

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PINWOOD RIVER BRIDGE REPLACEMENT
HIGHWAY 617, NORTH OF STRATTON, ONTARIO
RAINY RIVER DISTRICT
G.W.P. 6094-10-00, SITE 45-37**

Geocres Number: 52D-13

Report to

GENIVAR

Thurber Engineering Ltd.
2010 Winston Park Drive, Suite 103
Oakville, Ontario
L6H 5R7
Phone: (905) 829 8666
Fax: (905) 829 1166

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of a proposed bridge replacement crossing the Pinewood River. The existing bridge carries Highway 617 over the Pinewood River, approximately 14.7 km north of Stratton, Ontario, in the Rainy River District.

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

Thurber carried out the investigation as a sub-consultant to GENIVAR, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0012.

2 SITE DESCRIPTION

The Pinewood River bridge is located on Highway 617 between Brown Road and Neilson Road, approximately 14.7 km north of Stratton, Ontario.

Highway 617 is a paved two-lane road with narrow gravel shoulders. The existing structure consists of a five span bridge with a concrete and timber deck. The centre span is 8.0 m in length, while the other 4 spans are 5.0 m each. The total length and width of the existing bridge are 28.6 m and 9.1 m, respectively.

At this location, the Pinewood River flows from east to west. Based on the General Arrangement drawing (GA), the width of the river channel varies from 8.5 m to 11.5 m at the bridge location. The depth to the bottom of the river channel is 0.9 m below the water level (elevation 96.08).

The lands immediately surrounding the bridge site primarily consist of open fields with some forested areas. A photograph in Appendix C shows the general nature of the surrounding lands.

Based on the bedrock geology published by the Ontario Geological Society, the Pinewood River bridge site is underlain by felsic to intermediate metavolcanic rocks and minor sedimentary rocks. Locally, the bedrock is overlain by deposits of clays, silts and sands.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between August 14 and 16, 2011 and consisted of drilling and sampling six boreholes (identified as PINE-01 to PINE-06), at the existing bridge location. Boreholes PINE-01 and PINE-06 were drilled at the south and north approaches, respectively and were advanced to a depth of 11.3 m (elevations 88.5 and 88.6, respectively). Boreholes PINE-02 and PINE-03 were drilled near the south abutment and Boreholes PINE-04 and PINE-05 were drilled near the north abutment. These four boreholes were advanced to depths of 14.6 m to 22.7 m (elevation 85.3 to 77.1) where auger refusal was encountered. Bedrock was proved in Boreholes PINE-03 and PINE-04 by NQ size diamond coring. Boreholes PINE-03 and PINE-04 were advanced 3.0 m and 2.9 m into bedrock and terminated at 25.7 m and 19.5 m depth (elevations 74.1 and 80.4), respectively.

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

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

The drilling was carried out from the highway grade using a truck-mounted CME 75 drill rig. A combination of hollow-stem augers, casing and NQ coring methods were used to advance the boreholes. Overburden samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

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

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

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

Table 3.1 – Borehole Abandonment Details

Location	Borehole	Piezometer Tip Depth/ Elevation (m)	Abandonment Details
South Approach	PINE-01	None installed	Backfilled with bentonite from 11.3 m to 0.2 m, sand from 0.2 m to 0.04 m, then asphalt to surface.
South Abutment	PINE-02	None installed	Backfilled with bentonite from 15.8 m to 0.2 m, sand from 0.2 m to 0.04 m, then asphalt to surface.
	PINE-03	None installed	Backfilled with bentonite from 25.7 m to 0.1 m, sand from 0.1 m to 0.04 m, then asphalt to surface.
North Abutment	PINE-04	None installed	Backfilled with bentonite from 19.5 m to 0.1 m, sand and gravel from 0.1 m to 0.07 m, then asphalt to surface.
	PINE-05	14.0 / 85.9	Piezometer with 1.5 m slotted screen installed with sand filter from 14.0 m to 9.8 m, holeplug from 9.8 m to 0.1 m, 50 mm of sand with asphalt patch at surface.
North Approach	PINE-06	None installed	Backfilled with bentonite from 11.3 m to 0.7 m, sand from 0.7 m to 0.07 m, then asphalt to surface.

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 gradation analysis (hydrometer and sieve) and Atterberg Limits testing, where appropriate. The results of these tests are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

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

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil and rock stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions.

In general terms, the soil stratigraphy encountered at this site consists of a surficial layer of asphalt overlying sand fill. At the south abutment, a layer of silty clay fill was encountered below the sand fill. Organic clayey silt was encountered locally below the sand fill near the north abutment. The fill was found to be underlain by layers of native silty clay, clayey silt and a sequence of silts and sands and a layer of sand and gravel containing cobbles and boulders. These soils are underlain by moderately to slightly weathered metavolcanic bedrock. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt was encountered at the surface in all of the boreholes since they were all drilled through the existing highway. The thickness of the asphalt ranged from 35 mm to 75 mm.

5.2 Sand Fill

Granular fill was encountered directly below the asphalt in all boreholes. The granular fill consists of brown sand containing some gravel and occasional cobbles and boulders.

The thickness of the sand fill ranged from 0.9 m to 1.8 m, with the base of the fill encountered at depths of 1.0 m to 1.9 m (elevations 98.9 to 97.9).

SPT N-values recorded in the sand fill ranged from 9 to 21 blows for 0.3 m penetration, indicating a loose to compact relative density.

Moisture contents of samples of the sand fill ranged from 8% to 17%.

One sample of the sand fill underwent laboratory gradation analysis, the results of which are summarized below. These results are also presented on the Record of Boreholes sheets included in Appendix A. The grain size distribution curve for this sample is plotted on Figure B1 of Appendix B.

Soil Particles	Sand fill (%)
Gravel	13
Sand	82
Silt and Clay	5

5.3 Silty Clay Fill

Cohesive fill was encountered below the sand fill in Boreholes PINE-01 to PINE-03 drilled near the south abutment. The cohesive fill consists of dark grey to dark brown silty clay containing trace to some sand, trace gravel and occasional organics and wood fibres. Occasional cobbles and boulders were encountered within the silty clay fill at a depth of 2.7 m (elevation 97.1) in Borehole PINE-02. Coring through the boulder(s) was required to advance the borehole. It must be recognized that embankment fills are heterogeneous in

nature and may contain cobbles, boulders and rock fill in areas away from the borehole location.

The thickness of the silty clay fill varied from 1.1 m to 3.4 m.

The depth to the base of the silty clay fill ranged from 3.0 m to 4.6 m (elevations 95.2 to 96.7).

SPT 'N' values measured in the silty clay fill ranged from 2 to 7 blows per 0.3 m of penetration, indicating a soft to firm consistency.

The moisture contents of samples of the silty clay fill ranged from 22% to 38%.

One sample of the silty clay fill underwent laboratory grain size analysis testing and Atterberg Limits testing. The results of these tests are presented on the Record of Boreholes sheets included in Appendix A. The grain size distribution curve for this sample of silty clay fill is plotted on Figure B2 of Appendix B. The results of the Atterberg Limits test are plotted on Figure B7, Appendix B. The results of these laboratory tests are summarized as follows:

Soil Particle	Silty clay fill (%)
Gravel	0
Sand	21
Silt	44
Clay	35

Index Property	(%)
Liquid Limit	48
Plastic Limit	18
Plasticity Index	30

Results of the Atterberg Limits tests indicate that the silty clay fill is of medium plasticity with a group symbol of CI.

5.4 Organic clayey silt

A layer of dark brown organic clayey silt containing trace sand and clay was encountered locally in Borehole PINE-04 below the sand fill.

The layer of organic clayey silt was 1.2 m thick, with the lower boundary at 2.4 m depth (elevation 97.5).

A single SPT N-value of 6 blows for 0.3 m penetration was recorded in the organic clayey silt, indicating a loose relative density.

The moisture content of one sample of organic clayey silt was 26%.

5.5 Silty Clay

Native silty clay was encountered below the silty clay fill in Boreholes PINE-01 to PINE-03, below the layer of organic clayey silt in Borehole PINE-04 and below the sand fill in Boreholes PINE-05 and PINE-06. The silty clay is brown to grey and contains trace sand to sandy, trace gravel and occasional sand seams and wood fibres. The silty clay was also described as varved and contains silt layers. The thickness of the native silty clay layer ranged from 6.1 m to 9.5 m.

The depth to the base of the silty clay ranged from 10.1 m to 12.2 m depth (elevations 89.8 to 87.6).

SPT N-values recorded in the native silty clay ranged from 0 to 7 blows for 0.3 m penetration, indicating a very soft to firm consistency.

Moisture contents of samples of the native silty clay generally ranged from 21% to 49%. Three high moisture contents ranging from 66% to 73% were measured at approximate elevation 95.0 in Boreholes PINE-04 to PINE-06.

Several samples of the native silty clay underwent laboratory gradation analysis testing and 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. The grain size distribution curves for these samples are plotted on Figures B3 and B4 of Appendix B and the results of the Atterberg Limits tests are plotted on Figures B8 and B9.

Soil Particle	Silty clay (%)
Gravel	0 to 4
Sand	0 to 34
Silt	23 to 46
Clay	25 to 76

Index Property	(%)
Liquid Limit	31 to 87
Plastic Limit	13 to 29
Plasticity Index	18 to 58

Results of the Atterberg Limits tests indicate that the silty clay ranges from low to high plasticity with group symbols of CL, CI and CH.

5.6 Clayey Silt

Grey clayey silt containing trace sand was contacted below the silty clay at 10.7 m depth (elevation 89.1) in Borehole PINE-01.

The borehole was terminated within the clayey silt layer at 11.3 m depth (elevation 88.5).

An SPT N-value measured in the clayey silt was 11 blows per 0.3 m of penetration, indicating a stiff consistency.

The moisture content of the clayey silt was 38%.

A grain size distribution curve for a clayey silt sample is presented on the Record of Borehole sheets and on Figure B5 of Appendix B. The results of the Atterberg Limits tests are plotted on Figure B10, Appendix B. The results of the laboratory tests are summarized as follows:

Soil Particle	Clayey Silt (%)
Gravel	0
Sand	2
Silt	79
Clay	19

Index Property	(%)
Liquid Limit	39
Plastic Limit	18
Plasticity Index	21

Results of the Atterberg Limits tests indicate that the clayey silt is of medium plasticity with a group symbol of CI.

5.7 Gravelly Sand

Grey gravelly sand was contacted below the silty clay at 10.1 m depth (elevation 89.8) in Borehole PINE-04, drilled at the north abutment. The thickness of the layer was 1.0 m.

The depth to the base of the gravelly sand was 11.1 m (elevation 88.8).

An SPT N-value measured in the gravelly sand was 61 blows per 0.3 m of penetration, indicating a very dense relative density.

A moisture content of the gravelly sand was 9%.

5.8 Silt and Sand

A layer of greenish grey silt and sand was encountered below the silty clay in Boreholes PINE-02, PINE-03, PINE-05 and PINE-06 and below the thin layer of gravelly sand in Borehole PINE-04. The silt and sand layer contains trace clay, trace gravel and occasional layers of coarse sand and cobbles and boulders.

The silt and sand layer was fully penetrated in Boreholes PINE-03 and PINE-04 where it was 3.8 m thick. The depth to the base of the silt and sand was at 14.9 m in Borehole PINE-04 (elevation 85.0) and at 16.0 m (elevation 83.8) in Borehole PINE-03.

Boreholes PINE-02 and PINE-05 were terminated below the silt and sand layer upon refusal on probable bedrock or boulder at 15.8 m and 14.6 m depth (elevations 84.0 and 85.3). Borehole PINE-06 was terminated within the silt and sand at 11.3 m depth (elevation 88.6).

SPT N-values recorded in the silt and sand layer ranged from 41 blows for 0.3 m penetration to 50 blows for 0.075 m penetration, indicating a dense to very dense relative density. SPT N-values of 20 and 21 blows per 0.3 m of penetration were measured in Boreholes PINE-02 and PINE-03 near elevation 87.6.

The moisture content of samples of the silt and sand ranged from 11% to 21%.

Five samples of the silt and sand layer underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A. The grain size distribution curves for these samples are plotted on Figure B6, Appendix B.

Soil Particles	Silt and sand (%)
Gravel	1 to 3
Sand	38 to 54
Silt	39 to 57
Clay	4 to 6

5.9 Sand and Gravel with Cobbles and Boulders

A layer of grey sand and gravel containing cobbles and boulders was encountered below the silt and sand layer at 16.0 m and 14.9 m (elevations 83.8 and 85.0) in Boreholes PINE-03 and PINE-04, respectively.

The thickness of the sand and gravel layer was 6.7 m in Borehole PINE-03 and 1.7 m in Borehole PINE-04. The lower boundary of the sand and gravel layer was encountered at a depth of 16.6 m in Borehole PINE-04 (elevation 83.3) and a depth of 22.7 m in Borehole PINE-03 (elevation 77.1).

Due to the presence of cobbles and possibly boulders within the sand and gravel layer, coring methods were required to advance the boreholes and no SPT N-values were recorded in the sand and gravel layer. Samples of the layer of sand and gravel with cobbles were collected from the core barrel.

The moisture content of samples of the sand and gravel layer collected from the core barrel ranged from 6% to 15%.

Photographs of the samples recovered from coring through this layer are included in Appendix C. The cobbles ranged in size from 75 mm to greater than 120 mm.

5.10 Bedrock and refusal

The overburden soils described above are underlain by grey metavolcanic bedrock. The bedrock is generally described as moderately weathered to fresh. Sub-vertical fractures were noted throughout the bedrock cores.

Bedrock was proved by coring in Boreholes PINE-03 and PINE-04 drilled at the south and north abutments, respectively. Table 5.1 summarizes depths and elevations to the top of bedrock and auger refusal on probable bedrock or boulders.

Table 5.1 – Depths and Elevations of Top of Bedrock and Auger Refusal

Location	Borehole	Top of Bedrock/ Auger Refusal	
		Depth (m)	Elevation (m)
South Abutment	PINE -02	15.8	84.0
	PINE-03	22.7 ⁽¹⁾	77.1
North Abutment	PINE-04	16.6 ⁽¹⁾	83.3
	PINE-05	14.6	85.3

⁽¹⁾ Bedrock proved by coring

Total core recovery (TCR) in the bedrock cores ranged from 92% to 100%. The RQD value ranged from 17% to 37% in Borehole PINE-03, indicating very poor to poor rock quality. The RQD was 80% in Borehole PINE-04, indicating good rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally less than 7.

The average estimated unconfined compressive strength of the rock cores ranged from 74 MPa to 135 MPa, indicating a strong to very strong rock. These estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes. These results are summarized on the Record of Borehole sheets included in Appendix A. A summary of the Point Load Test Results is presented in Appendix B.

5.11 Water Levels

Water levels were not observed in the open boreholes during the drilling operations as water was introduced into the boreholes in order to complete and core them. One standpipe piezometer was installed in Borehole PINE-05 to monitor water levels after completion of drilling. However, this piezometer was destroyed prior to a subsequent site

visit for obtaining stabilized water level reading and no piezometric reading is therefore available.

GA drawings indicate that the water level in the Pinewood River was at elevation 96.08 m in May 2011.

6 MISCELLANEOUS

Borehole locations were selected in the field by Thurber Engineering Ltd. Borehole elevations and coordinates were provided by Genivar.

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

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a truck mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Ms. Eckie Siu of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng. Interpretation of the data and preparation of this report were carried out by Ms. Lindsey Blaine, E.I.T. and Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

Lindsey Blaine, E.I.T.
Project Manager

L. Blaine
Dec. 5/12

Rocío Palomeque Reyna, P.Eng.
Geotechnical Engineer



P. K. Chatterji, P.Eng.
Review Principal



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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 presents geotechnical recommendations for design of a new bridge to replace the existing bridge that carries Highway 617 over the Pinewood River, approximately 14.7 km north of Stratton, Ontario, in the Rainy River District.

Highway 617 is a two-lane paved road. The existing bridge consists of a five span concrete/timber deck structure supported on four piers and two abutments. The piers and abutments are supported on timber bents. The length and width of the bridge are 28.6 m and 9.14 m, respectively. The embankments are approximately 1.0 m to 4.6 m high.

Based on the General Arrangement (GA) drawing provided by Genivar, the proposed bridge consists of a two lane, single span structure with a precast concrete girders supported on a single row of driven steel H-piles. A sheet pile wall will be driven just behind the H-piles to retain the approach fill. The proposed length of the bridge is 26.3 m with a width of 8.5 m. It is anticipated that the replacement structure will be constructed along the existing horizontal alignment. The bridge approach will be raised by approximately 860 mm at the bridge location.

The discussions and recommendations presented in this report are based on the factual data obtained during the course of the investigation. The plans and profiles used for preparation of this report were provided by Genivar.

8 STRUCTURE FOUNDATIONS

The stratigraphy encountered in the six boreholes drilled at the north and south approaches and abutments revealed asphalt overlying loose to compact sand fill and firm to soft silty clay fill. The thickness of the fill varied from 3.0 m to 4.6 m at the south abutment and from 1.0 m to 1.9 m at

the north abutment. In Borehole PINE-02, drilled at the south abutment, coring through occasional cobbles and boulders was required to advance the borehole. A 1.2-m thick layer of organic clayey silt was encountered below the sand fill in one borehole (PINE-04) drilled at the north abutment. Below the fill, a layer of very soft to firm silty clay was contacted, varying in thickness from 6.1 m to 9.5 m. Locally at the south approach, a layer of clayey silt was encountered below the silty clay. Layers of compact to very dense gravelly sand, silt and sand and sand and gravel with cobbles and possibly boulders were encountered below the silty clay. The overburden is underlain by metavolcanic bedrock, at 22.7 m and 16.6 m depth (elevations 77.1 and 83.3) at the south and north abutments, respectively. Auger refusal on probable boulders or bedrock was met at depths ranging from 14.6 m to 15.8 m in two of the boreholes (elevations 85.3 and 84.0).

GA drawings indicate that the water level in the Pinewood River was at elevation 96.08 m in May 2011.

Based on borehole data, the possibility exists that the embankment fill may contain cobbles and boulders that were not identified during the field investigation and reported in the borehole logs.

Geotechnical recommendations for design of the proposed H-pile foundation system are presented in the following sections.

Consideration was also given to the following foundation types to support the bridge in the event that the foundation concept changes:

- Sheet pile foundation walls supporting the precast prestressed girder. Consideration of this option was requested by the designers.
- Spread footings on native soils.
- Augered Caissons (drilled shafts).

A comparison of the technical advantages and disadvantages of alternative foundation schemes (driven steel H-piles, sheet pile foundation, spread footings on native soil, and caissons/drilled shafts) is presented in Appendix D. A foundation scheme preferred from a foundations perspective is recommended.

8.1 Steel H-Pile Foundations

The ground conditions at the site are considered to be suitable for the support of abutments on driven steel H-pile foundations. In general, it is anticipated that the piles will encounter refusal at the bedrock surface. However, Boreholes PINE-03 and PINE-04 drilled at the south and north abutments, respectively, encountered a layer of sand and gravel with cobbles and possibly boulders at elevations 83.8 and 85.0, approximately 6.7 m to 1.7 m above the bedrock and the H-piles may encounter refusal in this layer.

The anticipated pile tip elevations, soil conditions, vertical, factored geotechnical resistance at Ultimate Limit States (ULS_f) and geotechnical resistance at Serviceability

Limit States (SLS) for H-piles driven to refusal on bedrock are presented in Table 8.1. The recommended geotechnical resistance values have been reduced to account for the possibility that some piles may encounter refusal in the sand and gravel above the bedrock.

Table 8.1 – Estimated Pile Tip Elevation and Recommended Pile Resistance Values

Foundation	Borehole	Soil Conditions at Pile Tip	Pile Tip Elevation	Pile Section HP 310 x 110	
				Factored Geotechnical Resistance at ULS (kN)	Geotechnical Resistance at SLS (kN)
South abutment	PINE-02 PINE-03	Bedrock	77.1	1,400	1,200
North abutment	PINE-04 PINE-05	Bedrock	83.3	1,400	1,200

* The GA drawing indicates that the lowest soffit is at elevation 99.36.

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

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

8.1.1 Pile Tips

For protection when contacting bedrock or driving to refusal on very dense sand and gravel with cobbles and boulders, the tips of all driven piles must be fitted with cast steel, H-section pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock.

8.1.2 Pile Installation

Pile installation should be in accordance with OPSS 903.

Pile installation should consider the following subsurface factors:

- The presence of cobbles and possibly boulders in the native sands and gravels. This has been discussed in Section 5.9 of the Factual Report.
- The possibility that piles may encounter refusal in cobbles and boulders at different elevations above the anticipated bedrock elevation.

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

8.1.3 Pile driven to bedrock

For piles installed for the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note “Piles to be driven to bedrock”.

8.1.4 Downdrag

Since the highway grade will be raised by 860 mm, downdrag forces will develop along the length of the pile embedded in the silty clay layer. Additionally, a 1.2-m thick layer of loose organic clayey silt was encountered at 1.2 m depth (elevation 98.7) in Borehole PINE-04, drilled at the northeast side of the north embankment.

Downdrag forces will develop along the length of pile embedded in the organic clayey silt and soft silty clay layer due to increased approach embankment loads. For design purposes, an unfactored downdrag load of 280 kN per pile is recommended to evaluate the impact of downdrag on the abutment piles. This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C8.6.4 to obtain a factored downdrag load.

In accordance with Section 6.8.4 of the CHBDC and clause C6.8.4 of the Commentary to CHBDC, in the structural design of a pile at the abutments, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag.

In geotechnical analysis of downdrag, live load effects should not be considered. The location of the neutral plane for a pile or groups of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

8.1.5 Lateral Resistance

The lateral resistance of a pile in the cohesionless soils encountered at this site 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

γ = 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

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

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

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

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \cdot L \cdot D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), D is the pile width (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 35 kN at SLS.

Table 8.2 – Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	S_u (kPa)	K_p	Unit Weight* (kN/m ³)	Soil Conditions
South abutment	OGL to 98.0	3,500	-	3.0	21	Sand fill, compact
	98.0 to 96.0	-	30	2.7	10*	Silty clay fill, very soft to stiff
	96.0 to 88.0	-	25	2.7	10*	Silty clay, very soft to firm
	88.0 to 84.0	6,000	-	3.3	11*	Sand and silt, compact to very dense
	84.0 to 77.0	10,000	-	3.7	11*	Sand and gravel, cobbles and boulders
North abutment	OGL to 98.0	3,500	-	3.0	21	Sand fill, compact
	98.0 to 97.5	1,000	-	2.7	9*	Organic clayey silt, loose
	97.5 to 89.6	-	25	2.7	10*	Silty clay, very soft to firm
	89.6 to 85.0	10,000	-	3.3	11*	Sand and silt, gravelly sand, dense to very dense
	85.0 to 83.3	10,000	-	3.3	11*	Sand and gravel, cobbles and boulders, compact to very dense

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

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.

8.2 Steel Sheet Pile Abutment Walls

Consideration was given to supporting the proposed abutments on steel sheet piles driven to refusal on bedrock. As indicated earlier, a layer of sand and gravel containing cobbles and possibly boulders was encountered above the bedrock surface at this site and it is possible that some of the sheet piles may encounter refusal on this layer.

Vertical, factored geotechnical resistance at Ultimate Limit States (ULS_f) and geotechnical resistance at Serviceability Limit States (SLS) for four sheet pile sections driven to refusal on bedrock are presented in Table 8.3. The geotechnical resistance values have been reduced to account for the possibility that some or all of the sheet piles may encounter refusal in the sand and gravel layer above the bedrock.

**Table 8.3 – Estimated Sheet Pile Tip Elevation and
 Recommended Sheet Pile Resistance Values**

Foundation	Soil Conditions at Pile Tip	Pile Tip Elevation	Sheet Pile Section							
			Geotechnical Resistance							
			EZ-88		XZ-100		JZ-127		AZ36-700N	
			Factored ULS per meter width (kN)	SLS per meter width (kN)	Factored ULS per meter width (kN)	SLS per meter width (kN)	Factored ULS per meter width (kN)	SLS per meter width (kN)	Factored ULS per meter width (kN)	SLS per meter width (kN)
South abutment	Bedrock	77.1	650	540	740	600	800	650	850	700
North abutment		83.3								

Sheet piles should be provided with sheet pile tip protector to minimize any tip damage.

The abutment reactions on steel sheet pile walls and service and ultimate loads were reported by Genivar to be 544 kN/m at ULS and 389 kN/m at SLS.

Steel sheet pile installation should be in accordance with OPSS 903.

Sheet piles should be driven to the specified elevations noted in Table 8.3.

The lateral resistance of sheet piles may be computed using the lateral earth pressure distribution and parameters presented in Section 9.

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause long term corrosion and reduce the service life of the structure.

8.3 Spread Footings on Native Soils

Spread footings founded on native soils are not recommended at this site due to the following reasons:

- The geotechnical resistance available in the native soils below the fill is relatively low and there is potential for large settlement.
- Excavation, up to 3.0 m deep, will be required to construct spread footings. Unwatering/groundwater control will be difficult for construction of footings below the water level in the Pinewood River.
- Spread footings could be subject to erosion or undermining/scour during high river flows.

8.4 Caissons

Augered caissons founded on bedrock were also considered for the support of the structure. However, the use of augered caissons is not recommended in view of the depth to bedrock which is in the order of 16.6 m to 22.7 m below existing ground surface and the presence of water-bearing cohesionless soils containing cobbles and boulders immediately above the bedrock at this site. The base of the caissons will be well below the groundwater level, resulting in difficulties in dewatering, base cleaning and base inspection. Construction of caissons will require the use of a liner sealed below the sand/silt and sand/gravel layers and/or slurry methods to control ground water, support the sidewalls of the shaft. Drilling of caissons through the sand and gravel layer with cobbles and boulders just above the bedrock will be difficult.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended.

8.5 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions steel H-pile foundation driven to refusal in the dense sand and gravel layer with cobbles and boulders immediately above the bedrock is the preferred foundation option at this site.

Supporting the bridge girders on the steel sheet pile abutment foundations driven to refusal on the sand and gravel layer above the bedrock is also a feasible option.

8.6 Frost Cover

The depth of frost penetration at this site is 2.3 m. The base of pile caps, if employed, must be provided with a minimum of 2.3 m of earth cover as protection against frost action.

9 EARTH PRESSURE ON SHEET PILE WALLS

The GA drawing indicates that steel sheet pile walls will be driven just behind the H-pile foundations at each abutment. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

The second design option for this site indicates that steel sheet pile walls will support the new bridge.

Driving of the sheet piles through the existing approach fill and into the underlying very soft silty clay and compact to very dense sands and silts is considered feasible based on the borehole data.

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 OPSS 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

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, the native silty clay and the underlain sand and silt layer. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

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

K = earth pressure coefficient (see Table 9.1)

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

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the 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 $\phi = 35^\circ, \gamma = 22.8$ kN/m ³		OPSS Granular B Type I or Type III, Native Sand and Silt $\phi = 32^\circ, \gamma = 21.2$ kN/m ³		Sand fill $\phi = 30^\circ$, $\gamma = 20$ kN/m ³		Native Silty Clay and Silty Clay Fill $\phi = 27^\circ$, $\gamma = 18$ kN/m ³	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall	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*	0.33	0.57*	0.37	0.77*
At rest (Restrained Wall)	0.43	-	0.47	-	0.5	-	0.55	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	3.0	-	2.7	-

* For wing walls.

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

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

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

The design of the sheet pile abutment walls must incorporate measures such as subdrains to permit drainage of the sheet pile abutment walls backfill, or alternatively the sheet pile abutment walls should be designed to withstand the potential build-up of hydrostatic pressures behind the walls.

10 EXCAVATION AND GROUNDWATER CONTROL

Any excavation required at this site must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and silty clay fill forming the existing approach embankment and the underlying native silty clay layer may be classified as Type 3 soil above the water table and Type 4 below the water table.

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

The selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. Excavations should be inspected regularly for evidence of instability.

The Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation. Excavation below the river level, if required for an alternate foundation system, would require dewatering within a cofferdam to lower the water level below the base of the excavation.

11 APPROACH EMBANKMENTS

Based on the GA drawing provided by Genivar, it was estimated that the existing approach embankment is 1.0 m to 5.0 m high. The foundation soils governing stability of the approach embankments consist generally of existing native very soft to firm silty clay over compact to very dense sand and silt and sand and gravel layers.

Comments regarding stability of the embankments and settlement of the foundations soils at the approaches due to the grade raise are provided in the following sections.

11.1 Slope stability

The global, internal and surficial stability of the approach embankment fills depends on the slope geometry and also to a large degree on the material used to construct the embankment. The existing embankments bearing on the foundation soils at this site appear to be performing satisfactorily under the existing conditions.

GA drawing provided by Genivar indicates that the existing Highway 617 grade will be raised approximately 860 mm at the bridge location. The GA drawing also indicate that the forward slopes are at an approximate inclination of 3H:1V. Placement of an additional 860 mm of new fill to raise the road grade is expected to have minimal impact on the stability of the embankments.

The additional approach fill to be placed behind the new abutment will be supported within a sheet pile enclosure and therefore the stability of the new approach will be governed by the sheet pile wall design. A global slope stability analysis was conducted to assess the embedment requirements for a sheet pile supporting the new approach fill including the 860 mm grade raise. The analyses were carried out using the Morgenstern-Price method of slope stability analysis.

The results of the analyses indicate that an adequate factor of safety for the long term conditions of 1.6 is achieved if the sheet pile is driven to elevation 95.0.

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

The slope stability computation outputs are included in Appendix E.

Embankment construction for a grade raise should be carried out in accordance with OPSS 206. It is recommended that earth fill should consists of granular materials in compliance with Special Provision 110S13, "Amendment to OPSS 1010 April 2004".

For widening of the embankment where placement of new fill is required, the existing slope surfaces should be appropriately benched, as per OPSD 208.010, after stripping of vegetation, topsoil, organic clayey silt, soft soils or otherwise unsuitable overburden materials.

In general, earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 804.

11.2 Settlement

The existing embankments have been in place for some years and it is anticipated that settlements of the native soils are completed.

However, placement of approximately 860 mm of new fill will induce some immediate (elastic) settlement in the existing non-cohesive fill as well as time dependent (consolidation) settlement in the underlying silty clay fill and native silty clay.

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 are as follows:

Location	Case	Elastic Settlement (mm)	Consolidation Settlement/Post Construction Settlement (mm)	Total Settlement (mm)
North Approach	Granular	10	30	40
South Approach	Granular	15	35	50

The immediate settlement will be essentially complete when construction of the fill is completed.

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.

The primary consolidation settlement is expected to be completed within 8 to 12 months of completion of fill placement.

If post construction settlement and maintenance is not acceptable, consideration should be given to using lightweight fill or EPS as backfill behind the sheet pile wall in order to reduce the settlements.

12 EROSION PROTECTION

Erosion protection should be provided along the toe 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.

13 ROADWAY PROTECTION

During the new bridge construction, temporary excavation of existing embankments will be required. 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 617 during bridge replacement operations.

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

Conventional steel soldier pile and timber lagging walls is a viable option 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:

γ	=	21 kN/m ³	(bulk unit weight)
γ_w	=	11 kN/m ³	(submerged unit weight under groundwater table)
K_a	=	0.33	(Active pressure coefficient for road embankment sand fill)
	=	0.37	(Active pressure coefficient for road embankment silty clay fill and native silty clay)
K_p	=	3.0	(Passive pressure coefficient for road embankment fill)
	=	2.7	(Passive pressure coefficient for road embankment silty clay fill and native silty clay)
h_w	=	96.08	(water level)

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

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

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

14 SEISMIC CONSIDERATIONS

14.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.02

The soil profile type at this site has been classified as Type III. Therefore, according to Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.5 should be used in seismic design.

14.2 Retaining Wall Dynamic Earth Pressures

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, Native Sand and Silt $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$	Sand fill, Native Sand/Silt $\phi = 30^\circ$ $\gamma = 20 \text{ kN/m}^3$	Native Silty Clay and Silty Clay Fill $\phi = 27^\circ$ $\gamma = 18 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32	0.34	0.38
Passive (K_{PE})	3.7	3.2	3.0	2.6
At Rest (K_{OE})**	0.45	0.50	0.52	0.57

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

** After Woods

14.3 Liquefaction Potential

The site overlies very soft to firm deposits and a high water table is present at this location.

A review of the subsurface conditions indicates that the site is not susceptible to liquefaction under current conditions. Localized liquefaction during a seismic event may result in local toe failure or minor embankment settlement, but this is expected to be readily repairable.

If the structure is supported on steel piles, the foundation loads will be transferred by the steel piles to bedrock. In this case, it is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

15 CONSTRUCTION CONCERNS

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

- The potential variability of pile lengths since some piles may reach refusal in a cobbles/boulders above the bedrock.
- H-piles and sheet pile tips must be protected and driving must be terminated before pile/sheet pile is damaged.
- Based on borehole data (Borehole PINE-02), cobbles and boulders may be encountered within the embankment fill. If the sheet piles encounter these conditions, the obstructions will have to be removed for driving sheet piles.
- Excavation, if required, should be maintained above the water level in the river.
- The approach embankment slopes should be protected from erosion.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, P.Eng.

The report was reviewed by Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Rocio Palomeque Reyna, P.Eng.
Geotechnical Engineer



P. K. Chatterji, P.Eng.
Review Principal



Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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


4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level






C_{pen} Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength (MPa) (psi)	Field Estimation of Hardness*	
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.

METRIC

[illegible]

+³, ×³ Numbers refer to Sensitivity

RECORD OF BOREHOLE No PINE-01

2 OF 2

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
DATUM Geodetic DATE 2011.08.16 - 2011.08.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
	Continued From Previous Page													
89.1	Silty CLAY , some sand Very Soft Grey Wet													
10.7	Clayey SILT , trace sand Stiff Grey		9	SS	11		89							0 2 79 19
88.5														
11.3	END OF BOREHOLE AT 11.3m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.2m, SAND TO 0.04m, THEN ASPHALT TO SURFACE.													

RECORD OF BOREHOLE No PINE-02

1 OF 2

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
 HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.08.16 - 2011.08.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
99.8	ASPHALT: (35mm)											
98.5	SAND, some gravel Compact Brown Moist (FILL)		1	SS	10		99					
1.3	Silty CLAY, trace to some sand, trace gravel, occasional organics and wood fibres Firm to Soft Brown (FILL)		2	SS	7		98					
			3	SS	3		97					
96.5	Cobble at 2.7m Cored through boulder (240mm) at 3.0m		4	CORE								
3.3	Silty CLAY, some sand to sandy Soft to Very Soft Dark Grey to Grey						96					
			5	SS	4		95					0 34 39 27
			6	SS	0		94					
			7	SS	0		93					
			8	SS	1		92					
							91					
							90					0 15 38 47

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PINE-02

2 OF 2

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
DATUM Geodetic DATE 2011.08.16 - 2011.08.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE	w _P w w _L				
	Continued From Previous Page														
	Silty CLAY , varved with silt layers Firm Grey		9	SS	6		89								
							88								
87.6															
12.2	SILT and SAND , trace clay, trace gravel Compact to Very Dense Grey Moist		10	SS	20		87								
	Occasional coarse sand seams		11	SS	57		86							2 48 46 4	
							85								
	Occasional cobbles		12	SS	50/ 0.075										
84.0							84								
15.8	END OF BOREHOLE AT 15.8m UPON CASING REFUSAL. BOREHOLE BACKFILLED WITH BENTONITE TO 0.2m, SAND TO 0.04m, THEN ASPHALT TO SURFACE.														

+³, x³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

METRIC

[illegible]

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PINE-03

2 OF 3

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
 HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.08.15 - 2011.08.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	Continued From Previous Page												
87.6	Silty CLAY , varved, silt layers Firm Grey		9	SS	4								
12.2	SILT and SAND , trace clay, trace gravel Compact Greenish Grey Moist		10	SS	21								1 38 57 4
	Dense		11	SS	48								
	Occasional cobbles		12	SS	45								
83.8	Casing refusal at 16.0m												
16.0	SAND and GRAVEL , with cobbles and boulders Grey Cored through cobbles (50mm to 120mm) from 16.0m to 22.7m		1	CORE									
			2	CORE									
	Mostly sand and gravel, occasional cobbles		3	CORE									

Continued Next Page



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

RECORD OF BOREHOLE No PINE-03

3 OF 3

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
 HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.08.15 - 2011.08.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L WATER CONTENT (%)		
	Continued From Previous Page						20 40 60 80 100									
77.1	SAND and GRAVEL, with cobbles and boulders Grey Frequent cobbles and boulders, some sand		4	CORE										RUN #1 TCR=100% SCR=47% RQD=17% UCS=100MPa (Average)		
22.7			BEDROCK, metavolcanic, grey			1	RUN									RUN #2 TCR=100% SCR=68% RQD=37% UCS=74MPa (Average)
74.1						2		RUN								
25.7	END OF BOREHOLE AT 25.7m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.1m, SAND TO 0.04m, THEN ASPHALT TO SURFACE.															

RECORD OF BOREHOLE No PINE-04

1 OF 3

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
 HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.08.14 - 2011.08.14 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE							w _p w w _L		
								● QUICK TRIAXIAL × LAB VANE									
99.9 0.0 0.1	ASPHALT: (75mm) SAND, some gravel Compact Brown Wet (FILL)						20	40	60	80	100	20	40	60	GR SA SI CL		
98.7			1	SS	14												
1.2	ORGANIC, clayey silt, trace sand, trace clay Loose Dark Brown Wet		2	SS	6												
97.5																	
2.4	Silly CLAY, some sand to sandy Firm to Very Soft Grey		3	SS	7												
			4	SS	4												
			5	SS	0												
	Occasional sand seams		6	SS	0												
			7	SS	0												
			8	SS	0												

Continued Next Page

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

METRIC

SOIL PROFILE						SAMPLES
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	
	Continued From Previous Page					
89.8 10.1	Gravelly SAND, coarse grained Very Dense Grey Moist	[Pattern]	9	SS	61	
88.8 11.1	SILT and SAND, trace clay, trace gravel Very Dense Grey Moist	[Pattern]	10	SS	64	
	Cobbles and boulders	[Pattern]	11	SS	63	
85.0 14.9	SAND and GRAVEL, with cobbles and boulders Cored through cobbles and boulders from 14.9m to 16.6m	[Pattern]	1	RUN		
83.3 16.6	BEDROCK, metavolcanic, grey Sub-vertical fractures (25mm to 75mm long) from 16.4m to 16.8m and 17.2m to 17.5m Clay seam (50mm thick) at 17.4m Sub-vertical fractures (25mm to 50mm long) at 17.9m, 18.1m, 18.2m, 18.6m 125mm at 18.3m 100mm at 18.5m 300mm at 18.6m 150mm at 19.1m 100mm at 19.2m Quartz interbeds (25mm to 75mm thick) at 18.3m, 18.9m, 19.1m, 19.3m 175mm at 18.0m	[Pattern]	2 3	RUN RUN		
80.4 19.5	END OF BOREHOLE AT 19.5m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.1m, SAND AND	[Pattern]				

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PINE-04

3 OF 3

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
DATUM Geodetic DATE 2011.08.14 - 2011.08.14 CHECKED BY RPR

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page GRAVEL TO 0.07m, THEN ASPHALT TO SURFACE.													

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PINE-05

2 OF 2

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
DATUM Geodetic DATE 2011.08.14 - 2011.08.14 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ KN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	Continued From Previous Page													
89.6	Silly CLAY													
10.3	Very Soft Grey													
	SILT and SAND, trace gravel, trace clay Very Dense to Dense Greenish Grey Moist		9	SS	85		89							2 53 39 6
	Layer of coarse sand		10	SS	44		88							
	Occasional cobbles		11	SS	117		86							
85.3														
14.6	END OF BOREHOLE AT 14.6m UPON REFUSAL ON PROBABLE BEDROCK OR BOULDERS. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. UNABLE TO TAKE READINGS AS PIEZOMETER WAS DESTROYED.													

METRIC

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	W _P	W		
20.0												
							SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100					

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PINE-06

2 OF 2

METRIC

W.P. 6094-10-00 LOCATION Pinewood River Bridge ORIGINATED BY ES
HWY 617 BOREHOLE TYPE Casing COMPILED BY AN
DATUM Geodetic DATE 2011.08.16 - 2011.08.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE		WATER CONTENT (%) w _p w w _L				
	Continued From Previous Page							20 40 60 80 100						
89.4	Silty CLAY Very Soft Grey													
10.5	SILT and SAND , trace gravel Very Dense Greenish Grey Moist		9	SS	41		89					○		
88.6														
11.3	END OF BOREHOLE AT 11.3m. BOREHOLE BACKFILLED WITH BENTONITE TO 0.7m, SAND TO 0.07m, THEN ASPHALT TO SURFACE.													

+³, X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

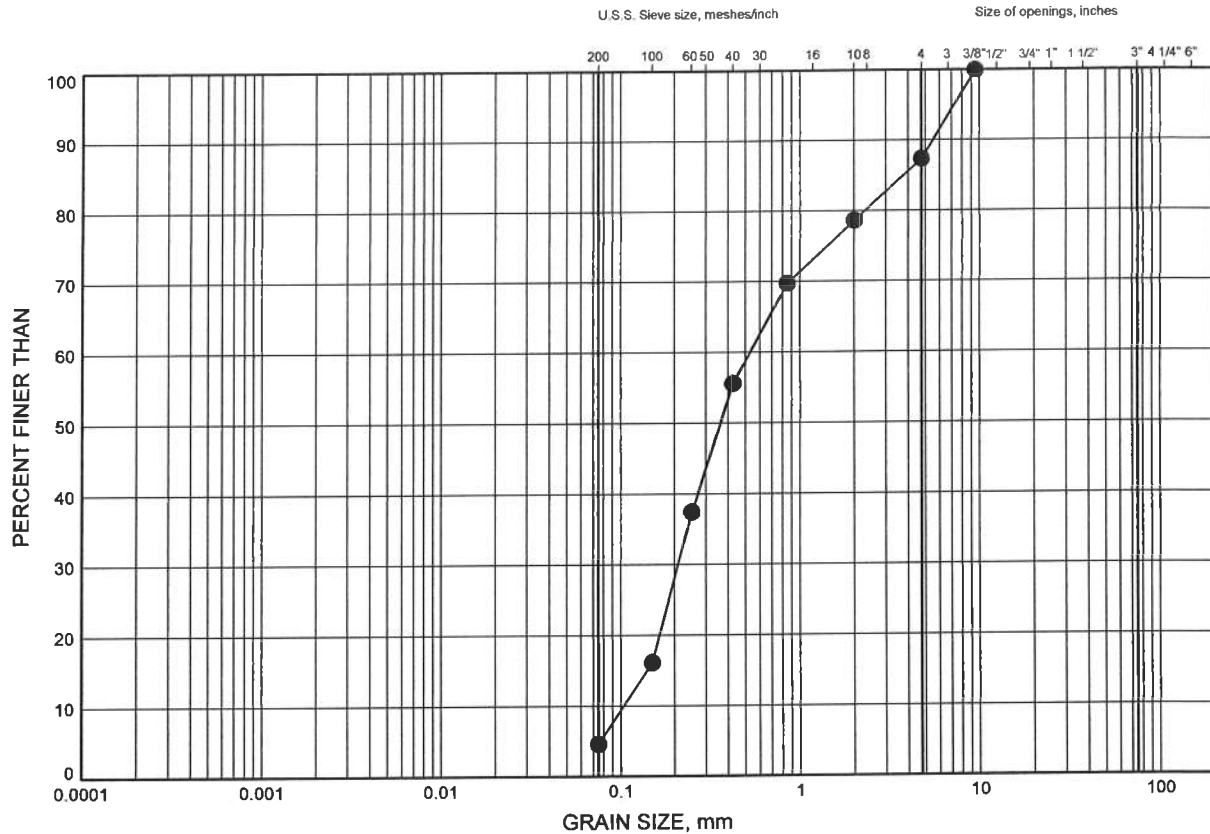
Appendix B

Laboratory Test Results

NWR HWY 11 Bridge 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)
●	PINE-05	1.07	98.79

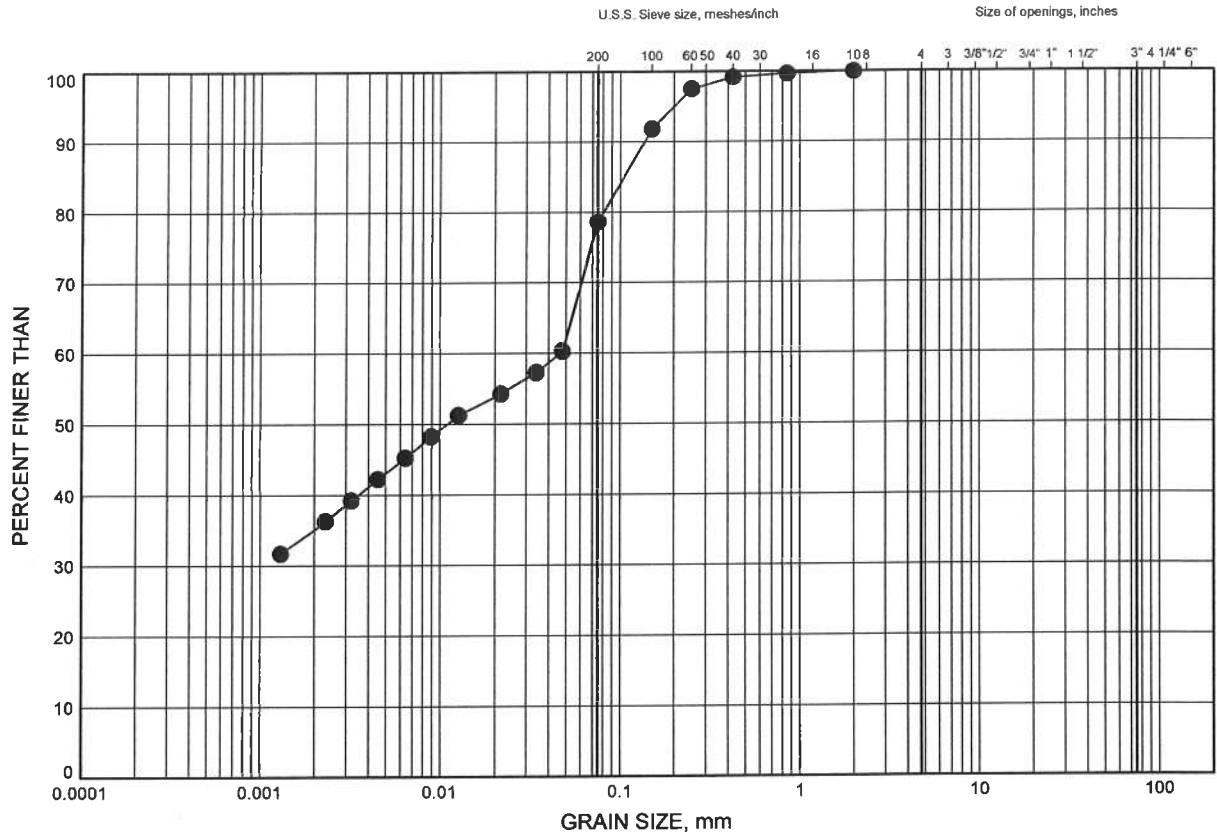


W.P.# 6094-10-00
Prepared By AN
Checked By RPR

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY FILL



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

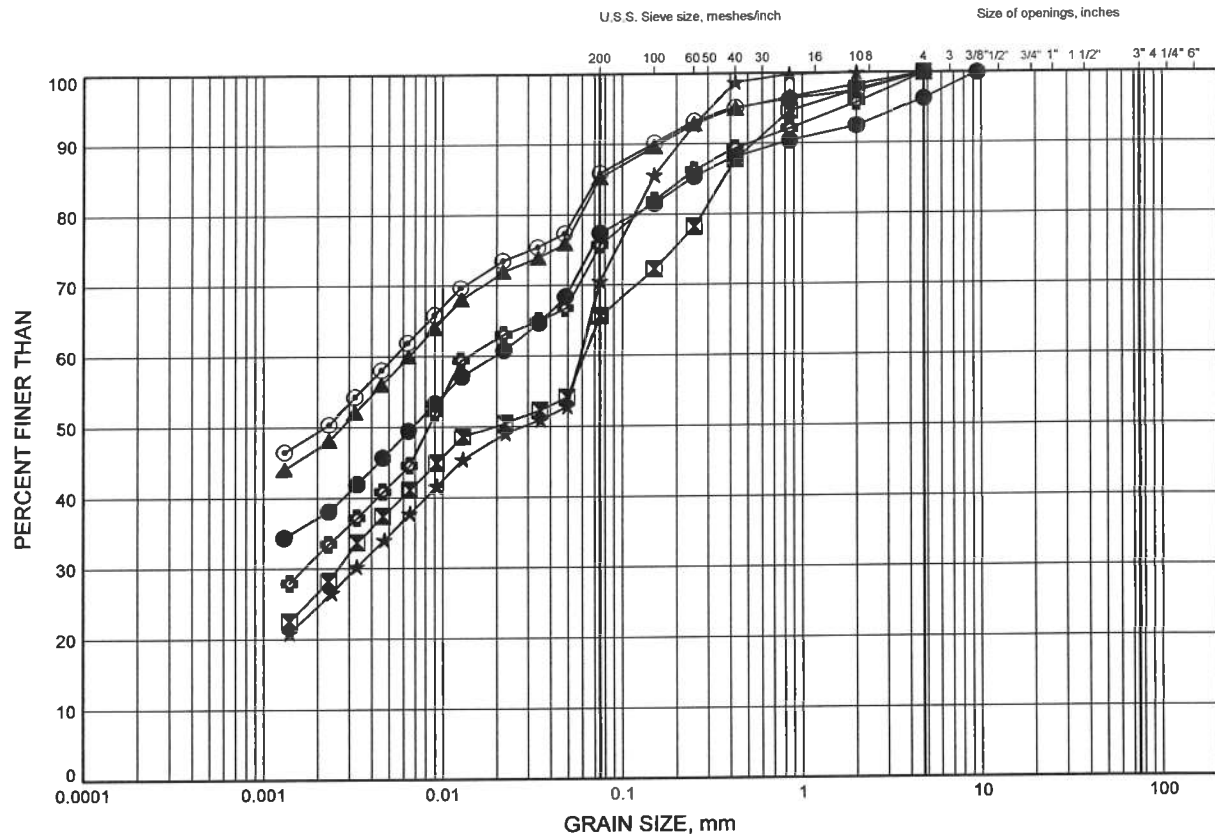
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-01	2.59	97.17

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-01	6.40	93.36
⊠	PINE-02	4.88	94.89
▲	PINE-02	9.45	90.32
★	PINE-03	3.35	96.41
⊙	PINE-03	7.92	91.83
⊗	PINE-04	6.40	93.47

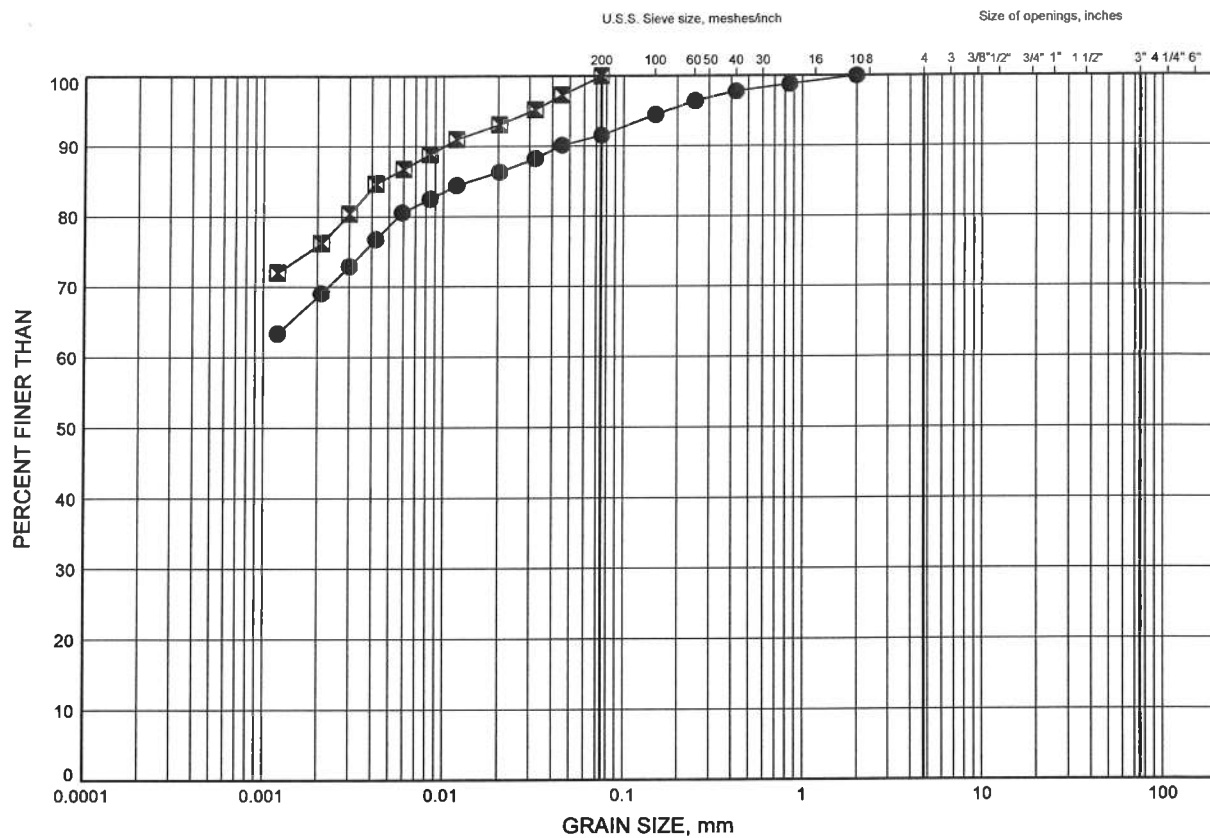


W.P.# 6094-10-00
Prepared By AN
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NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-05	4.88	94.98
■	PINE-06	2.59	97.31

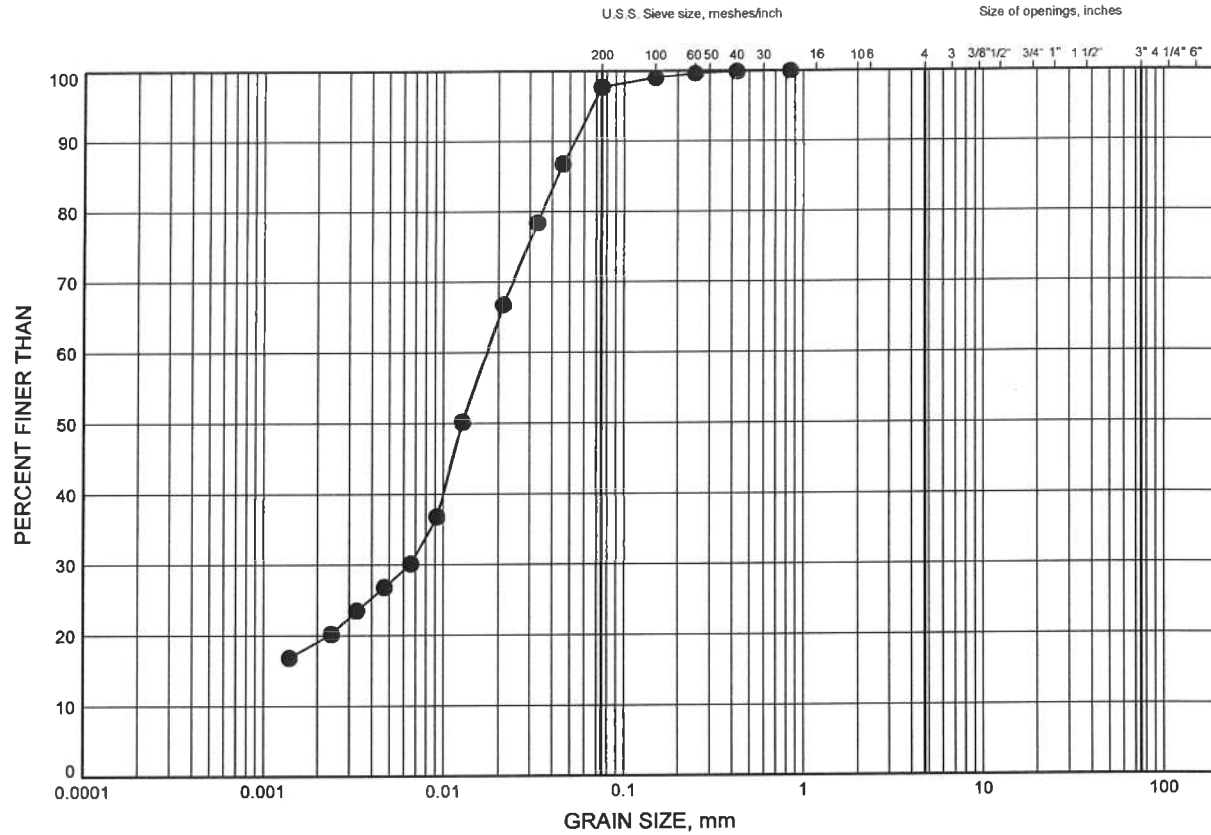


W.P.# 6094-10-00
Prepared By AN
Checked By RPR

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B5

CLAYEY SILT



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-01	10.97	88.79

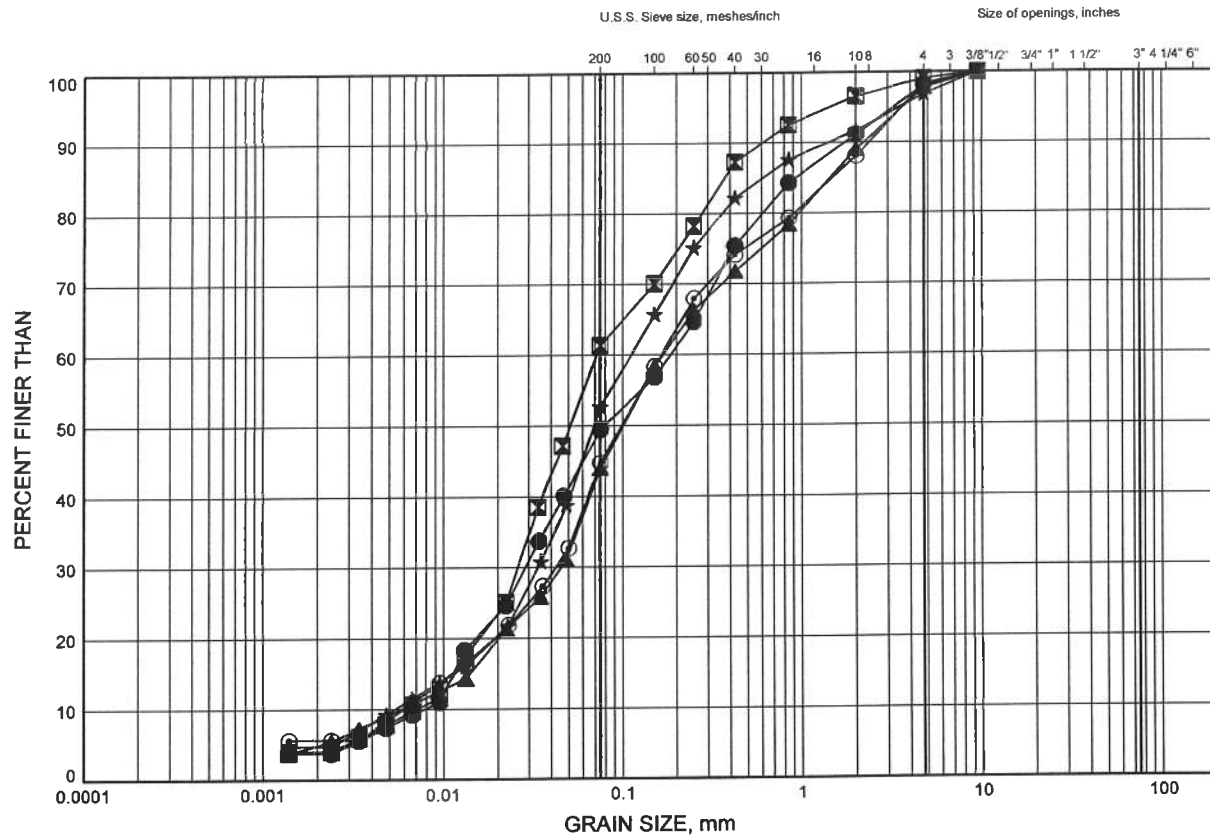


W.P.# 6094-10-00
Prepared By AN
Checked By RPR

NWR HWY 11 Bridge GRAIN SIZE DISTRIBUTION

FIGURE B6

SILT & SAND



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

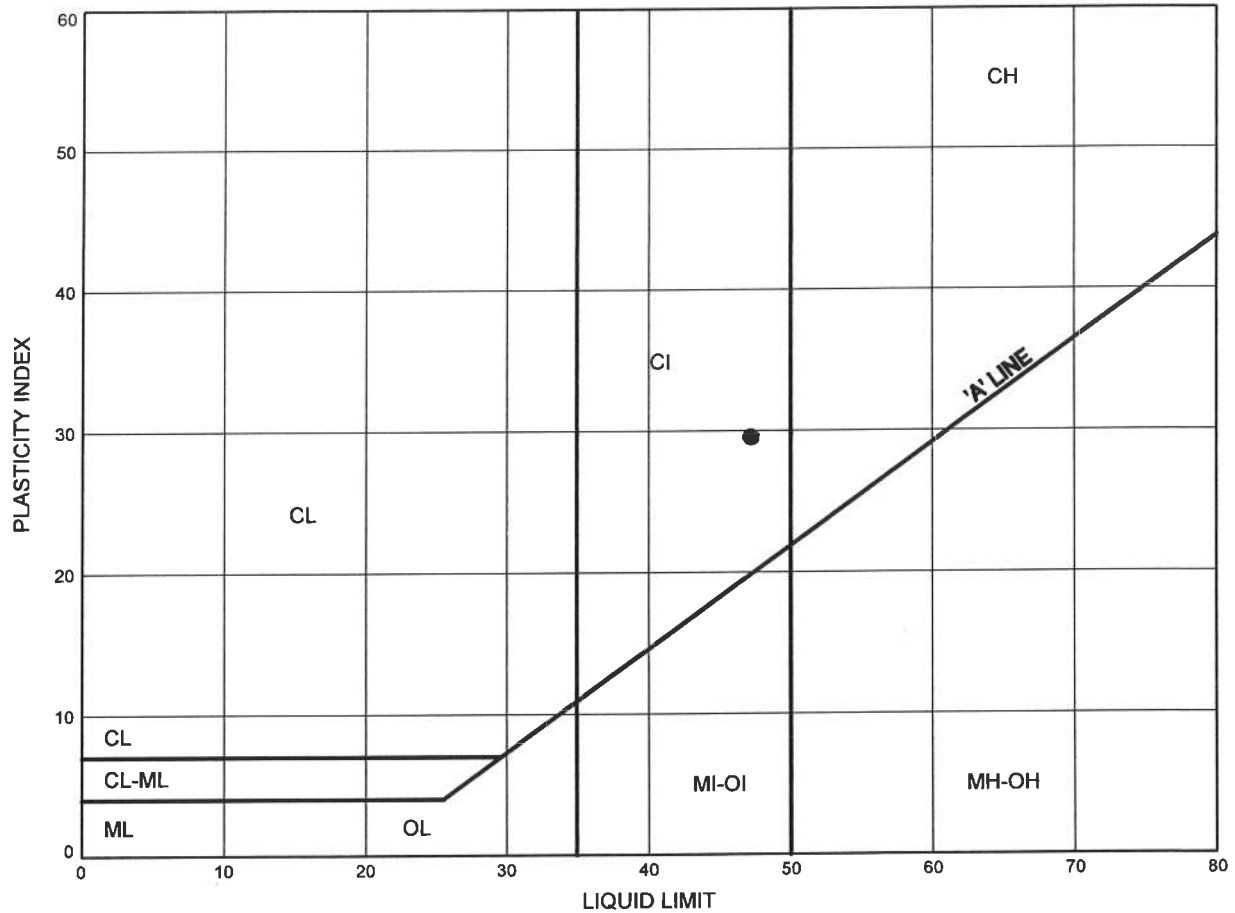
LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	PINE-02	14.02	85.75
⊠	PINE-03	12.50	87.26
▲	PINE-04	11.13	88.74
★	PINE-04	12.50	87.37
⊙	PINE-05	10.97	88.89

NWR HWY 11 Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B7

SILTY CLAY FILL

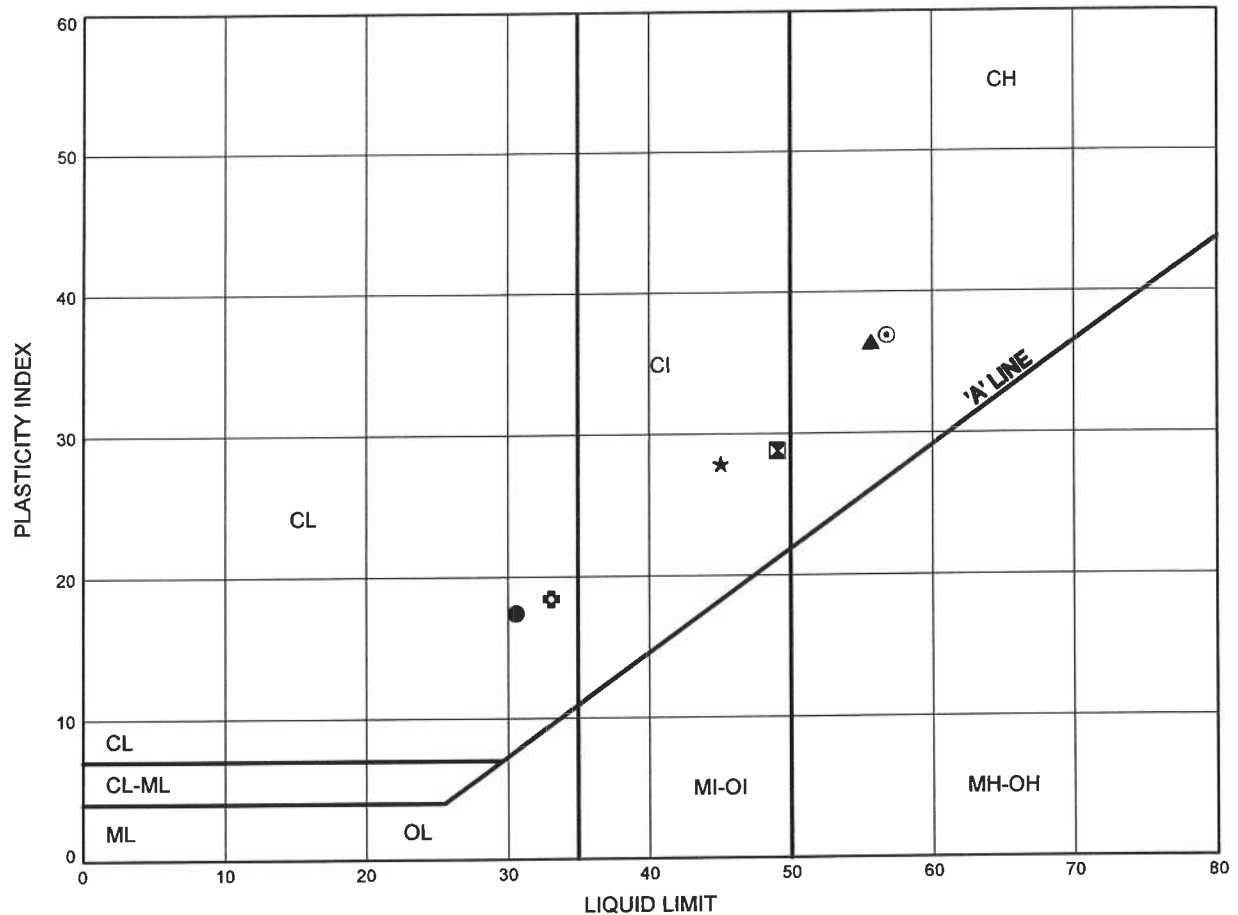


SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	PINE-01	2.59	97.17

NWR HWY 11 Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B8

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	PINE-01	6.40	93.36
⊠	PINE-02	4.88	94.89
▲	PINE-02	9.45	90.32
★	PINE-03	3.35	96.41
⊙	PINE-03	7.92	91.83
⊕	PINE-04	6.40	93.47

Date April 2012
 Project 6094-10-00

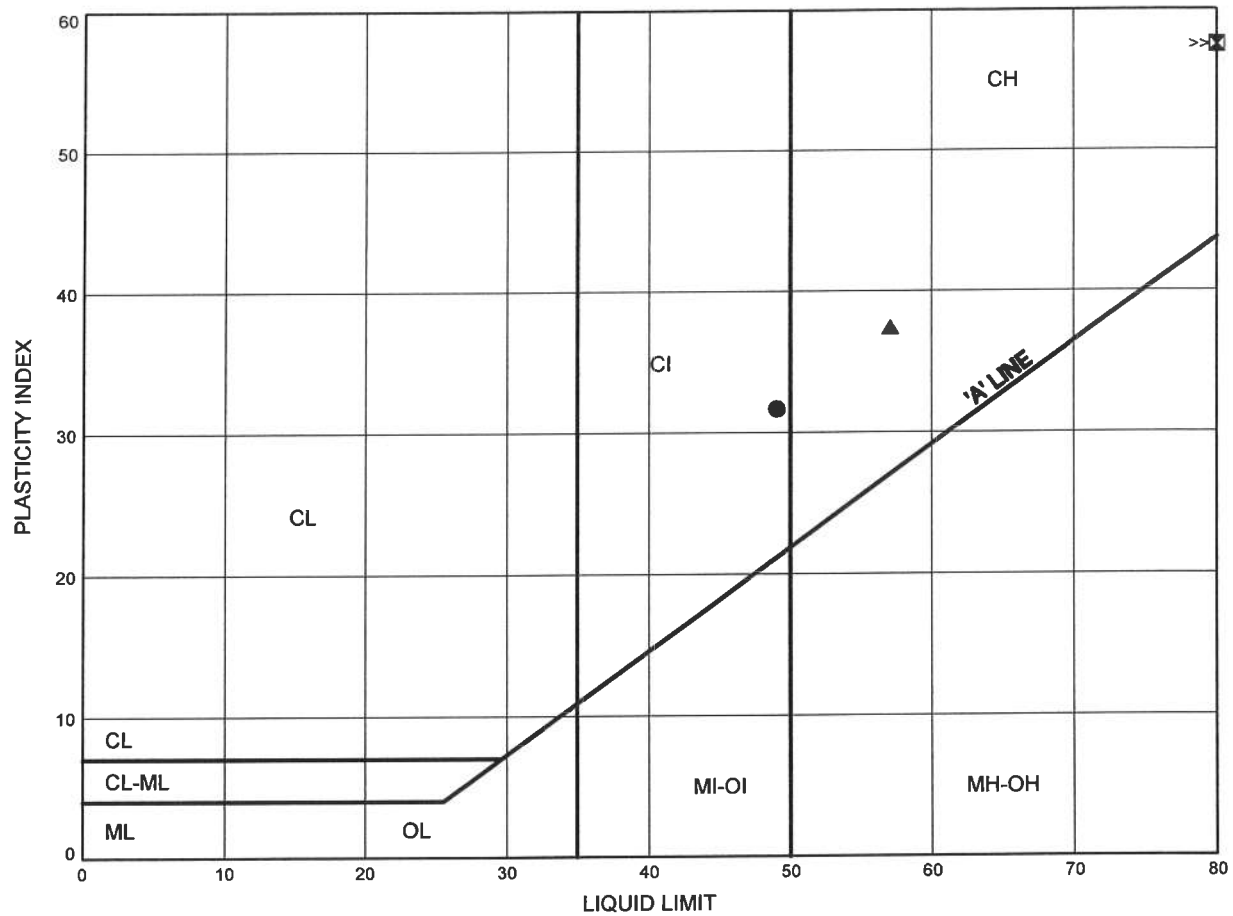


Prep'd AN
 Chkd. RPR

NWR HWY 11 Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B9

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	PINE-05	4.88	94.98
⊠	PINE-06	2.59	97.31
▲	PINE-06	7.92	91.97

Date April 2012
 Project 6094-10-00

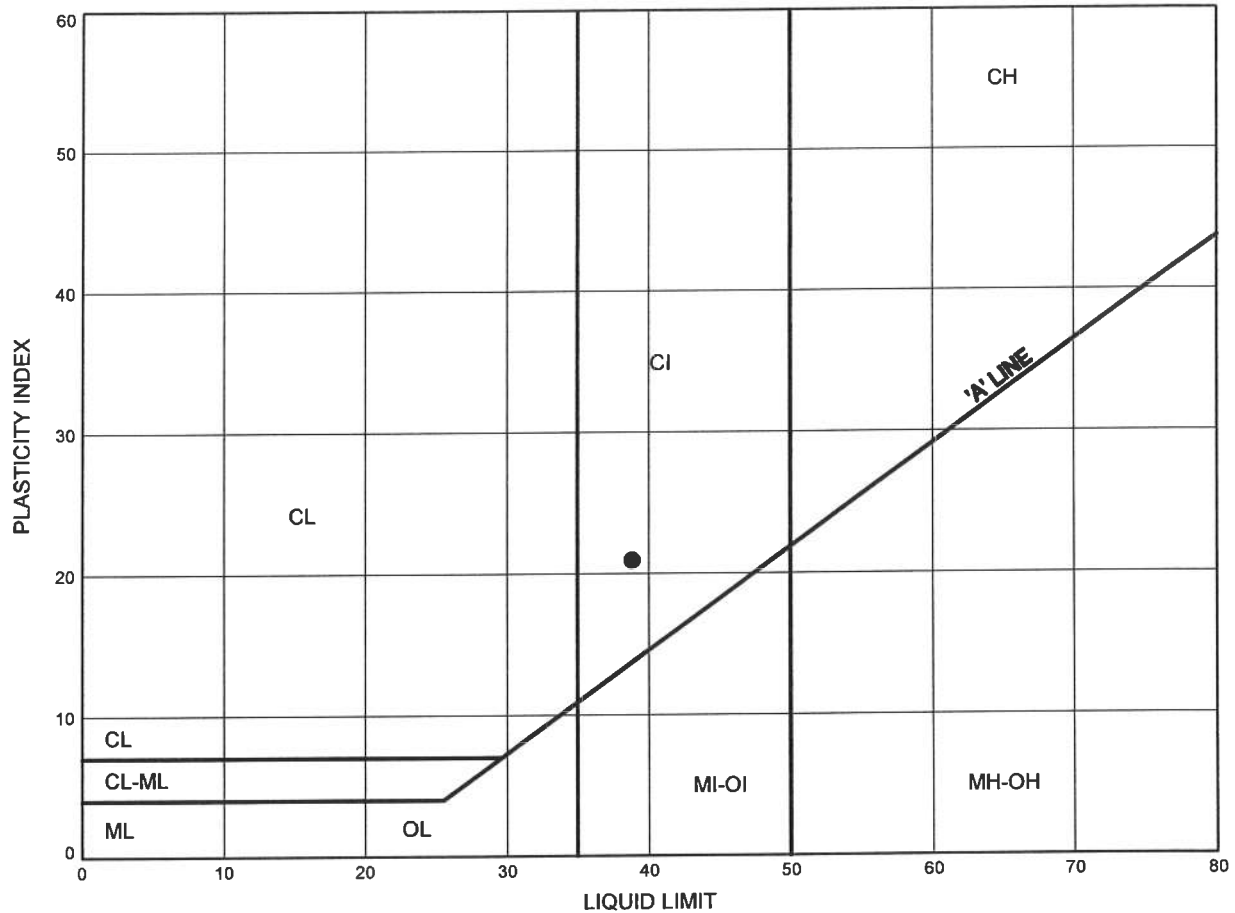


Prep'd AN
 Chkd. RPR

NWR HWY 11 Bridge
ATTERBERG LIMITS TEST RESULTS

FIGURE B10

CLAYEY SILT



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	PINE-01	10.97	88.79



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-5308-40 Client : GENIVAR
Date Drilled : August 15, 2011
Project Name : Pinewood River Bridge Date Tested : September 02, 2011
Core Size : NQ BH No : PINE-03 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	23.5	D	11.9	47.4	84.5	124.4		Very Strong
2	1	23.7	D	6.8	45.8	70.8	74.9		Strong
3	2	24.3	D	7.1	47.4	71.0	74.2		Strong
4	2	25.2	D	7.3	47.7	77.1	75.6		Strong
5	2	25.5	D	6.8	47.5	47.3	70.8		Strong
6									
7									
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1

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

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



THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

POINT LOAD TEST SHEET

Job No : 19-5308-40 Client : GENIVAR
Date Drilled : August 14, 2011
Project Name : Pinewood River Bridge Date Tested : September 02, 2011
Core Size : NQ BH No : PINE-04 Tester : DB

Test No.	Run No.	Depth (m)	Axial or Diametral	Force (kN)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	15.2	D	21.5	47.3	82.0	225.1		Very Strong
2	2	16.6	D	13.4	47.4	80.6	139.8		Very Strong
3	2	17.0	D	19.9	47.2	82.8	208.5		Very Strong
4	2	17.9	D	5.3	47.4	68.8	55.3		Strong
5	3	18.0	D	13.2	47.5	67.3	137.7		Very Strong
6	3	18.7	D	7.3	47.3	61.0	76.4		Strong
7	3	19.3	D	10.0	47.4	66.0	104.1		Very Strong
8									
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									

- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.

Appendix C

Site Photographs and

**Photographs of samples recovered from coring through the layer of sand and gravel with
cobble and boulders**



Photograph 1 – Pinewood River Bridge, looking south.

Pinewood River Bridge Replacement, Site 45-37
Highway 617, North of Stratton, Ontario



Photographs 2 and 3 – West side of Pinewood River Bridge



Photograph 4 – East side of Pinewood River Bridge



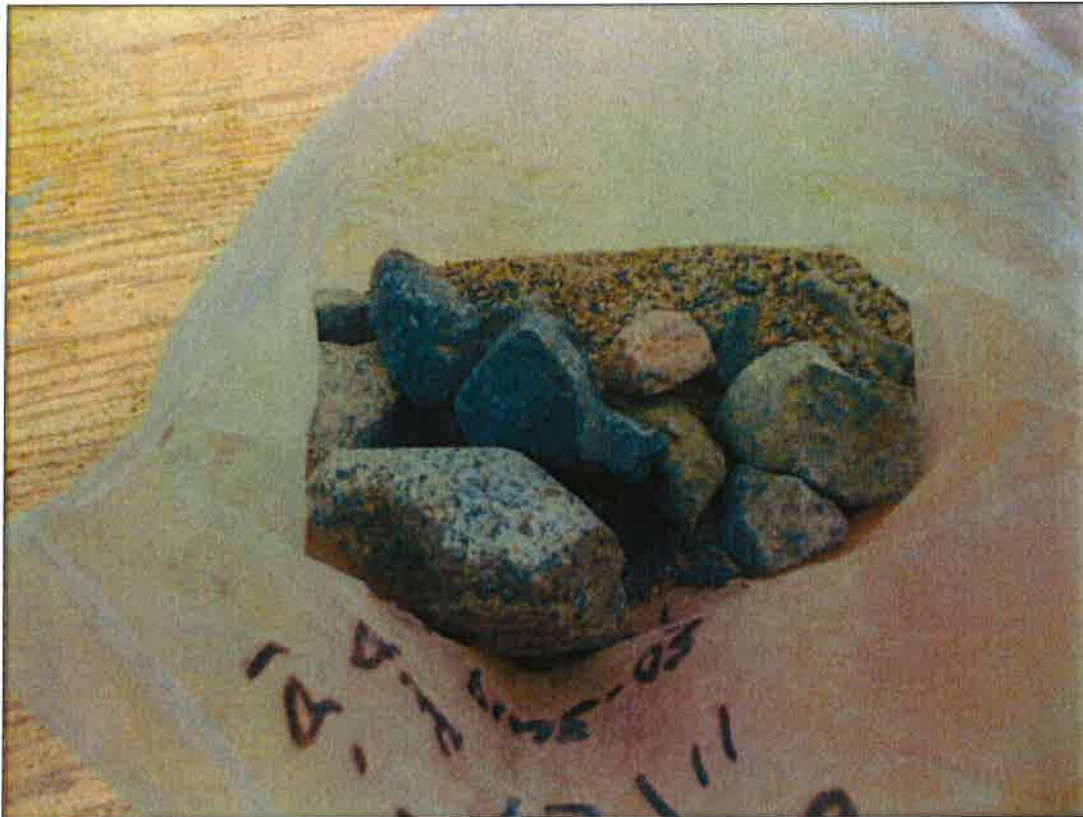
Photograph 5 - Sample from Borehole PINE-03, Run 2, Depth: 17.4 m to 18.9 m.
Soil description: Sand and gravel with cobbles.
Cobble diameter: 75 mm



Photograph 6- Sample from Borehole PINE-03, Run 3, Depth: 18.9 m to 20.4 m depth
Soil description: Sand and gravel with cobbles.
Cobble diameter: 75 mm



Photograph 7 - Sample from Borehole PINE-03, Run 4, Depth: 20.4 m to 21.9 m
Soil description: Sand and gravel with cobbles
Cobble diameter: 75 mm to 120 mm



Photograph 8 - Sample from Borehole PINE-03, Run 1, Depth: 15.9 m to 17.4 m

Soil description: Sand and gravel with cobbles

Cobble diameter: 75 mm

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

H-Piles Driven to Refusal	Sheet Piles Driven to refusal	Footings on Native Soil	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to refusal. ii. Installation of piles could continue in freezing weather. iii. Foundation construction may require less volume of excavation than footings. iv. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Pile lengths required to achieve design resistance may vary. <p style="text-align: center;">RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Provides shoring and foundation elements in one operation. iii. Installation of piles could continue in freezing weather. iv. Potentially minimizes volume of excavation and roadway protection requirements. v. Minimizes potential for disturbance of streambed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Unconventional design. ii. Cost of sheet piles. iii. Sheet pile lengths may vary due to variable depths of refusal. <p style="text-align: center;">FEASIBLE</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Low available geotechnical resistance in native cohesive deposits. ii. Potential for large settlements. iii. Dewatering will be required due to the high groundwater levels. iv. Potential disturbance of river during excavation. <p style="text-align: center;">NOT RECOMMENDED</p>	<p>Advantages:</p> <ul style="list-style-type: none"> i. Construction of caissons could continue in freezing weather. ii. High geotechnical resistance available for units founded on bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher cost than spread footings ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting bases. <p style="text-align: center;">NOT RECOMMENDED</p>

Appendix E

Slope Stability Outputs

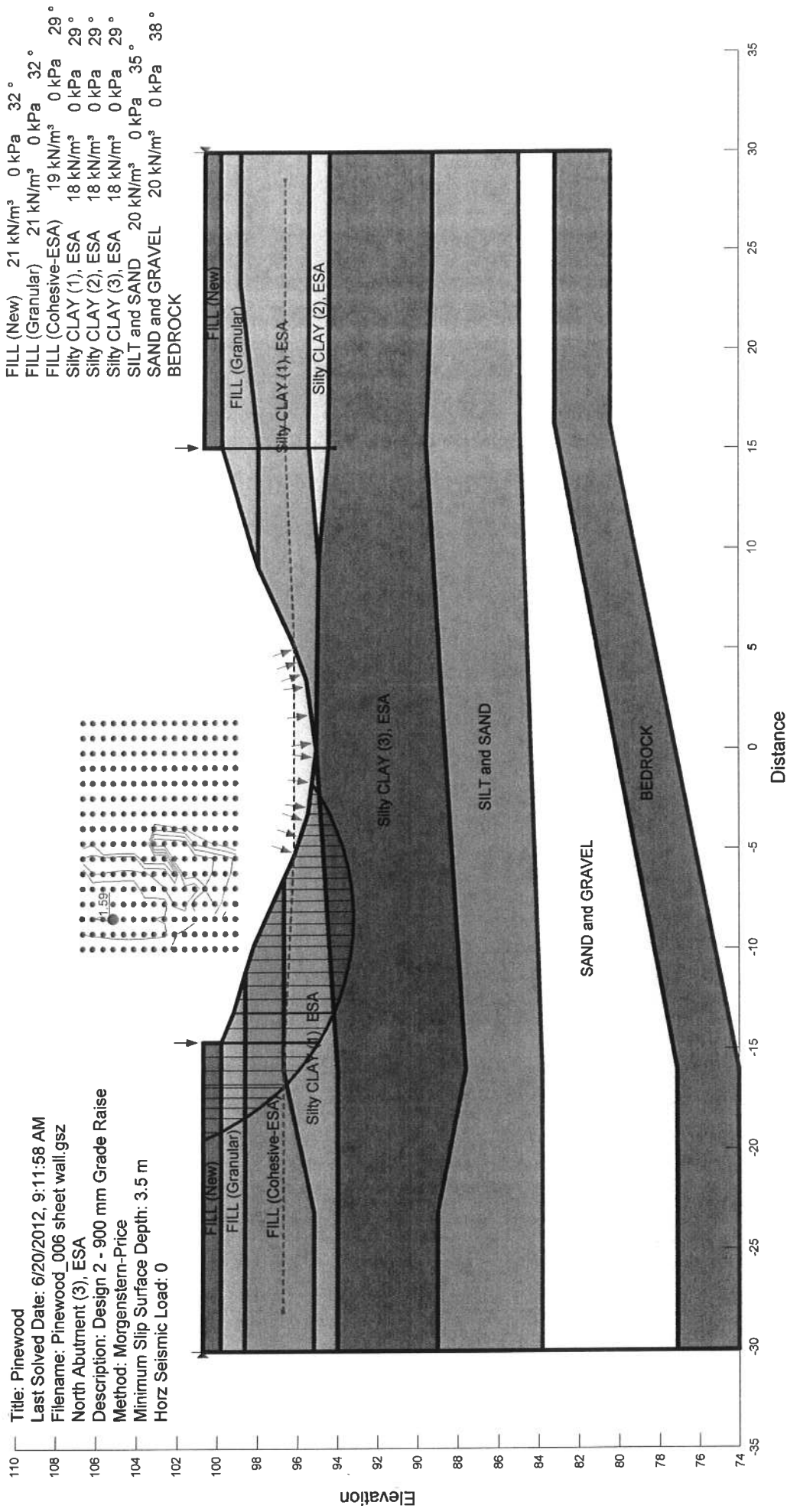


Figure 01

Directory: H:\19\5308\40 NWR 11 Bridge 3 Culvert Rehabs\Reports & Memos\Pinewood River Bridge\Analysis\Slope stability\

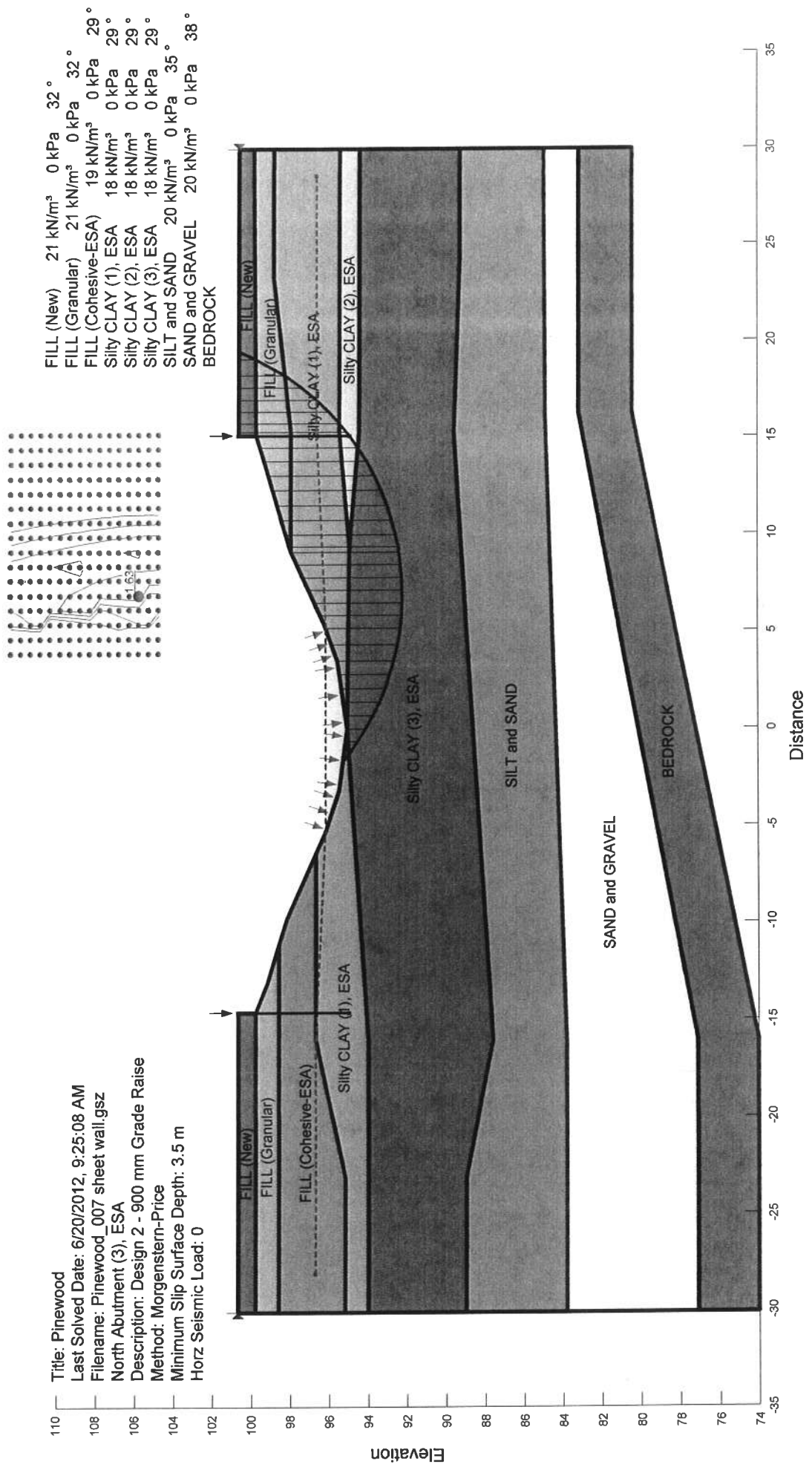


Figure 02

Directory: H:\19\5308\40 NWR 11 Bridge 3 Culvert Rehabs\Reports & Memos\Pinewood River Bridge\Analysis\Slope stability\

Appendix F

List of SPs and OPSS, and Suggested Text for NSSP

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

- OPSS 903
- OPSS 804
- OPSS 902
- OPSS 3101.150.
- Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.
- OPSS 539
- OPSS 501
- OPSS 208.010
- OPSS 1010
- OPSS 206

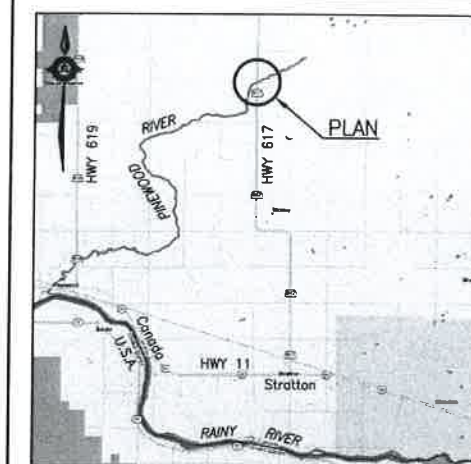
Appendix G

Borehole Locations and Soil Strata Drawings






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

CONT No
WP No 6094-10-00

HWY 617
PINWOOD RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA



KEYPLAN

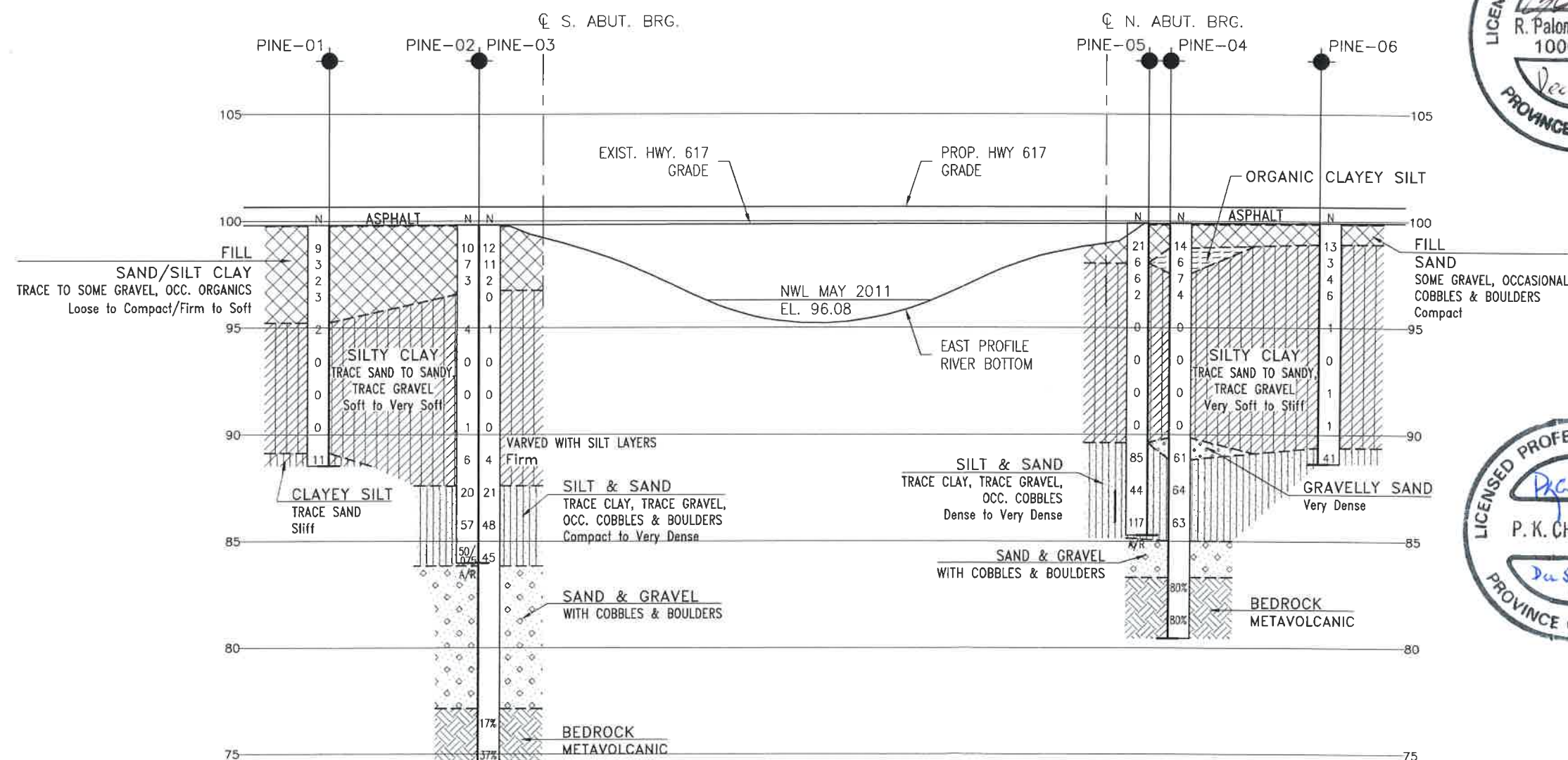
- | | |
|---|---------------------------------------|
|  | Borehole |
|  | Borehole and Cone |
| N | Blows /0.3m (Std Pen Test, 475J/blow) |
| CONE | Blows /0.3m (60° Cone, 475J/blow) |
| PH | Pressure, Hydraulic |
|  | Water Level |
|  | Head Artesian Water |
|  | Piezometer |
| 90% | Rock Quality Designation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

[illegible]

-NOTES-

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

GEOCRES No. 52D-13



PROFILE ALONG C HWY 617

[illegible]

