



DRAFT
Foundation Investigation and Design Report

Atikokan River Tributary Culvert

WP 526-00-00
Site No. 45-268C
Geocres No. xxxx

Highway 11B
2.8 km North of Highway 11

Prepared for
Earth Tech Canada Inc.

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Part A FOUNDATION INVESTIGATION REPORT

1 Introduction

TBT Engineering has been retained by Earth Tech Canada Inc. to provide Foundation Engineering services for the proposed Atikokan River Tributary Culvert replacement. The design work is being undertaken as part of a Total Project Management assignment for Detail Design services between Earth Tech Canada Inc. and the Ministry of Transportation, Northwestern Region.

The site is located on Highway 11B just south of Atikokan, Ontario and is approximately 2.8 km north of Highway 11.

The existing culvert is to be replaced with a new culvert-type structure.

A foundation investigation was carried out to investigate subsurface conditions at the site. This investigation consisted of a number of boreholes drilled in the vicinity of the proposed new structure location, laboratory testing and geotechnical analysis of the data. This report provides a summary of that work and of the conditions encountered.

The foundation section has assigned GEOCRES No. XXXX to this site.

2 Site Description

The site is located on Highway 11B near the south limit of Atikokan, Ontario, approximately 2.8 km north of Highway 11. At this location Highway 11B runs generally in a north-south direction. The Atikokan River Tributary runs in a southwesterly through the culvert. The existing timber culvert is currently almost completely submerged.

A high voltage power line crosses the highway on the south side of the culvert, from a sub-station approximately 100 m to the west of the highway.

The area surrounding the ^{culvert} bridge site is rural covered with irregular bush. The terrain in the immediate surrounding area is undulating, with the Tributary running through a localized low area. The quaternary geology of the site is mapped as glacio-lacustrine deposits, silt and clay with minor, quiet water deposits (Ministry of Northern Development and Mines, Ontario Geological Survey, Map 2554)

The Tributary and its flood plain are about 4 m below the road grade with the current tributary channel being about 4 to 6 m wide at the water level. The water level in the tributary was at approximately 389.1 m elevation at the time of our investigation (November 2002).

The road embankment is about four metres high with fore slopes close to 2h:1v. Highway 11B is a two lane paved section through the culvert site.



Culvert Site, West Wide November 2002



Culvert Site, East Side, Towards North August 2002

3 Investigation Procedures

A site investigation was undertaken between November 20 through 22, 2002. Three boreholes were drilled for the project at the locations indicated on the Borehole Location Plan (Enclosed). The boreholes drilled at this site were numbered as Boreholes 11, 12 and 13 to differentiate from boreholes drilled at another site under the same assignment.

Borehole 11 was drilled north of the tributary, on the west side of the highway. It was located near the toe of the highway embankment slope. Borehole 12 was drilled south of the tributary, on the east side of the highway, partway down the slope of the embankment on a platform cut into the embankment fore-slope to avoid the very soft and wet conditions at the toe of slope. Borehole 13 was drilled through the embankment on the west side of Highway 11A and south of the tributary.

Paddock Drilling was contracted to provide the drilling services. A skidder mounted Acker SX70 drill was utilized. The boreholes were drilled to bedrock using hollow stem augers. Bedrock was cored using NQ diamond core equipment.

Soil samples were obtained with a split spoon sampler as a part of the Standard Penetration Tests (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.

Bedrock samples were stored in core boxes for review with recovery and rock quality designation values determined.

The borehole characteristics and drill techniques utilized are summarized in Table 1.

Table 1. Drill Summary

Location	Surface Elevation	Bedrock (Elevation/Depth)	Bottom of Hole (Elevation/Depth)	Comments
BH 11	389.7	379.7/10.0 m	376.6/13.0 m	NQ Core obtained
BH 12	391.4	377.4/14.0 m	374.1/17.3 m	NQ Core obtained
BH 13	392.1	378.4/13.7 m	378.4/13.7 m	Bedrock Depth inferred from auger refusal

Locations and drawings are referenced to site plans prepared by Engineering Northwest Ltd.

Soil samples were transported to TBT Engineering's laboratory in Thunder Bay for testing. Routine testing included moisture content, grain size analysis and Atterberg Limits. The results of this testing are shown on the Borehole Logs (Appendix A) and on the laboratory data reports (Appendix B).

4 General Site Geology and Sub-Surface Conditions

4.1 Site Geology

The quaternary (surficial) geology of the site is mapped as glacio-lacustrine deposits, silt and clay with minor, quiet water deposits.

The bedrock geology of the area is illustrated on Ontario Geological Survey Map No 2543 and may be summarized as follows. The site is situated in the Quetico sub-province of the Superior province of the Precambrian Canadian Shield. The bedrock types contained in the Quetico sub-province consist predominantly of metamorphosed sedimentary rocks, mainly greywacke with minor amounts of metamorphosed volcanic and intrusive rocks (Williams, 1991).

The Quetico Fault, a regional scale dextral shear zone and fault is located approximately 2-3 km to the north of the site, passing in a general east-west sense through the town of Atikokan.

4.2 Subsurface Conditions

Details of the subsurface conditions are provided on the Borehole Logs, Appendix A, and on the Section Plans, Drawing 1. In general, the native subsurface conditions at the site consist of firm to stiff clays overlying bedrock, which is present near elevations 377 to 380 m, approximately 10 to 12 m below the Tributary level. The embankment fill is approximately 4 m thick and includes both earth and rock fill.

Groundwater levels were similar to the level of the Atikokan River Tributary at the time of the investigation.

4.2.1 Fill

The embankment fill below the pavement structure includes both clay soils as noted at Borehole 12 and granular materials as noted at Borehole 13. Both boreholes drilled through the embankment encountered an approximately one-meter thick zone of rock fill.

Borehole 12, drilled on the side of the embankment, encountered clay fill over rock fill. The clay fill consists of firm to stiff red/brown silty clay. Rock fill was present from about 1.7 m below embankment grade (elevation 389.7) and was about 0.8 m in thickness at the borehole location.

Borehole 13, drilled in the south bound shoulder, encountered 3.0 m of granular fill overlying rock fill. The granular fill varied from silt sand to sand and gravel. Standard Penetration tests indicated the granular fill is in a compact condition. The granular fill is underlain with 1.1 m of rock fill at about 3 m, below grade (Elevation 389.9 to 388.0). The rock fill is in turn underlain with 1.1 m of sand and gravel with cobbles and boulders, which also may be fill material.

4.2.2 Clay

Clay is present from below the base of the fill, or from a thin organic surfacing at Borehole 11, to the bedrock surface.

The clay varies from low to high plastic. At Borehole 11 soil of low plasticity (CL-ML) material are present. Near about 383 to 385 m elevation, the deposit becomes medium to high plastic and frequently has a nuggetty structure, changing gradually into to a varved clay.

Standard Penetration Test blow counts were recorded to be in the range of 3 to 9 blows per 0.3 m. Shear strengths of the clay were consistently in excess of 50 kPa, based on field vane shear tests and field penetrometer tests.

4.2.3 Bedrock

Bedrock was cored at both ends of the culvert, on either side of the Highway (Boreholes 11 & 12). Bedrock was encountered at the site at depths ranging from 10 to 14 m below grade at the boreholes, (Elevation 377.4 to 379.7) or about 10 to 12 m below the Atikokan River Tributary water level.

The bedrock encountered in the boreholes consisted of greywacke of variable degrees of alteration and texture. At Borehole 11 the rock was very weakly altered, with a weak foliation/slatey cleavage and fair to good rock quality (RQD >50%). The top of the core was found to be approximately 45 degrees to the horizontal, indicating a steeply sloping bedrock surface. At Borehole 12 the rock was strongly altered by silicification and sericitization, had a strongly developed foliation or schistosity, and exhibited poor to very poor rock quality (RQD <50%).

Bedrock in this area is known to vary considerably. Bedrock surface elevation, amount of fracturing, bedding and type of bedrock can all change markedly over a small distance.

Table 3 Bedrock Surface at Boreholes

Location	Bedrock Surface Depth (m)	Bedrock Surface Elevation (m)
Borehole 11 West Side	10.0	379.7
Borehole 12 East Side	14.0	377.4
Borehole 13 East (Northbound) Shoulder	13.7	378.4

4.2.4 Ground Water

The ground water levels at the site generally correspond to water levels within the Atikokan River Tributary at the time of the investigation. (Elevation 389.4 m)

Part B Foundation Design Recommendations

5 Discussions and Engineering Recommendations

5.1 Introduction

The Atikokan River Tributary Culvert across Highway 11B, just south of Atikokan is to be replaced under WP 526-00-00. The design project is being carried out under a TPM Agreement between the Ministry of Transportation and Earth Tech Canada Inc. TBT Engineering is providing Foundation Engineering services as a sub-consultant to Earth Tech Canada Inc.

The new culvert is anticipated to be of an open footing Super-Cor type with a span of at least 6 m. The design of the culvert will be conducted in accordance with the Canadian Highway Bridge Design Code [2].

The horizontal and vertical alignment of the Highway will remain essentially unchanged.

A foundation investigation was carried out to investigate subsurface conditions at the site. This investigation consisted of a number of boreholes drilled 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.

5.2 Culvert Foundations

The foundation system for the proposed structure must support the design loads within acceptable settlement tolerances and must accommodate all anticipated loadings. The soil conditions at this site include firm to stiff clays between the invert level and the bedrock surface at a depth about 13 m below the foundation level.

The design configuration for the culvert involves the use of a Super-Cor type, open bottom structure. The TPM Consultant investigated alternate designs during the preliminary design phase with Super-Cor being chosen for environmental, navigation and structural reasons.

It is understood the proposed Super-Cor structures will impose factored vertical lineal foundation loadings in excess of 200 kN/m in part due to the anticipated depth of cover over the culvert.

The use of conventional shallow footings would require very wide footings and could result in uneven settlements at the foundation level. Given the depth to bedrock, a piled foundation is the most practical configuration.

The use of friction piles was reviewed but was not considered practical because of the relatively low capacities available and the relatively shallow bedrock.

Competent bedrock is present at this site at a reasonable depth below the foundation level, therefore steel piles founded in the bedrock are suitable for the support of the foundation loadings. These piles may be conventional driven steel "H" piles. Care will be required during driving of the piles to attain suitable bearing on the sloping bedrock. Alternatively, the piles may be pre-drilled and set into the underlying bedrock after drilling a cased hole into the rock. Bedrock in this area is known to vary markedly over very short distances. The contractor should be prepared to install piles of varying length.

The design frost depth for this project is 2.4 m [3].

5.3 Piling

5.3.1 Driven "H" Piles

Piles driven to effective refusal on bedrock may be designed using the full structural resistance of the pile. Factored axial geotechnical resistance at ULS, as below, are appropriate:

Table 6 - Pile Design Capacities

Pile Designation	Factored Geotechnical Resistance at ULS (KN)
HP 310 x 110	2000
HP 310 x 79	1450

Serviceability limits (SLS) are not given as the bedrock is considered to be unyielding.

Piles should be driven to a set consistent with current MTO criteria and Ontario Provincial Standard Specification (OPSS) 903. Pile set should be verified in the field for the specific pile and pile driver combination in use.

As a preliminary estimate, piles should be driven using a minimum Rated Driving energy of 50 kJ (37,000 ft-lbs) to a "set" of 10 blows/25 mm for the final 30 blows. The piles will be driven into sloping and/or fractured bedrock, and may penetrate a significant distance into the rock to achieve the required capacity. The depth to competent material should be expected to vary and the contractor should be prepared to drive piles of variable lengths.

It is anticipated that effective refusal will be encountered near 379 m elevation, on the north side and near 376.5 m on the south, about 10 to 13 m below the foundations.

The surface of the bedrock was found to slope at about 45 degrees at one of the boreholes. To achieve the set required to achieve the required capacity, the pile will have to be driven in such a manner that the pile is able to obtain a "bite" into the bedrock. Where a hard sloping surface is encountered the pile can slip during driving

resulting in excessive lean. The use of extra piles may be required during piling to counteract excessive lean and/or to provide suitable capacity.

The use of Oslo rock points in accordance with Ontario Provincial Standard Drawing (OPSD) 3304.00 is recommended.

The use of a dynamic pile analyzer is recommended at the beginning of the driving program to accurately measure the stresses in the pile under the particular driver/pile combination being used. The use of the pile analyzer will also allow optimization of the required energy (set) to achieve capacity.

Do we need that?

The elevation of the tops of driven piles should be measured immediately after driving. If uplift occurs in any piles during the driving of adjacent piles, the displaced piles should be re-driven to at least their previous final elevation and final set.

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

5.3.2 Pre-Drilled Piles

To reduce the uncertainty associated with the sloping bedrock, the use of a temporary casing and pre-drilled bedrock socket may be considered. A temporary casing is typically advanced to bedrock, and the bedrock drilled (likely with a down hole hammer) to form a socket in the rock. The hammer is then removed and an "H" pile placed through the casing to the base of the hole in the bedrock. A concrete plug is placed around the "H" pile to the bedrock surface. The remaining void may be filled with sand and the casing withdrawn. The geotechnical pile capacities are equivalent to those achieved by the driven piles. Based on a one metre rock socket depth, factored axial geotechnical resistance at ULS as below are appropriate:

Table 6 - Pile Design Capacities

Pile Designation	Factored Geotechnical Resistance at ULS (KN)
HP 310 x 110	2000
HP 310 x 79	1450

Serviceability limits (SLS) are not given as the bedrock is considered to be unyielding.

Piles should be installed in accordance with current MTO criteria and Ontario Provincial Standard Specification (OPSS) 903.

✓ 5.3.3 Negative Skin Friction

Drag loads caused by negative skin friction will not be induced on the piles for this project since there is no grade raise anticipated for the embankment. Vertical stresses in the soil will be equal to, or less than existing stresses.

5.3.4 Lateral Pile Resistance

Lateral pile resistance may be estimated using the methods provided in the Canadian Foundation Engineering Manual [7], including ultimate lateral resistance and lateral deformation characteristics. The clays below the pile cap level will provide lateral pile resistance at this site. Lateral capacities available may be estimated using the methods provided in the Canadian Foundation Engineering Manual [7], including ultimate lateral resistance and lateral deformation characteristics.

Ultimate lateral resistance may be calculated using Broms' method [4, Section 20.4.1, Fig 20.10] and the following soil parameters:

$$C_u = 40 \text{ kPa}$$

Lateral deformations may be calculated using the methods provided within the Canadian Foundation Engineering Manual [4], or using computer modeling. In either case, a coefficient of lateral sub-grade reaction may be calculated from:

$$K_s = 67c_u/d \text{ kN.m}^3$$

where: K_s = co-efficient of lateral sub-grade reaction per unit volume (kN.m^3)
 c_u = shear strength (use 40 kPa)
 d = pile width (m)

Reductions in estimates of lateral resistance due to group effects for a single row of piles are not required where the pile spacing is at least 2.5 pile diameters [8].

5.4 Soil Backfill

The culvert foundations for the Atikokan River Tributary will be constructed using an open footing Super-Cor type culvert structure.

Design of the culvert is to be carried out by the supplier. Construction shall conform to the supplier's requirements. Where a light weight fill section is not used (see below), fill outside of the Granular A required by the Super-Cor manufacturer should be sloped using a frost taper, with $d=700$ mm and $f=2.4$ m, (similar to frost taper shown on OPSD 3501.000)

Design unit weights for granular backfill as follows are applicable:

Granular A	22 kN/m ³
Granular B (Type 1) modified	21 kN/m ³

Backfill behind the culvert must be free draining. The fill may be specified as Granular B, Type I (Modified).

A frost depth of 2.4 m [3] is appropriate for design purposes.

5.5 Stability – Lightweight fill

The Super-Cor type culvert will be designed by the supplier to support vertical and horizontal loadings above the foundation level. The culvert supplier does not normally carry out global stability design. The inclusion of an open bottom structure within the highway embankment can induce stability failures into the base of the channel. The driving forces of the failure are the weights of the embankment fill. These must be

resisted by the mobilized shear strength of the embankment fill and base materials, or with structural members.

Stability analyses of the culvert configuration indicates the soils at this site have insufficient shear strength to resist the full weight of the embankment fill, and are subject to shear failure, in the form of a rotational failure into the open channel.

Various methods to improve the stability conditions were reviewed, including batter piles, laterally loaded piles, struts and lightweight fill. The use of lightweight fill was found to be the most practical.

The driving force of the failure is due to the weight of the embankment above the channel invert level. A reduction of this weight through the use of lightweight fill can be used to provide a stable configuration at the culvert. Various configurations and layouts were investigated to determine effective stability control.

Stability calculations were carried out using Slope-W software and soil parameters determined by laboratory testing and from published correlations. The stability analysis considered a variety of loading conditions that included:

- Short Term, Un-Drained Conditions
- Long Term, Drained Conditions
- Traffic Loadings
- High Water Levels
- Rapid Draw-Down Conditions

A target factor of safety of 1.3 was utilized. The analysis was carried out using the Morgenstern-Price method and verified with Bishops simplified analyses.

Lightweight Fill

A lightweight fill configuration, as shown on Figure 1 (Appendix C), is suitable for the various loading conditions. Polystyrene foam should be utilized for the lightweight fill. Type 2 or 3 foam (CAN/ULC-S701-97) as provided by Plasti-Fab (or approved equivalent) is suitable. The polystyrene should be placed in accordance with manufacture's recommendations, which include the staggering of joints, longitudinal orientation of blocks and temporary restraint during construction.

5.6 Embankments

The highway alignment and grading are to remain essentially unchanged. No revisions to the embankment section are anticipated. Finished fore-slopes should be constructed to a slope of 2h:1v or flatter.

The embankment loadings at the culvert will be unchanged, or slightly reduced if the culvert opening area is increased. Where lightweight fill is used the embankment loads beyond the culvert will also be reduced. Further settlements due to the embankment loadings are not anticipated, and a camber is not required along the culvert profile.

The face of the slope that borders on or into the tributary should be provided with a 0.6 m thickness of rock protection (OPSS 1004.05.06.02) to the high water level. Elsewhere, the exposed slope should be seeded to provide protection from erosion and surficial sloughing.

The embankment fill used to restore the roadway to the sub-grade level should consist of Granular B, Type 1 (modified), placed in 200 mm lifts and compacted to at least 95% of Standard Proctor Maximum Dry Density (SPMDD). Above this the pavement fill structure should be placed as per the Geotechnical Design Report.

In general, temporary cuts in the existing embankment fills may be trimmed to 1.5h:1v or flatter. Isolated areas of soft or loose materials should be anticipated, and the temporary slopes flattened in these areas. Where surcharge loads (including traffic) are present temporary slopes should be reduced to 2h:1v or flatter.

Both granular fills and rock fill are present near the proposed foundation level. Below the water level, these materials are very pervious and the granular materials subject to sloughing and erosion. Depending on the configuration used for the culvert construction, temporary shoring of the existing embankment may be required.

5.7 Temporary Shoring - De-watering

Temporary shoring, if required, should be designed to resist lateral loadings and to minimise deformations. Temporary shoring may be designed using the methods provided in Canadian Foundation Engineering Manual [7]. The loading configurations will depend on the type of shoring system used. In general, temporary shoring may be designed using the following parameters:

Table 9 - Embankment Soil Parameters (unfactored)

Soil	Active Earth Pressure Co-efficient (Ka)	Passive Earth Pressure Co-efficient (Kp)	Unit Weight (KN/m ³)	Phi (Degrees)
Granular A	0.27	3.7	22	35
Granular B	0.30	3.4	21	33
Clay	0.4	2.5	18	26
Sand Fill	0.31	3.3	20	32

The estimated permeabilities of the soils below the groundwater table vary from 10^{-8} cm/sec in the clay to more than 10 cm/sec in the rock fill. Because of the high permeability values in the coarser soils, conventional de-watering of excavations below the water table using only sumps and pumps is not expected to be successful, particularly where the rock fill is below the water table. Should de-watering be required, the use of sheet piling is anticipated. The piling will have to be driven to sufficient depth to provide toe support, control seepage and will be completely around the area to be dewatered.

5.8 Temporary Road Protection

The culvert will likely be constructed in two portions with traffic routed along one lane of the highway. It is anticipated some form of longitudinal retaining system will be used to support the embankment fill near the centreline of the highway. Details of this staging are normally the responsibility of the contractor. It is anticipated that a form of sheet piling will be used with tie-backs and/or bracing.

5.9 Red Flag Issues

Specific attention should be paid to un-conventional aspects of this project, as previously discussed. The sloping bedrock surface is expected to provide challenging pile driving conditions. Excavations below the water table may be complicated by the presence of a layer of rock fill.

6 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 used in this report are 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 conditions that could affect the type and scope of dewatering procedures, which may be considered, cannot be readily determined from boreholes. This includes 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.

7 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

Prepared by:



Wayne Hurley, P.Eng
Vice-President, Engineering

Reviewed By



Dieter Eigenbrod, P.Eng

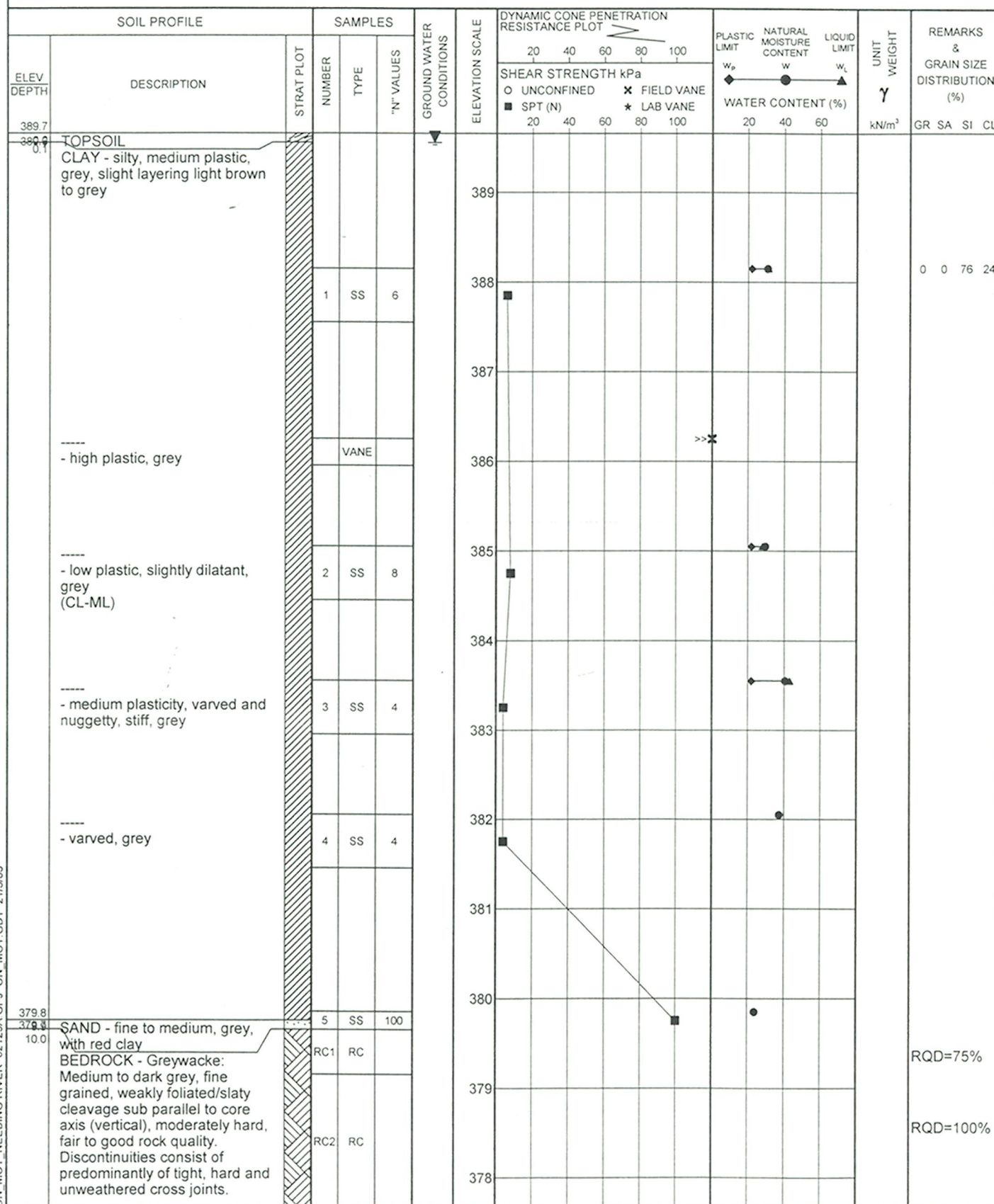
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4. Ontario Ministry of Transportation, Surveys and Design Office, Pavement Design and Rehabilitation Manual, The Queens Printer for Ontario, 1990, ISBN 0-7729-6379-7
5. Canadian Geotechnical Society, Canadian Foundation Engineering Manual, Third Edition, BiTech Publishers Ltd., 1992, ISBN 0-920505-09-0
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APPENDIX A

BOREHOLE LOGS

TBT Engineering
W.P. 526-00-00 PROJECT Atikokan River Tributary Culvert SITE NO. 45-268C ORIGINATED BY WH
DIST 61 HWY 11B LOCATION STA. 12+734 12.8m Lt TBTE JOB# 02-129A COMPILED BY JN
DATE 20 November 2002 BOREHOLE TYPE HS Auger/Core DATUM 389.235 CHECKED BY CZ



ON MOT. NEEDING RIVER 02129A GPJ ON MOT GDT 21/3/03

Continued Next Page

× 3 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

TBT Engineering		RECORD OF BOREHOLE No 11		2 OF 2	METRIC
W.P.	526-00-00	PROJECT	Atikokan River Tributary Culvert	SITE NO.	45-268C
DIST	61	HWY	11B	LOCATION	STA. 12+734 12.8m Lt
DATE	20 November 2002	BOREHOLE TYPE	HS Auger/Core	DATUM	389.235
				ORIGINATED BY	WH
				COMPILED BY	JN
				CHECKED BY	CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			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 (%)				

ON MOT NEEBING RIVER 02129A.GPJ ON MOT.GDT 21/3/03



TBT Engineering

RECORD OF BOREHOLE No 12

1 OF 2

METRIC

W.P. 526-00-00 PROJECT Atikokan River Tributary Culvert SITE NO. 45-268C ORIGINATED BY WH
DIST 61 HWY 11B LOCATION STA. 12+712 9.8m Rt TBTE JOB# 02-129A COMPILED BY JN
DATE 21 November 2002 BOREHOLE TYPE HS Auger/Core DATUM 391.448 CHECKED BY CZ

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			SHEAR STRENGTH kPa						
391.4								20 40 60 80 100						
0.0	FILL - Clay, stiff, red/brown													
			1	SS	25									
	- shot rock													
389.0														
2.4	SAND - silty, grey													
388.3														
3.1	CLAY - silty, trace sand, medium plasticity, firm to stiff, grey		2	SS	4									
	- low plastic, slightly dilatant, stiff (CL-ML)		3	SS	7									
	- nuggetty		4	SS	4									
	- medium to high plasticity, stiff		5	SS	9									
	- varved		6	SS	8									
379.4														

ON MOT NEEDING RIVER 02129A.GPJ ON MOT GDT 21/3/03

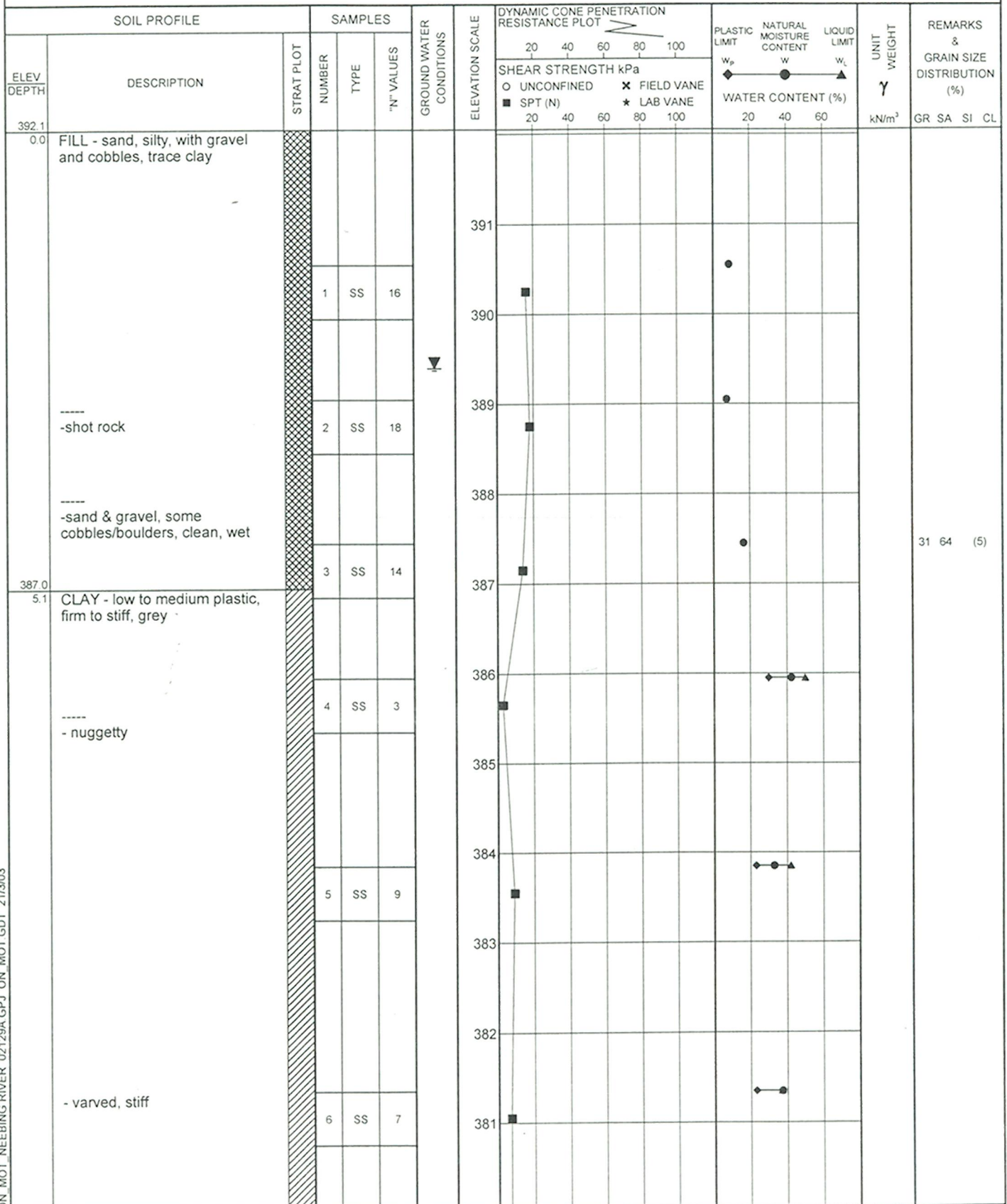
Continued Next Page

× 3 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

TBT Engineering		RECORD OF BOREHOLE No 12				2 OF 2		METRIC						
W.P. <u>526-00-00</u>		PROJECT <u>Atikokan River Tributary Culvert</u>				SITE NO. <u>45-268C</u>		ORIGINATED BY <u>WH</u>						
DIST <u>61</u> HWY <u>11B</u>		LOCATION <u>STA. 12+712 9.8m Rt</u>				TBTE JOB# <u>02-129A</u>		COMPILED BY <u>JN</u>						
DATE <u>21 November 2002</u>		BOREHOLE TYPE <u>HS Auger/Core</u>				DATUM <u>391.448</u>		CHECKED BY <u>CZ</u>						
ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	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
			NUMBER	TYPE	"N" VALUES			20	40					
12.0	CLAY - silty, trace sand, medium to high plasticity, stiff, grey		7	SS	6									
377.4														
14.0	BEDROCK: Greywacke (altered): Light to medium greenish grey, fine grained, strongly foliated to schistose with foliation tightly folded and undulating, generally sub parallel to core axis (vertical), moderately hard Discontinuities consist of cross joints occurring in at least two sets plus random orientations. Fair to poor rock quality.		RC1	RC										RQD=36%
			RC2	RC										RQD=0%
			RC3	RC										RQD=81%
374.1														
17.3	End of Borehole @ 17.3m													

ON MOT. NEEDING RIVER 02129A.GPJ ON MOT.GDT 21/3/03

TBT Engineering		RECORD OF BOREHOLE No 13		1 OF 2	METRIC
W.P. 526-00-00	PROJECT Atikokan River Tributary Culvert	SITE NO. 45-268C	ORIGINATED BY WH		
DIST 61 HWY 11B	LOCATION STA. 12+729 5.4m Lt	TBTE JOB# 02-129A	COMPILED BY JN		
DATE 22 November 2002	BOREHOLE TYPE HS Auger	DATUM 392.051	CHECKED BY CZ		



Continued Next Page

× 3 * 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ON MOT NEEDING RIVER 02129A GPJ ON MOT GDT 21/3/03

TBT Engineering		RECORD OF BOREHOLE No 13		2 OF 2	METRIC
W.P. <u>526-00-00</u>	PROJECT <u>Atikokan River Tributary Culvert</u>	SITE NO. <u>45-268C</u>	ORIGINATED BY <u>WH</u>		
DIST <u>61</u> HWY <u>11B</u>	LOCATION <u>STA. 12+729 5.4m Lt</u>	TBTE JOB# <u>02-129A</u>	COMPILED BY <u>JN</u>		
DATE <u>22 November 2002</u>	BOREHOLE TYPE <u>HS Auger</u>	DATUM <u>392.051</u>	CHECKED BY <u>CZ</u>		

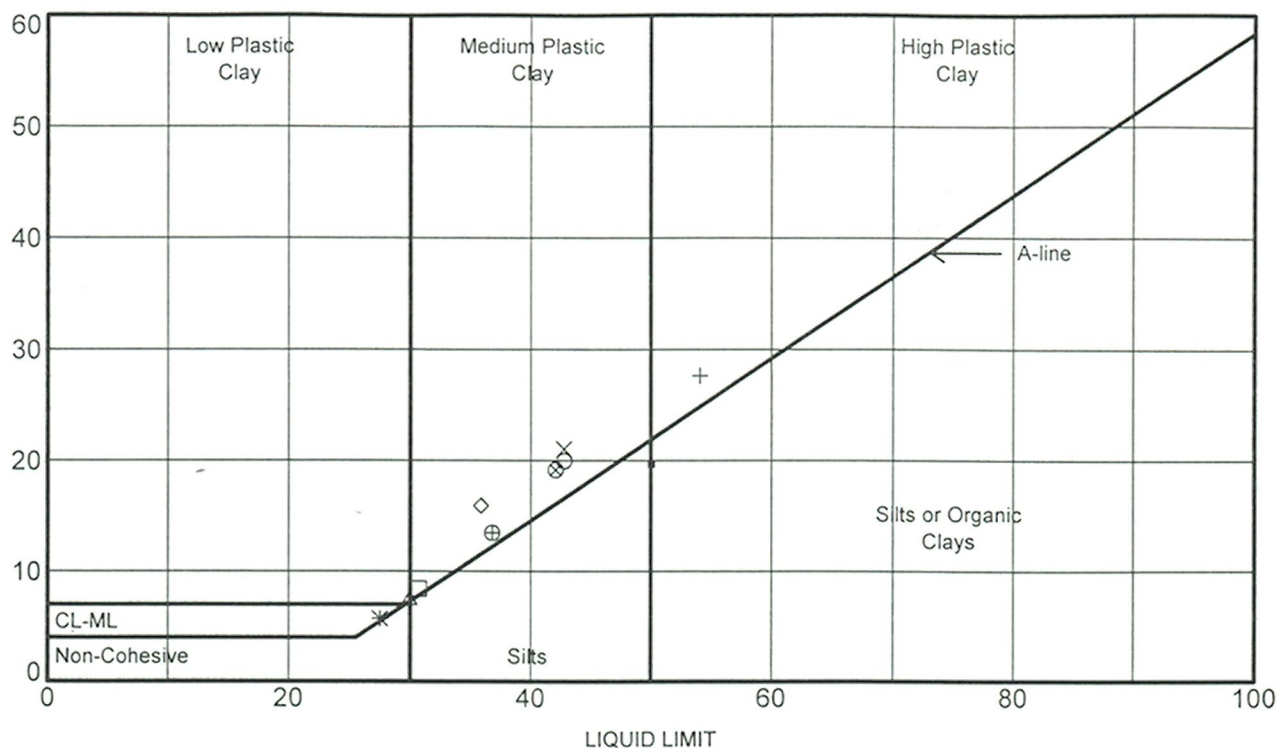
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100	20 40 60 80 100	W _p	W	W _L		
								<div><div></div> UNCONFINED</div> <div><div></div> SPT (N)</div>	<div><div></div> FIELD VANE</div> <div><div></div> LAB VANE</div>					
378.4	- red/grey varved, slightly dilatant		7	SS	7		379	<div><div></div></div>		<div><div></div></div>				0 28 (72)
13.7	End of Borehole @ 13.7m Auger refusal, SPT bouncing.													

ON MOT NEEBING RIVER 02129A.GPJ ON MOT.GDT 21/3/03

APPENDIX B

Laboratory Test Data

PLASTICITY INDEX



Location	Depth (m)	Sample No.	LL	PL	PI	M/C
□ 11	1.50	1	31	22	9	31
✱ 11	4.60	2	28	22	6	30
✕ 11	6.10	3	43	22	21	41
+ 12	0.90		54	26	28	32
◇ 12	3.00	2	36	20	16	26
△ 12	4.60	3	30	22	8	30
○ 12	8.40	5	43	23	20	34
◻ 13	6.10	4	50	30	20	42
⊗ 13	8.20	5	42	23	19	33
⊕ 13	10.70	6	37	23	14	37

TBT ATTERBURG NOTE 02129A GPJ CAN LAB.GDT 21/3/03

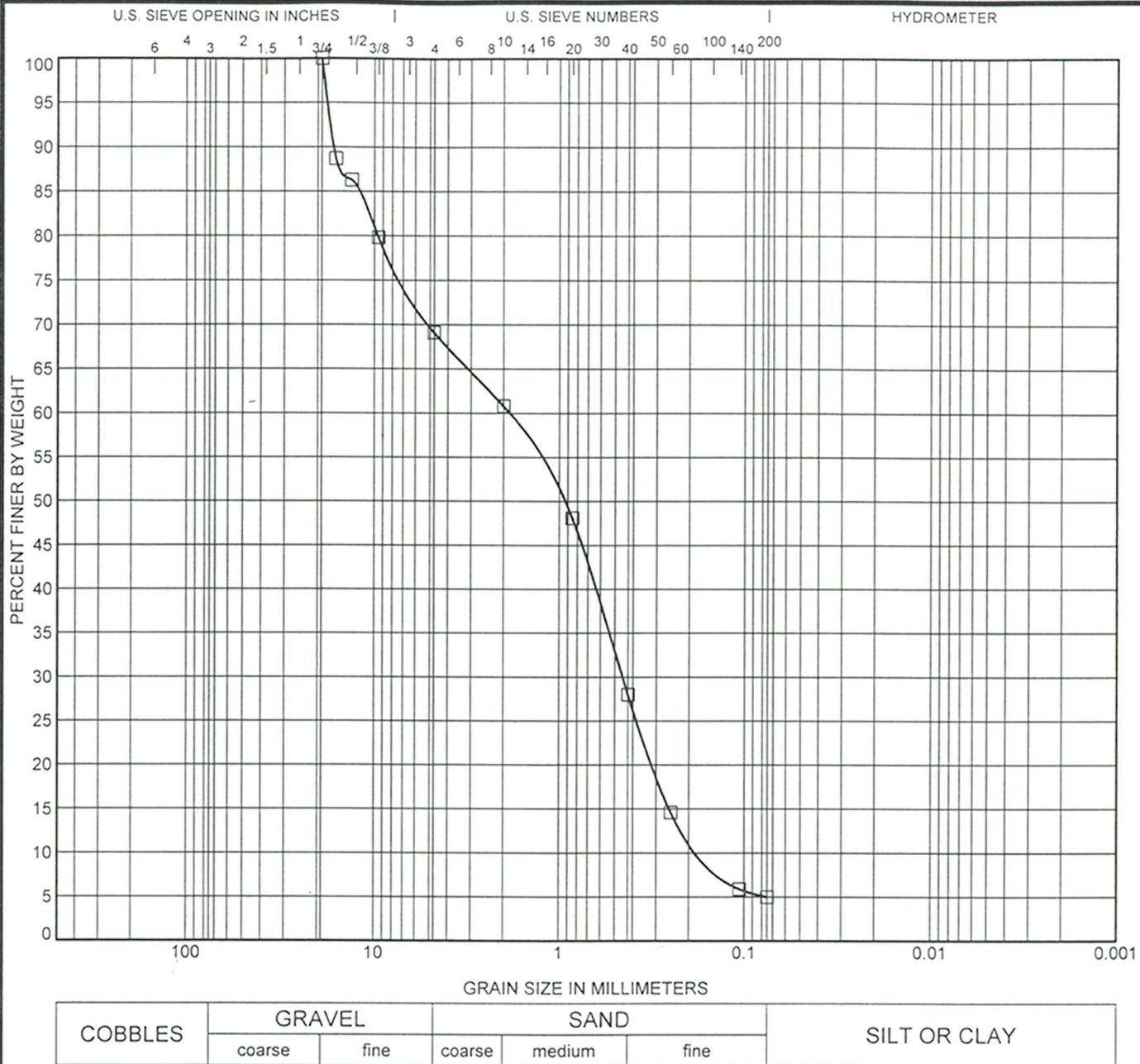


TBT Engineering
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 Telephone: (807) 624-5160
 Fax: (807) 624-5161

ATTERBURG LIMIT RESULTS

Project: Atikokan River Tributary Culvert
 Location: 11B
 Number: 526-00-00

ENCLOSURE 1



Remarks:
SAND FILL

Test Hole	Sample	Depth (m)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
<input checked="" type="checkbox"/> 13	3	4.60	19	1.895	0.455	0.159	30.9	64.1	5.0	



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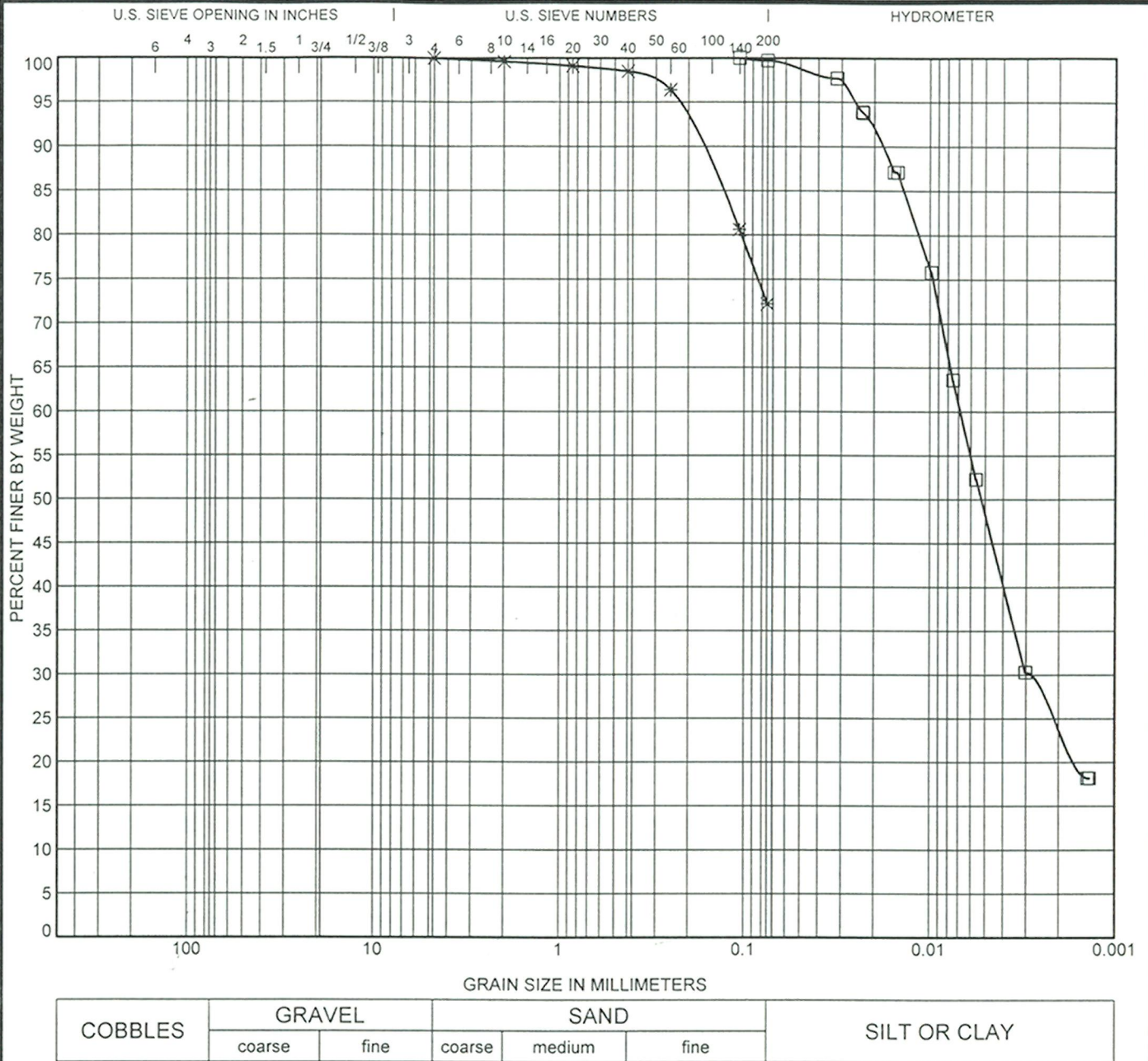
GRAIN SIZE DISTRIBUTION

Project: Atikokan River Tributary Culvert

Location: 11B

Number: 526-00-00

ENCLOSURE 2



Remarks:
SILTY CLAY

Test Hole	Sample	Depth (m)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
□ 11	1	1.50	0.106	0.007	0.003		0.0	0.3	75.8	23.9
* 13	7	13.10	4.75				0.0	27.8	72.2	



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GRAIN SIZE DISTRIBUTION

Project: Atikokan River Tributary Culvert

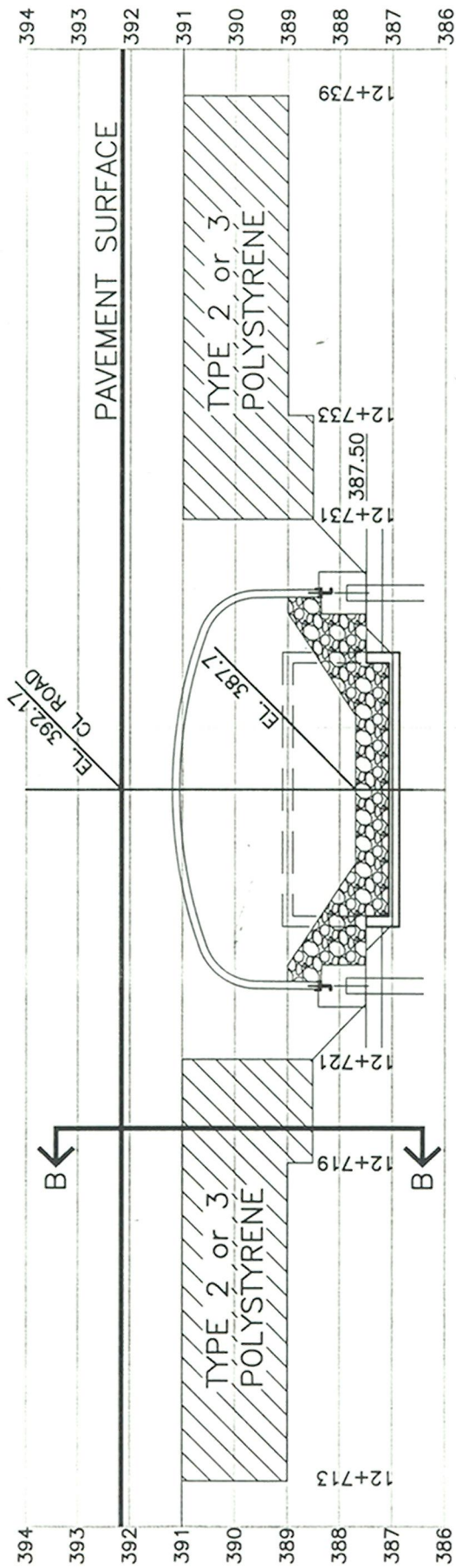
Location: 11B

Number: 526-00-00

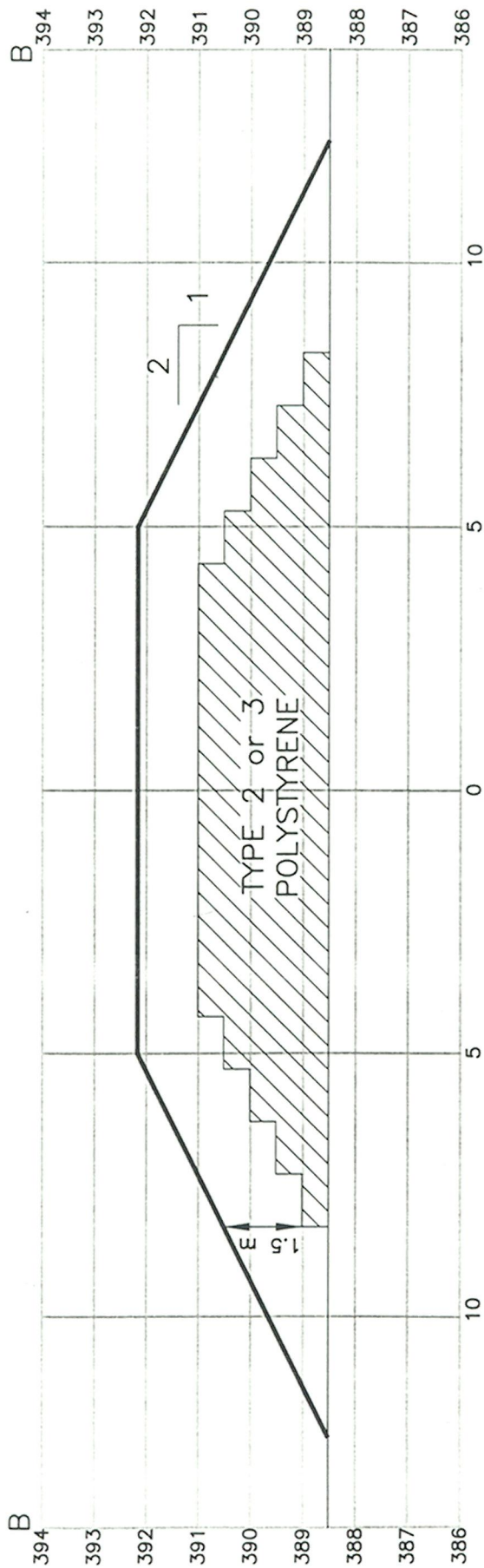
ENCLOSURE 3

APPENDIX C

DRAWINGS



CULVERT PROFILE



CROSS-SECTION B-B



CLIENT:

MINISTRY OF
TRANSPORTATION

CAD REF. NO.:

BASE.DWG

DWG TITLE:

LIGHTWEIGHT FILL DETAIL

PROJECT:

ATIKOKAN RIVER CULVERT REPLACEMENT
HIGHWAY 17

ATIKOKAN

ONTARIO

DATE:

FEB 2003

SCALE:

1:125

PROJECT NO.:

J02-129

ENCLOSURE

1

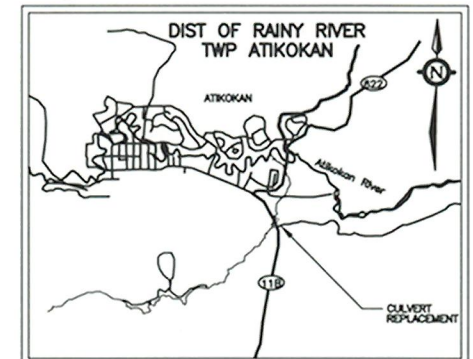
HWY 622

CONT No
WP NO 526-00-00

ATIKOKAN CULVERT REPLACEMENT
HWY 621, Approx. 3.6km North of HWY 11 Jct.
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET
1



KEY PLAN
1.0 km 0 1.0 km

SOIL STRATA SYMBOLS			
	SAND		SILT some sand
	BEDROCK		SILT & SAND
	FILL		GRAVEL
	CLAY		

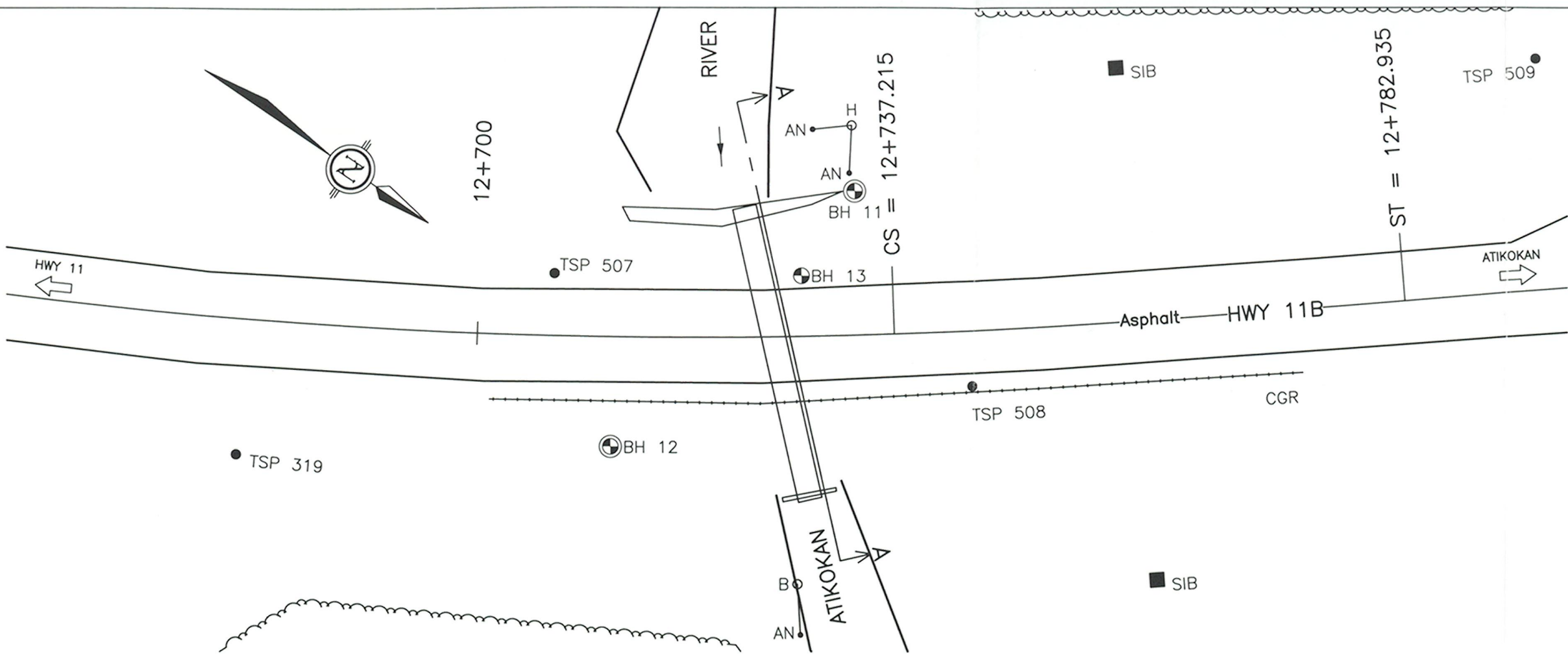
LEGEND			
	Borehole		
	Borehole with Rock Core		
'N'	Std Pen Test (Blows/0.3m)		
	WL at time of investigation Nov 2002		
No	ELEVATION	BOREHOLE LOCATION STA.	O/S
11	389.4	12+734	12.8 LT
12	391.3	12+712	9.8 RT
13	391.9	12+729	5.4 LT

HORIZONTAL DATUM
Assumed

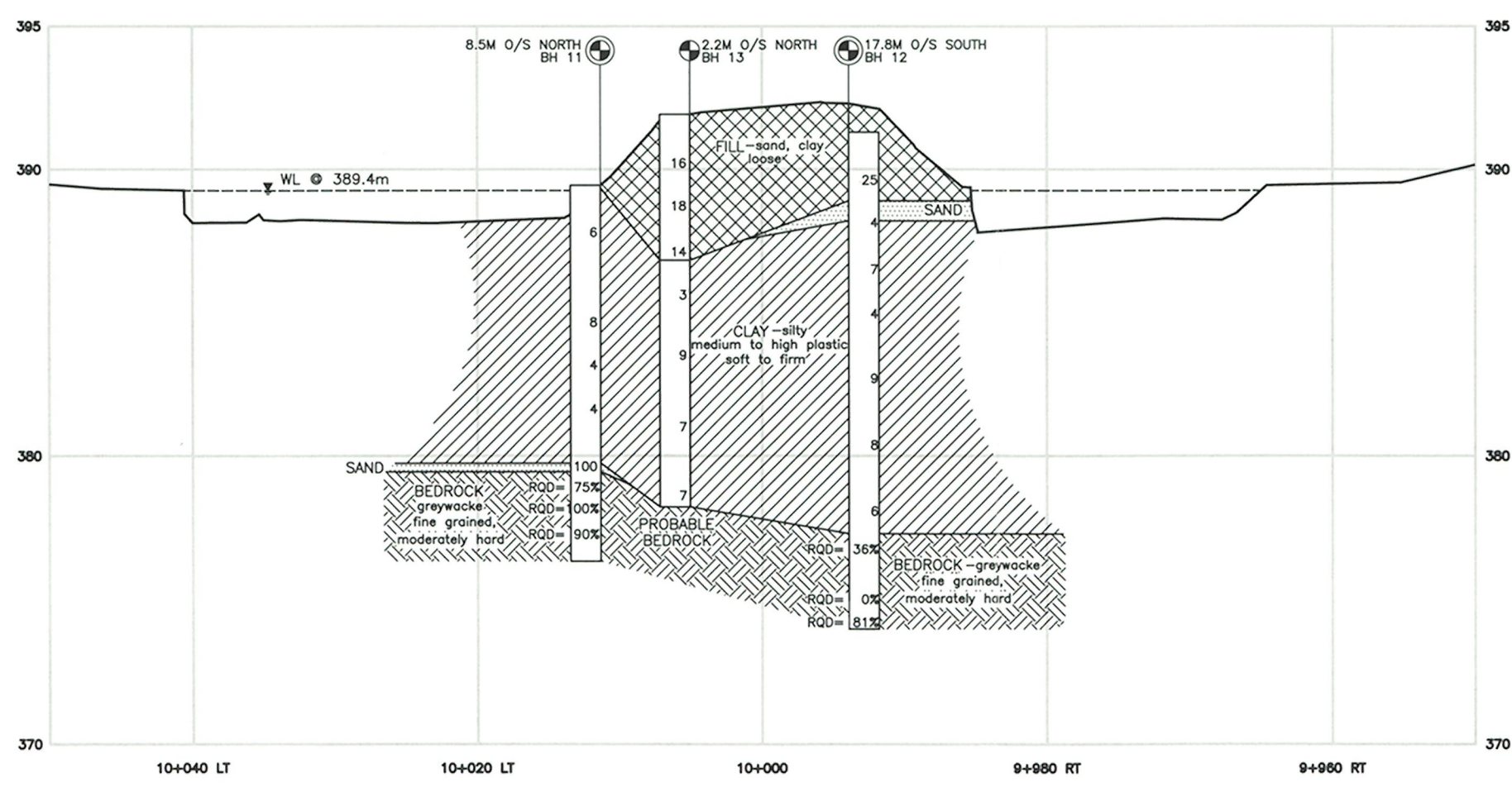
VERTICAL DATUM
Canadian Geodetic Vertical Datum
1928 Adjustment, Geodetic Elevations

NOTE
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS			



PLAN
SCALE
0 1 2 3 4 5 10m



SECTION A-A
SCALE
HOR 0 1 2 3 4 5 10m
VERT 0 1 2 3 4 5m