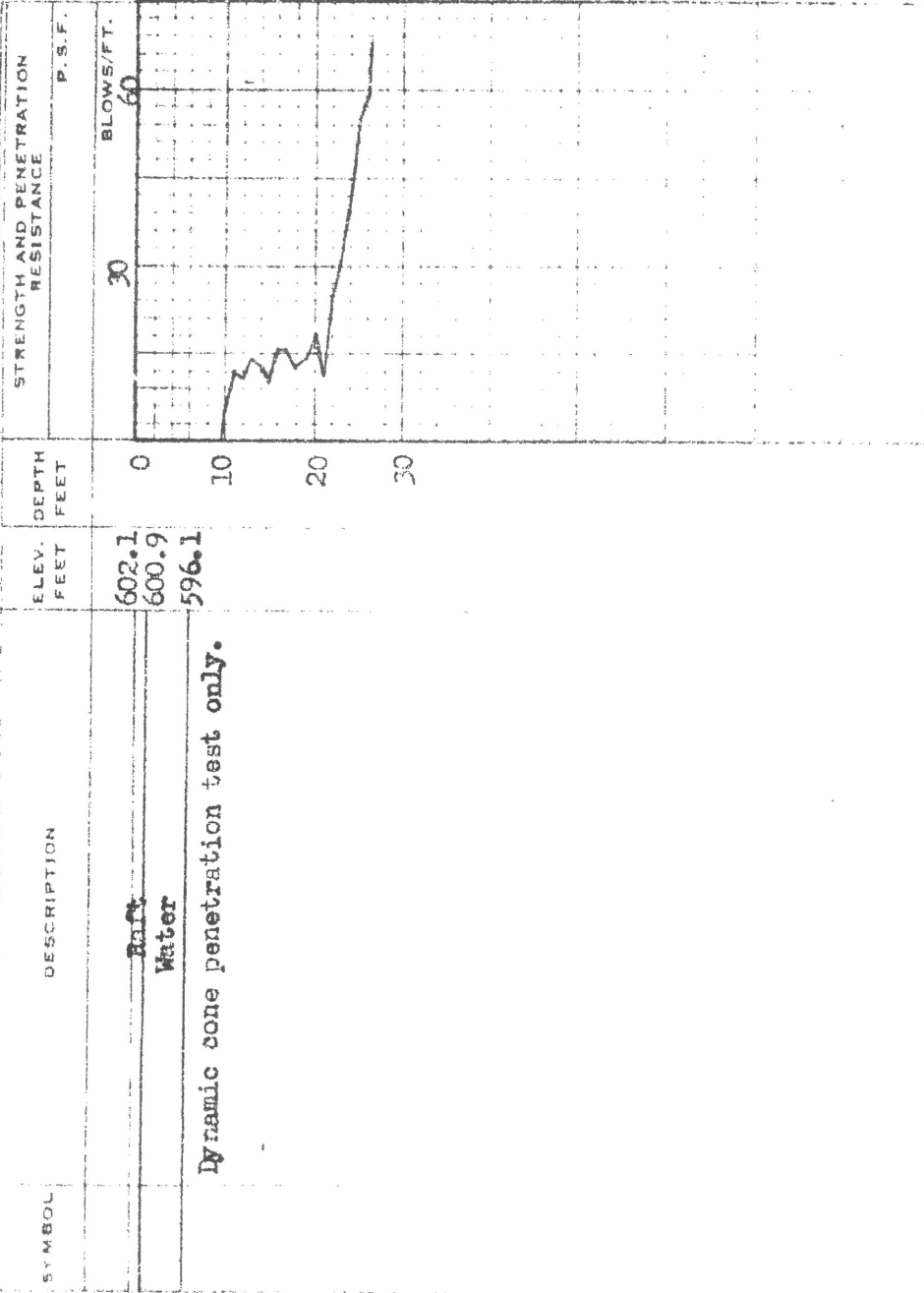
PROJECT Jackfish River Crossing, Hwy #17

LOCATION T.C.H. near Nipigon, Ont.

HOLE LOCATION See drg. No. 1

HOLE ELEVATION AND DATUM 602.1

River elevation = 600.9



## TROW SODERMAN AND ASSOCIATES

SITE INVESTIGATIONS AND SOIL MECHANICS CONSULTATION

PROJECT Jackfish River Crossing, Hwy #17  
LOCATION T.C.H. near Ripison, Ont.

HOLE LOCATION See Dwg. No. 1

HOLE ELEVATION AND DATUM 602.2  
Water elevation = 600.9BOREHOLE NO. 10  
FIELD SUPERVISOR D.S.  
DRILLER H.J.  
PREP. D.S.

## LEGEND

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

CASING

2" SHELBY

1/2 UNCONFINED COMPRESSION (Qu)

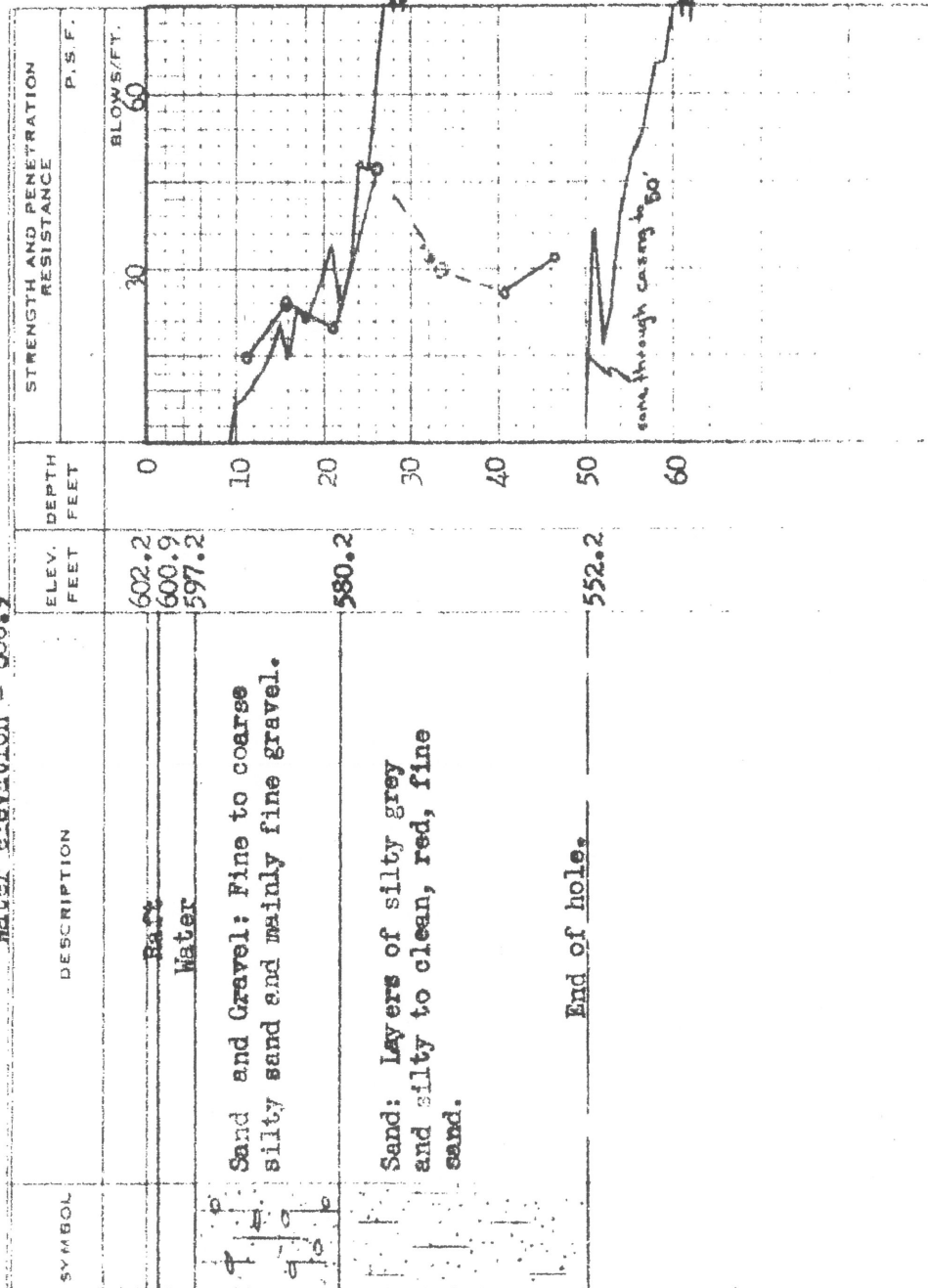
VANE TEST (C) AND SENSITIVITY (S)

NATURAL MOISTURE AND

LIQUIDITY INDEX

LIQUID LIMIT

PLASTIC LIMIT



CONSISTENCY	SAMPLE	NATURAL UNIT WT. P.C.F.
MOIST. CONTENT - % DRY WT.		
	SS1	
	SS2	
	SS3	
	SS4	
	SS5	
	SS6	
	SS7	

**Appendix E**  
**Foundation Comparison**

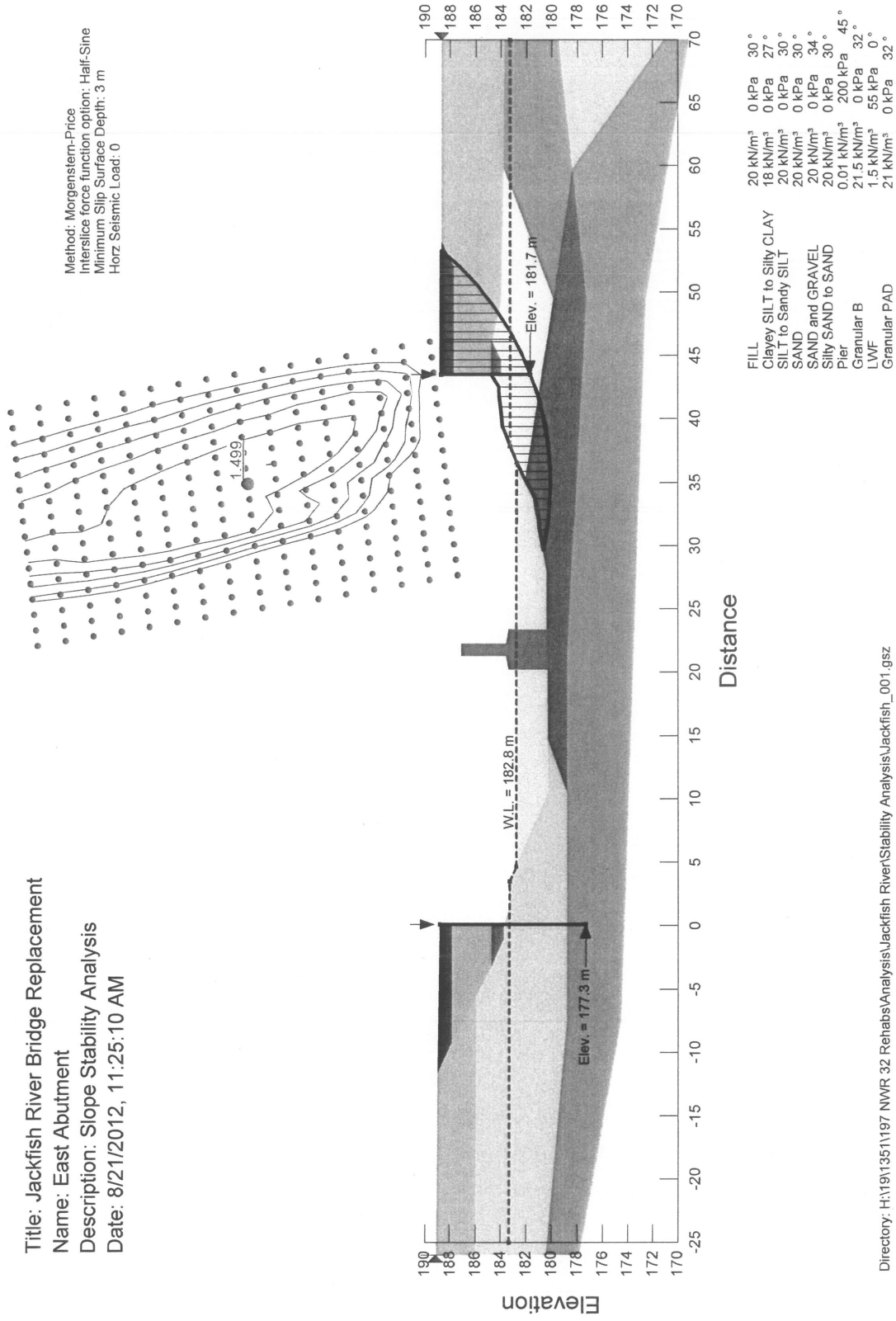
### COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footings on Native Soil	Augered Caissons (drilled shafts) socketed into bedrock	Pipe Piles socketed drilled to bedrock	Driven Piles to Bedrock or to layer of sand and gravel with cobbles/boulders
<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Generally less costly construction than deep foundation elements.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Low available geotechnical resistance in native soils.</li> <li>ii. Potential for settlements.</li> <li>iii. Relatively deep excavation in cohesionless soils extending below the groundwater level is required. This will necessitate prior dewatering.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance available for caissons socketed into bedrock</li> <li>ii. Construction of caissons could continue in freezing weather.</li> <li>iii. Subexcavation of fill not required.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit cost compared to spread footings.</li> <li>ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table.</li> <li>iii. Potential difficulty in cleaning and inspecting rock sockets.</li> <li>iv. Installation through cobbles and boulders will be difficult.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistance available for units into bedrock.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Possibly higher unit cost compared to other foundation options such as footings.</li> <li>ii. Difficulties in obtaining seal below the liner to pour concrete in dry conditions.</li> <li>iii. Specialized installation.</li> <li>iv. Potential difficulty in cleaning and inspecting rock socket.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>ii. High geotechnical resistance available by driving piles to achieve resistance on the bedrock or upon refusal the sand and gravel layer containing cobbles and boulders.</li> <li>iii. Installation of piles could continue in freezing weather.</li> <li>iv. Independent of groundwater conditions.</li> <li>v. Foundation construction requires less volume of excavation than footings</li> <li>vi. Readily installed.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Higher unit costs than footings.</li> <li>ii. Pile lengths required to achieve design resistance may vary.</li> <li>iii. Pre-augering might be required if cobbles and boulders are encountered within the fill.</li> </ul>
<b>NOT RECOMMENDED</b>	<b>NOT RECOMMENDED</b>	<b>FEASIBLE</b>	<b>RECOMMENDED</b>

**Appendix F**  
**Slope Stability Output**

Title: Jackfish River Bridge Replacement  
 Name: East Abutment  
 Description: Slope Stability Analysis  
 Date: 8/21/2012, 11:25:10 AM

Method: Morgenstern-Price  
 Interslice force function option: Half-Sine  
 Minimum Slip Surface Depth: 3 m  
 Horiz Seismic Load: 0



Directory: H:\191\3511197 NWR 32 Rehab\Analysis\Jackfish River\Stability Analysis\Jackfish\_001.gsz

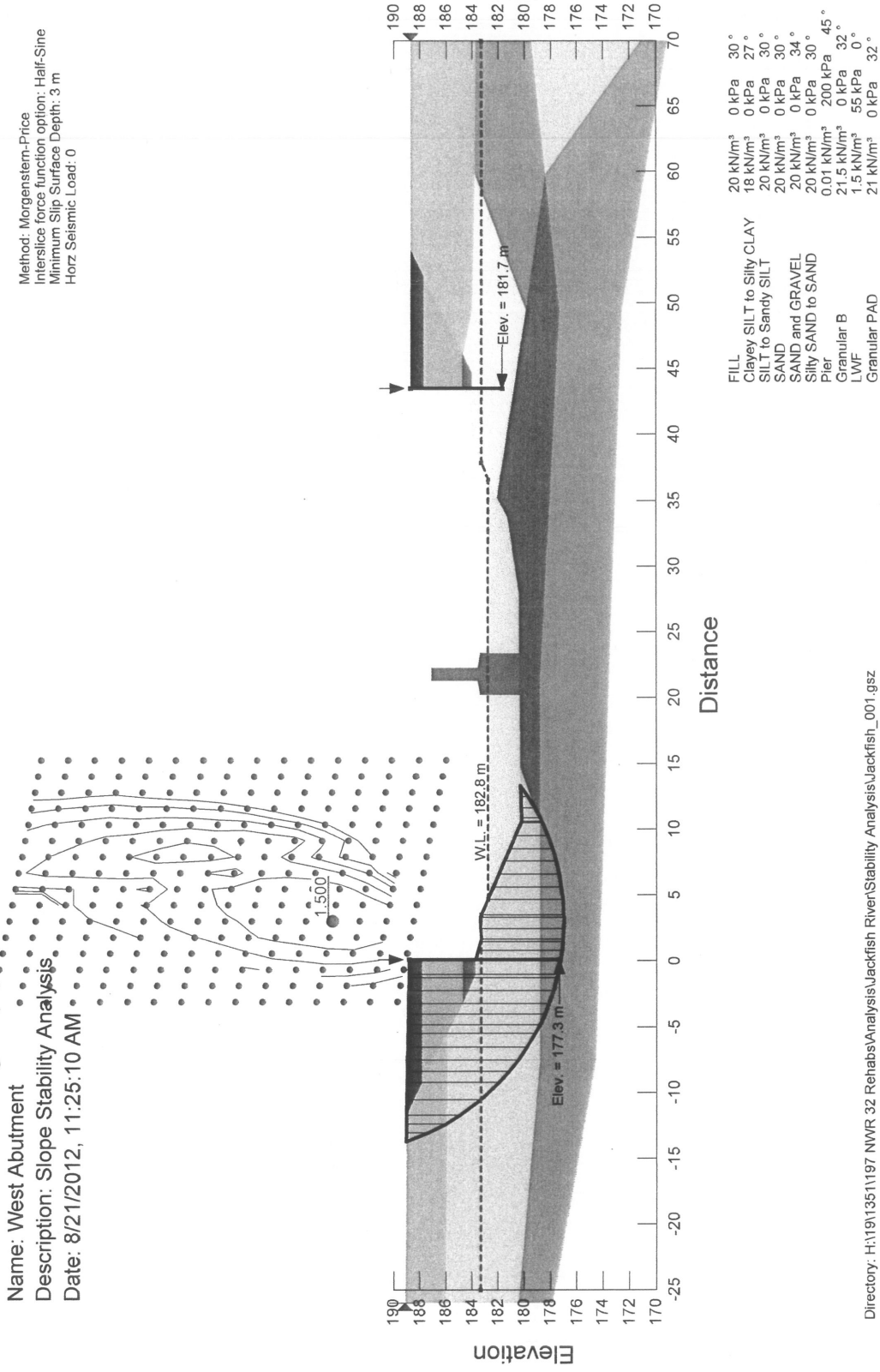
Title: Jackfish River Bridge Replacement

Name: West Abutment

**Description:** Slope Stability Analysis.

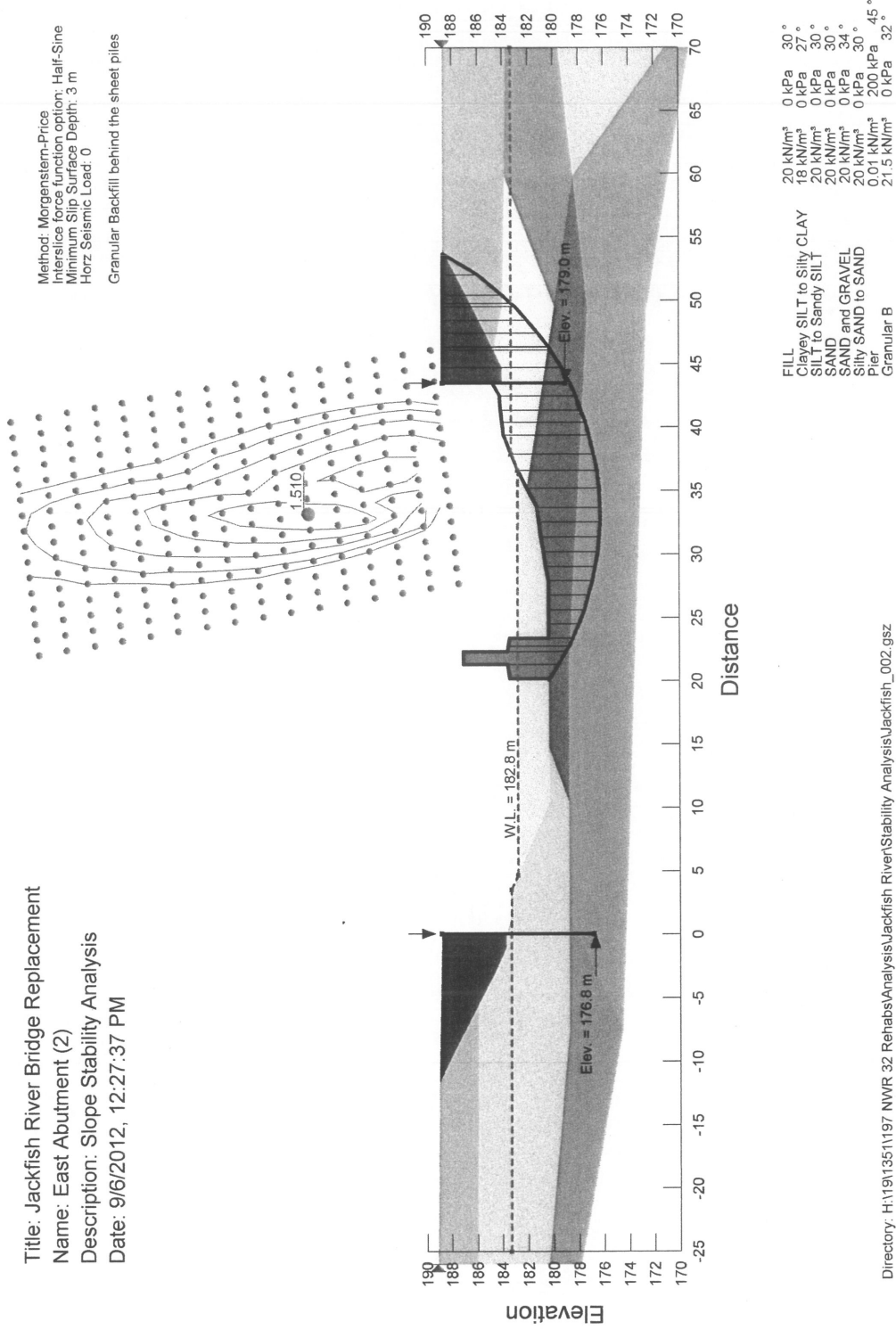
Date: 8/21/2012, 11:25:10 AM

Method: Morgenstern-Price  
Interslice force function option: Half-Sine  
Minimum Slip Surface Depth: 3 m  
Horz Seismic Load: 0



Title: Jackfish River Bridge Replacement  
 Name: East Abutment (2)  
 Description: Slope Stability Analysis  
 Date: 9/6/2012, 12:27:37 PM

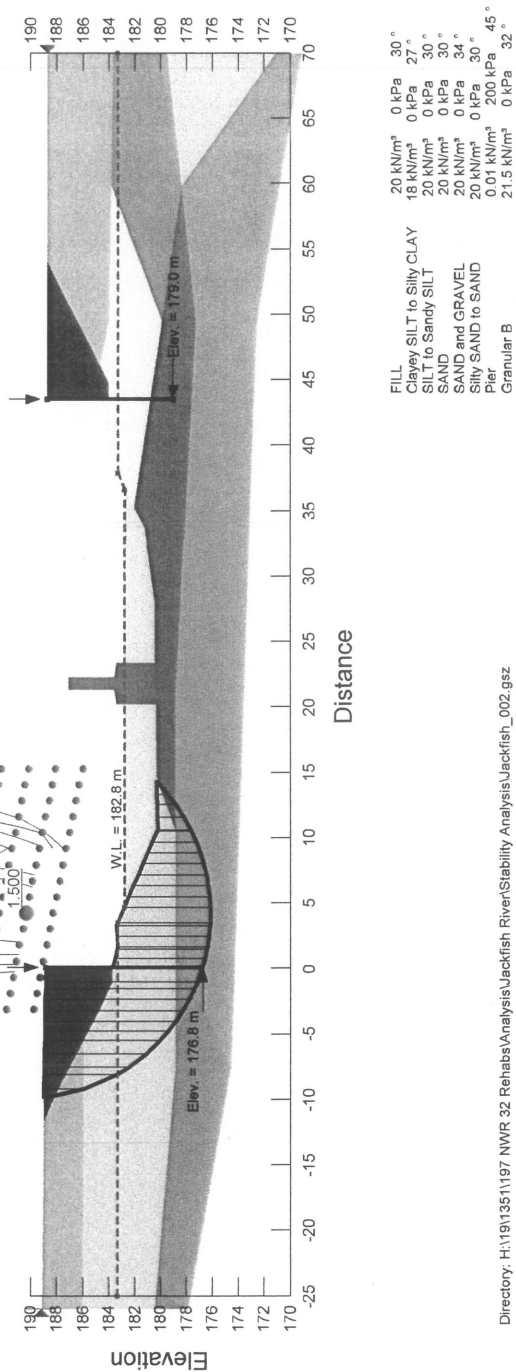
Method: Morgenstern-Price  
 Interslice force function option: Half-Sine  
 Minimum Slip Surface Depth: 3 m  
 Horiz Seismic Load: 0  
 Granular Backfill behind the sheet piles



Directory: H:\19135\1197 NWR 32 Rehabs\Analysis\Jackfish River\Stability Analysis\Jackfish\_002.gsz



Method: Morgenstern-Price  
Interslice force function option: Half-Sine  
Minimum Slip Surface Depth: 3 m  
Horz Seismic Load: 0  
Granular Backfill behind the sheet piles



**Appendix G**  
**List of SPs and OPSS**

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

- OPSS 903
- OPSS 902
- OPSS 1010
- OPSS 501
- OPSS 804
- OPSD 208.010
- OPSD 3101.150
- OPSS 539
- Special Provision 110S13 "Amendment to OPSS 1010, April 2004".

**Appendix H**  
**Site Photographs**



**Photograph 1H** – Highway 17 at Jackfish River bridge, looking west



**Photograph 2H** – Jackfish River bridge east abutment





**Photograph 3H** – Jackfish River bridge west abutment

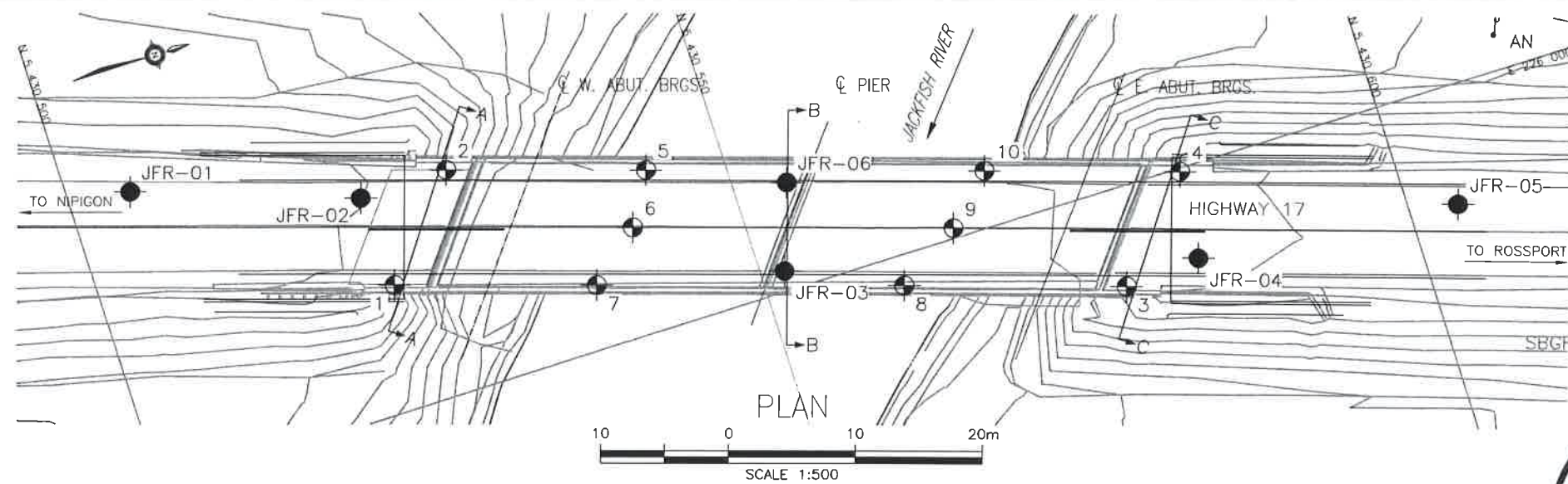


**Photograph 4H** – Jackfish River bridge pier

**Appendix I**

**Drawing titled "Borehole Locations and Soil Strata"**





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



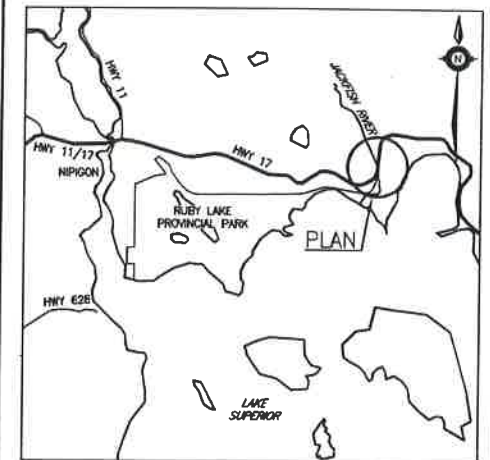
CONT No  
WP No 465-00-01

HIGHWAY 17  
JACKFISH RIVER  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET  
33

**MRC** MCCORMICK RANKIN  
A member of MRM GROUP

**THURBER ENGINEERING LTD.**



# LEGEND

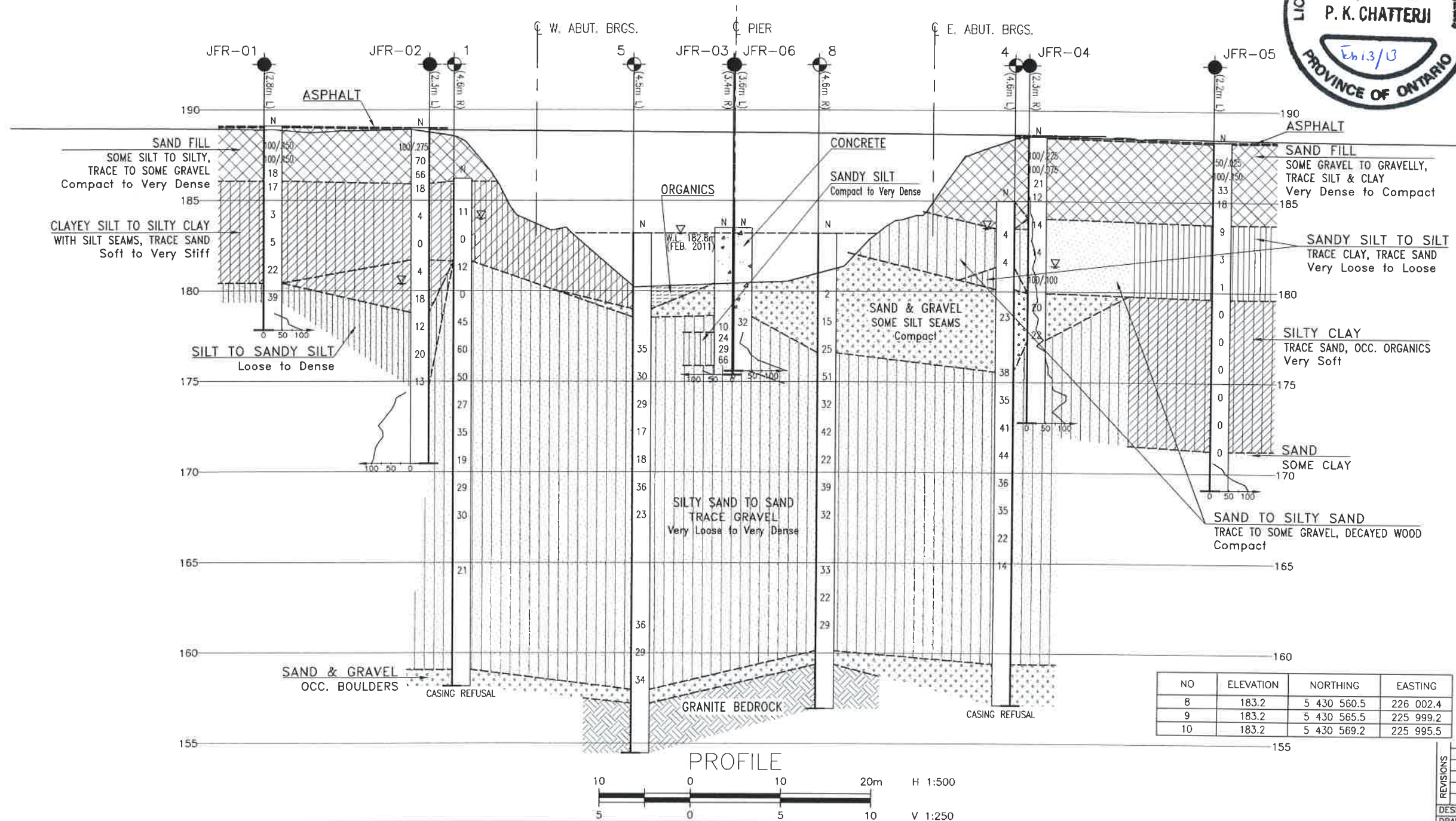
- ◆ Borehole (Current investigation by Thurber)
- ◆ Borehole (Previous investigation by others)
- N Blows /0.3m (Std Pen Test, 475J/blow)
- CONE Blows /0.3m (60° Cone, 475J/blow)
- PH Pressure, Hydraulic
- W Water Level
- W Head Artesian Water
- W Piezometer
- 90% Rock Quality Designation (RQD)
- A/R Auger Refusal

NO	ELEVATION	NORTHING	EASTING
JFR-01	189.1	5 430 504.9	225 976.5
JFR-02	189.0	5 430 522.0	225 982.6
JFR-03	183.5	5 430 551.9	225 998.4
JFR-04	188.6	5 430 583.1	226 007.3
JFR-05	188.3	5 430 603.8	226 009.3
JFR-06	183.5	5 430 554.2	225 991.7
1	186.2	5 430 522.4	225 990.0
2	185.1	5 430 529.1	225 982.5
3	184.6	5 430 577.0	226 007.7
4	185.0	5 430 583.8	226 000.3
5	183.2	5 430 544.0	225 987.4
6	183.2	5 430 541.6	225 991.4
7	183.2	5 430 537.5	225 994.9

## 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.
- Locations of Boreholes 1 to 10 are approximate

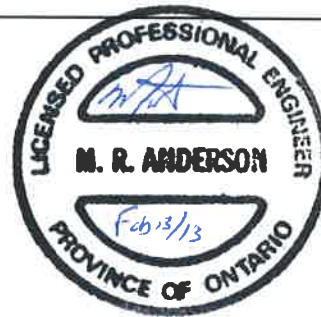
**GEOCRES No. 52H-18**



NO	ELEVATION	NORTHING	EASTING
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9	183.2	5 430 565.5	225 999.2
10	183.2	5 430 569.2	225 995.5

DATE	BY	DESCRIPTION
DESIGN	LRB	CHK PKC
DRAWN	MFA	CHK LRB
DATE	JAN. 2013	
STRUCT	DWG 2	



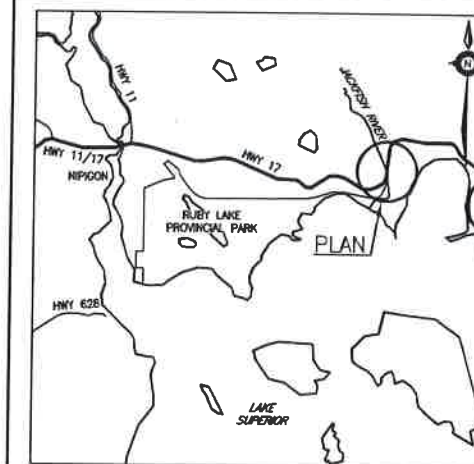


**METRIC**  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

CONT No  
WP No 465-00-01

HIGHWAY 17  
JACKFISH RIVER  
BRIDGE REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET  
34



### KEYPLAN

### LEGEND

	Borehole (Current investigation by Thurber)
	Borehole (Previous investigation by others)
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)
A/R	Auger Refusal

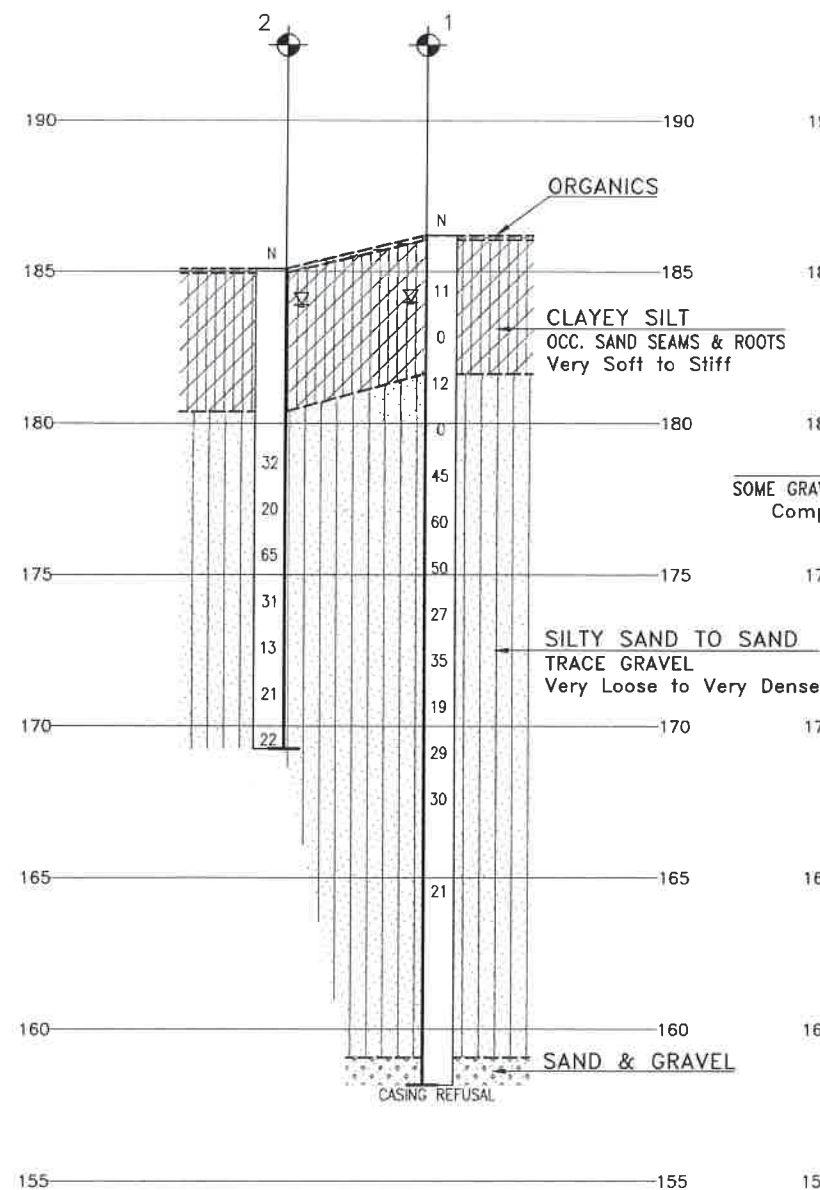
NO	ELEVATION	NORTHING	EASTING
JFR-01	189.1	5 430 504.9	225 976.5
JFR-02	189.0	5 430 522.0	225 982.6
JFR-03	183.5	5 430 551.9	225 998.4
JFR-04	188.6	5 430 583.1	226 007.3
JFR-05	188.3	5 430 603.8	226 009.3
JFR-06	183.5	5 430 554.2	225 991.7
1	186.2	5 430 522.4	225 990.0
2	185.1	5 430 529.1	225 982.5
3	184.6	5 430 577.0	226 007.7
4	185.0	5 430 583.8	226 000.3
5	183.2	5 430 544.0	225 987.4
6	183.2	5 430 541.6	225 991.4
7	183.2	5 430 537.5	225 994.9

### NOTES

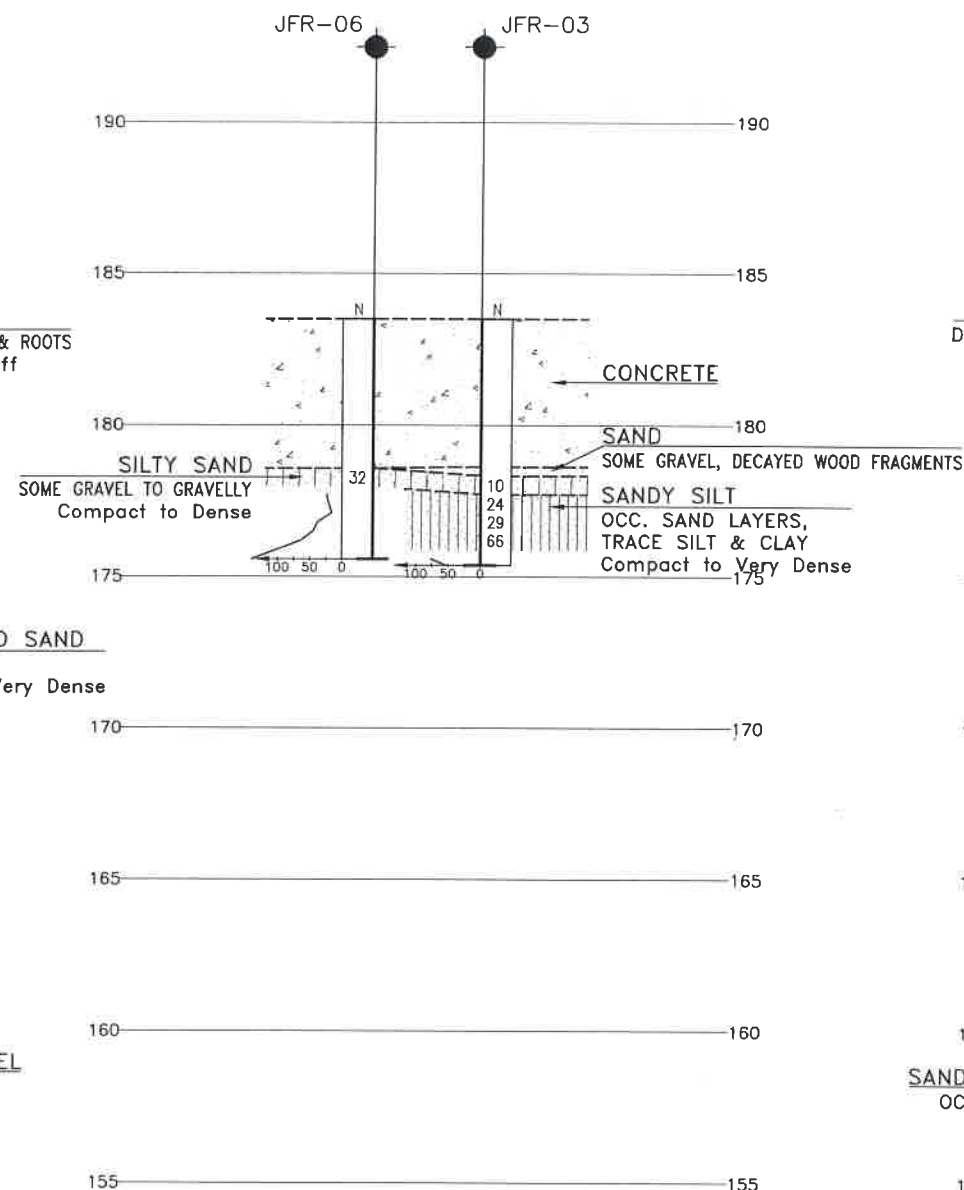
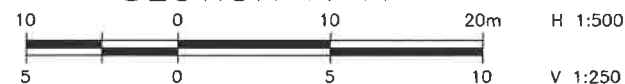
- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Locations of Boreholes 1 to 10 are approximate.

**GEOCREs No. 52H-18**

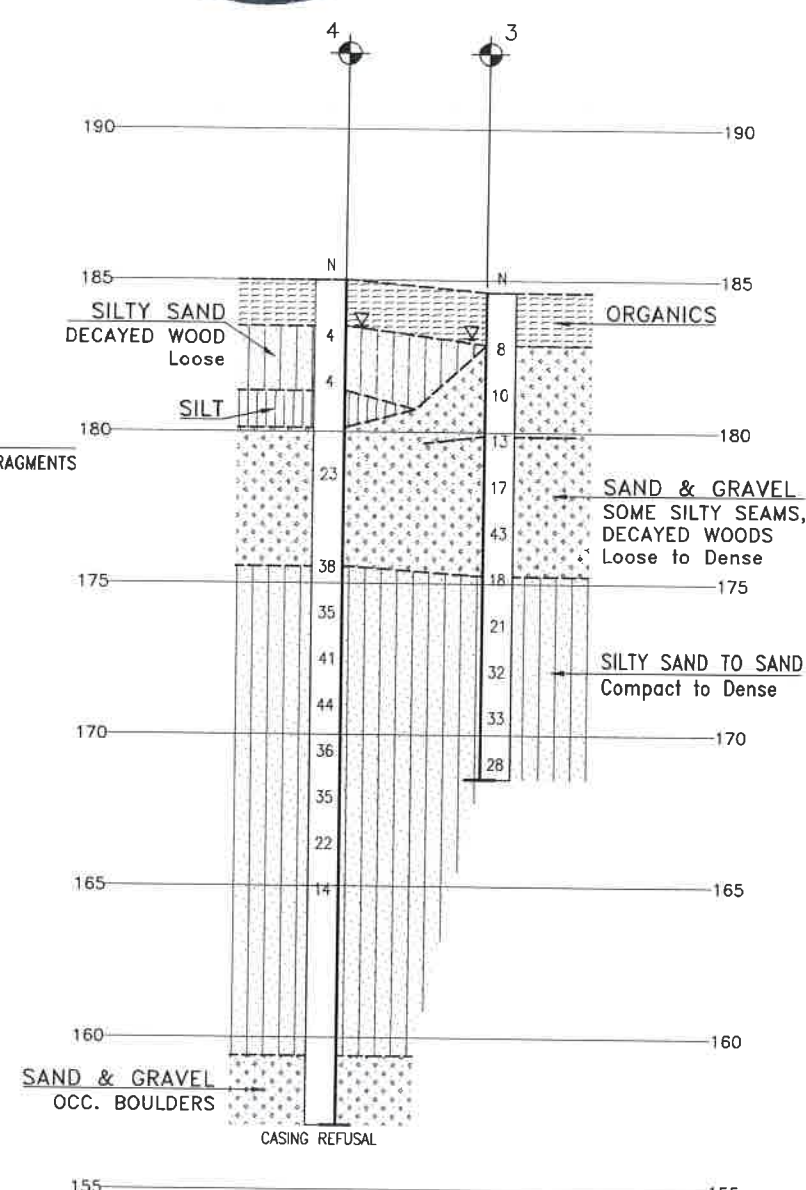
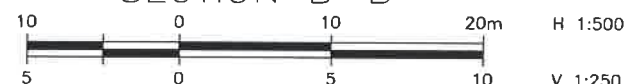
DATE	BY	DESCRIPTION
DESIGN	LRB	CHK PKC CODE
DRAWN	MFA	CHK LRB SITE
		LOAD
		STRUCT
		DWG 3
		DATE JAN. 2013



### SECTION A-A



### SECTION B-B



### SECTION C-C



NO	ELEVATION	NORTHING	EASTING
8	183.2	5 430 560.5	226 002.4
9	183.2	5 430 565.5	225 999.2
10	183.2	5 430 569.2	225 995.5