

# **Foundation Investigation and Design Report Frazer River Bridge Replacement**

**GWP 476-00-00**

**Highway 585  
17 km north of the 585 and 11/17 Highway Intersection**

**Prepared for  
Ministry of Transportation, Northwestern Region**

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## Part A - FOUNDATION INVESTIGATION REPORT

### 1 Introduction

TBT Engineering has been retained by the Ministry of Transportation to provide foundation investigation services for the bridge replacement at Frazer River.

The site is located on Highway 585, approximately 17 km north of the Highway 11/17 intersection, within Booth Township, Ontario.

The foundation investigation was carried out to investigate subsurface conditions at the site. This investigation consisted of four boreholes drilled in the vicinity of the proposed bridge replacement, and laboratory testing. This report provides a summary of that work and of the conditions encountered.

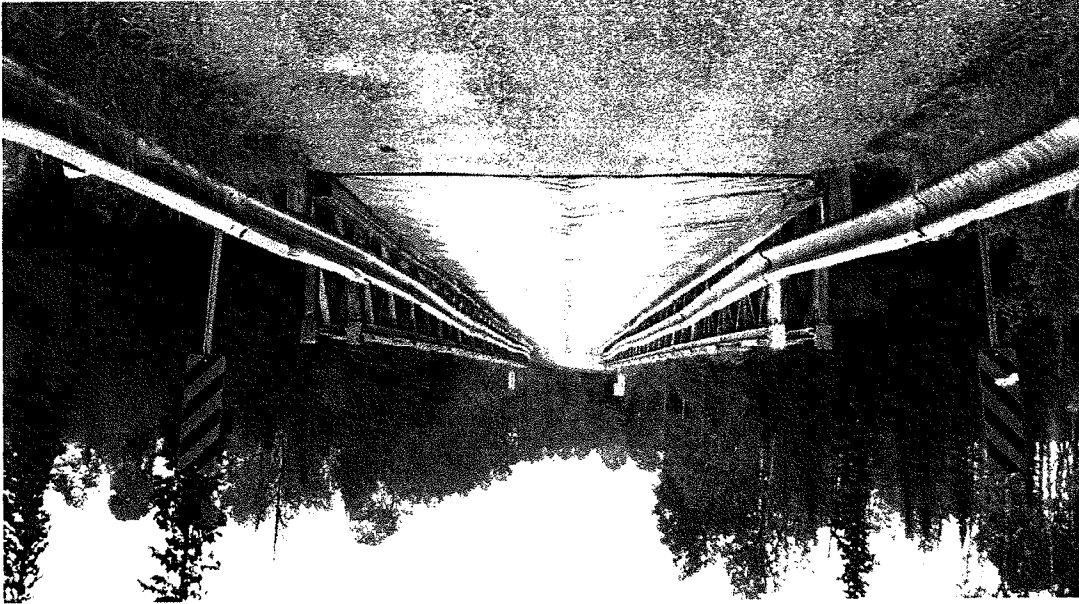
The site is identified as Site No. 48C-03B.

## 2 Site Description

The site is located on Highway 585, approximately 17 km north of the Highway 11/17 intersection. At this location Highway 585 runs locally in an east-west direction.

The existing bridge is a single lane Bailey structure supported on existing railway piers and is approximately 46 m long. The approach embankments have side slopes of approximately 1 horizontal to 1 vertical or steeper as the Highway approaches the Frazer River. Historical drawings show original river levels from 1930 and the corresponding fill operations to 1931. The drawings indicate significant amounts (approximately 10 m) of fill have been placed on the site.

The site is within Ontario's boreal forest region. Highway 585 in the bridge location is generally level with the surrounding area. The Frazer River water level at the time of the survey was approximately of 5.5 m below the bridge deck (elevation 93.5 m).



Frazer River Existing Bailey Bridge  
Booth Township

### 3 Investigation Procedures

A site investigation was undertaken on July 29, August 11 and 26, 2008. Four boreholes were drilled for this project along the approach (two on either side of the river). The investigation was carried out using a CME drill rig equipped for geotechnical testing. The boreholes were drilled to depths ranging between 8.3 and 21.5 m below existing ground surface. Borehole locations and depths were determined through consultation with the client during an onsite meeting. Potential borehole locations were limited due to the narrow bridge and road width as well as the need to allow traffic to cross the bridge during drilling operations. Refer to the Record of Borehole drawings for specific depths and comments concerning each borehole.

Soil samples were obtained at the boreholes with a split spoon sampler as a part of the Standard Penetration Testing (SPT). The SPT involves driving a thick walled sampler into the soils under a standardized energy (63.5 kg, falling 760 mm). The number of blows required to drive the sampler 0.3 m, known as the SPT blow count (N), was recorded. In addition to SPTs, shear vane testing was conducted within cohesive soils. Thin walled tube samples were taken within cohesive materials to obtain relatively undisturbed samples. Bedrock was sampled utilizing diamond core techniques.

Borehole location and elevations were surveyed in the field by Cook Engineering and referenced to a local benchmark. Number 300, a RR spike driven in a 0.45 m diameter poplar at station 20+146.770 12.365 LT offset, with an elevation of 99.675 m.

A summary of the borehole location data is provided on the enclosed Borehole Location Plan and Soil Strata Drawing 1.

The borehole characteristics and drill techniques utilized are summarized for the various borehole locations in the following table.

Frazer River  
Booth Township

The boreholes were backfilled to ensure the environmental integrity of the site, utilizing appropriate bentonite/cement mixtures for the soils encountered (at individual locations).

(Appendix B).

## **4 General Site Geology and Sub-Surface Conditions**

### **4.1 Site Geology**

The Frazier Creek flows approximately south-southeast from Elizabeth Lake, located approximately 11 km north of the bridge site, to the Nipigon River, just south of the site. The Nipigon River flows generally south and reports to Lake Superior near the town site of Red Rock.

Available surficial geology mapping (OGS NOEGTS Map 5052 – Frazier Lake) shows a north-south oriented sand and gravel esker ridge terminating in the area of the east bank of the Frazier Creek, adjacent to the bridge site. Local relief is expected to be elevated on that side, compared to the lower-lying silt-clay lacustrine plain and drift-veneer over bedrock deposits mapped on the west side of the creek. Borehole data acquired at the site generally confirm the mapped deposits, showing variability in materials both aerially and with depth.

Bedrock in the area of the Frazier Creek is of the Archean-aged Quetico geological subprovince. Available mapping (OGS Map 2232 – Nipigon-Schreiber Compilation) shows paragneiss and metasedimentary-derived migmatite underlying the area of the bridge. Associated granitic units may also be present to the north and east of the bridge. Regional deformation is generally aligned east-west; however, faulting is locally aligned approximately north-south. One such fault underlies the Frazier Creek valley in area of the bridge site, between Elizabeth Lake and the Nipigon River. A Proterozoic-aged Nipigon diabase sill crops out approximately 500 m to the northwest of the bridge site.

### **4.2 Subsurface Conditions**

Details of the subsurface conditions are provided on the Borehole Logs, Appendix A, and on the Borehole Location Plan and Strata Drawing 1.

#### **4.2.1 General**

In general, the subsurface stratigraphy consists of fill, underlain by a strata of clay above lower silts. The lower silt strata overlies sand and gravel till which extends to bedrock.

Historically this section of the river has received a large amount of fill placement dating back to the 1930's. An estimated 10 m of fill has been placed on the site based on historical drawings.

The subsurface stratigraphy has been interpreted based on the results of the boreholes and has been illustrated on the Borehole Location Plan and Soil Strata Drawing 1.

#### **4.2.1.1 Fill**

Fill consisting of various mixtures of sand, gravel and silt were encountered at surface of all the boreholes. The fill varies in thickness from 1.2 to 7.3 and extends to a depth ranging from 1.2 to 7.3 m (elevation 97.6 to 91.7 m). Historical drawings indicate the bottom of the fill is more than 10 m below the water level in Frazer River. Based on three samples tested, the fill can consist of 7 - 48 % gravel, 48 - 75% sand, and 4 - 10 % silt sized particles with occasional cobbles. The material is very loose to dense as indicated by "N" values ranging from 1 to 31 blows per 0.3 m.

Within the fill at Borehole FR08-03 a void was encountered at a depth of 4.6 to 6.0 m (elevation 94.4 to 93.0 m). At the 6.0 m the SPT was driven through 50 mm of wood, before continuing into more fill.

#### **4.2.1.2 Silt – Possible Fill**

Sandy silt was encountered beneath the fill at Boreholes FR08-01, 02, and 04. This material may be fill when its location and elevation are compared to the historic fill drawings, however there is not enough information currently available to confirm this. Grain size analyses indicate the sandy silt can consist of 0 % gravel, 23 - 26 % sand, and 71 to 74 % silt sized particles. The silt varies in thickness from 0.8 to 4.3 m in thickness and extends to a depth ranging from 3.4 to 7.2 m (elevation 95.4 to 91.4 m). The material is very loose to loose as indicated by "N" values ranging from 1 to 5 blows per 0.3 m. This material is considered to be frost susceptible. Atterberg limit testing conducted on sample from Boreholes FR08-01 indicate the silt is non-plastic.



#### 4.2.1.3 Clay

Clay is present beneath the sandy silt at Boreholes FR08-01, 02, and 04 and beneath the fill at FR08-03. This clay layer is present at all borehole locations. The clay varies in thickness from 1.4 to 5.4 m in thickness and extends to a depth of ranging from 8.6 to 10.5 m (elevation 90.4 to 88.1 m). The material is firm to stiff as indicated by field vane tests ranging from 30 to 86 kPa. Atterberg limits testing indicate the clay ranges from silty clay (CL-ML) to medium plastic clay (CI) with natural moisture contents generally above the liquid limit.

#### 4.2.1.4 Lower Silt

A layer of silt was encountered below the clay at all boreholes with the exception of FR08-04 in which the borehole was terminated within the clay. Based on a sample from Borehole FR08-01 the silt can consist of 0 % gravel, 0 % sand, and 100 % silt sized particles. The silt varies in thickness from 0.9 to 6.0 m in thickness and extends to a depth ranging from 9.5 to 16.5 m (elevation 99.0 to 82.1 m). The material is loose to compact as indicated by "N" values ranging from 8 to 25 blows per 0.3 m. Atterberg limits tests conducted on a sample from Borehole FR08-01 indicate the silt is non-plastic.

#### 4.2.1.5 Discontinuous Silt and Sand

A discontinuous silt and sand layer was encountered below the silt within Borehole FR08-01. Based on a sample from Boreholes FR08-01 the silt and sand can consist of 0 % gravel, 41 % sand, and 59 % silt sized particles. The silt and sand layer has a thickness of 6.3 m and extends to a depth of 22.8 m (elevation 75.8 m). The material is compact to very dense as indicated by "N" values ranging from 13 to 100 blows per 0.3 m.

#### 4.2.1.6 Till

A heterogeneous mixture of sand, gravel and silt was encountered below the discontinuous silt and sand at Borehole FR08-01 and below the lower silt at Borehole FR08-02. This till can consist of 32 % gravel, 41 % sand, and 27 % silt sized particles. This material varies in thickness from 1.5 to 3.4 m and extended to a depth ranging from 11.7 to 26.2 m (elevation 89.3 to 72.4 m). Numerous cobbles and boulders were

encountered within this stratum at Borehole FR08-01. The till is dense to very dense as indicated by "N" values ranging from 48 to greater than 100 blows per 0.3 m.

#### 4.2.1.7 Bedrock

Bedrock was confirmed at Borehole FR08-02 at a depth of 11.7 to 14.8 m from ground surface (elevation 84.0 to 87.1 m). The bedrock is a dark grey to black biotite-schist, mostly fine-grained, quite weathered and fractured with some alteration along the fracture surfaces from depths 11.7 to 13.8 m. The bedrock becomes more competent from 13.8 to 14.8 m with some tendency for vertical fracture, and spaced horizontal jointing. The intact portion of the bedrock can be considered extremely strong based on the point load tests performed on core samples from borehole FR08-02 at depths of 14.0 and 14.3 m with estimated uniaxial compressive strengths ranging from 300 to 375 MPa. The estimated uniaxial compressive strengths are based on published correlations with point load testing.

The rock quality designation (RQD) is an indirect measure of the number of fractures and the amount of jointing in the rock mass. The RQD is expressed as a percentage of the ratio of summed core lengths (greater than 100 mm) to the total length cored. The RQD index is used to provide a classification for the rock quality according to the following limits.

Table 2 – RQD Index

RQD %	Rock Quality
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

The RQD for the four core lengths at Borehole FR08-02 were 0 %, 0 %, 14 % and 46 %, indicating a trend of increasing RQDs with depth, from elevation 87.1 to 84.0 m. The rock quality however varies from very poor to poor quality.

The bedrock surface grades are known to vary considerably. Significant elevation changes are common within short distances.

#### **4.2.1.8 Ground Water**

The water level of the Frazer River at the time of the survey was at elevation 92.4 m. Groundwater levels upon completion of drilling had not stabilized. Ground water levels will be highly dependant on the water level within the river, and will generally rise in elevation away from the river. Ground water levels will also fluctuate with precipitation events.

#### **5 Miscellaneous**

The field drilling services for this project were provided by TBT Engineering. Laboratory testing was carried out at the TBT Engineering laboratory in Thunder Bay. The field operations were supervised by H. Finke. This report was prepared by S Seller, P.Eng. and G. Maki, P.Eng., and reviewed by W. Hurley, P.Eng.

## Part B FOUNDATION DESIGN RECOMMENDATIONS

### 6 Discussions and Engineering Recommendations

#### 6.1 Introduction

The current Frazer River Bridge was constructed in 1986 and is a double span Bailey structure supported on an old railway structure. The railway structure is supported by concrete piers, reportedly founded on bedrock. The proposed replacement structure will be a single span single lane Bailey structure. A detour will not be utilized during the construction of the replacement bridge.

The foundation investigation as described in Part A, was carried out to investigate subsurface conditions at the site. This investigation consisted of a number of boreholes advanced in the vicinity of the new structure, laboratory testing and geotechnical analysis of the data. The purpose of this section of the report (Part B) is to provide geotechnical design recommendations for the project. These are based on the conditions encountered at the test locations and our interpretation of the subsurface conditions at the site.

#### 6.2 Bridge Foundations

The foundation system for the proposed structure must support the design loads within acceptable settlement tolerances and must accommodate all anticipated loadings. The existing soil strata at this site includes fills, over clay which lies over silt followed by a sand and gravel till to bedrock varying from 22 to 12 metres below grade. Bedrock is encountered on the east side of the bridge at an elevation of 87.1 m. The preliminary design concepts for the bridge and abutment involve the use of Bailey Bridge founded on piles.

Shallow foundations are not suitable at this site due to the presence of fill material to depths in excess of seven meters at the existing abutment locations and a void found in the fill on the west side of the River.

Deep driven piles may be designed to bear within the till or on bedrock. It is expected the piles could vary from 12 to 26 m in length with tip elevations varying from 87 to 72 m. The actual depth of pile penetration will depend on localized till, conditions and thickness and potentially the under lying bedrock condition and elevation.

### 6.3 Driven Piles

For the anticipated design loads and with the presence of compressible overburden soils over till, the most feasible foundation will consist of piles driven into the underlying till. A heavy steel pile such as a HP 310 x 110 piles is recommended. H-piles are expected to penetrate the till to varying depths. Piles are to be driven under the requirements of Special Provision 903.S01.

**Table 3 - Pile Design Capacities**

Pile Designation	ULS Factored Geotechnical Axial Resistance	HP 310x110
SLS Geotechnical Resistance for 25 mm of Settlement	1800 kN	1450 kN

It is anticipated that effective refusal (pile tip elevation) will be encountered between 87 and 72 m elevation, approximately 12 to 26 m below the top of piles. For refusal within the till material, geotechnical axial resistance (ULS) verification shall be determined through the use of the Hiley Formula (Standard drawing SS1030-11) as per Section 903.08.01.02 of SSP903S01.

**Table 4 – Estimated Effective Pile Tip Elevation**

Abutment	Estimated Effective Pile Tip		Comments
	Depth m	Elevation m	
East	12	87	From Borehole FR08-02 approx. 30 m from existing abutment location
West	26	72	From Borehole FR08-01 approx. 22 m from abutment location

Pile set should be verified in the field for the specific pile and pile driver combination in use. The depth to refusal should be expected to vary and the contractor should be prepared to drive piles of variable lengths.

Due to the presence of boulders in the till stratum, and high stresses induced during driving through very dense till the piles should be equipped with a driving shoe to prevent damage to the pile tip, such as a Titus Standard "H" Bearing Pile Point. The behaviour of the piles should be monitored during driving for any signs indicative of pile damage.

Existing concrete piers may be adjacent/near to the new piles. The new piles must avoid existing foundations during driving. The presence of numerous cobbles and boulders within the till and fill may cause the piles to "walk" during driving. Displaced piles will require review and potential replacement.

Piles should be spaced at least 2.5 pile widths apart (centre to centre).

Drag loads caused by negative skin friction are not anticipated at this site since no sustained wide spread and/or heavy concentrated surface loads are anticipated.

## **7 Embankments**

It is understood the approach embankments will not be raised from their existing elevations and no revisions/upgrades to the existing embankments will be done. Based on the survey data provided, the existing embankment slopes are over-steepened and have factors of safety in terms of slope stability marginally greater than unity.

If any alteration/upgrading of the existing embankments/slopes are considered, additional investigation and testing of the embankment and historic fill will be required as well as hydraulic investigations of the River. Caution should be taken during construction as to not disturb the existing embankment/slope configurations.

## **8 Frost Protection**

Based on the Ontario Provincial Standard Drawing 3090.1 "Foundation Frost Depth for Northern Ontario" the estimated frost depth penetration within the expected embankment fill is 2.6 m.

The soils within the frost depth at the proposed abutment locations are considered to be of low frost susceptibility (MTO Pavement Design and Rehabilitation Manual).

## **9 Dewatering and Excavations**

Excavations for pile installations should be carried out and sloped in accordance with the requirements of the Occupational Health and Safety act. Dewatering will not be required for the small excavations anticipated to be needed for pile and pile cap construction.

## **10 Red Flag Issues**

The installation of piles will be into a till deposit with numerous cobbles and boulders. Driving piles to or through materials containing cobbles and boulders has a high potential for damage and or misalignment of the piles. The contractor should take precautions to avoid over driving the piles, and should monitor piles for signs indicative of damage.

During drilling operations occasional cobbles were encountered within the fill. Due to the unknown nature of the existing fill and its varying thickness, there is a potential for this material to contain numerous cobbles, boulders or other obstructions. Hard driving conditions may also be experienced within the fill strata, depending on the materials encountered.

The embankments are marginally stable at the current configuration. Actions which could decrease stability should be avoided. (i.e. steeping of the bank, vegetation removal etc.).

The above comments are not intended to include all critical issues that may become apparent during the construction of this project. The responsibility to deliver acceptable construction quality remains with the contractor.

## 11 Limitations

Conclusions and recommendations presented in this report are based on the information determined at the test hole locations. Subsurface and groundwater conditions between and beyond these locations may differ from those encountered. Conditions may become apparent during construction that were not detected and could not be anticipated at the time of the site investigation.

The design recommendations provided in this report are based on the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments given in this report on potential construction problems and possible methods of construction are intended only for the guidance of the designer.

Benchmarks and elevations referred to in this report are used primarily to establish relative elevation differences between the test hole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

Groundwater levels indicated are based on the information described within the report. The presence of all conditions that could affect the type and scope of dewatering procedures which may be considered cannot readily be determined from boreholes. These include local and seasonal fluctuations of the groundwater level, changes in soil conditions between test locations, thin and/or discontinuous layers of highly permeable soils, etc.

The information contained within this report in no way reflects any environmental aspect of the site or soil.

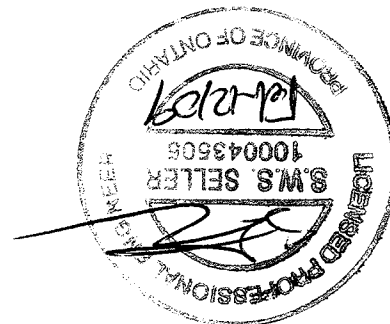


## 12 Closure

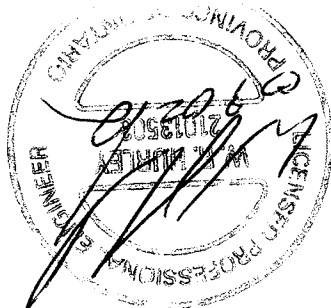
We trust the above addresses your project requirements at this time. Should you have any questions or comments, please do not hesitate the contact us at your convenience.

Yours truly,

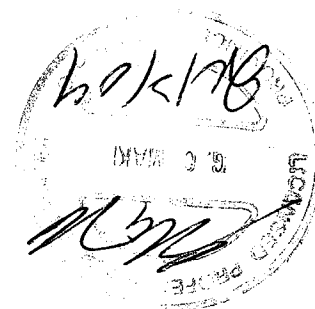
For TBT ENGINEERING



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Vice-President, Engineering



Gordon Maki, P. Eng  
Geotechnical Engineering

## References

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4. Canadian Geotechnical Society, Canadian Foundation Engineering Manual, Fourth Edition, Bitech Publishers Ltd., 1992, ISBN 0-920505-28-7
5. Hunt, Roy E., Geotechnical Engineering Analysis and Evaluation, McGraw Hill Inc, 1986, ISBN 0-07-031310-5
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## MTO Special Provisions and Drawings

Special Provision 903.S01 (Pile Installations)

## APPENDIX A BOREHOLE LOGS

TBT Engineering

# EXPLANATION OF TERMS USED IN REPORT

N VALUE The standard penetration test (SPT) N value is the number of blows required to cause a standard 50mm O.D. split barrel sampler to penetrate 0.3m into undisturbed ground in a borehole when driven by a hammer with a mass of 63.5kg, falling freely a distance of 0.76m for penetrations of less than 0.3m. N values are indicated as the number of blows for the penetration achieved average N value is denoted thus N.

DYNAMIC CONE PENETRATION TEST Continuous penetration of a conical steel point (50mm O.D. 60° cone angle) driven by 47.5 J impact energy on a size drill rods. The resistance to cone penetration is measured as the number of blows for each 0.3m advance of the conical point into the undisturbed ground.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH ( $c_u$ ) AS FOLLOWS

$c_u$ (kPa)	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD
0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200	

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS

N (BLOWS/0.3m)	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE
0 - 5	5 - 10	10 - 30	30 - 50	> 50	

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN  
 MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN  
 THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	VERY POOR	POOR	FAIR	GOOD	EXCELLENT
0 - 25	25 - 50	50 - 75	75 - 90	90 - 100	

JOINTING AND BEDDING:

SPACING	50mm	50 - 100mm	100 - 200mm	> 200mm
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK
	VERY THIN	THIN	MEDIUM	THICK

## ABBREVIATIONS AND SYMBOLS

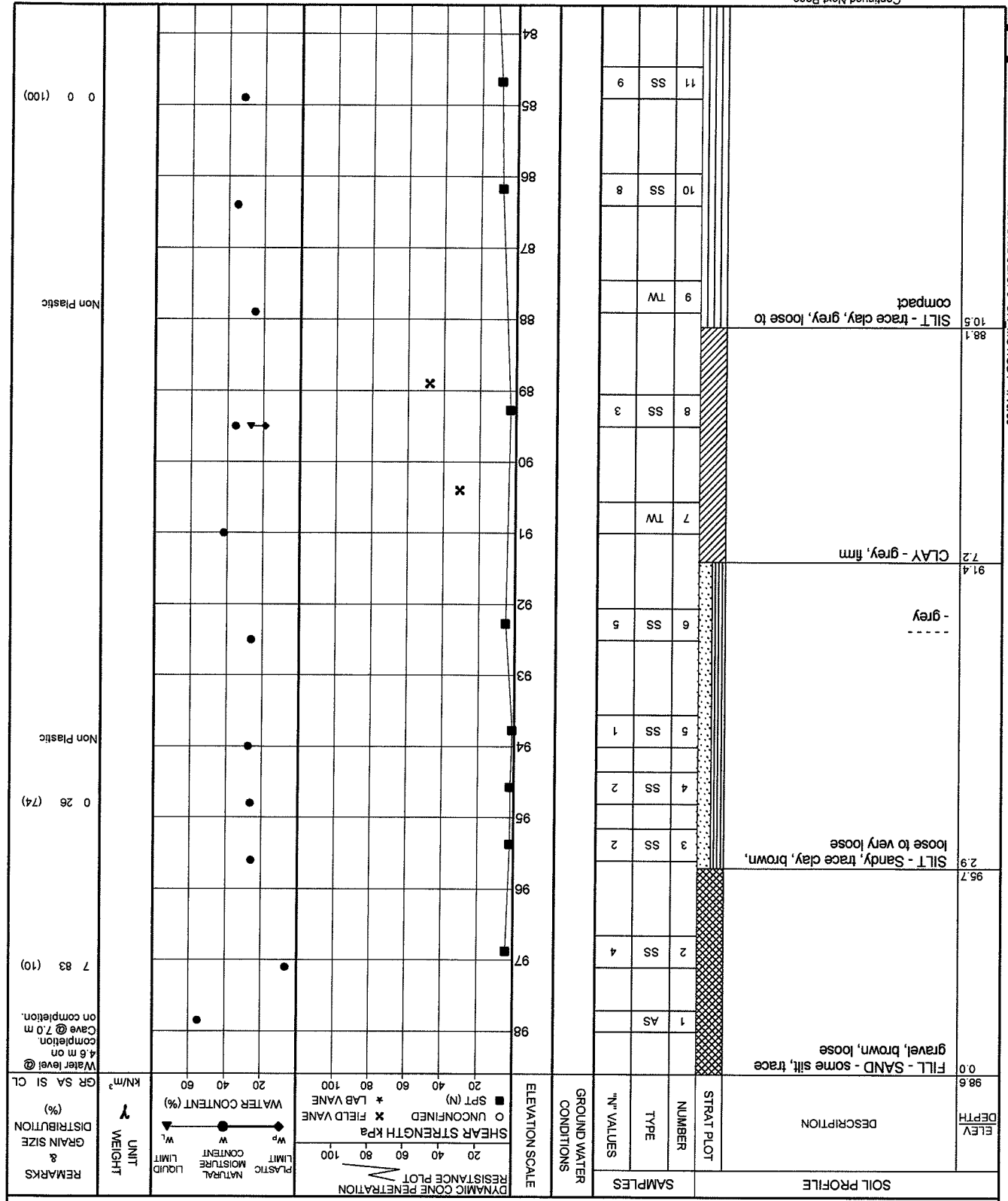
### MECHANICAL PROPERTIES OF SOIL

5 S	SPLIT SPOON	T F	THINWALL PISTON	$m_v$	COEFFICIENT OF VOLUME CHANGE
W 5	WASH SAMPLE	O 5	OSTERBERG SAMPLE	$c_c$	COMPRESSION INDEX
3 1	SLOTTED TUBE SAMPLE	R C	ROCK CORE	$c_s$	SWELLING INDEX
8 5	BLOCK SAMPLE	P H	1 W ADVANCED HYDRAULICALLY	$c_{\alpha}$	RATE OF SECONDARY CONSOLIDATION
C 5	CHUNK SAMPLE	P M	1 W ADVANCED MANUALLY	$c_{\alpha}$	COEFFICIENT OF CONSOLIDATION
F 5	FOIL SAMPLE	F 5	FOIL SAMPLE	$m$	DRAINAGE PATH
STRESS AND STRAIN					
$u_w$	PORE WATER PRESSURE	$\sigma'_v$	EFFECTIVE VERTICAL STRESS	$\sigma'_v$	EFFECTIVE VERTICAL STRESS
$\sigma$	TOTAL NORMAL STRESS	$\sigma_v$	TOTAL VERTICAL STRESS	$\sigma_v$	TOTAL VERTICAL STRESS
$\sigma'$	EFFECTIVE NORMAL STRESS	$\sigma'_v$	EFFECTIVE VERTICAL STRESS	$\sigma'_v$	EFFECTIVE VERTICAL STRESS
$\tau$	SHEAR STRESS	$\tau$	SHEAR STRESS	$\tau$	SHEAR STRESS
$\sigma_1, \sigma_3$	PRINCIPAL STRESSES	$\sigma_1, \sigma_3$	PRINCIPAL STRESSES	$\sigma_1, \sigma_3$	PRINCIPAL STRESSES
$\epsilon_1, \epsilon_3$	PRINCIPAL STRAINS	$\epsilon_1, \epsilon_3$	PRINCIPAL STRAINS	$\epsilon_1, \epsilon_3$	PRINCIPAL STRAINS
$E$	MODULUS OF LINEAR DEFORMATION	$E$	MODULUS OF LINEAR DEFORMATION	$E$	MODULUS OF LINEAR DEFORMATION
$C$	MODULUS OF SHEAR DEFORMATION	$C$	MODULUS OF SHEAR DEFORMATION	$C$	MODULUS OF SHEAR DEFORMATION
$\mu$	COEFFICIENT OF FRICTION	$\mu$	COEFFICIENT OF FRICTION	$\mu$	COEFFICIENT OF FRICTION

### PHYSICAL PROPERTIES OF SOIL

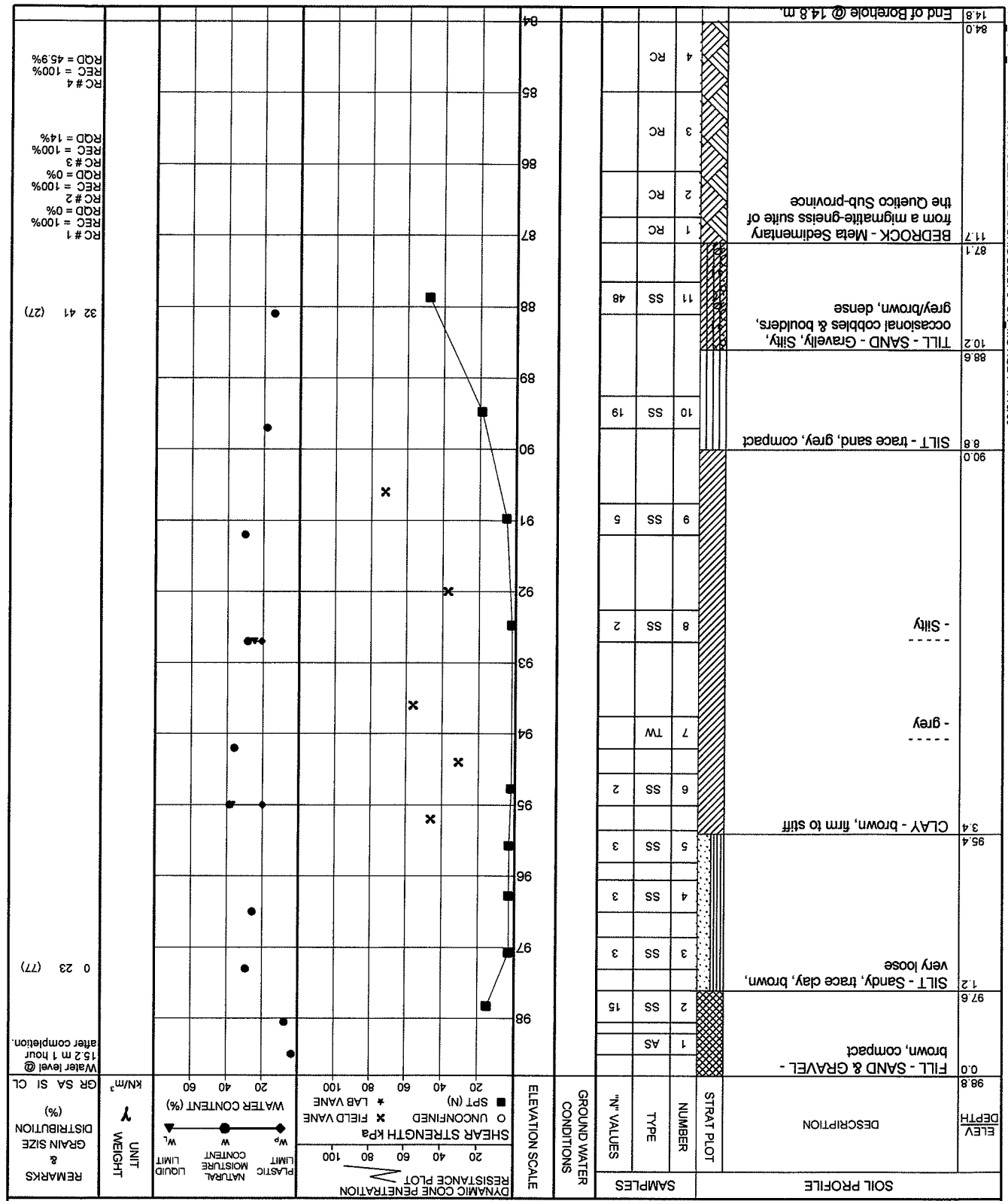
$\rho_s$	DENSITY OF SOLID PARTICLES	$e$	VOID RATIO	$\epsilon_{min}$	VOID RATIO IN DENSEST STATE
$\gamma_s$	UNIT WEIGHT OF SOLID PARTICLES	$n$	POROSITY	$i_D$	DENSITY INDEX
$\rho_w$	DENSITY OF WATER	$w$	WATER CONTENT	$D$	GRAIN DIAMETER
$\gamma_w$	UNIT WEIGHT OF WATER	$s_r$	DEGREE OF SATURATION	$O_n$	PERCENT DIAMETER
$\rho$	DENSITY OF SOIL	$w_L$	LIQUID LIMIT	$C_u$	UNIFORMITY COEFFICIENT
$\gamma$	UNIT WEIGHT OF SOIL	$w_p$	PLASTIC LIMIT	$h$	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	DENSITY OF DRY SOIL	$w_s$	SHRINKAGE LIMIT	$q$	RATE OF DISCHARGE
$\gamma_d$	UNIT WEIGHT OF DRY SOIL	$I_p$	PLASTICITY INDEX	$v$	DISCHARGE VELOCITY
$\rho_{sat}$	DENSITY OF SATURATED SOIL	$I_L$	LIQUIDITY INDEX	$i$	HYDRAULIC GRADIENT
$\rho'$	DENSITY OF SUBMERGED SOIL	$I_c$	CONSISTENCY INDEX	$k$	HYDRAULIC CONDUCTIVITY
$\gamma'$	UNIT WEIGHT OF SUBMERGED SOIL	$i_{max}$	VOID RATIO IN LOOSEST STATE	$j$	SEEPAGE FORCE

TBT Engineering Consulting Group		RECORD OF Borehole No FR08-01		1 OF 2		METRIC	
PROJECT Frazer River Bridge		SITE NO. 48C-03B		ORIGINATED BY HF			
LOCATION Stn. 20+227.8 o/s 2.8 Lt.-Booth Twp		TBTE JOB# 08-085		COMPILED BY TB			
DATE July 29, 2008		BOREHOLE TYPE Hollow Stem Auger		DATUM Local		CHECKED BY SS	



\* 3, \* 3: Numbers refer to Sensitivity  
 NP Non Plastic  
 ○ 3% STRAIN AT FAILURE

TBT Engineering Consulting Group		RECORD OF Borehole No FR08-02		1 OF 1		METRIC	
PROJECT Frazer River Bridge		SITE NO. 48C-03B		ORIGINATED BY HF		W.P. 476-00-00	
LOCATION Sm. 20+326.1 o/s 3.3 Lt. - Booth Twp		TBTE JOB# 08-085		COMPILED BY TB		DIST 61 HWY 585	
BOREHOLE TYPE Hollow Stem Auger		DATUM Local		CHECKED BY SS		DATE August 11, 2008	



TBT Engineering Consulting Group		RECORD OF Borehole No FR08-03		1 OF 1		METRIC	
W.P. 476-00-00		PROJECT Frazer River Bridge		SITE NO. 48C-03B		ORIGINATED BY HF	
DIST 61		LOCATION Stn. 20+246.4 CL - Booth Twp		TBTE JOB# 08-085		COMPILED BY TB	
DATE August 26, 2008		BOREHOLE TYPE Hollow Stem Auger		DATUM Local		CHECKED BY SS	

SOIL PROFILE				ELEVATION SCALE				GROUND WATER CONDITIONS				SAMPLES			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	RESISTANCE PLOT	SHEAR STRENGTH KPa	WATER CONTENT (%)	UNIT WEIGHT	REMARKS	GRAIN SIZE DISTRIBUTION (%)	GR SA	SI	CL	
99.0	SURFACE TREATMENT - 20 mm		1	AS											
	FILL - SAND & GRAVEL - brown, very loose to compact		2	SS	29										
	- occasional cobbles		3	SS	2										
			4	SS	11										
			5	SS	2										
			6	SS	1										
94.4	- organics														
4.6	VOID - drill rod and spoon fell with no resistance from a depth of 4.6m to 6m		7	SS											
6.0	- wood (50mm) FILL - SAND & GRAVEL - brown, very loose		8	SS	4										
91.7	- wood														
7.3	CLAY - Sandy, trace gravel, grey, firm		9	SS	4										
89.3			10	SS	5										
9.7	End of Borehole @ 9.7 m.														

3 \* 3 Numbers refer to Sensitivity Non Plastic

3% STRAIN AT FAILURE

1 27 (72)

Dry on completion.



TBT Engineering Consulting Group		1 OF 1		METRIC	
PROJECT <b>Frazer River Bridge</b>		SITE NO. <b>48C-03B</b>		ORIGINATED BY <b>HF</b>	
LOCATION <b>Stn. 20+300.4 CL - Booth Twp</b>		TBTE JOB# <b>08-085</b>		COMPILED BY <b>TB</b>	
DATE <b>August 26, 2008</b>		BOREHOLE TYPE <b>Hollow Stem Auger</b>		CHECKED BY <b>SS</b>	

W.P. <b>476-00-00</b>		PROJECT <b>Frazer River Bridge</b>		SITE NO. <b>48C-03B</b>		ORIGINATED BY <b>HF</b>	
DIST <b>61</b>		LOCATION <b>Stn. 20+300.4 CL - Booth Twp</b>		TBTE JOB# <b>08-085</b>		COMPILED BY <b>TB</b>	
DATE <b>August 26, 2008</b>		BOREHOLE TYPE <b>Hollow Stem Auger</b>		CHECKED BY <b>SS</b>			

SOIL PROFILE		ELEVATION SCALE		DYNAMIC CONE PENETRATION		RESISTANCE PLOT		SHEAR STRENGTH KPa		SPT (N)		WATER CONTENT (%)		UNIT WEIGHT		REMARKS	
DESCRIPTION		GROUND WATER CONDITIONS		RESISTANCE PLOT		RESISTANCE PLOT		RESISTANCE PLOT		RESISTANCE PLOT		RESISTANCE PLOT		RESISTANCE PLOT		RESISTANCE PLOT	
SURFACE TREATMENT - 20mm		FILL - SAND & GRAVEL to dense		GRAVELLY - trace silt, brown, loose		GRAVELLY - trace silt, brown, loose		GRAVELLY - trace silt, brown, loose		GRAVELLY - trace silt, brown, loose		GRAVELLY - trace silt, brown, loose		GRAVELLY - trace silt, brown, loose		GRAVELLY - trace silt, brown, loose	
- trace wood		- occasional cobbles		- trace wood		- occasional cobbles		- trace wood		- occasional cobbles		- trace wood		- occasional cobbles		- trace wood	
SILT - Sandy, Gravelly, grey		CLAY - Silty, embedded sand, trace organics, grey, stiff		SILT - some sand, trace gravel, very stiff		SILT - some sand, trace gravel, very stiff		SILT - some sand, trace gravel, very stiff		SILT - some sand, trace gravel, very stiff		SILT - some sand, trace gravel, very stiff		SILT - some sand, trace gravel, very stiff		SILT - some sand, trace gravel, very stiff	
11		10		9		8		7		6		5		4		3	
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25		1		31		16		6		6		12		10		4	
11		10		9		8		7		6		5		4		3	
SS		SS		SS		SS		SS		SS		SS		SS		SS	
25		1		31		16		6		6		12		10		4	
11		10		9		8		7		6		5		4		3	
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## APPENDIX B

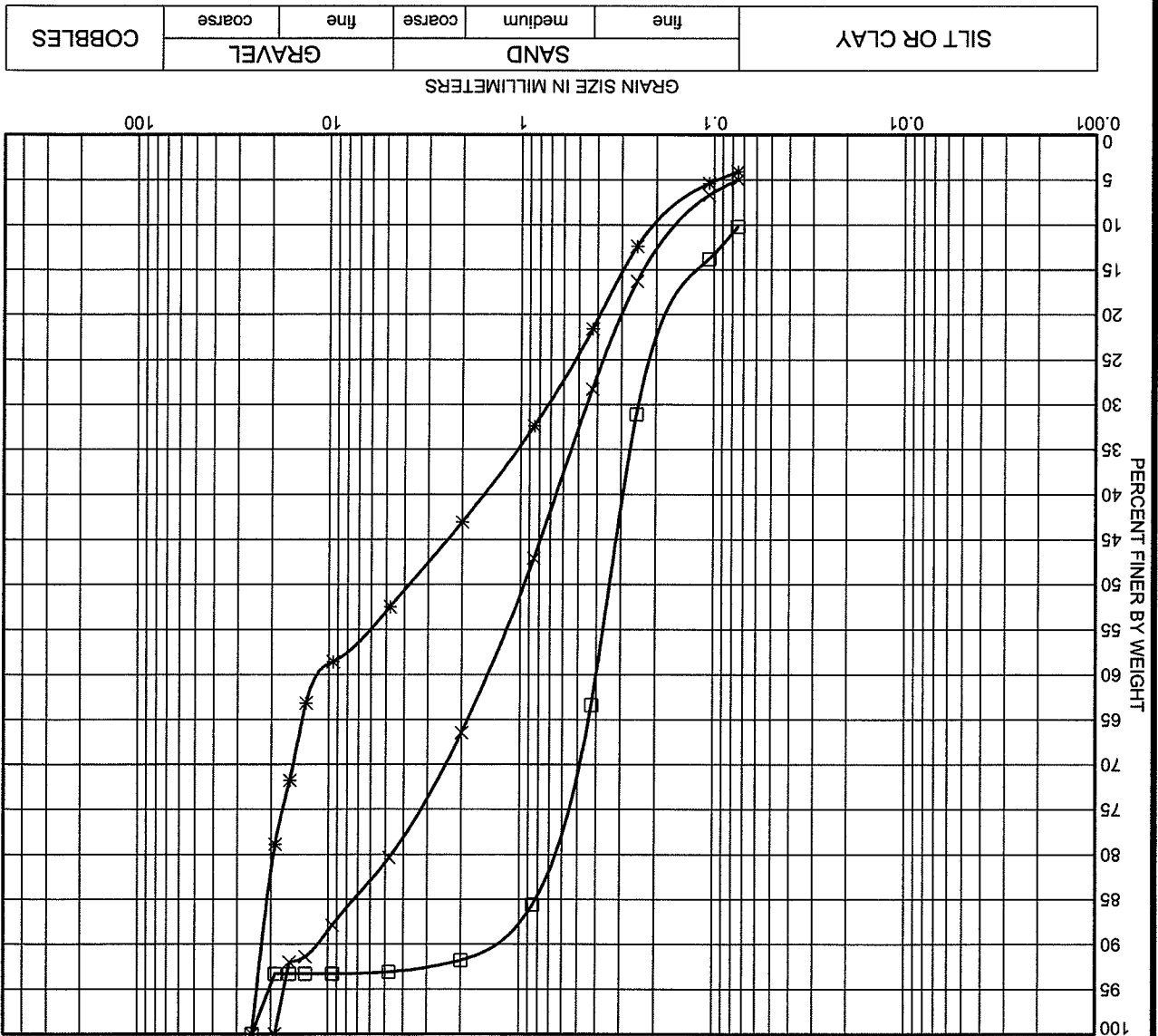
### Laboratory Test Data

TBT Engineering

## GRAIN SIZE DISTRIBUTION

[illegible]

Remarks: FILL





TBT Engineering Consulting Group  
101 Syndicate Avenue North  
Thunder Bay, Ontario P7C 3V4  
PH: 807-624-5160  
FX: 807-624-5161  
Email: tbt@tbt.ca  
Web: www.tbt.ca

Project: Frazer River Bridge  
W P: 476-00-00  
DIST: 61 HWY: 585

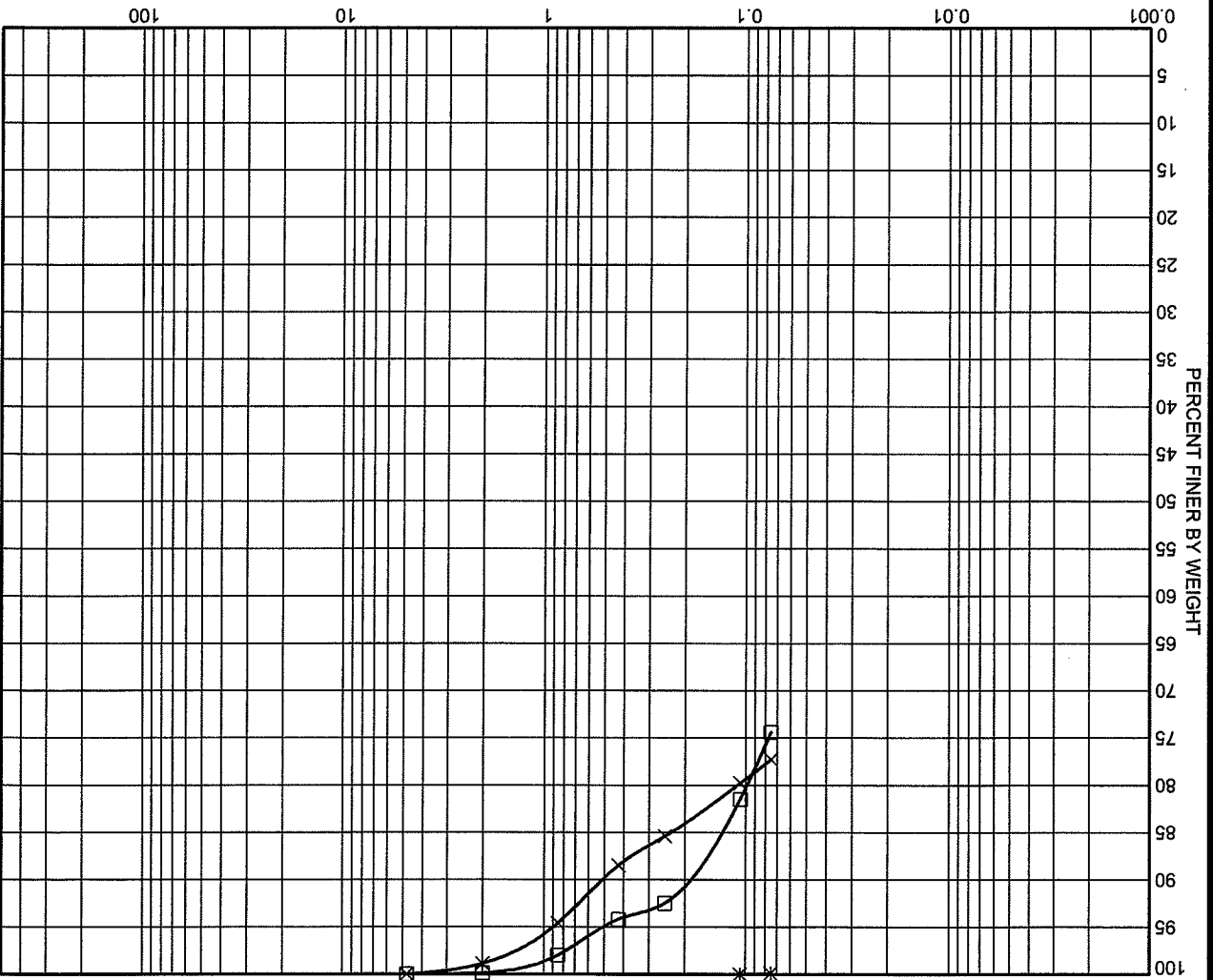
### GRAIN SIZE DISTRIBUTION

Test Hole	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
FR08-01	3.80	4.75				0.0	25.6	74.4	
FR08-01	13.70	0.106				0.0	0.1	99.9	
FR08-02	1.50	4.75				0.0	22.7	77.3	

Remarks: SILT

SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

GRAIN SIZE IN MILLIMETERS





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Thunder Bay, Ontario P7C 3V4  
PH: 807-624-5160  
FX: 807-624-5161  
Email: tbt@tbt.ca  
Web: www.tbt.ca

Project: Frazer River Bridge  
W P: 476-00-00  
DIST: 61 HWY: 585

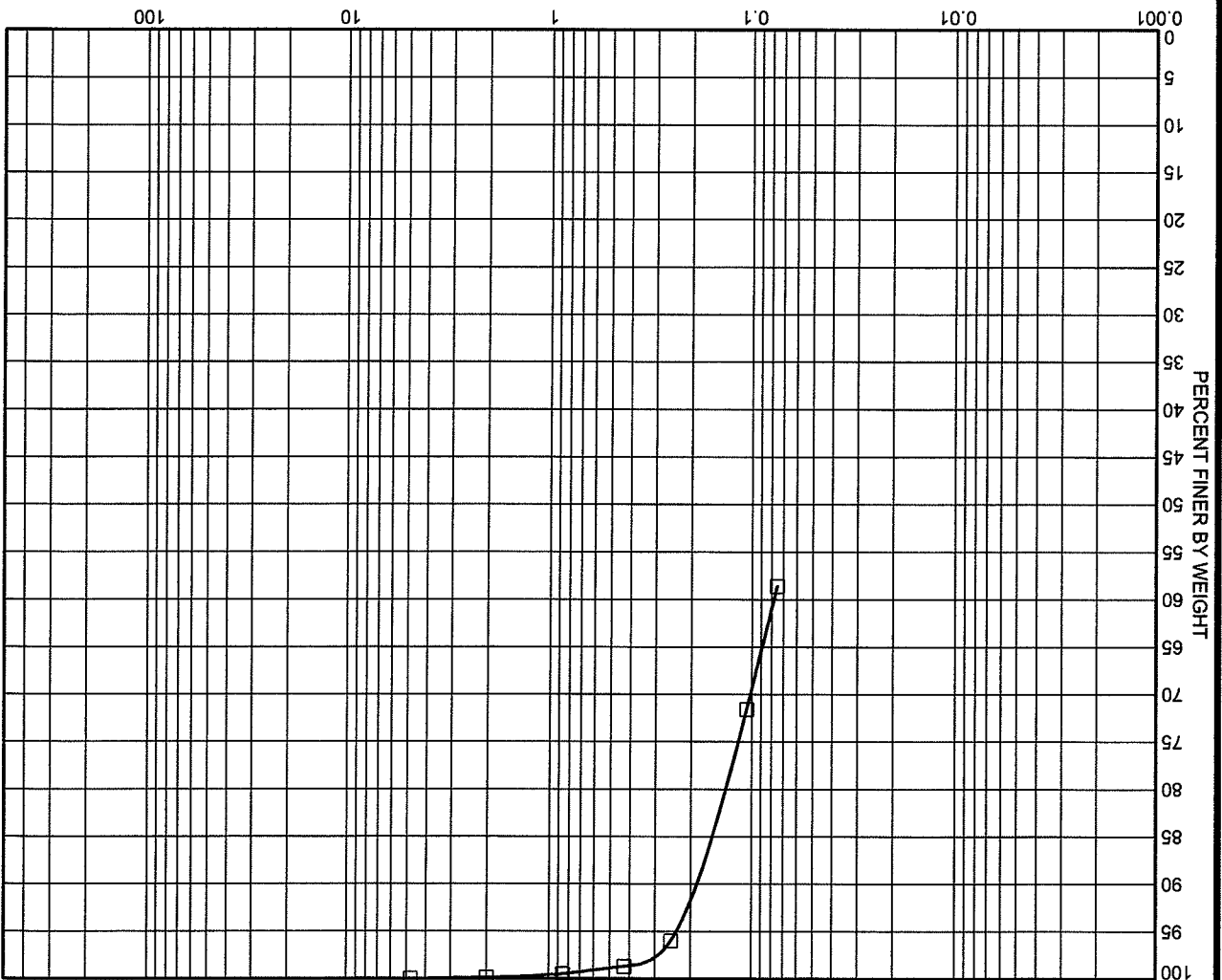
### GRAIN SIZE DISTRIBUTION

Test Hole	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
FR08-01	21.30	4.75	0.078			0.0	41.4		58.6

Remarks:  
SILT & SAND

SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

GRAIN SIZE IN MILLIMETERS





TBT Engineering Consulting Group  
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Thunder Bay, Ontario P7C 3V4  
PH: 807-624-5160  
FX: 807-624-5161  
Email: [tbt@tbt.ca](mailto:tbt@tbt.ca)  
Web: [www.tbt.ca](http://www.tbt.ca)

Project: Frazer River Bridge  
W P: 476-00-00  
DIST: 61 HWY: 585

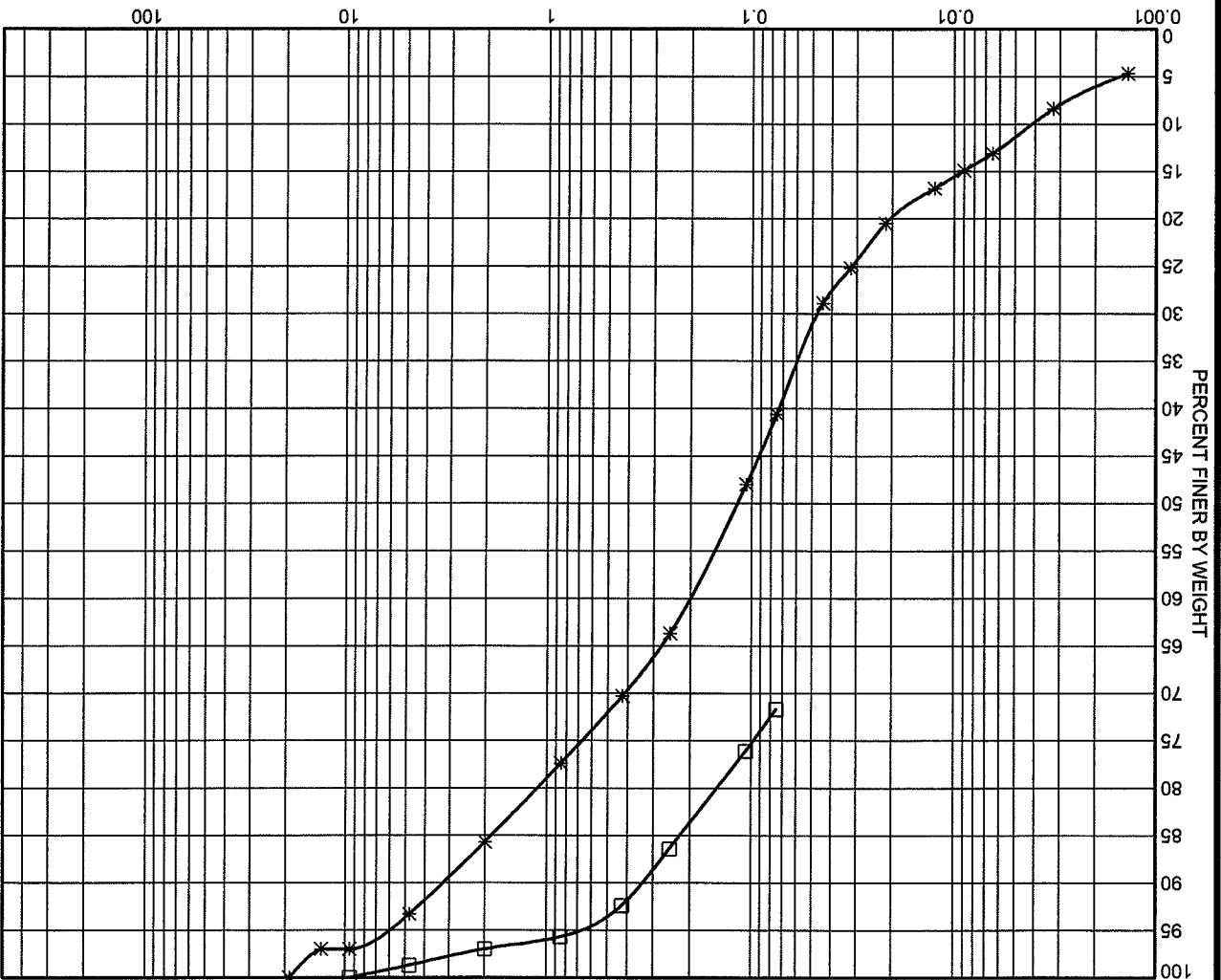
## GRAIN SIZE DISTRIBUTION

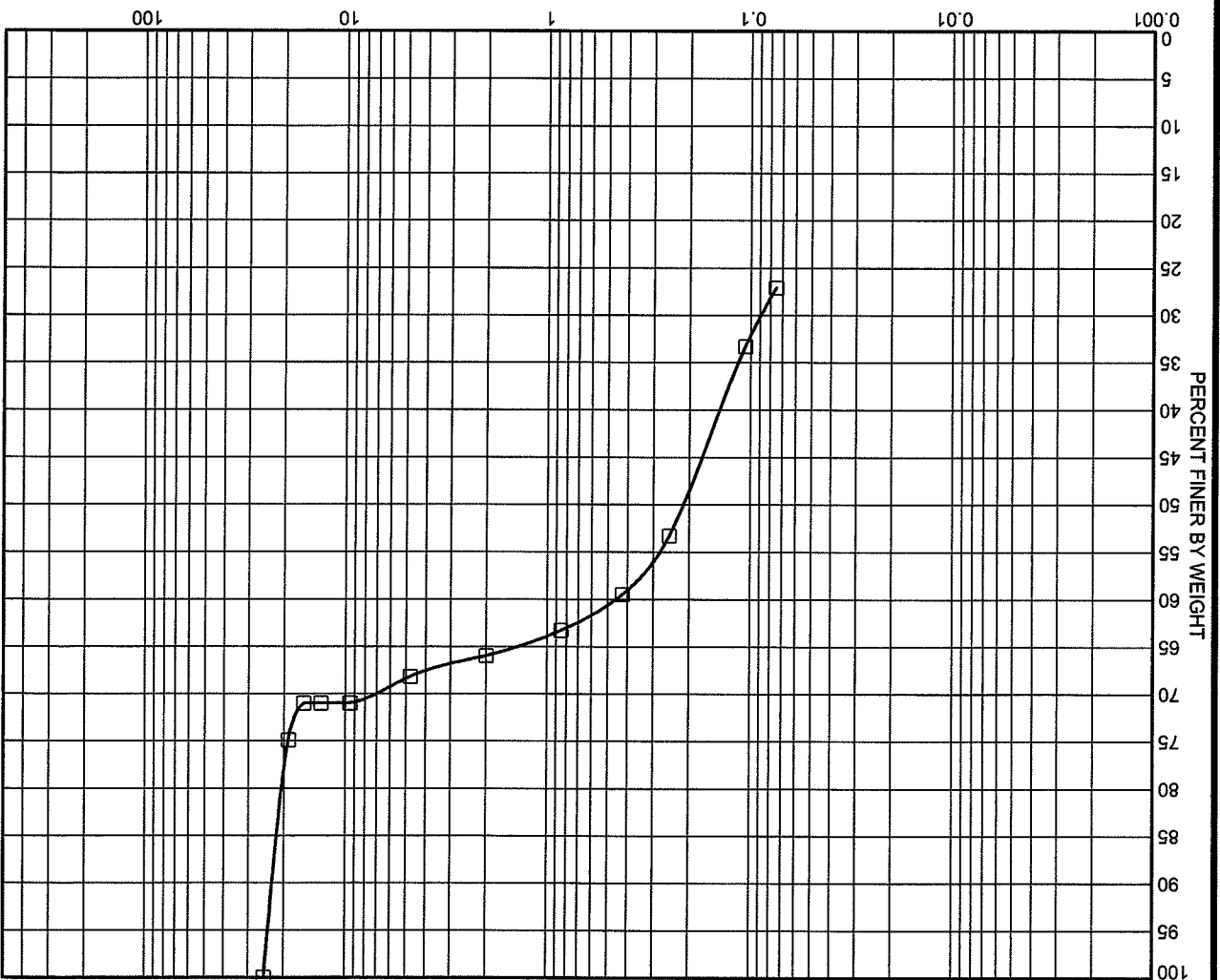
Test Hole	Depth	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
FR08-03	7.60	9.5				1.3	27.0	71.7	
FR08-04	7.60	19	0.204	0.046	0.004	6.7	52.7	40.6	

Remarks:  
CLAY

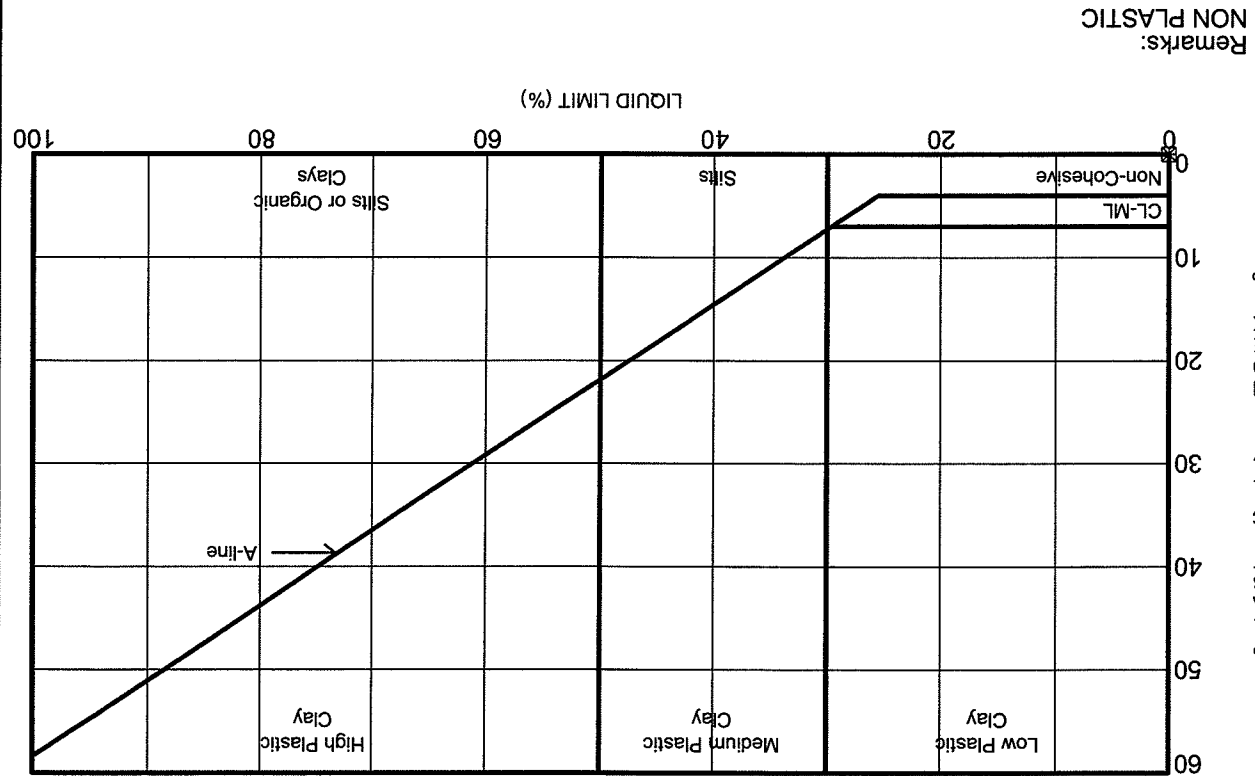
SILT OR CLAY	SAND			GRAVEL		COBBLES
	fine	medium	coarse	fine	coarse	

GRAIN SIZE IN MILLIMETERS



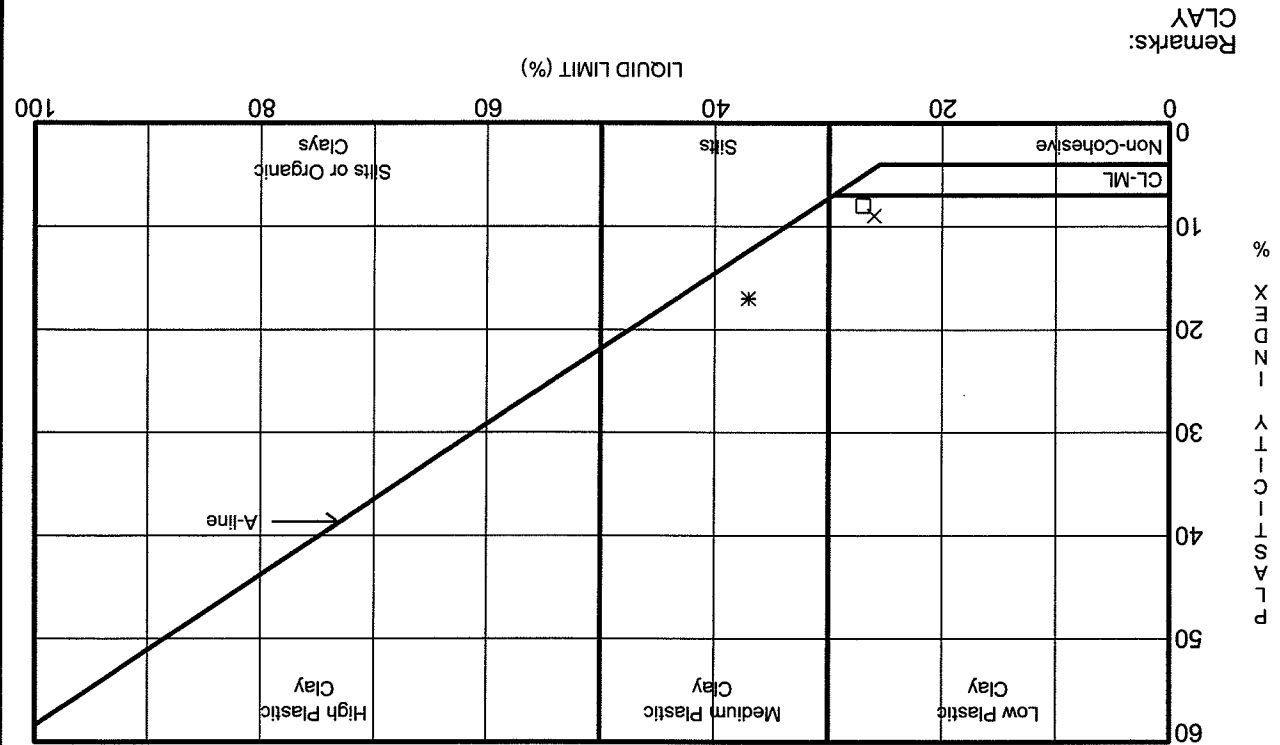


## ATTERBERG LIMIT RESULTS









## APPENDIX C

### Borehole Locations and Soil Strata Drawings

