

G.I.-30 SEPT. 1976

GEOCRES No. 31L-72DIST. 54 REGION W.P. No. GWP 398-91-00CONT. No. W. O. No. STR. SITE No. HWY. No. 17LOCATION Retaining Wall, Station 10+350 to
10+600, Commanda TownshipNo of PAGES -=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

**PRELIMINARY
FOUNDATION INVESTIGATION REPORT FOR
PROPOSED RETAINING WALL, COMMANDA TWP.
HIGHWAY 17 FROM NORTH BAY TO STURGEON FALLS
W.P. 812-76-00 & 398-91-00,
DISTRICT 54, SUDBURY**

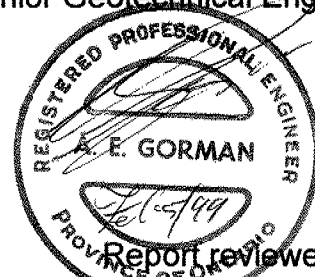
Report

to

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February 5, 1999
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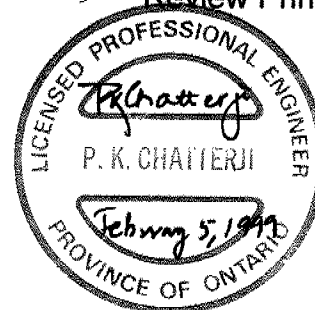


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DRAWINGS

19-2847-0B-01 Borehole Location Plan

APPENDICES

Appendix A Borehole Logs
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**PRELIMINARY
FOUNDATION INVESTIGATION FOR
PROPOSED RETAINING WALL, COMMANDA TWP.
HIGHWAY 17 FROM NORTH BAY TO STURGEON FALLS
W.P. 812-76-00 & 398-91-00,
DISTRICT 54, SUDBURY**

1. INTRODUCTION

This report presents the results of the foundation investigation carried out by Thurber Engineering Ltd. (Thurber) at the site of a proposed retaining wall on Highway 17. The proposed retaining wall is located between approximately Station 10+350 and 10+600 in Commanda Twp. The purpose of the investigation was to explore the subsurface soil and groundwater conditions at the site and based on the data obtained provide borehole logs, laboratory test data, soil profile and a written description of the subsurface conditions.

Thurber carried out the investigation as a sub-consultant to Stantec Consulting Ltd. (Stanley) under Ministry of Transportation (MTO) Agreement 9750 - 7411 - 5298.

2. SITE DESCRIPTION

2.1 Site Location

The site lies between approximately Station 10+350 and 10+600 in Commanda Twp. on Highway 17 approximately 11 km west of Gormanville Road, North Bay. It is located on the south side of the ROW where the ROW closely approaches the now abandoned CN ROW.

The existing wall retains the Highway 17 pavement structure and roadway fill approximately 5 m above the level of the ground immediately to the south. The north edge of the highway abuts a natural rock slope and rock cut face.

2.2 Physiography

Physiographically, the site lies within the Canadian Shield in an area characterized by rock outcrops and rock covered by shallow drift. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

Rock is exposed in the slope/cut face on the north side of the highway and also in a shallow railway cut immediately west of the site. To the east of the site, the flat topography, wet conditions and composition of the vegetation suggest an area of muskeg. The abandoned rail bed lies a few metres to the south of the existing retaining wall and the intervening ground surface is rough and largely covered by boulders and broken rock. Surface ponding of water was evident.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

On November 9 and 10, 1998, a Nodwell track mounted drill rig was used on site for drilling and Standard Penetration Testing (SPT, following the procedures of ASTM D 1586). Four boreholes were drilled and sampled to obtain data for foundation design for the proposed retaining wall. The approximate locations of the boreholes are shown on Drawing 19-2847-0B-01.

The holes were advanced using hollow stem augers and SPTs were carried out at selected intervals as the holes were advanced. The holes were advanced to the point of effective refusal to further auger penetration.

The boreholes were numbered 98-1 through 98-4. The depths of sampling in the four boreholes were as follows:

Borehole No.	Depth of Sampling (m)
98-1	4.7
98-2	3.2
98-3	4.7
98-4	4.7

Samples were recovered at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT). Samples were generally recovered at intervals of 0.75 m in the upper 3.0 m and thereafter at intervals of 1.5 m.

The results of the drilling, sampling and insitu testing are summarized on the borehole logs in Appendix A.

Standpipe piezometers were installed in Boreholes 98-3 and 98-4 to monitor the groundwater levels. All boreholes were backfilled with drill cuttings on the completion of drilling and sampling, with the exception of the borehole intervals where piezometers were installed and bentonite seals were also installed.

3.2 Laboratory Analysis

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In addition, grain size analyses were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figure B1 in Appendix B.

Two soil samples were selected and submitted for analysis for sulphate and pH testing. The results are shown in Table 1 in Appendix B.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the subsoil conditions encountered in the boreholes are presented in the borehole logs in Appendix A. The stratigraphic profile inferred from borehole information is shown on Drawing 19-2847-0B-01.

Based on visual identification and the results of laboratory testing, two native soil types were identified, together with fill. These are described in the following paragraphs.

Fill

Borehole 98-2 encountered 75 mm of topsoil overlying approximately 600 mm of fill consisting of crushed rock mixed with sand, trace gravel. The fill is described as compact, brown and moist.

Some 25 to 100 mm of topsoil was encountered in Boreholes 98-1, 98-3 and 98-4.

Sand

Below a thin cover of topsoil in Boreholes 98-1, 98-3 and 98-4, the soil consisted of sand, fine to medium grained, clean to silty, trace to some gravel. Based on SPT values ranging from 3 to 34 for 0.3 m penetration, the sand is described as very loose to compact, occasionally dense. This sand layer is 0.7 to 2.3 m thick and the elevation of the base of the layer ranges from 205.1 to 206.8.

The sand is brown to grey in colour and is generally described as moist with natural moisture contents ranging from 18 to 25%. A wet condition was encountered in Borehole 98-1 and this is attributed to the presence of ponded water at the surface.

Sand (Glacial Till)

Below the fill and upper layer of sand described above, the soil encountered is described as sand, silty, some gravel (glacial till). The gradation results varied from sand, some gravel, trace silt at one extreme to silt and sand at the other. These extreme gradations represents pockets of variation within the till deposit.

The glacial till deposit is described as very dense with SPT blows as high as 100 blows for 0.125 m penetration. The upper elevation of the very dense soil is interpreted to range from Elevation 205.1 to 206.8.

The soil is described as dry to moist, with measured natural moisture contents ranging from 5 to 20%. The colour of the soil is dark brown.

4.2 Groundwater

The following data was recorded in the boreholes drilled at the site:

Date	Groundwater Depth/Elevation (m)			
	BH 98-1	BH 98-2	BH 98-3	BH 98-4
Nov 10, 1998	2.4 (completion)	Dry on completion	Dry on completion	Dry on completion
Nov 10, 1998			Piezometer installed	Piezometer installed
Nov 10, 1998			3.7	1.0

The water levels reported for November 10, 1998, were measured only a few hours after installation of the piezometers. Within this short time, the levels are not expected to be stabilized and the stabilized levels may be higher.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

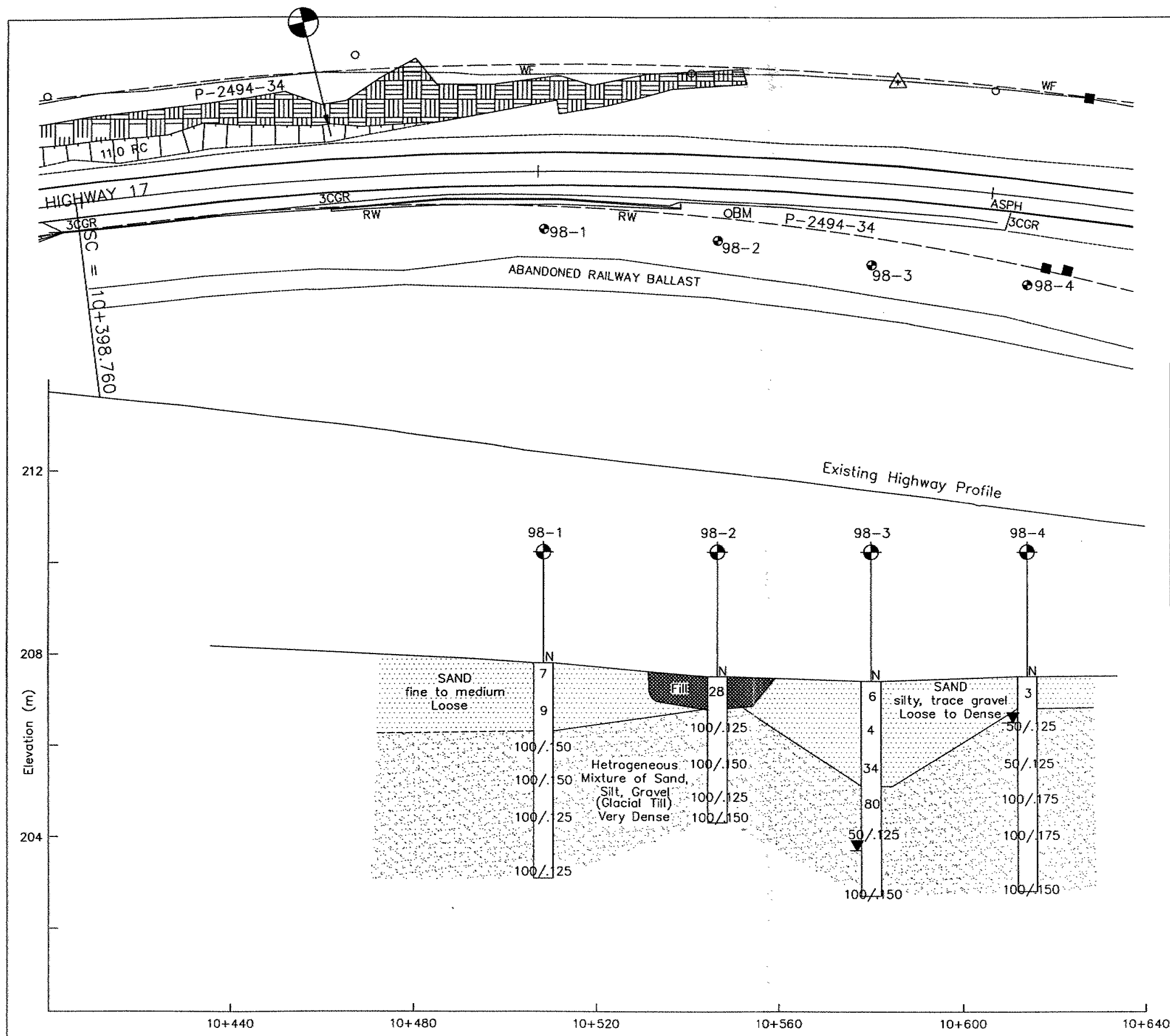
The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.


8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT


The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.





Stanley Consulting

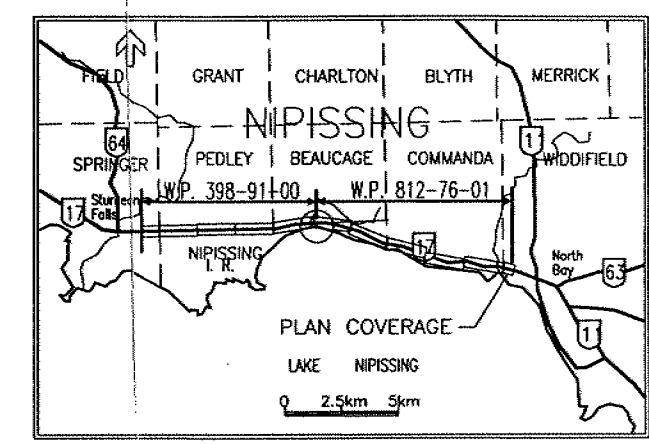
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THURBER ENGINEERING LTD.

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



KEY PLAN

LEGEND			
98-3		Borehole	
▼		WL November 10, 1998	
'N'	Blows/0.3m (Std. Pen Test)		
No	ELEV.	LOCATION	
		NORTHING	EASTING
98-1	207.8	5134867.669	295235.054
98-2	207.5	5134865.104	295273.054
98-3	207.4	5134859.699	295307.054
98-4	207.5	5134854.608	295341.054

APPENDIX A
BOREHOLE LOGS

- Borehole Logs 98-1 to 98-6

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE		
	Shelby Tube	A - Casing
	SPT	 Grab/Auger sample
	No Recovery	 Core

● MC - Moisture Content (% by Weight) as determined by sample]

W Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

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			

RECORD OF BOREHOLE No 98-1

1 OF 1

METRIC

W.P. 19-2847-08

LOCATION HWY 17, RETAINING WALL

ORIGINATED BY GA

DIST 54 HWY 17

BOREHOLE TYPE 210mm HOLLOW STEM AUGERS

COMPILED BY WM

DATUM Geodetic

DATE 98.11.10 - 98.11.10

CHECKED BY AEG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y 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					
207.8 0.0	<i>Ground surface</i> TOPSOIL-75mm over SAND, fine to medium grained, clean, loose, brown, wet (SW)		1	SS	7											
			2	SS	9											
206.3 1.5	SAND, silty, some gravel (Glacial Till), very dense, brown, dry		3	SS	100/ .150											
			4	SS	100/ .150											
			5	SS	100/ .125											
203.1 4.7	END OF BOREHOLE AT 4.7m. BOREHOLE OPEN TO 4.7m. ON COMPLETION. WATER LEVEL AT 2.44m ON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.		6	SS	100/ .125											

RECORD OF BOREHOLE No 98-2

1 OF 1

METRIC

W.P. 19-2847-08 LOCATION HWY 17, RETAINING WALL ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM Geodetic DATE 98.11.10 - 98.11.10 CHECKED BY AEG

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		
207.5														
0.0	TOPSOIL-75mm over		1	SS	28		207							
206.8	Fill-crushed rock, mixed with sand, trace gravel, compact, moist		2	SS	100/ .125		206							
0.7	SAND, silty, some gravel (Glacial Till), very dense, brown, dry		3	SS	100/ .150		205							10 58 24 8
			4	SS	100/ .125									
204.3			5	SS	100/ .150									
3.2	END OF BOREHOLE AT 3.2m. BOREHOLE DRY ON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.													

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 98-3

1 OF 1

METRIC

W.P. 19-2847-08 LOCATION HWY 17, RETAINING WALL ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM Geodetic DATE 98.11.10 - 98.11.10 CHECKED BY AEG

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		
207.4													
0.0	TOPSOIL-25mm over SAND, fine grained, silty, trace gravel, loose to dense, brown, moist		1	SS	6	207							2 75 22 1
			2	SS	4	206							
			3	SS	34	205							
205.1													
2.3	SAND, some gravel, trace silt (Glacial Till), very dense, brown, dry		4	SS	80	204							15 79 5 1
			5	SS	50/ .125	203							
202.7			6	SS	100/ .150								
4.7	END OF BOREHOLE AT 4.72m. BOREHOLE OPEN TO 4.72m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. Borehole dry on completion. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 10/11/98 3.71												

RECORD OF BOREHOLE No 98-4

1 OF 1

METRIC

W.P. 19-2847-08 LOCATION HWY 17, RETAINING WALL ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM Geodetic DATE 98.11.09 - 98.11.09 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
							20 40 60 80 100					
207.5												
0.0	TOPSOIL-100mm over		1	SS	3							
206.8	SAND, trace gravel, very dense, brown to grey, moist		2	SS	50/ .125							
0.7	SAND and SILT, trace gravel (Glacial Till), very dense, brown, dry		3	SS	50/ .125							
			4	SS	100/ .175							
			5	SS	100/ .175							
202.8			6	SS	100/ .150							
4.7	END OF BOREHOLE AT 4.72m. BOREHOLE OPEN TO 4.72m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. Borehole dry on completion. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 10/11/98 1.01											

APPENDIX B

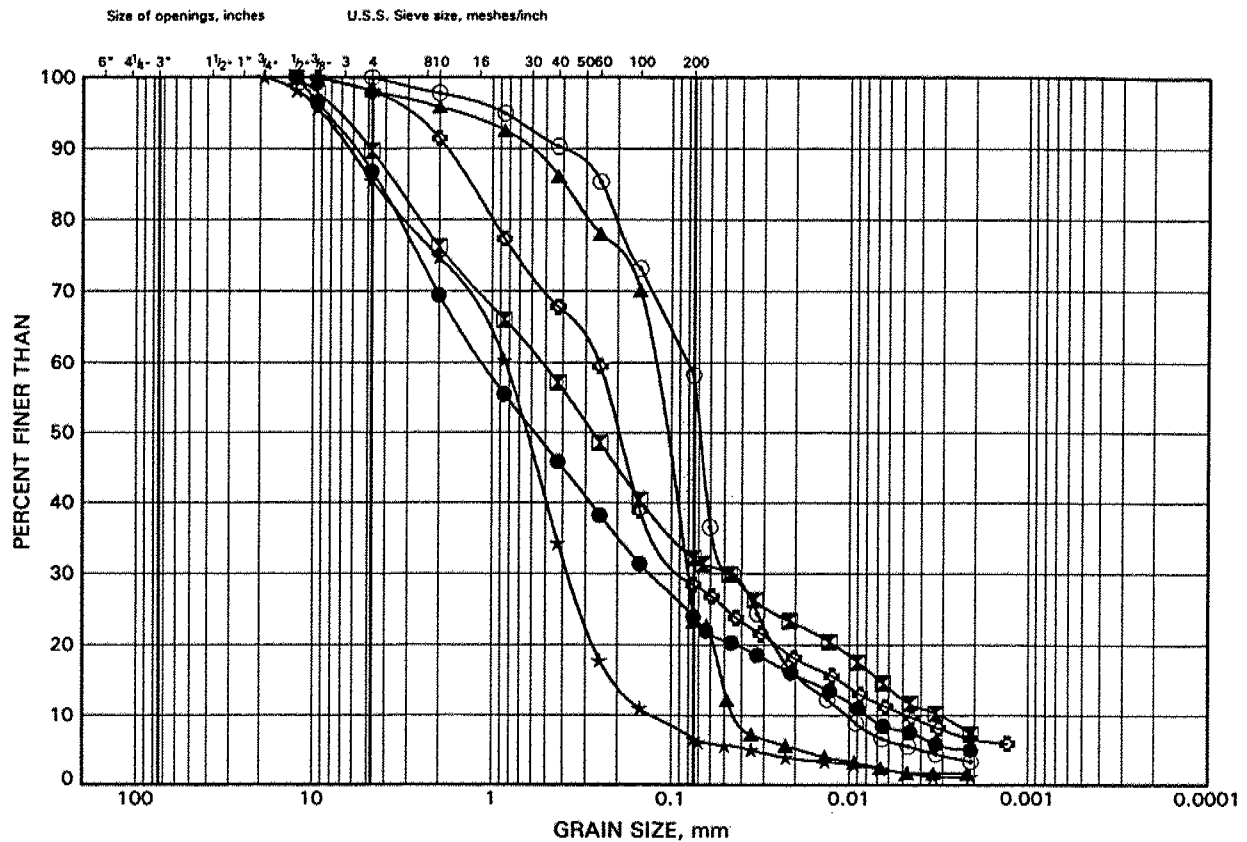
LABORATORY TEST RESULTS

- Figure B1 - Grain Size Analyses

- Table 1 - pH and Sulphate

HWY 17, RETAINING WALL GRAIN SIZE DISTRIBUTION

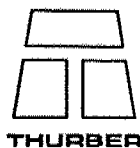
FIGURE B1



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-1	1.83	205.93
⊠	98-2	1.07	206.45
▲	98-3	0.30	207.08
★	98-3	2.59	204.79
⊙	98-4	1.83	205.65
⊛	98-4	4.52	202.96

Date February 1999
Project 19-2847-0B



Prep'd WM
Chkd. AEG

Table 1

Results of pH and Sulphate Testing

Sample	Depth (m)	pH	Sulphates (ppm)
98-1, Sa 2	1.0	8.5	4
98-4, Sa 3	1.8	8.5	131

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT FOR
PROPOSED BRIDGE OVER LARONDE CREEK
HIGHWAY 17 FROM NORTH BAY TO STURGEON FALLS
W.P. 812-76-00 & 398-91-00,
DISTRICT 54, SUDBURY

Report
to
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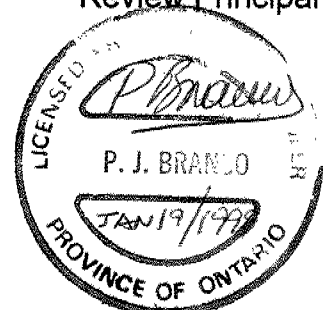


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1. INTRODUCTION

This report presents the results of the foundation investigation and design analysis carried out by Thurber Engineering Ltd. (Thurber) at the site of the proposed replacement bridge structure to carry Highway 17 across Laronde Creek in Beaucage Twp. The purpose of the investigation was to explore the subsurface soil and groundwater conditions at the site and based on the data obtained, provide borehole logs, laboratory test data, soil profile and a written description of the subsurface conditions. The purpose of the analysis of the data obtained during the investigation was to produce recommendations for the design and construction of the structure foundations and associated earth works.

Thurber carried out the investigation as a sub-consultant to Stantec Consulting Group Ltd. (Stanley) under Ministry of Transportation (MTO) Agreement 9750 - 7411 - 5298.

2. SITE DESCRIPTION

2.1 Site Location

The future Laronde Creek Bridge site is located at Station 13+600 in the Township of Beaucage, approximately 17 km west of Gormanville Road, North Bay. The future structure will be located partially within and outside the existing right-of-way limits. The highway runs approximately east-west and the creek flows from north to south towards Lake Nipissing.

At the site, the highway is carried across the creek on an existing bridge deck approximately 5 m above creek level. Based on drawings provided to Thurber, the existing structure consists of a concrete rigid frame founded on timber piles. The bridge has a span of 13.7 m and a width of 10.1 m.

At the site, the Highway is paralleled by railway tracks operated by Ottawa Valley Rail Link. The tracks lie approximately 26 m to the south of the centreline of the highway and are carried across Laronde Creek on a steel girder bridge at approximately the same elevation as the highway. At the creek, the highway and

railway embankments are separated by a ditch. On the east side of the creek, the ditch is deep and V-shaped. To the west, the ditch broadens out and is several metres wide at the base.

A short distance to the east of the site at Station 13+900, the highway is crossed by an abandoned CNR ROW.

The land in the northwest quadrant of the highway and the creek is occupied by a commercial operation known as NBissing Lodge. The lodge has a boat ramp approximately 20 m upstream from the north face of the existing bridge. The ground to the northeast is forested.

2.2 Physiography

Physiographically, the site lies within the Canadian Shield in an area where the bedrock is overlain by deep overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed. The area around the site is part of a former glacial lake basin and is underlain by lake sediments.

In the wider area, the bedrock is exposed as low, rounded hills with a sporadic covering of thin glacial drift. The site lies in an essentially flat plain which starts approximately 0.7 km to the east and extends many km to the west. It also stretches from the present shore of Lake Nipissing to several km north of the highway.

Visual inspection of the creek banks, the shore of lake Nipissing and local roads and construction sites indicated that the near surface soils are predominantly fine sand and silt. This was confirmed by examination of the logs of boreholes drilled a few km to the west at Little Sturgeon River and available in the Ministry's GEOCRESS library. These logs indicated that the near surface soils are underlain by a deposit of clay approximately 40 m thick. The clay is underlain by very dense sand and gravel (till).

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

Between October 15 and 26, 1998, a Nodwell track mounted drill rig was used on site for drilling, Standard Penetration Testing (SPT, following the procedures of ASTM D 1586), undisturbed tube sampling, in-situ vane shear tests and rock coring. Four deep boreholes were drilled and sampled to obtain data for

foundation design and two shallow holes were drilled for approach fill considerations. The approximate locations of the boreholes are shown on Drawing 19-2847-0A-01.

The holes were advanced using hollow stem augers and SPTs were carried out at selected intervals as the holes were advanced. At selected intervals, in-situ vane shear strength tests were conducted and at other selected intervals undisturbed tube samples of the cohesive soils were obtained. When refusal to further auger penetration was encountered, coring equipment was set up and each hole was advanced a minimum of a further 3.0 m by coring the bedrock with NQ series core equipment.

The boreholes were numbered 98-1 through 98-6. The depths of sampling in the four boreholes were as follows:

Borehole No.	Depth of Sampling (m)
98-1	18.3 OBD* 4.3 BDR**
98-2	20.9 OBD* 3.1 BDR**
98-3	21.0 OBD* 3.2 BDR**
98-4	26.9 OBD* 3.1 BDR**
98-5	5.2 OBD*
98-6	5.2 OBD*

* Overburden

** Bedrock

Samples were recovered at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT). Samples were generally recovered at intervals of 0.75 m in the upper 3.0 m and thereafter at intervals of 1.5 m to the top of bedrock.

The results of the drilling, sampling and rock coring are summarized on the borehole logs in Appendix A.

Standpipe piezometers were installed in Boreholes 98-1 and 98-4 to monitor the groundwater levels. Due to the presence of artesian groundwater encountered in Borehole 98-1, all boreholes were grouted on the completion of drilling and sampling, with the exception of the borehole interval where a piezometer was installed.

3.2 Laboratory Analysis

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In addition, grain size analyses and Atterberg limit tests were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figures B1 to B12 in Appendix B.

Two soil samples were selected and submitted for analysis for sulphate and pH testing. The results are shown in Table 1 in Appendix B.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the subsoil conditions encountered in the boreholes are presented in the borehole logs in Appendix A. The stratigraphic profile inferred from borehole information is shown on Drawing 19-2847-0A-01.

The findings from the borehole investigation show that the general stratigraphy at the subject site consists of low to intermediate plasticity clays and silts over granitic bedrock. Three of the four deep holes encountered a layer of sand, gravel, cobbles and boulders immediately overlying the bedrock. At the ground surface, the boreholes encountered silt and construction fill.

Fill

Borehole 98-4 was drilled from the shoulder level of the existing highway and encountered sand and silt fill comprising the approach embankment. The fill extended from the ground surface (Elevation 201.4) to a depth of 3.0 m (Elevation 198.4). Based on SPT values of 3 to 6, the fill is classified as being in a very loose to loose state of relative density. The measured in-situ moisture contents ranged from 5 to 15%.

Borehole 98-3 was drilled in the low area between the highway and railway embankments. This hole encountered silt and sand fill extending from the original ground surface (Elevation 197.5) to a depth of 2.3 m (Elevation 195.2). The fill is very loose to loose, based on the recorded SPT values. In-situ moisture contents of 15 and 16% were measured.

Borehole 98-5, drilled in the west approach area of the north alignment, encountered loose to compact sand and silt, possible fill from past construction activities. This fill is brown and moist. In the area south of the west approach for the existing alignment, Borehole 98-6 encountered 2.3 m of very loose to loose, sandy silt fill. The fill is moist to wet and brown to black in colour.

Silt

Silt was encountered in Boreholes 98-1 and 98-2 from the ground surface (Elevations 196.8 and 199.6, respectively,) to depths of 0.8 and 3.0 m, corresponding to Elevations 196.1 and 196.5, respectively. In borehole 98-6, the silt was encountered between depths of 2.3 and 4.0 m. The silt is low plastic to non-plastic and is in a very loose to loose state, based on recorded SPT values of 2 to 5. The natural moisture contents measured for Borehole 98-2 ranged from 22 to 31%. The one silt sample collected from Borehole 98-1 had a measured moisture content of 61%, possibly due to local clayey or organic inclusions. The silt was found to be brown and wet.

Clay and Silt

Below the fill and silt described above, all boreholes encountered the clay and silt which forms the predominant element of the subsoil stratigraphy at the subject site. The clay and silt ranges from low plasticity (CL) to intermediate plasticity (CI) and on the borehole logs the deposit has been subdivided between the two classifications where there appeared to be a distinct difference.

The clay and silt appears to be a lacustrine deposit, probably of late glacial age, and visual examination indicated the presence of thin seams and partings of silt and fine sand in some places. The clay and silt layer extends from the underside of the fill and silt as described above, down to elevations ranging from 178.5 to 181.0. The measured vane shear strengths are plotted against elevation in Figure B13 and show that the clay and silt is firm down to Elevation 188± and stiff from there to Elevation 181.5±. Below the latter elevation, the consistency is again firm. The sensitivity of the clay and silt is low, the measured values being 7 or less.

The clay and silt is grey and wet for its full depth. The measured natural moisture contents ranged from 23 to 61%. The plasticity indices measured on selected samples are plotted in Figures B7 to B12

Sand

Below the clay and silt, three of the four deep boreholes encountered a layer of sand with gravel, cobbles and boulders. Based on visual examination of the samples, the composition of this layer is highly variable and based on the SPT values recorded in classed as compact to dense, with some instances of very high blows attributed to the presence of cobbles and boulders. The sand deposit is grey and wet.

Borehole 98-1 did not encounter this layer of sand. However, visual examination of the last few metres of sampling indicated an increasing frequency and thickness of silt and sand seams.

Bedrock

Below the soil strata described above, all four deep boreholes encountered bedrock at elevations ranging from 174.5 to 178.7. The rock is hard, undifferentiated, granitic rock of the Canadian Shield.

4.2 Groundwater

The following data was recorded in the two piezometers installed at the site:

Date	Groundwater Depth/Elevation (m)	
	BH 98-1*	BH 98-4
21/10/98	+2.44 (199.2)	N/A
22/10/98	+2.44 (199.2)	N/A
23/10/98	+2.29 (199.1)	N/A
24/10/98	+2.29 (199.1)	4.1 (197.3)
25/10/98	+2.29 (199.1)	4.1 (197.3)
26/10/98	+2.29 (199.1)	4.3 (197.1)
27/10/98	+2.31 (199.1)	4.2 (197.2)

* Artesian groundwater head.

5. RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

It is understood that the preferred location for the permanent bridge lies immediately to the north of the existing structure. A preliminary general arrangement drawing supplied to Thurber in November 1998 illustrates this option and has been used as the basis of the Borehole Location Plan, Drawing 19-2847-0A-01.

As a secondary measure, if property cannot be obtained to accommodate a new highway alignment of the existing structure, the existing structure could be replaced at its existing location. This section of the report provides foundation design recommendations for the replacement of the Laronde Creek structure at either location (i.e. north of the existing structure or at its present location). Assuming that there will be no traffic restrictions applied to the detour bridge, the same foundation design recommendations apply to the detour bridge as to the permanent bridge.

5.1 Type of Structure

It is understood that the structure replacement of Laronde Creek Bridge will consist of a 30 m single span, slab-on-steel girder superstructure consisting of five 1,000 mm deep steel I-beams with integral abutments (i.e. jointless deck) with supports founded on steel H-piles driven to refusal on rock. Several different span lengths for the future structure, centred on Laronde Creek, were also considered in the foundation analysis.

It is further understood that the bridge may be designed as an integral abutment structure.

5.2 Recommended Foundation Type

The soil stratigraphy encountered at this site consists of a thick layer of soft to stiff clay and silt overlying a comparatively thin layer of sand and gravel with artesian water pressures over bedrock.

The support of the bridge structure on spread footing was considered. However, it is apparent that the characteristics of the clay soils encountered at this site would lead to very large footings and unacceptably large settlements. As a consequence, and in light of the possibility of integral abutment design, it is recommended that the structure be supported on steel H-piles driven to bedrock. HP 310X110 steel piles are recommended.

5.3 Axial Pile Resistance

Steel H-piles driven to bedrock may be considered to be end bearing and the bedrock present at this site may be taken as unyielding. Accordingly, the capacity of the pile may be taken to be equal to the factored structural capacity of the pile at ULS. It is understood that the MTO Structural Manual limits this capacity to 1,600 kN.

For SLS verification, the deformation of the pile head may be taken as equal to the elastic deformation of the pile shaft. The SLS condition may be assumed not to govern. ✓

5.4 Downdrag

Due to the compressive nature of the clay and silt layer, construction of an approach fill on the native soil will cause long term settlements which will create downdrag forces in the piles.

For piles driven to bearing on unyielding bedrock, the neutral plane will lie close to the bottom of the clay and silt layer. Thus, it is conservative to assume that downdrag forces will develop over the full length of embedment in the clay and silt. From the projected top of asphalt at Elevation 203±, the underside of the abutment stem is assumed to be at Elevation 197.0 and the maximum length of pile over which negative skin friction may develop is 19.5 m. Based on these figures, the maximum unfactored downdrag force on a single HP310X110 pile should be taken as 1,000 kN. Verification of the pile structural resistance at the neutral plane should be carried out as recommended by the OHBDC Clause 6 - 9.11 (and as illustrated in the Commentary to the OHBDC, Table C6 - 9.11.1) using the design dead loads and the 1,000 kN downdrag force.

It should be noted that the downdrag force is to be used only for verification of the structural capacity as described above. It does not impact on the available geotechnical resistance of the pile.

What's your recommended pile resistance?

5.5 Lateral Resistance

The lateral resistance of the piles may be analyzed using the following coefficient of horizontal subgrade reaction k_s and ultimate pressure at the interface of the soil and the pile:

Depth Below Surface (m)	Value of k_s (MN/m ³)	Ultimate Soil/Pile Contact Pressure (kPa)
Above Elevation 188.0	6.5	270
below 188.0	13.0	540

5.6 Pile Driving

Driving of the piles through the soft to firm to stiff cohesive soils is expected to meet little resistance. In fact, the contractor may experience difficulty in keeping a conventional diesel pile driver firing if he elects to drive the upper lengths of the piles in that manner. The piles may, however, encounter increased resistance in a layer of sand, gravel, cobbles and boulders overlying the bedrock as evident in Boreholes 98-2, 98-3 and 98-4. The presence of random boulders in the overlying cohesive soils also cannot be completely ruled out.

Selection of the equipment used for the final driving of the piles must be based on penetrating a very dense layer of sand, gravel and boulders and seating the piles in the bedrock. For this reason, equipment capable of delivering driving energy approaching 70 kJ is probably appropriate.

The range of elevations to which pile tips may be expected to be driven is as follows:

Foundation Element	Range of Tip Elevations
New alignment, west abutment	179 to 177
New alignment, east abutment	179 to 176
Present alignment, west abutment	177 to 176
Present alignment, east abutment	176 to 174

Due to the presence of boulders in a very dense granular matrix and the requirement to seat into hard igneous or metamorphic bedrock, it is recommended that the piles be equipped with rock points.

If the final design results in piles being driven at or close to the location of the existing bridge foundations or the foundations of any older structure, obstructions in the form of remnants of the previous construction may be encountered. Possible obstructions include, but are not limited to, the timber piles believed to support the existing structure.

5.7 Pile Driving Note

The pile driving note to be added to the drawings is Note 4 in Clause 2.5.11 of the Structural Manual - "Piles to be driven to bedrock."

6. EARTH PRESSURES ON ABUTMENT WALLS

The lateral earth pressures to be used in design should be computed in accordance with Section 6-7 of the OHBDC .

Granular backfill should be placed behind the abutment walls and wing walls to conform to the minimum requirements illustrated in OPSD 3501.00. The granular backfill should conform to Ontario Provincial Standard Specifications (OPSS) 1010 for Granular B, Type 1. The fill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3501.00 to maintain the granular fill in a drained condition. The subdrain should be provided with a positive outlet

to the highway drainage system.

For the above backfill and drainage conditions, the abutment walls and wing walls may be designed based on the following unfactored earth pressure distributions:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table below.

γ = unit weight of soil, = 21.2 kN/m³ for Granular B

h = depth below top of wall, m

Case	Earth Pressure Coefficient (K)	
	OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Passive	3.0	-
At Rest	0.50	-
Active (Unrestrained Wing Wall)	0.33	0.55

If an integral abutment design is used, the abutments will be cast integrally with the deck and as a result, the abutment walls will thrust against the backfill under certain circumstances. Accordingly, the abutment walls must be designed to resist an earth pressure between the at rest and passive conditions, as described in the OHBDC. As a conservative assumption, the walls should be designed to resist the passive earth pressure. If the wing walls will not be connected to the abutments and therefore will be able to accommodate some rotation they may be treated as unrestrained and be designed for the active earth pressure case. The above also assumes a horizontal ground surface behind the abutment walls. If concrete approach slabs are not provided, an additional load equivalent to 600 mm of fill should be superimposed on the wall loadings to account for traffic surcharge loading.

Additional lateral pressure must be added to account for compaction induced forces. The additional pressure must be computed in accordance with Section 6-7.4.3 of the OHBDC.

7. APPROACH EMBANKMENT DESIGN

Depending on the final design, there may be two different scenarios to be considered for embankment construction:

- a new permanent alignment to the north of the existing highway
- a temporary detour to the north of the existing highway with reinstatement of the existing alignment after construction.

If a new permanent alignment to the north of the existing highway is chosen, new approach embankments will be constructed and at the completion of construction the existing embankments are expected to be abandoned. If the new bridge is constructed on the existing alignment, new approach embankments will be required to the north for a temporary detour and the existing approach embankments will be widened and raised.

7.1 Settlements

Construction of a new embankment, or enlargement of the existing embankment, on top of the soils encountered at this site will result in settlement of the embankment as the underlying clay consolidates under the new loading. Any such consolidation and settlement would be expected to create a number of negative effects, including:

- settlement of the approach pavement surface relative to the structure
- development of negative skin friction, or downdrag, in the piles supporting the structure (the structural implications of downdrag are dealt with in Section 5.4 of this report)
- development of lateral forces on the piles due to lateral movements as the clay consolidates.

7.1.1 New Embankment

Based on the results of the laboratory consolidation tests conducted on two selected samples, the calculated settlement of the existing soil due to the loading from a 6 m high new embankment is in the order 1,000 mm at the abutment.

To reduce future settlement of the pavement surface relative to the structure, it is recommended that the embankments be constructed under an advance contract to allow the majority of the expected consolidation and settlement of the embankment to occur prior to pile driving and bridge construction. Surcharging of the embankment is recommended to accelerate the settlement and advance consolidation of the underlying soil beyond the primary consolidation stage prior to

construction.

It is recommended that the embankment initially be constructed to an elevation 2.5 m above the final profile grade. That total height of fill is expected to induce settlement in the order of 1,300 mm. When construction of the bridge starts, the over-built embankment should be stripped down to the required subgrade elevation. Removal of the surcharge will unload the underlying soil and reduce the magnitude of long term secondary consolidation effects.

Based solely on the laboratory consolidation tests, it is calculated that more than 3 years would be required to complete the primary consolidation of the clay deposit. However, visual examination of undisturbed soil samples indicated that there are numerous varves or minor layers of silt and fine sand throughout the deposit. These layers provide drainage and greatly accelerate the consolidation process. Based on the presence of the silt and fine sand layers and from a literature review that indicated that settlement due to consolidation typically occurs significantly faster in the field than predicted using laboratory test results, it is estimated that primary consolidation will be complete in a period of less than 1 year.

It is recommended that the settlement of the fill be monitored on a monthly basis and the results plotted to assess the progress of consolidation. Monitoring should consist of accurate level survey of settlement points installed at the embankment centreline near the top of the embankment but anchored below frost depth and protected from the effects of frost heave.

Construction of the bridge should not commence until the consolidation of the clay has reached at least 97% of the calculated total settlement.

7.1.2 Existing Embankment

It is understood that if the permanent structure is constructed on the existing alignment, the grade will be raised by approximately 1.5 m. Assuming that conventional fill is placed, the increased loading due to raising the grade will result in further consolidation of the underlying soil. Based on the values obtained in the laboratory, the addition of 1.5 m of fill will induce surface settlement in the order of 250 mm. It is possible that actual settlement will be less than this figure due to the consolidation that has already occurred under the weight of the existing embankment.

With respect to the timing, it is recommended that the embankment grade be raised as soon as possible and that it be over-built by 0.5 m in order to accelerate

consolidation of the underlying clay. If this is done, it is estimated that by the end of a 6 month period consolidation will be sufficiently advanced to allow removal of the surcharge and completion of the pavement structure. This period of pre-loading and consolidation can coincide with the construction of the bridge.

If it will be necessary to re-open the highway in less than 6 months, provision should be made to accommodate settlement due to the remaining primary consolidation of the clay. It is estimated, for example, that at the end of a 3 month period under the 0.5 m pre-load, there will be approximately 40 mm of primary consolidation left to occur.

If the estimated 40 mm of settlement cannot be tolerated, consideration could be given to alternatives such as the use of lightweight fill to allow the grade to be raised without increasing the stress on the underlying compressible soil. To use lightweight fill effectively, the existing embankment should be removed to some depth below existing pavement and then rebuilt using the lightweight fill. The depth 'd' (m) which should be removed depends on the unit weight of the lightweight fill proposed for use. The value of 'd' required to compensate for raising the grade 1.5 m can be obtained from the equation:

$$d = (1.5\gamma_F + 11)/(20 - \gamma_F)$$

where:

d = depth of removal (m)

γ_F = the unit weight of the lightweight fill (kN/m³)

20 kN/m³ = assumed unit weight of existing fill in place

the term 11 kPa is assumed to account for the new pavement structure.

7.2 Stability

how about construct stability?

The foundation stability of an embankment constructed with side slopes inclined at 2H:1V has been analyzed. The results of the analysis show a global factor of safety against failure of 1.3 at completion of construction. This value is considered acceptable for earthworks. The underlying clay will gain strength as it consolidates, leading to an increased factor of safety. It is also noted that the existing highway embankment and the adjacent railway embankment appear to be performing satisfactorily.

8. EXCAVATION AND GROUNDWATER CONTROL

Excavation of the native cohesive soils and the fill encountered in the boreholes will be accomplished with conventional excavating equipment. Bidders should be warned that there is no information regarding the construction and backfilling of the existing abutments or what conditions may be encountered immediately behind the abutment walls. Similarly, bidders should be warned of the possibility of encountering remnants of some previous structure on the site.

It is anticipated that excavation for construction of the new abutment stems will not penetrate below the level of the water in the creek. In that situation, it is not anticipated that prior dewatering of the site will be a necessity. However, it is anticipated that there will be continuing seepage from the silt seams and fill encountered at or near the present ground surface and some sloughing of the sides of the excavation may occur.

The artesian groundwater head measured at Borehole 98-1 originated in the granular deposits close to the bedrock surface and are not expected to have any influence on the shallow excavations required for the abutment stems.

All excavations must be carried out in accordance with the Occupational Health and Safety Act of Ontario (OHSA). For the purposes of OHSA, the soils encountered on this site are classified Type 4 on the basis of the soft and very loose conditions encountered and the possibility of running soils. Type 4 soils require excavations to be supported or to be sloped at 3H:1V from the base of the excavation.

Excavations to the north of the existing highway to construct new abutments will probably encroach into the existing approach fills. In this case, temporary shoring will be required to support the embankment. For design purposes, the following assumptions are recommended:

- the retained soils are loose, fine grained, non-cohesive soils
- the soils providing toe resistance are soft cohesive soils.

Assuming support will be provided by means of a temporary, cantilevered retaining wall, the active earth pressure may be treated as having a triangular distribution and may be computed on the basis of a coefficient of active earth pressure of $K_a = 0.4$. Toe resistance should be computed on the basis of an undrained shear strength of 30 kPa. Design of the shoring should be carried out by a professional engineer experienced in such designs.

9. FROST PROTECTION

The design depth of frost penetration for this project is 1.8 m. All pile caps and footings designed for this site must be provided with at least 1.8 m of soil cover as protection against frost penetration.

10. CONSTRUCTION CONCERNS

The principal construction concerns at this site relate to the compressibility of the clay and silt and to the presence of boulders in the few metres above bedrock. The main points to be monitored are:

- construction of the approach embankments
- monitoring of embankment settlements to determine when settlement is essentially complete
- monitoring of pile driving to ensure piles are reaching the anticipated tip elevation to bear on rock and are not being impeded by boulders.

11. CONSTRUCTION INSPECTION AND MONITORING

During construction, all foundation installation, excavation and approach embankment construction activities should be monitored by geotechnical personnel to ensure that the foundation recommendations and design are being correctly implemented and that soil conditions do not differ materially from the interpretation used in this report.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS


The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.




DIST 54

CONT No.

WP No. 812-76-00 & 398-91-00

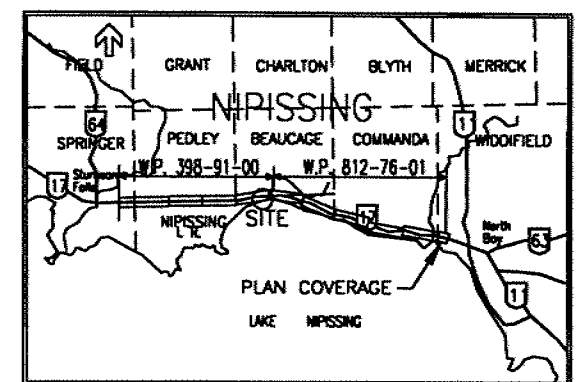
LARONDE CREEK BRIDGE HIGHWAY 17




SHEET

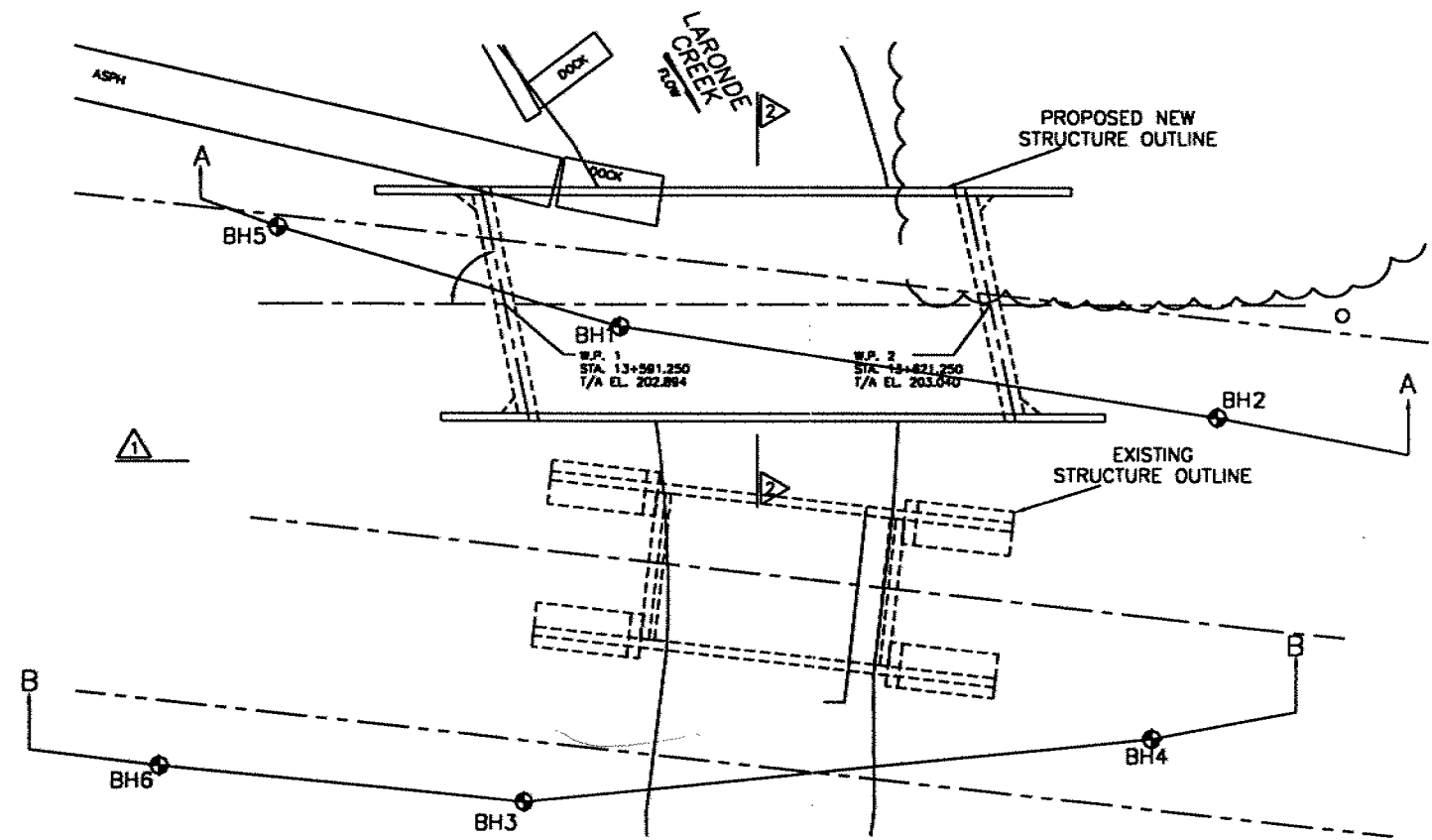
THURBER ENGINEERING LTD.

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




KEY PLAN

LEGEND			
<div> <div>BH1</div> <div>  <div>Borehole</div> </div> </div>			
No	ELEV.	LOCATION	
		NORTHING	EASTING
98-1	196.8	5136763.609	288575.314
98-2	199.6	5136763.377	288612.703
98-3	197.5	5136733.585	288573.659
98-4	201.4	5136743.057	288611.487
98-5	197.3	5136767.711	288557.288
98-6	198.4	5136733.414	288556.528

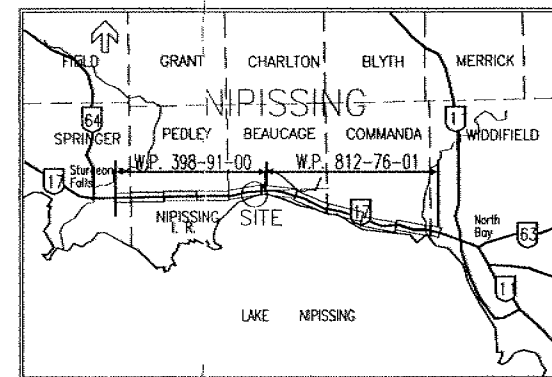


BOREHOLE LOCATION PLAN

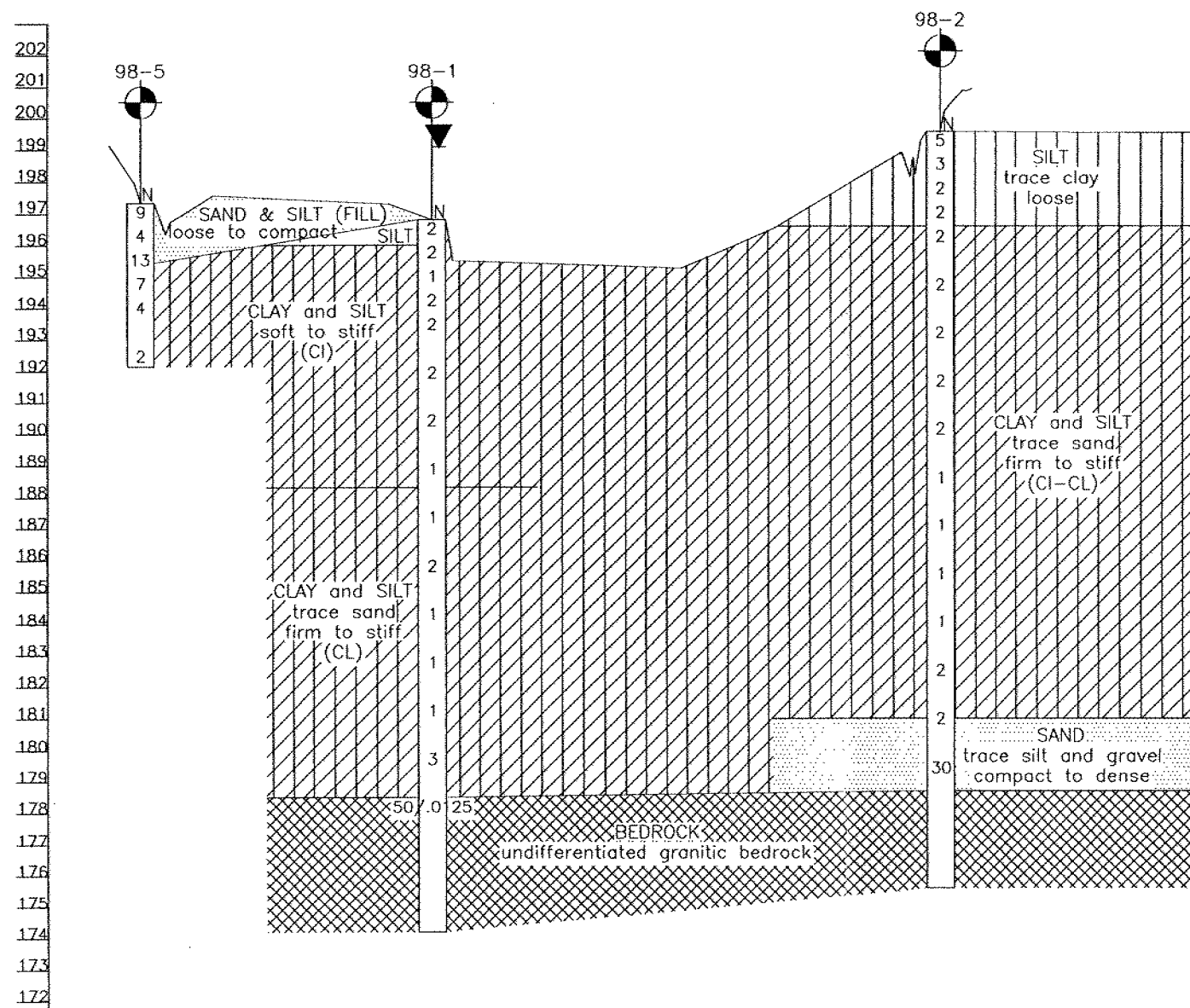
TABLET IN FACE OF W. ABUTMENT
 OF CPR RAILWAY BRIDGE 24.6 RT.
 AT STA. 13+598.2
 (EXISTING HIGHWAY 17 L ALIGNMENT)

	DIST 54	SHEET
	CONT No.	
	WP No. 812-76-00 & 398-91-00	
LARONDE CREEK BRIDGE HIGHWAY 17		
THURBER ENGINEERING LTD.		



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



KEY PLAN



SOIL PROFILE A-A

LEGEND			
98-1		Borehole	
		WL October 27, 1998	
	'N'	Blows/0.3m (Std. Pen Test)	
No	ELEV.	LOCATION	
		NORTHING	EASTING
98-1	196.8	5136763.609	288575.314
98-2	199.6	5136763.377	288612.703
98-3	197.5	5136733.585	288573.659
98-4	201.4	5136743.057	288611.487
98-5	197.3	5136767.711	288557.288
98-6	198.4	5136733.414	288556.528

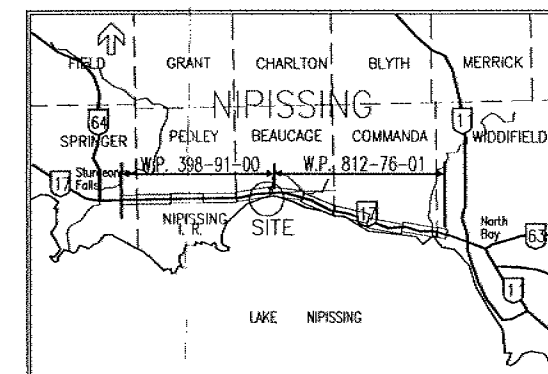
19-2847-0-2

TABLET IN FACE OF W. ABUTMENT
OF CPR RAILWAY BRIDGE 24.6 RT.
AT STA. 13+596.2
(EXISTING HIGHWAY 17 L ALIGNMENT)



DIST 54
CONT No.
WP No. 812-76-00 &
398-91-00
ARONDE CREEK BRIDGE
HIGHWAY 17
THURBER ENGINEERING LTD.

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



KEY PLAN

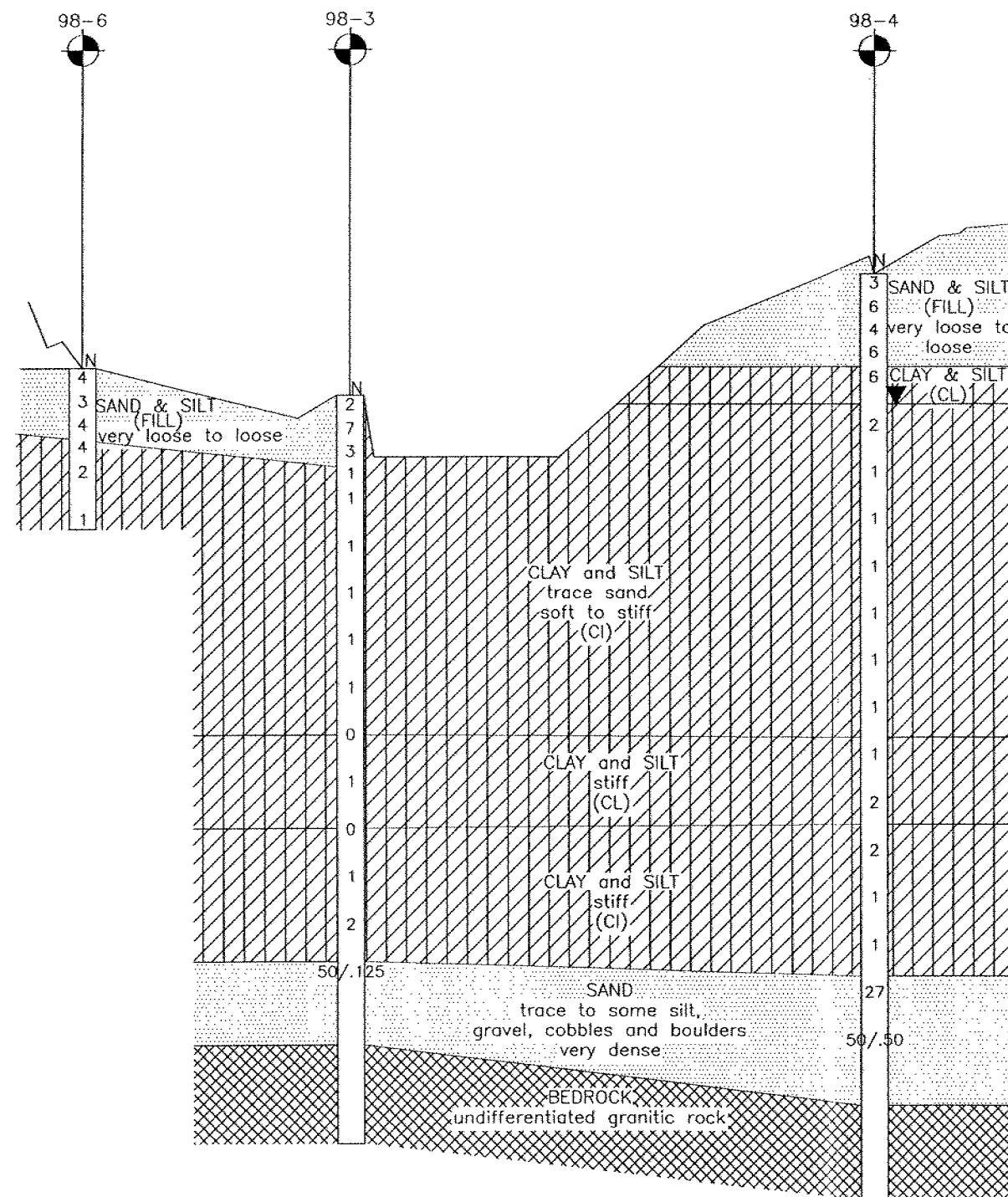
LEGEND

- 98-3 Borehole
WL October 27, 1998
'N' Blows/0.3m (Std. Pen Test)

No	ELEV.	LOCATION	
		NORTHING	EASTING
98-1	196.8	5136763.609	288575.314
98-2	199.6	5136763.377	288612.703
98-3	197.5	5136733.585	288573.659
98-4	201.4	5136743.057	288611.487
98-5	197.3	5136767.711	288557.288
98-6	198.4	5136733.414	288556.528

ELEVATION (M)

202
201
200
199
198
197
196
195
194
193
192
191
190
189
188
187
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185
184
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182
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173
172
171
170



SOIL PROFILE B-B

PLOT-DATE

TABLET IN FACE OF W. ABUTMENT
OF CPR RAILWAY BRIDGE 24.6 RT.
AT STA. 13+598.2
(EXISTING HIGHWAY 17 L. ALIGNMENT)

19-2847-0-3

APPENDIX A
BOREHOLE LOGS

- Borehole Logs 98-1 to 98-6

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE		
	Shelby Tube	A - Casing
	SPT	 Grab/Auger sample
	No Recovery	 Core

● MC - Moisture Content (% by Weight) as determined by sample]

⏏ Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

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			

RECORD OF BOREHOLE No 98-1

1 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 763.6 E 288 575.3 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
 DATUM Geodetic DATE 98.10.15 - 98.10.15 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
196.8																		
0.0	SILT, sandy, very loose, grey, wet		1	SS	2								60.62					
196.1																		
0.8	Silty CLAY, medium plasticity, soft to firm, grey, wet (CI)		2	SS	2		196						○					
			3	SS	1		195						41.7	0 0 41 59				
			4	SS	2		194						45.0	0 0 42 58				
			5	SS	2		193						○					
							192						41.69					
			6	SS	2		191											
	some varves evident		7	SS	2		190						48.48					
							189			+			46.45	0 0 43 57				
			8	SS	1		188											
188.3																		
8.5	Silty CLAY, trace sand, low plasticity, firm to stiff, grey, wet (CL)						187						○					
			10	SS	2		186		+				○	0 1 61 38				
							185											
	some varves evident		11	SS	1		184						○					
							183						50.99					
			12	SS	1		182											

Continued Next Page

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

RECORD OF BOREHOLE No 98-1

2 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 763.6 E 288 575.3 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
DATUM Geodetic DATE 98.10.15 - 98.10.15 CHECKED BY AEG

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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
178.5			13	SS	1		181	+					52.830	
			14	SS	3		180						42.840	0 1 63 36
18.3	BEDROCK undifferentiated granitic rock, very dense		15	SS	50/ .0		178							
174.2							177							
22.6	END OF BOREHOLE AT 22.6m. Piezometer installation consist of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 21/10/98 2.44(above surface) 22/10/98 2.44(above surface) 23/10/98 2.29(above surface) 24/10/98 2.29(above surface) 25/10/98 2.29(above surface) 26/10/98 2.29(above surface) 27/10/98 2.31(above surface)						176							
							175							

RECORD OF BOREHOLE No 98-2

1 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 763.4 E 288 612.7 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
 DATUM Geodetic DATE 98.10.20 - 98.10.20 CHECKED BY AEG

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						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT							
							w _p w w _L							
							10 20 30							
199.6														
0.0	SILT, trace clay, loose, grey to brown, wet (ML)		1	SS	5		199							
			2	SS	3									
			3	SS	2									
			4	SS	2									
196.5														
3.0	CLAY and SILT, trace sand, medium plasticity, firm to stiff, grey, wet (CI)		5	SS	2		196							
	some varves evident		9	SS	2		190							
	(possibly CL between 10 & 12m)		10	SS	1		189							

Continued Next Page

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

METRIC

ORIGINATED BY GA

COMPILED BY WM

CHECKED BY AEG

+ 3, x 3: Numbers refer to Sensitivity

1 OF 2

METRIC

LOCATION

Laronde Creek, N 5 136 733.6 E 288 573.7

ORIGINATED BY GA

DIST

HWY 17

BOREHOLE TYPE

Hollow Stem Augers, N Core

COMPILED BY WM

DATUM Geodetic

DATE _____

98.10.24 - 98.10.24

CHECKED BY AEG

[illegible]

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 98-3

2 OF 2

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 733.6 E 288 573.7 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
DATUM Geodetic DATE 98.10.24 - 98.10.24 CHECKED BY AEG

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							WATER CONTENT (%)			
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
							20 40 60 80 100					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L						
179.2			13	SS	1		182											
							181			+								
			14	SS	2		180											
18.3	SAND, some silt, some cobbles and boulders, very dense, grey, wet		15	SS	50/.125		179											
							178											
							177											
176.5																		
21.0	BEDROCK, undifferentiated granitic rock, very dense						176											
							175											
							174											
173.3																		
24.2	END OF BOREHOLE AT 24.2m. BOREHOLE BACKFILLED AS FOLLOWS: 0-18m GROUT 18-21m BENTONITE & SAND 21-24.2m BENTONITE																	

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

RECORD OF BOREHOLE No 98-4

2 OF 3

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 743.1 E 288 611.5 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
 DATUM Geodetic DATE 98.10.22 - 98.10.22 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100					
15.0	CLAY and SILT, low plasticity, stiff, grey, wet (CL)		13	SS	1		186								0 0 63 37
			14	SS	2		185								
183.6							184								
17.8	CLAY and SILT, medium plasticity, stiff, grey, wet (CI)		15	SS	2		183								43.230
			16	SS	1		182								
	some varves evident		17	SS	1		181								
							180								
178.7							179								
22.7	SAND, trace gravel, cobbles and boulders, compact to very dense, grey, wet		18	SS	27		178								46.590
			19	SS	50/ .050		177								
							176								
							175								
174.5							174								
26.9	BEDROCK undifferentiated granitic rock						173								
							172								
171.5															

Continued Next Page

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

RECORD OF BOREHOLE No 98-4

3 OF 3

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 743.1 E 288 611.5 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Core COMPILED BY WM
DATUM Geodetic DATE 98.10.22 - 98.10.22 CHECKED BY AEG



SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT			UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					PLASTIC LIMIT W _p	W	LIQUID LIMIT W _L		
								20	40	60	80	100					
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
								20	40	60	80	100					
													10	20	30		
30.0	END OF BOREHOLE AT 29.97m. Piezometer installation consist of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 24/10/98 4.1 25/10/98 4.1 26/10/98 4.3 27/10/98 4.2																

RECORD OF BOREHOLE No 98-5

1 OF 1

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 767.7 E 288 557.3 ORIGINATED BY GA
DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Coring COMPILED BY WM
DATUM Geodetic DATE 98.10.19 - 98.10.19 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20	40	60	80	100						
0.0	SAND and SILT, loose to compact, brown, moist: (FILL)		1	SS	9													
			2	SS	4													
			3	SS	13													
1.9	CLAY, trace silt, firm to stiff, grey, wet (CL)		4	SS	7													
			5	SS	4													
			6	SS	2													
5.2	END OF BOREHOLE AT 5.18m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.																	

RECORD OF BOREHOLE No 98-6

1 OF 1

METRIC

W.P. 812-76-01,398-91-00 LOCATION Laronde Creek, N 5 136 733.4 E 288 556.5 ORIGINATED BY GA
 DIST 54 HWY 17 BOREHOLE TYPE Hollow Stem Augers, N Coring COMPILED BY WM
 DATUM Geodetic DATE 98.10.26 - 98.10.26 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
0.0	SILT, sandy, trace gravel, very loose to loose, brown to black, moist to wet: (POSSIBLE FILL)		1	SS	4												0 3 83 14
			2	SS	3												
			3	SS	4												
2.3	SILT, clayey, soft, grey, moist to wet: (ML)		4	SS	4												0 0 50 50
			5	SS	2												
4.0	CLAY and SILT, low plasticity, grey, wet: (CL)		6	SS	1												0 0 50 50
5.2	END OF BOREHOLE AT 5.18m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.																

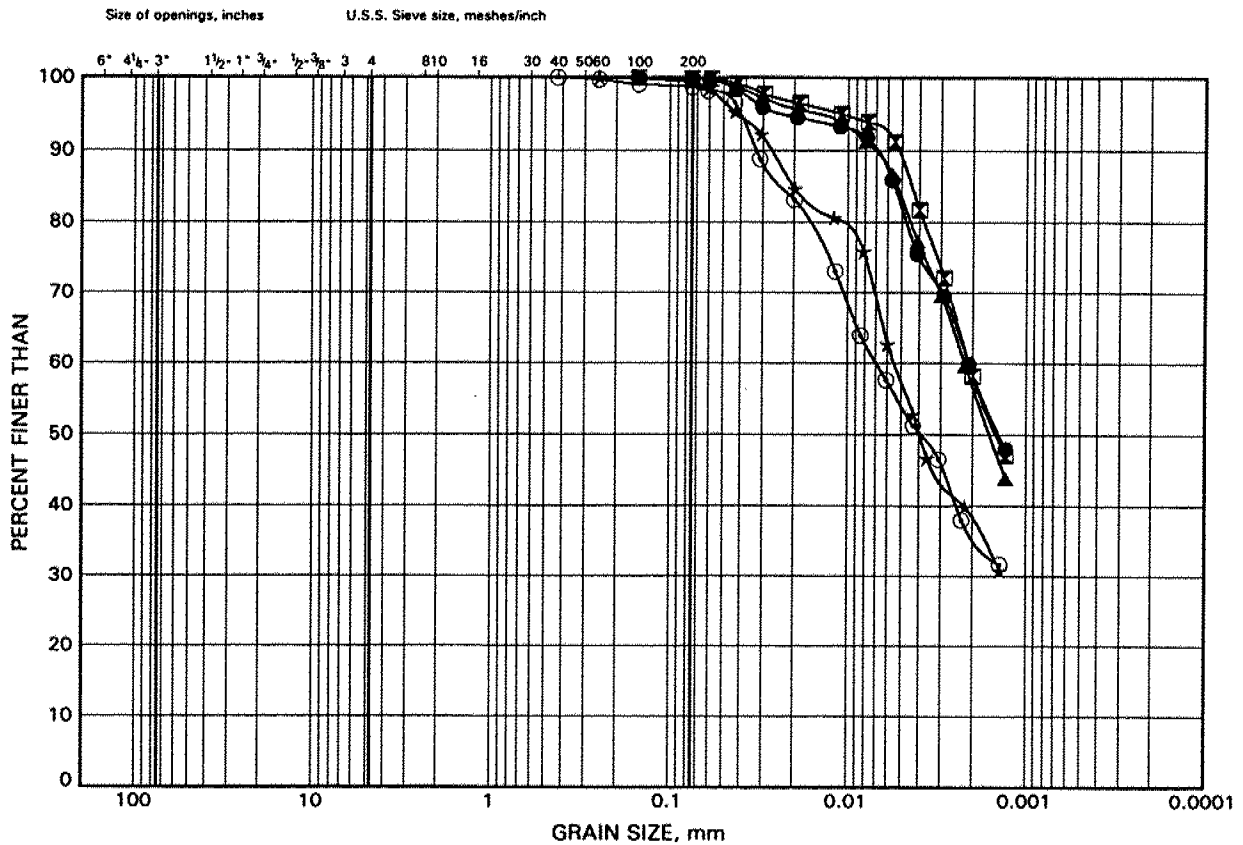
APPENDIX B

LABORATORY TEST RESULTS

- Figures B1 to B6 - Grain Size analyses
- Figures B7 to B12 - Plasticity Charts
- Figure B13 - Vane Shear Strengths
- Table 1 - pH and Sulphate

HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B1



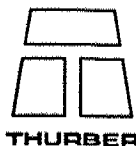
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	98-1	1.83	194.98
⊠	98-1	2.59	194.22
▲	98-1	7.92	188.89
★	98-1	10.97	185.84
⊙	98-1	17.07	179.74

Date December 1998

Project 812-76-01,398-91-00

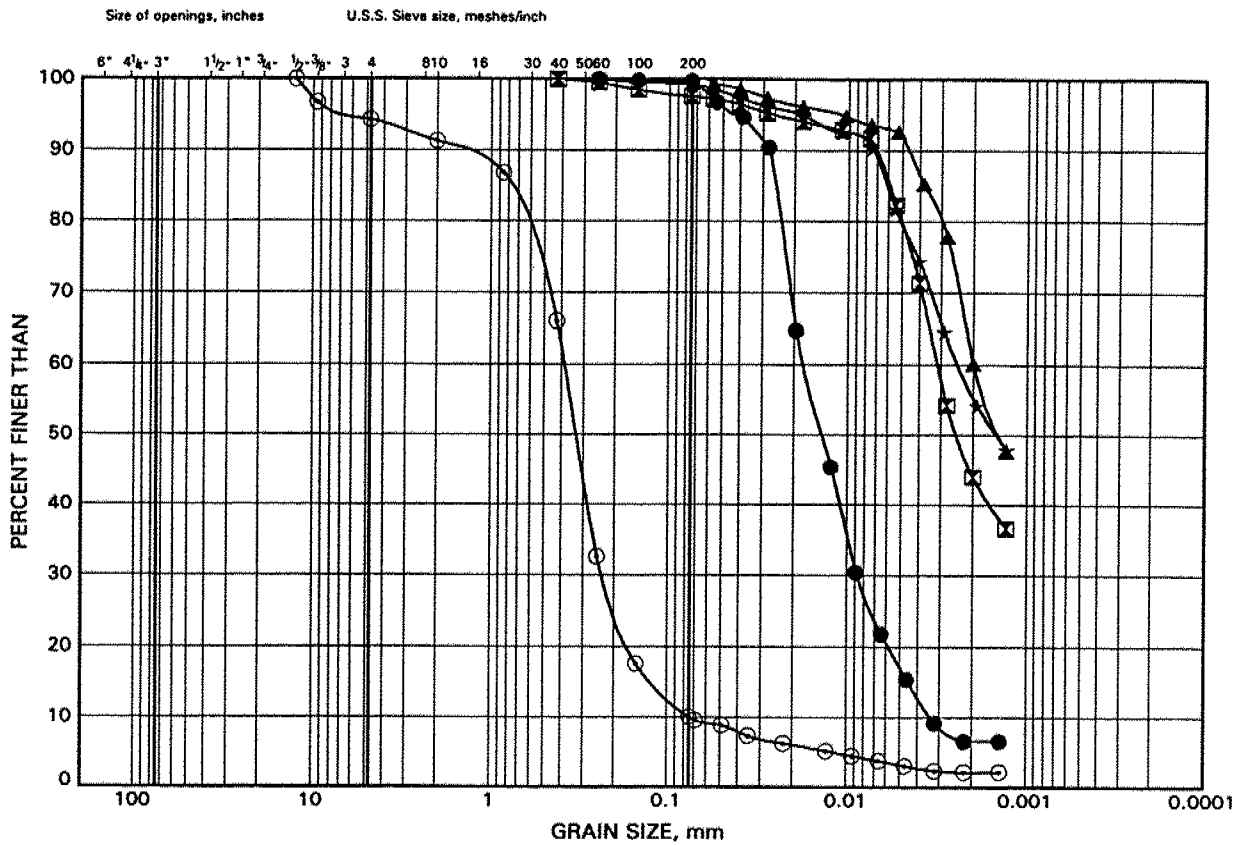


Prep'd WM

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HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B2



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-2	1.07	198.48
⊠	98-2	3.35	196.20
▲	98-2	9.45	190.10
★	98-2	15.54	184.01
⊙	98-2	20.12	179.43

Date December 1998

Project 812-76-01,398-91-00

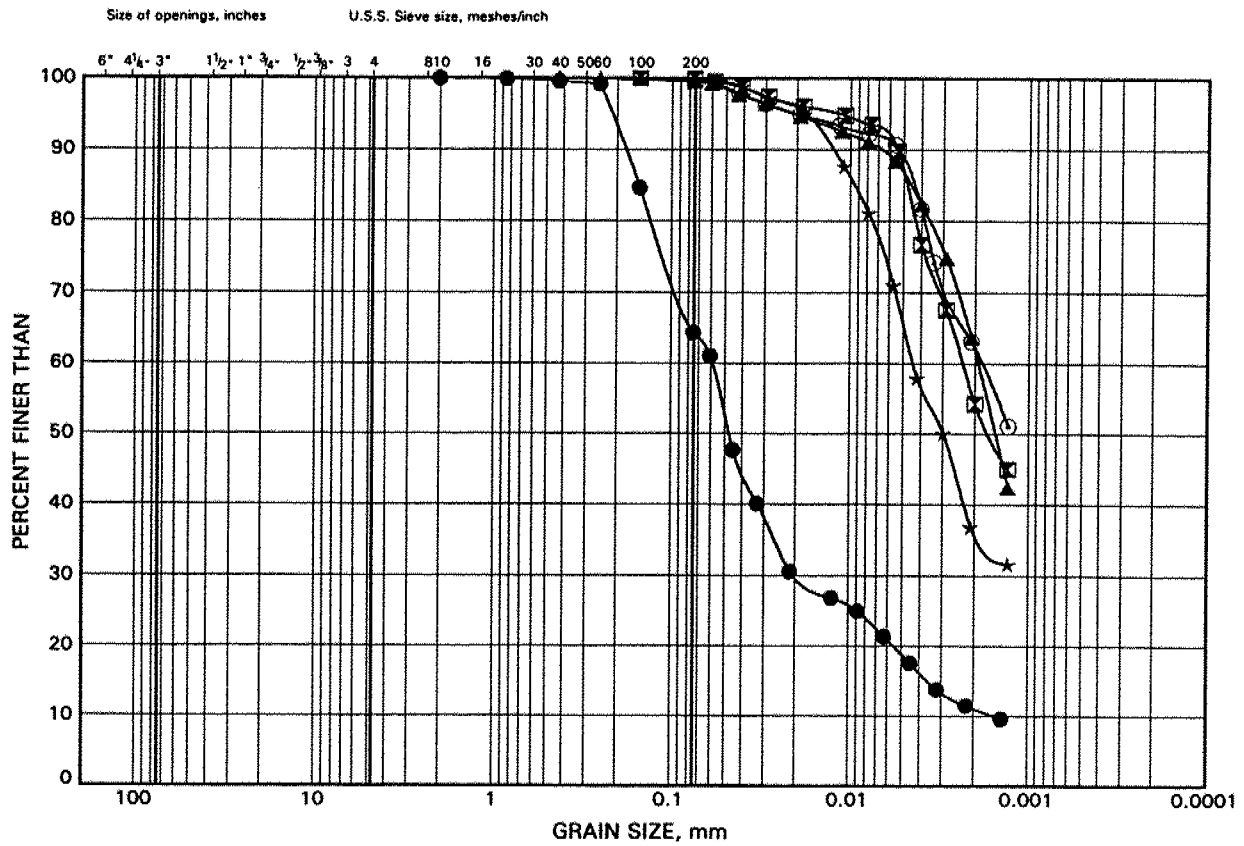


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B3



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-3	1.07	196.38
⊠	98-3	2.59	194.86
▲	98-3	6.40	191.05
★	98-3	12.50	184.95
⊙	98-3	17.07	180.38

Date December 1998

Project 812-76-01,398-91-00

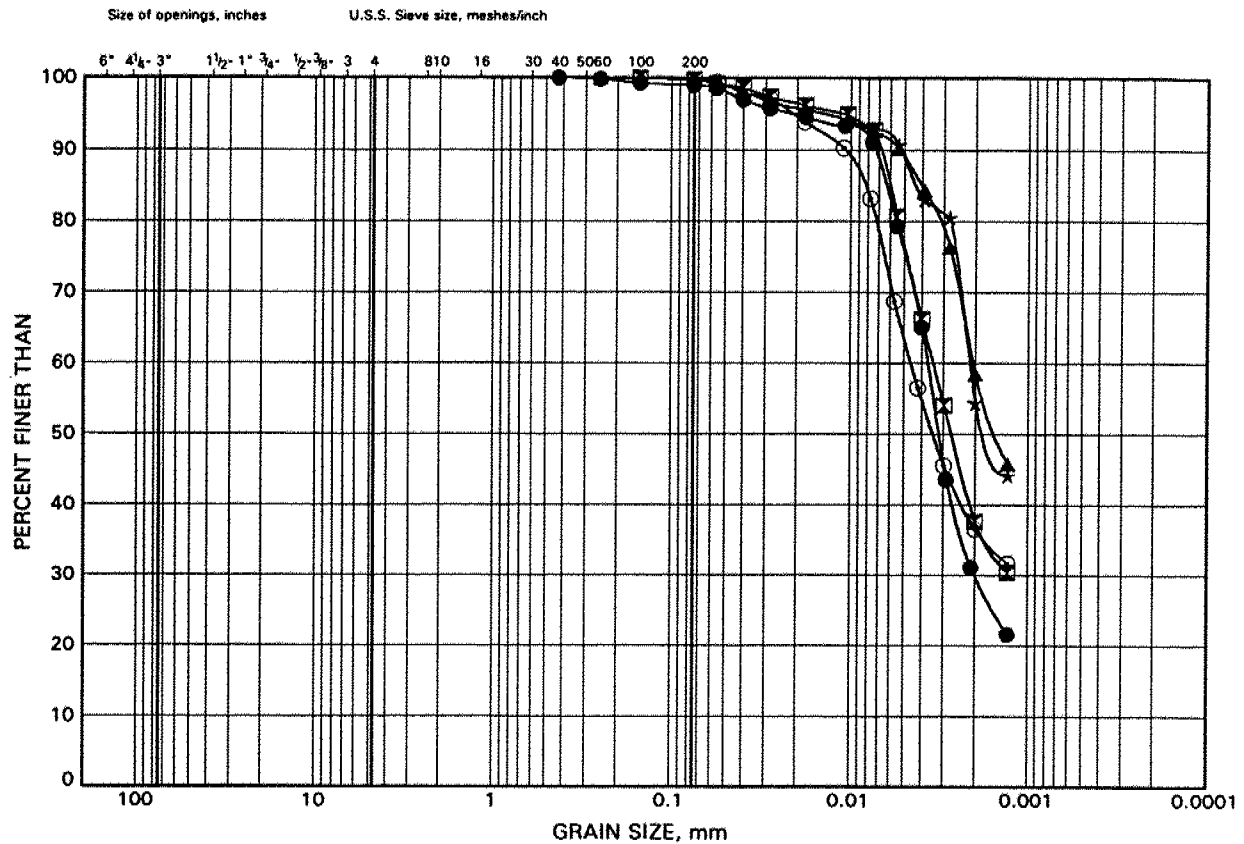


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK GRAIN SIZE DISTRIBUTION

FIGURE B4



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-4	3.35	198.08
⊠	98-4	4.88	196.55
▲	98-4	10.97	190.46
★	98-4	14.02	187.41
⊙	98-4	17.07	184.36

Date December 1998
Project 812-76-01,398-91-00

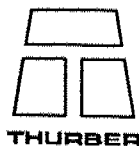


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FIGURE B5




Date December 1998
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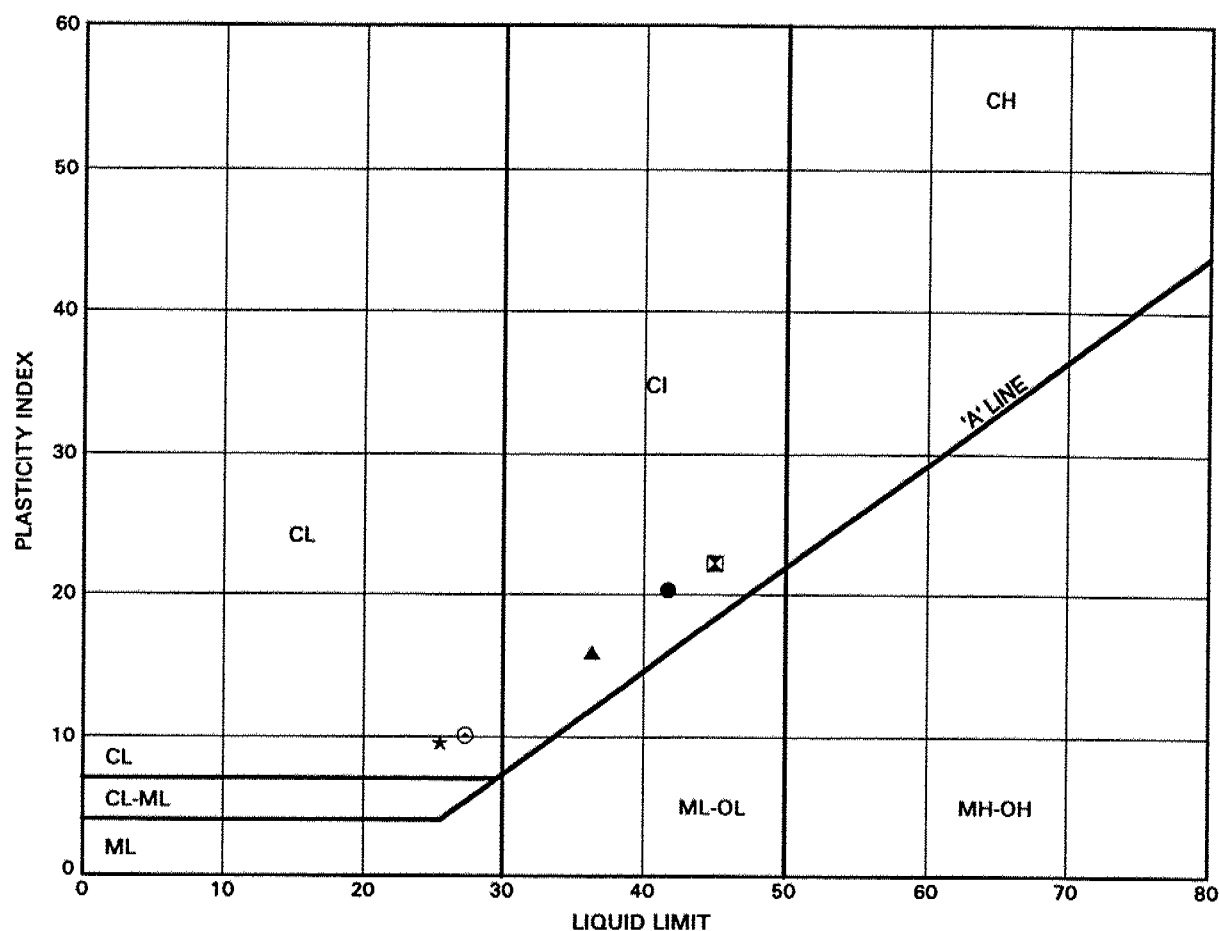
FIGURE B6



Prep'd WM
Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

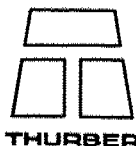
FIGURE B7



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-1	1.83	194.98
⊠	98-1	2.59	194.22
▲	98-1	7.92	188.89
★	98-1	10.97	185.84
⊙	98-1	17.06	179.75

Date December 1998

Project 812-76-01,398-91-00



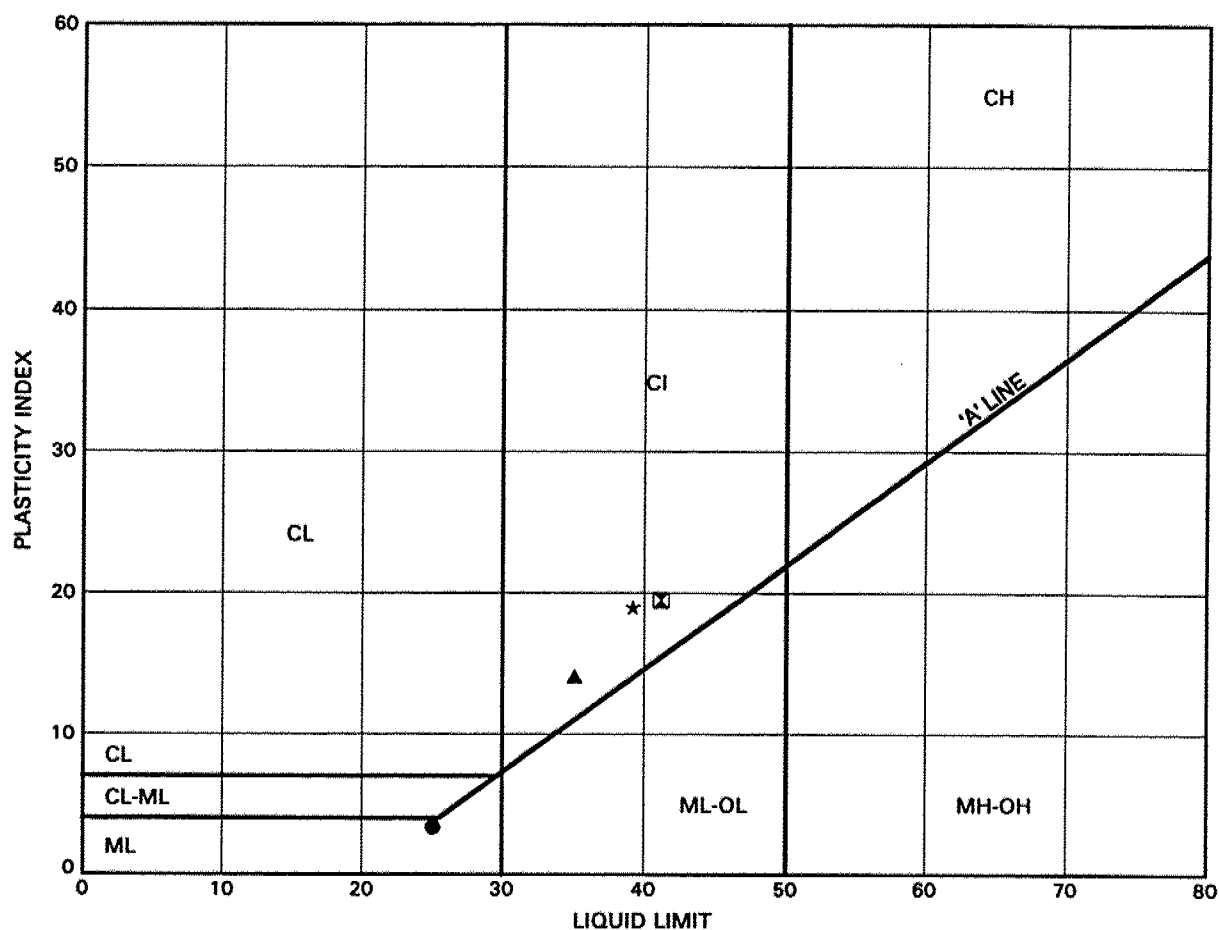
THURBER

Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

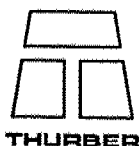
FIGURE B8



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-2	1.07	198.48
⊠	98-2	3.35	196.20
▲	98-2	9.45	190.10
★	98-2	15.54	184.01

Date December 1998

Project 812-76-01,398-91-00

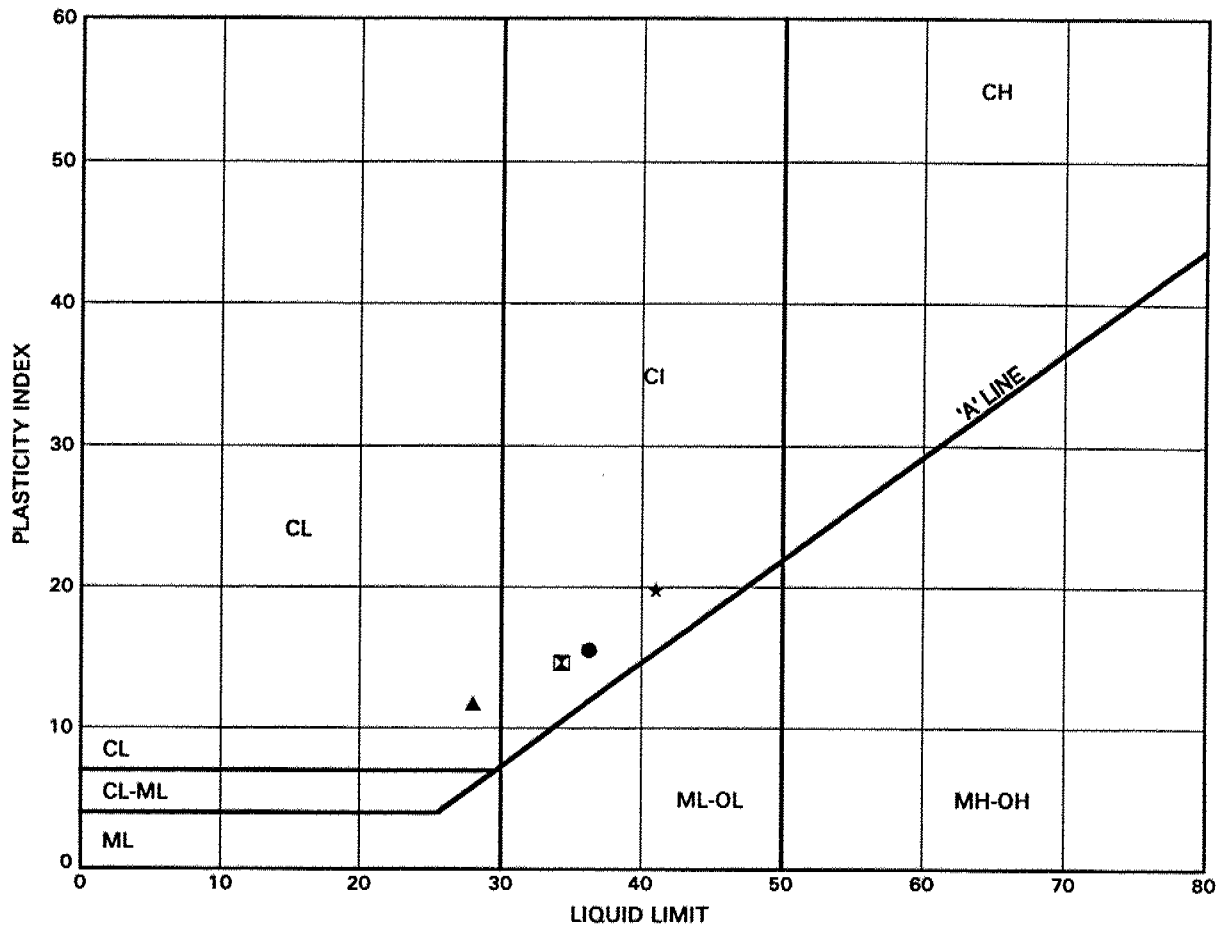


Prep'd WM

Chkd. AEG

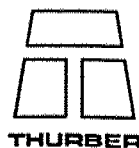
HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

FIGURE B9



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-3	2.59	194.86
☒	98-3	6.40	191.05
▲	98-3	12.50	184.95
*	98-3	17.07	180.38

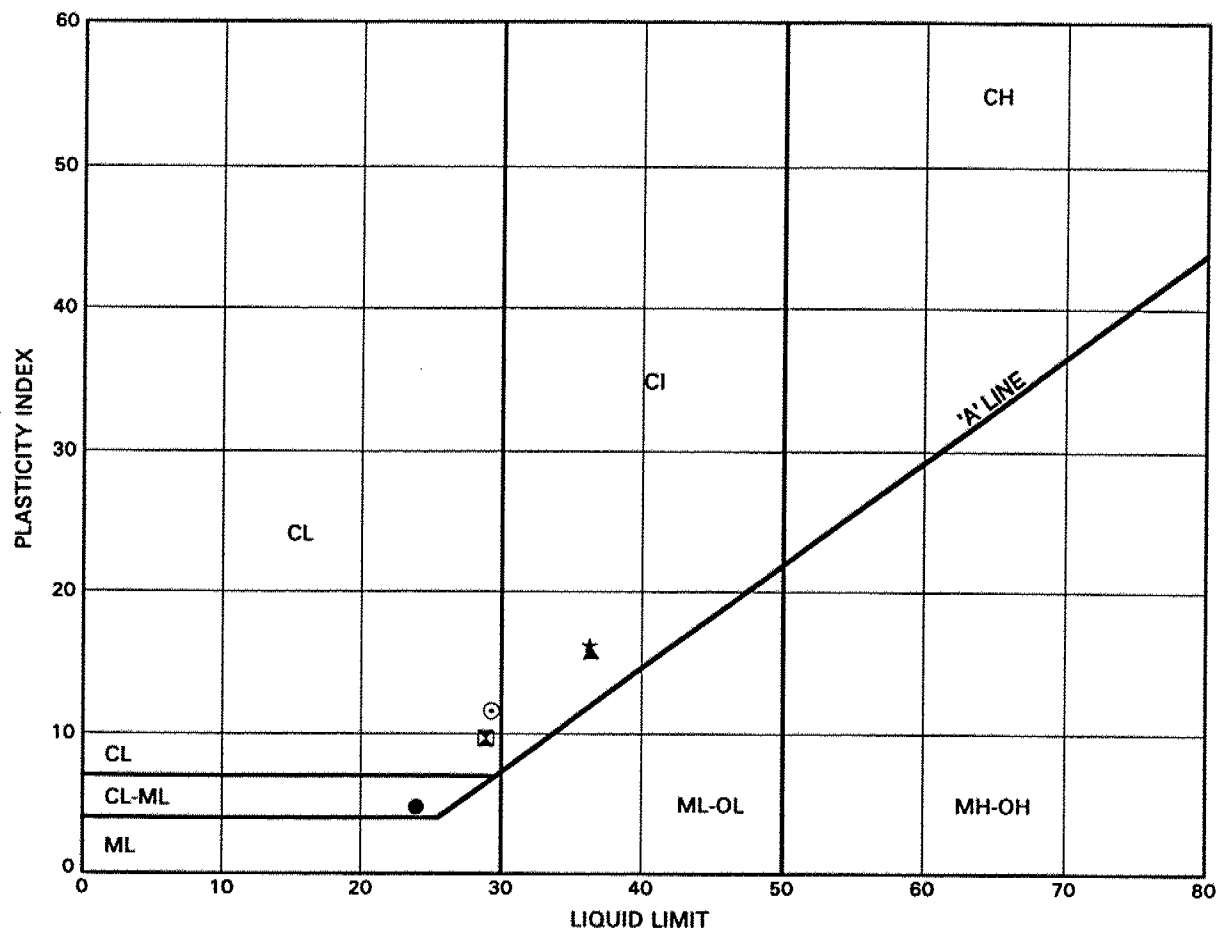
Date December 1998
 Project 812-76-01,398-91-00



Prep'd WM
 Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

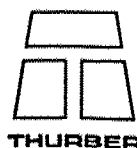
FIGURE B10



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-4	3.35	198.08
⊠	98-4	4.88	196.55
▲	98-4	10.97	190.46
★	98-4	14.02	187.41
⊙	98-4	17.07	184.36

Date December 1998

Project 812-76-01,398-91-00

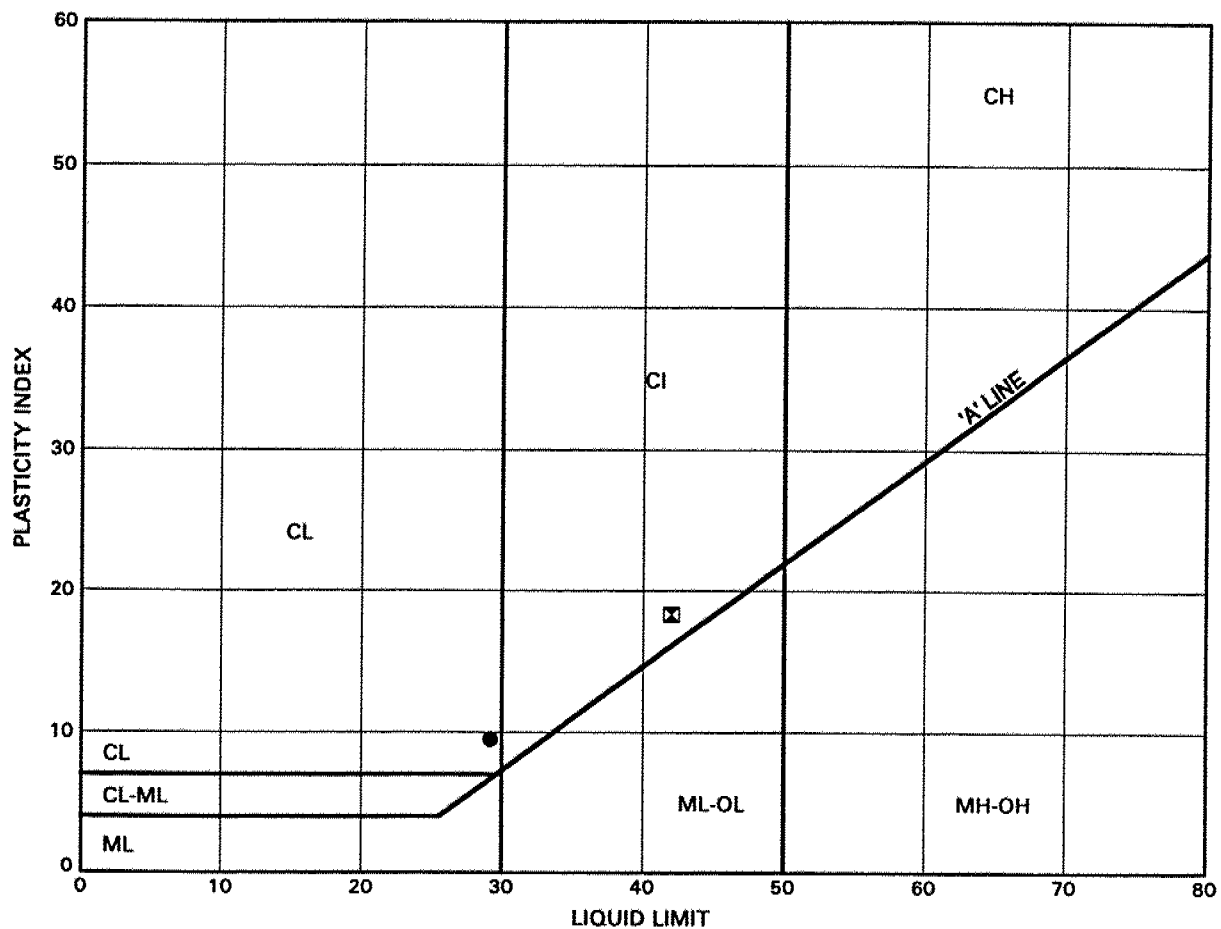


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

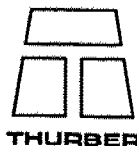
FIGURE B11



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	98-5	2.60	
⊠	98-5	4.88	

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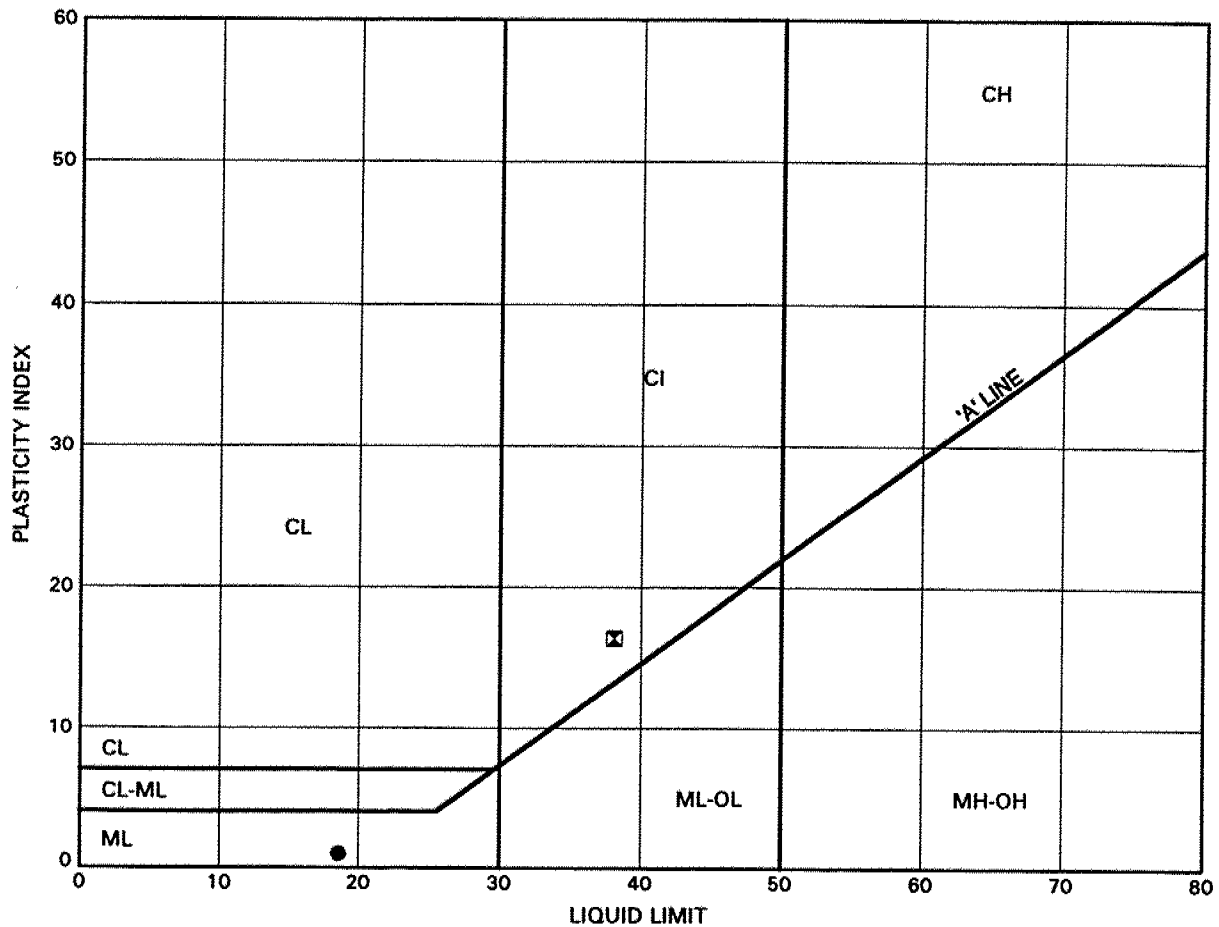


Prep'd WM

Chkd. AEG

HWY 17, LARONDE CREEK ATTERBERG LIMITS TEST RESULTS

FIGURE B12

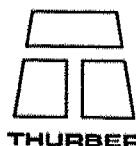


SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
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●	98-6	1.83	
⊠	98-6	4.88	

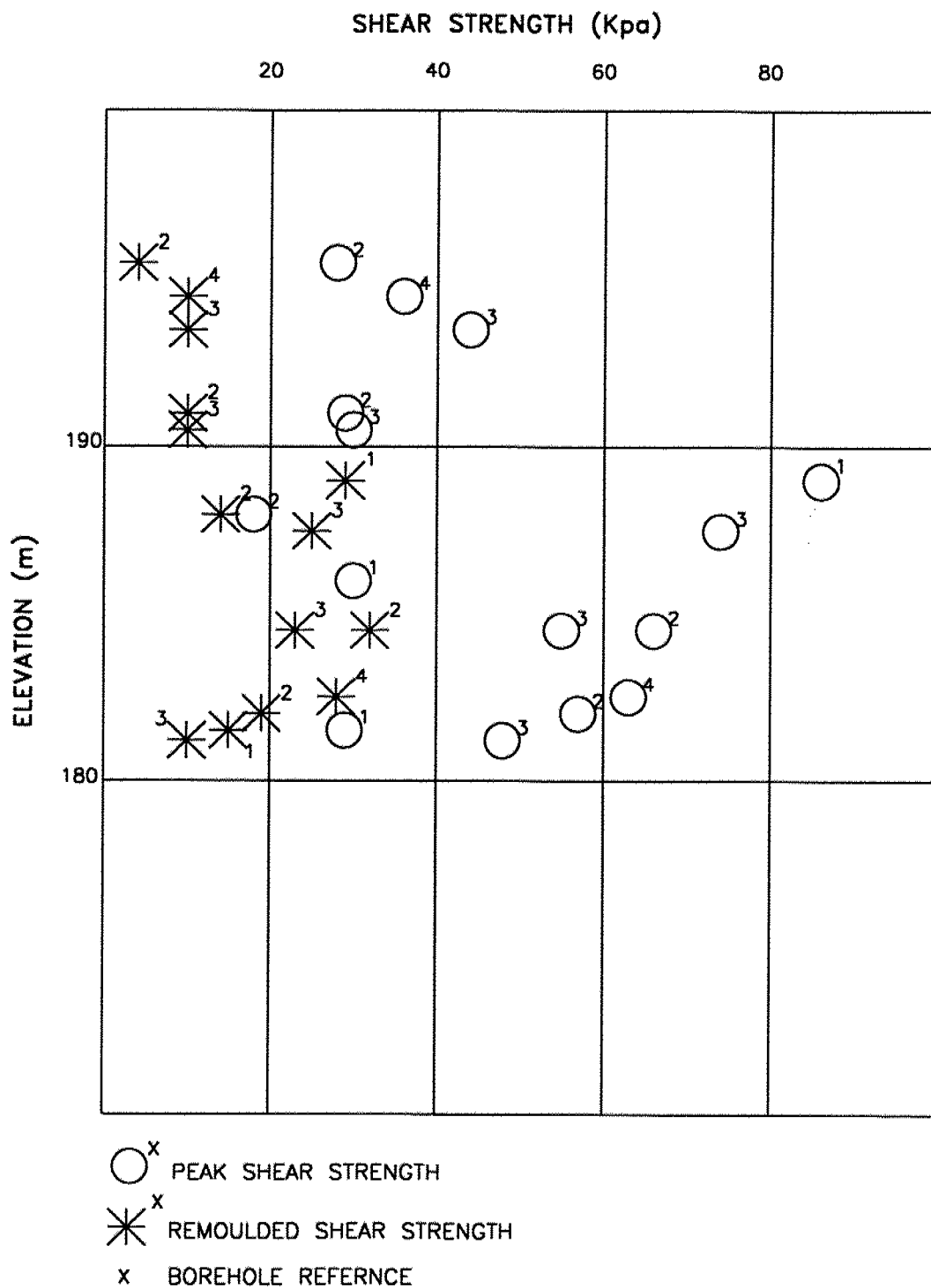
Date December 1998

Project 812-76-01,398-91-00

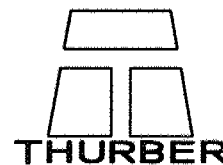


Prep'd WM

Chkd. AEG



IN-SITU VANE SHEAR STRENGTHS
LARONDE CREEK BRIDGE SITE



DRAWING No.

19-2847-0 FIG. B13

Table 1

Results of pH and Sulphate Testing

Sample	Depth (m)	pH	Sulphates (ppm)
98-2, Sa 3	1.8	8.6	36
98-3, Sa 3	2.0	8.0	7.2