



THURBER ENGINEERING LTD.

**FOUNDATION REPORT FOR DESKTOP STUDY
HIGHWAY 401 / SUCKER CREEK STRUCTURE
AS PART OF HIGHWAY 401 / COUNTY ROAD 41
INTERCHANGE AND MEDIAN IMPROVEMENTS
GREATER NAPANEE, ONTARIO**

SITE NO. 17-054

G.W.P. 4459-04-00

Latitude: 44.265922

Longitude: -76.961117

Geocres Number: 31C-275

Report to

AECOM

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August 7, 2018
File: 10035

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Memos\Sucker Creek\Desktop Study - Sucker Creek.doc

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents a summary of the factual subsurface information for the existing Highway 401 crossing of Sucker Creek in the geographic Township of Richmond, in the Town of Greater Napanee, Ontario. This work is associated with the rehabilitation and/or widening of the County Road 41 interchange and structures (EBL and WBL).

The purpose of this desktop study report is to summarize currently available subsurface information pertinent to the foundation aspects of the proposed structural rehabilitation works. The information includes previous foundation reports, preliminary design report, preliminary staging report, general arrangement and foundation layout drawings available from the Ministry of Transportation Ontario (MTO), geological maps, and a site reconnaissance visit. It also presents preliminary geotechnical recommendations for use in assessment of the existing foundations and for preliminary design at the site. It is noted that the proposed structural alternatives are not yet defined. Additional investigation and analysis may be required in any subsequent detail design phase of the project.

Thurber carried out this desktop study as a sub-consultant to AECOM under the MTO Consultant Assignment Number 4015-E-0003.

2 SITE DESCRIPTION

2.1 General

The Sucker Creek structure is located approximately 2.1 km north of Napanee and 0.3 km east of the existing County Road 41 overpass structure. At this location, the existing Sucker Creek Bridge consists of a single span rigid frame structure with a total span length



of about 15.9 m perpendicular to the creek and a width of about 46.6 m perpendicular to Highway 401. The structure was originally constructed in 1959 and widened in 2004 to accommodate a new E-N Ramp.

The structure accommodates 4 traffic lanes (2 westbound and 2 eastbound lanes), 1 eastbound speed change lane, one westbound speed change lane on the E-N ramp structure, a paved median and paved shoulders on Highway 401 over Sucker Creek. Select photographs of the structure are included in Appendix C.

2.2 Geology

The project area is situated within the physiographic region known as the Napanee Plain. The Napanee Plain is characterized by a thin veneer of glacial till underlain at relatively shallow depths by limestone bedrock of the Bobcaygeon Formation. Thick glacial sediments are present in the deep river and stream valleys in the region.

2.3 Topography and Land Use

The Highway 401 corridor addressed in this project generally runs in an east to west orientation along relatively flat terrain. There are commercial and institutional developments in the vicinity of the structure. These are concentrated to the south of Highway 401 with less intensive development to the north. Sucker Creek meanders in the general north east to south west direction at the bridge location. It is noted that Sucker Creek was re-aligned to accommodate the construction of the Highway 401 structure.

3 STUDY PRODECURES

The desktop study is based on geotechnical data gathered from available sources with no borehole drilling and sampling in this phase of the work.

Information on existing surface and subsurface conditions relevant to the foundations of the existing structures and embankments have been collected from the following sources:

- Review of existing foundation investigation and design reports for the structure available from the MTO GEOCREs system, and selected information from archived contract files.
- Review of contract drawings, a Preliminary Design Report and Preliminary Staging Report for the project site.
- Review of published geological information for the study area.
- Site reconnaissance visit by the Thurber project engineer to observe and document the existing structures, cuts, embankments and any visible geological/geotechnical features.

Imperial units in the original GEOCREs files have been converted to metric.

3.1 MTO GEOCRES Files

Existing foundation/geotechnical information relevant to the subject sites has been obtained from the MTO GEOCRES library. The documents used for the desktop study are listed below, and are included in Appendix A:

Reference 1 (GEOCRES 31C00-061): Highway 401 Line 'C' & Selby Creek Diversion Crossing, Lot 22, Concession II, Twp. Of Richmond, Approx. 1 Mile North of Napanee, W.P. 29-59, W.J. F-59-25, dated June 1959.

Reference 2 (GEOCRES 31C00-165): Foundation Investigation and Design, Bridge Structure for E-N Ramp over Sucker Creek, Detail Design – Short-Term Intersection Improvements at Highway 401 and County Road 41. G.W.P. 331-99-0, Agreement No. 4005-A-000206. Town of Greater Napanee, Eastern Region, dated April 2003.

3.2 Contract Drawings

Contract drawings and quantity sheets were prepared by the MTO Eastern Region for the Highway 401 and County Road 41 Interchange in 2004 that included widening of the Sucker Creek Bridge to accommodate a speed change lane for the proposed E-N Ramp.

3.3 Preliminary Design Report

A Preliminary Design Report was prepared by the MTO Eastern Region for the Highway 401 interchange at County Road 41 (W.P. 31-99-00) in April 2001. Appendix G in the Preliminary Design Report included a Structural Design Report with accompanying General Arrangement Drawings for the Sucker Creek Bridge structure prepared by Cole Sherman in October 2000.

3.4 Preliminary Staging Report

A Preliminary Staging Report for the structure rehabilitation of Highway 401 at County Road 41 and Sucker Creek Bridge was prepared by the Ainley Group in September 2011.

3.5 Geological Information

During the preparation of this report, reference was made to Chapman and Putnam, "The Physiography of Southern Ontario", Third Edition, Ontario Geological Survey, Special Volume 2, Ministry of Natural Resources, 1984.

3.6 Site Reconnaissance Visit

A site reconnaissance visit was carried out by Thurber's project engineer during the preparation of this report. The site was visited and documented for visible

geological/geotechnical features and for assessing structure, cut and embankment performance.

The sight inspection revealed no visible bedrock outcrops.

No distress was noted that would suggest a problem with the foundations of the structure.

Visually, the side slopes of approach fills on the EB lanes appeared to be flatter than 2H:1V and appeared to be stable, though there was some evidence of historic erosion or minor settlement. There are a number of animal burrows in forward slopes at the wing walls on the south side of the structure, see Photo 2 at the end of the report. These burrows gradually undermine the slope and lead to accelerated erosion of the slope.

The structure was extended to the north in 2004 to accommodate a new E-N Ramp. Accordingly the north side of the structure is in very good condition and there is no evidence of foundation issues, see Photo 3. The side slopes and forward slopes of the WB approach fills are protected by rock sheeting, visible in Photos 3 and 5. Visually these slopes appear to be steeper than 2H:1V. The slopes are in good condition, with no evidence of instability or erosion. The toe ditch along the E-N Ramp is also lined with rock sheeting, see Photo 6.

4 DESCRIPTIONS OF SUBSURFACE CONDITIONS

A foundation investigation report was completed by the MTO Department of Highways Materials and Research Section in January 1959 (Reference 1) for the Highway 401 bridge structure over Sucker Creek. A subsequent foundation investigation report was prepared by Golder Associates in April 2003 (Reference 2) for the Highway 401 E-N Ramp structure over Sucker Creek. The Golder report makes reference to the MTO report as part of their investigation. A summary of both field investigations is summarized in the sections below.

4.1 Highway 401 Bridge Structure over Sucker Creek

The field investigation outlined in Reference 1 consisted of 1 borehole (labelled 4) with an adjacent dynamic cone penetration test (DCPT) and three separate DCPTs advanced for the construction of the existing Highway 401 bridge structure over Sucker Creek. The report stated that no sampling was conducted in the overburden due to the shallow bedrock depth. Bedrock was confirmed below the overburden by coring in one of the boreholes from a depth of 1.1 m to a depth of 2.7 m below the original ground surface (Elevations 92.1 to 90.6 m). The DCPTs were terminated upon refusal on probable bedrock at depths ranging from 1.1 to 1.5 m below the existing ground surface (Elevations 91.8 to 91.6 m).

The site was originally overlain by a layer of alluvial topsoil that ranged in thickness from 0.5 to 1.2 m.

With the exception of the south west borehole, the topsoil was underlain by an alluvium deposit of sand and gravel with some clay. The thickness of the deposit ranged from 0.6 to 0.9 m, with the base of the layer lying at Elevations 92.1 to 91.6 m.

Bedrock was encountered below the topsoil in the south west quadrant of the site at a depth of 1.2 m (Elevation 91.8 m), and underlying the sand and gravel in the remaining boreholes at depths ranging from 1.1 to 1.5 m below the existing ground surface (Elevations. 92.1 to 91.6 m). The original investigation described the bedrock as fine-grained limestone in a “very sound” condition with no sign of fracture or weathering.

Although not specified in the borehole logs, the report stated that the water table was encountered at the ground surface at approximate Elevation 93.3 m. This was a short term observation made prior to the formation of existing approach cuts for this structure, and therefore likely did not represent stabilized groundwater conditions.

4.2 Highway 401 Bridge Structure for E-N Ramp over Sucker Creek

The field investigation outlined in Reference 2 consisted of 4 sampled boreholes (labelled 7 to 10) advanced to the north of the Highway 401 bridge over Sucker Creek. The four boreholes were drilled and sampled through the overburden in conjunction with Standard Penetration Tests (SPTs) to depths ranging from 0.3 to 0.8 m below the original ground surface (Elevations 92.7 to 92.5 m). Bedrock was confirmed below the soil sampling depth by coring in two of the boreholes to depths ranging from 3.5 to 3.8 m below the original ground surface (Elevation 89.5 m). The remaining two boreholes were terminated upon refusal on probable bedrock at depths ranging from 0.6 to 0.8 m below the existing ground surface (Elevation 92.7 m).

Silty clay till was encountered from the ground surface to the west of Sucker Creek. The thickness of the till ranged from 0.3 to 0.6 m, with the base of the layer encountered at Elevation 92.7 m. The till was brown to grey in colour and contained trace to some sand and gravel. SPT ‘N’ values in the till ranged from 7 to 9 blows per 0.3 m penetration indicating a firm to stiff consistency.

Silty clay with topsoil was encountered from the ground surface to the east of Sucker Creek. The thickness of the silty clay was 0.8 m and had a base elevation that ranged from 92.7 to 92.5 m. The silty clay was brown in colour. SPT ‘N’ values in the silty clay ranged from 5 to 9 blows per 0.3 m penetration indicating a firm to stiff consistency. Higher ‘N’ values were encountered in this layer directly overlying bedrock and are not representative of the layer.

Bedrock was encountered below the silty clay with topsoil to the east of Sucker Creek at a depth of 0.8 m (Elevations 92.7 to 92.5 m) and encountered underlying the silty clay till on

the west side of Sucker Creek at a depths ranging from 0.3 to 0.6 m (Elevation 92.7 m). Where cored on both sides of the creek, the bedrock was described as fresh, massive, medium to fine crystalline, grey, strong to very strong limestone with interbeds of finely laminated, dark grey to black shaley limestone.

With the exception of a 0.3 m thick zone of highly fractured rock at a depth of 1.7 m below the rock surface near the creek, the fracture index ranged from 1 to 3 fractures per 0.3 m. The measured Rock Quality Designation (RQD) ranged from 42 to 85% indicating poor to good quality.

Strength testing carried out on the rock yielded unconfined compressive strength (UCS) values ranging from 105 to 125 MPa indicating very strong rock. The strength of the rock generally increased with depth.

All boreholes with the exception of the borehole to the immediate west of Sucker Creek was dry upon completion of drilling. The base of the remaining borehole was observed as wet prior to coring bedrock. A piezometer was sealed in the bedrock at this borehole location. The water level was recorded as frozen 0.2 m above the ground surface 5 weeks after completion of drilling (Elevation 93.2 m) and recorded as frozen at the ground surface 3 months after completion of drilling (Elevation 93 m).

The borehole location and strata drawing derived from a GA provided to Golder Associates stated that the normal water level in Sucker Creek is at Elevation 92.6 m with a high water level at Elevation 94.0 m.

It is anticipated that the water level at the site is at or just below the ground surface and is influenced by the creek level. These are short term observations made prior to the formation of existing approach cuts for this structure, and therefore likely did not represent stabilized groundwater conditions.

5 EXISTING FOUNDATIONS

5.1 Highway 401 Bridge Structure over Sucker Creek

The foundation investigation report completed by the MTO Department of Highways Materials and Research Section in January 1959 (Reference 1) recommended that the Highway 401 structure over Sucker Creek should be supported by spread footings 0.6 m into bedrock (Elevations 92.0 to 91.7).

Based on the original construction drawings, all of the spread footings for the structure were constructed at a founding Elevation of 91.3 m.

At this elevation, the footings bear on the bedrock just below the bedrock surface and it is assumed that the design bearing pressure did not exceed 1,000 kPa.

The bearing capacities given in working stress analysis are analogous to the SLS reaction given in Limit State Design.

5.2 Highway 401 Bridge Structure for E-N Ramp over Sucker Creek

The foundation investigation report completed by Golder Associates in April 2003 (Reference 2) recommended that the Highway 401 E-N Ramp structure over Sucker Creek should be supported by spread footings directly on the bedrock surface.

Based on the original construction drawings, all of the spread footings for the structure were constructed at a founding Elevation of 92.4 m.

Where the bedrock elevation was higher than the specified founding elevation, the bedrock was excavated to accommodate the footing. Where the bedrock depth was lower than the required founding elevation, fill concrete was used to raise the founding elevation to accommodate the footing.

A factored Ultimate Limit States (ULS) value of 3,000 kPa was given for footings supported on or within bedrock, or founded on concrete with a strength of at least 25 MPa.

6 FUTURE ANALYSIS OF EXISTING FOUNDATIONS

Based on the subsurface stratigraphy provided in the foundation investigation reports and using present day accepted methods of foundation analysis, the following values may be used in assessing the structure:

Structure	Footing	Assumed Founding Elevation (m)	SLS (kPa)	Factored ULS (kPa)
Highway 401	East	91.3	*	1,000
	West	91.3	*	1,000
Highway 401 N-E Ramp	East	92.4	*	1,000
	West	92.4	*	1,000

* The bedrock may be considered to be an unyielding stratum and the SLS condition will not govern foundation design.

It is recommended that evaluation of the footings on bedrock be based on a factored ULS resistance of 1,000 kPa because the founding elevations are very close to the bedrock surface and within a depth where weathering effects may be more pronounced.

6.1 Structure Rehabilitation

In general, if the load demands of the rehabilitated structure can be met by the geotechnical resistances shown above, no further foundation analysis is required. However, if the factored ULS resistance or SLS reaction given above are exceeded as a result of the rehabilitation, supplementary site investigation and analysis will be required.

6.2 Structure Widening

For widening of the structure, it is recommended that the new construction be supported on spread footings bearing on bedrock. If the new footings will be close to the bedrock surface, as in the existing case, they may be designed on the basis of a factored ULS geotechnical resistance of 1,000 kN and the SLS condition may be assumed not to govern. Higher bearing resistances, typically in the order of 2,000 kPa could be used for footings founded on proven sound bedrock.

6.3 Embankment Widening

If embankment widening is considered, then for preliminary design purposes the widening may be considered to be stable at side slopes of 2H:1V. The shallow depth to bedrock also will restrict settlement under the widening. New construction must be keyed into the existing embankment slope in accordance with OPSD 208.010.

It is recommended that any widening or rehabilitation of the embankment address the issue of burrowing animals. This could be achieved by facing the affected areas of the slopes with rock sheeting similar to that used for the E-N Ramp.

7 FUTURE WORK

Supplementary geotechnical investigation and analysis may be required in the following situations:

- Widening of the structure to further define the bedrock surface.
- Construction activities, including rehabilitation that will require roadway protection to be designed.

8 MISCELLANEOUS

Ms. Deanna Pizycki, E.I.T. and Mr. Alastair Gorman, P.Eng. prepared the Desktop Foundation Investigation Report. Dr. P.K. Chatterji, P.Eng. a Designated Principal Contact for MTO Foundations projects, reviewed the report.

THURBER ENGINEERING LTD.

Deanna Pizycki
Geotechnical Engineer-in-Training



Alastair Gorman, P.Eng.
Senior Associate / Senior Geotechnical Engineer



P.K. Chatterji, Ph.D., P.Eng.
Review Principal, Designated MTO Contact

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or 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 Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together 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. THURBER IS NOT 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 purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

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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 judgmental in nature. 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 the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. 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. If 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.
- 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 Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

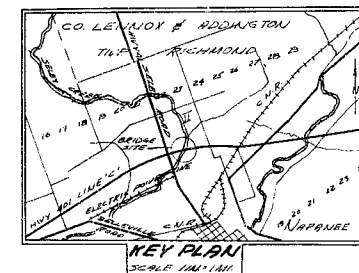
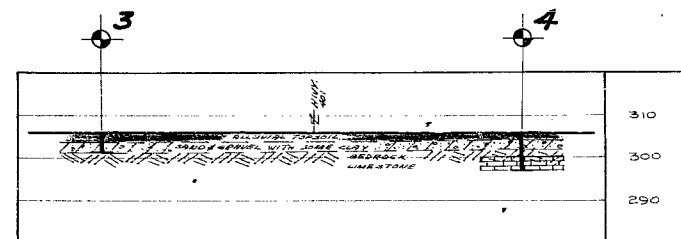
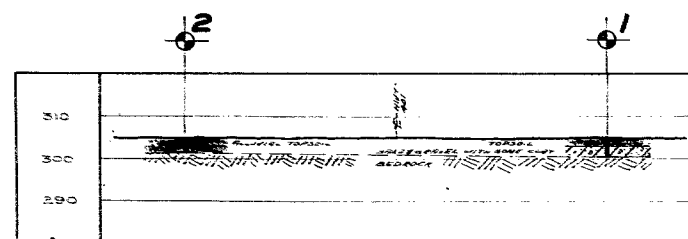
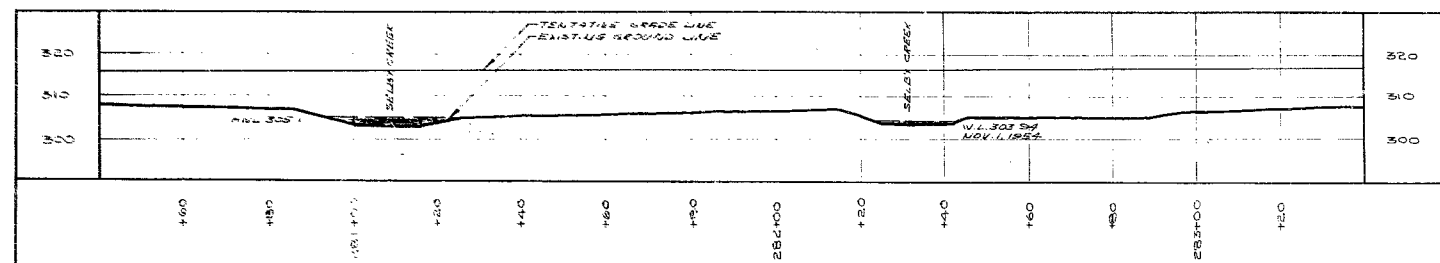
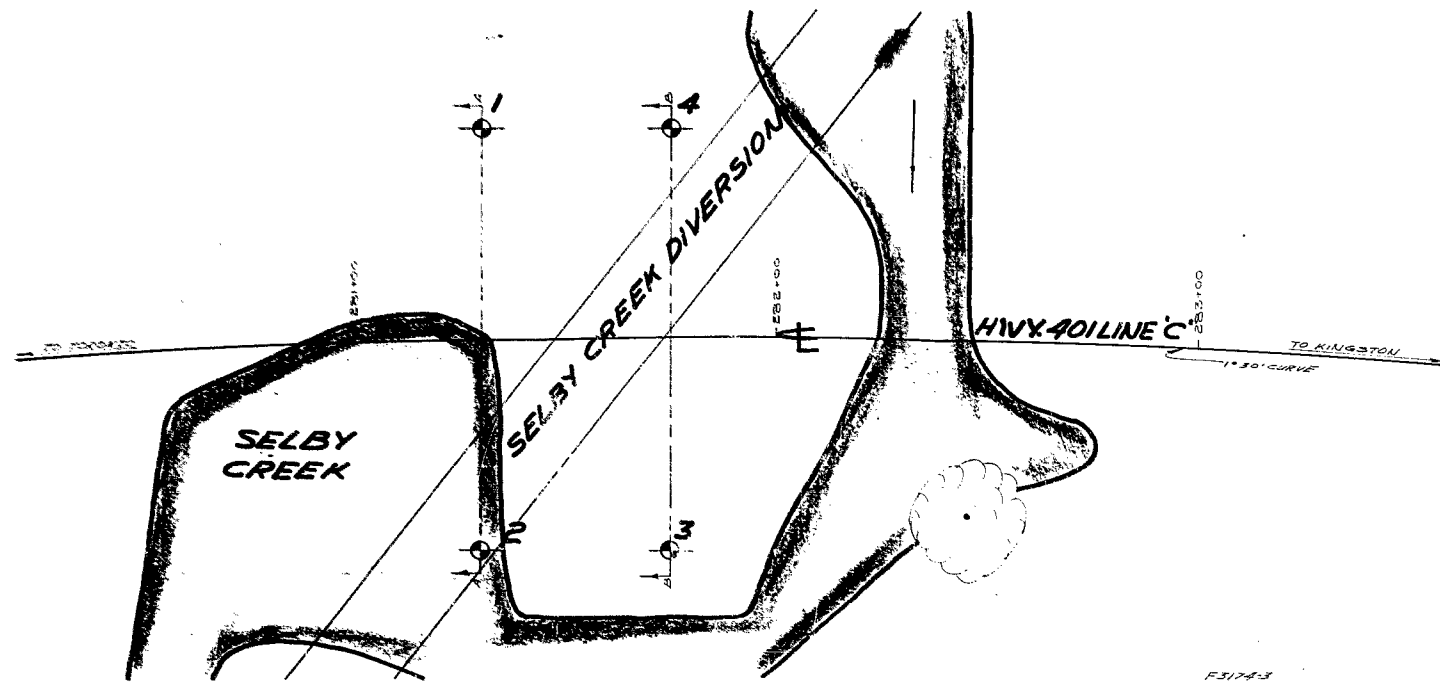
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Appendix A
Geocres Files (Reference 1)

59-F-25
W.P. # 29-59
Hwy. # 401
SELBY CREEK
DIVERSION
CON. # 2
1 MILE N. OF
NAPANEE



LEGEND			
BORE HOLE			
PENETRATION HOLE			
BORE & PENETRATION HOLE			
HOLE NO.	ELEVATION	STATION	DISTANCE FROM E
1	305.0'	281+30	50 FT.
2	305.0'	281+30	50 FT.
3	306.0'	281+75	50 FT.
4	306.0'	281+75	50 FT.

NOTE

THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BORE HOLES THE BOUNDARIES ARE AS SHOWN. EVIDENCE AND MAY BE SUBJECT TO ERROR.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH SECTION

**SELBY CREEK DIVERSION
PROPOSED CROSSING**

SHOWING POSITIONS & ELEVATIONS OF HOLES

HWY 401	DISTRICT 8	COUNTY LENOX & ADDINGTON
TOWNSHIP RICHMOND	LOT 22	CON. II
LOCATION R.R. 1 MI. N. OF WARRICK		
DRAWN BY T. MELLOWS	CHECKED BY	W.P. 23-59
DATE AUGUST 1959	APPROVED BY	DRAWING NO.
SCALE 1/4" = 20 FT.		F59-25A

cc: Mr. A. M. Toye

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Section.

June 18, 1959.

FOUNDATION REPORT -

Attention: Mr. S. McCombie.

Re: Hwy. 401 Line 'C' & Selby Creek
Diversion Crossing -
Approx. One Mile N. of Napanee,
Lot 22, Con. II, Twp. of Richmond,
W.P. 29-59.

Enclosed herewith is our report on the subsoil conditions existing at the above noted creek crossing. Reference to the contents of this report shows that the subsoil conditions consist of a shallow deposit of alluvial material overlying sound limestone bedrock. Spread footings, founded directly upon the bedrock are recommended. A conservative bearing capacity of 10 tons/sq. ft. for footings placed directly upon the limestone has been indicated.

No problems need be anticipated with respect to embankment foundation instability. The topsoil should be removed prior to placing embankment fill.

Should any queries arise in connection with the above structure site, please contact our office.

L. G. Soderman

L. G. Soderman,
PRINCIPAL FOUNDATION & SOILS ENGR.

LGS/Mdef
Encl.

cc: Messrs. A. Toye ✓
H. A. Tregaskes
D. G. Ramsay
S. Markiewicz
L. E. Walker
J. Gruspier
Dr. P. Karrow
Foundation Section.
Gen. Files.

FOUNDATION REPORT

on

Hwy. 401 Line 'C' & Selby Creek Diversion Crossing,
Approximately One Mile North of Napanee,
Lot 22, Con. II, Township of Richmond.

Plan No: F-3174-3

Profile No: F-3174-2

Chainage: 281+60.

Distribution:

Mr. A. M. Toye,
Bridge Engineer.

Mr. H. A. Tregaskes,
Construction Engineer.

Mr. D. G. Ramsay,
Design Engineer.

Mr. S. Markiewicz,
Proj. Design Engineer.

Mr. L. E. Walker,
District Engineer.

Mr. J. Gruspier,
Regional Soils Engineer.

Dr. P. Karrow,
Department of Mines.

Foundation Section.

Gen. Files.

W.J. F-59-25

W.P. 29-59

INTRODUCTION:

An investigation has been carried out to determine the competence of the subsoil layers for supporting the foundation of the proposed structure located at approximately one mile North of Napanea, where Hwy. 401 Line 'C' crosses Selby Creek Diversion in Lot 22, Con. II, Twp. of Richmond (Station 281+60, Profile No. F-3174-2).

The field work commenced on April 16, 1959 and was completed on April 17, 1959.

DESCRIPTION OF THE SITE & GEOLOGY:

The site is located on the flood plain of Selby Creek. The topography is generally level. Selby Creek meanders at the site, flowing at a velocity of approximately one foot per second. The area surrounding the site is presently in pasture and woods. Limestone rock outcrops are visible in the vicinity of the site. Bedrock is outcropped at the bed of the creek at, and half a mile upstream from the site. During spring run-off, the site has been reported to be inundated.

Physiographically, the site is located on the Napanea Plain, a flat to undulating plain of limestone from which the glaciers stripped most of its overburden. At this site the limestone bedrock is covered by a shallow overburden of alluvial and stream-bed deposits.

FIELD WORK:

Field work consisted of 1 borehole with dynamic cone penetration test adjacent to the hole, and 3 separate dynamic cone penetration tests. The exploration programme was carried

FIELD WORK: (cont'd.) ...

out by a standard diamond drill adapted for soil sampling. Because of the shallow bedrock surface encountered, no sampling was carried out in the overburden. Bedrock was drilled and cored 5 feet to determine its quality and soundness. No sudden drops during pressure drilling were noted, indicating that no mud seams exist in the formation. Rock core samples were visually examined and identified in the field, as well as in the laboratory.

SUBSOIL CONDITIONS:

The site is underlain by shallow alluvial & stream-bed deposits of sand & gravel with some clay overlying bedrock. Bedrock is composed of fine-grained limestone of the Black River Series. The limestone is in a very sound condition with no sign of fracture or weathering. An allowable bearing value of 10 t.s.f. can be used for footing design.

Water table at the site was encountered at the ground surface - i.e., Elev. 306'. Bedrock is at approximately Elev. 301' to 302'.

CONCLUSIONS & RECOMMENDATIONS:

- (1) A creek diversion or channel improvement, as shown on Drawing No. F-59-25A, is necessary at this site.
- (2) The proposed structure can be founded on spread footings placed on sound bedrock at approximately Elev. 301' to 302'. To avoid undermining of the footings due to stream erosion and scour, footings should be founded 2 ft. into sound bedrock. An allowable bearing value of 10 t.s.f. can be used.

cont'd. /3 ...

CONCLUSIONS & RECOMMENDATIONS: (cont'd.) ...

- (3) No serious ground water problems are anticipated, but provision should be made for dewatering or pumping operations during footing excavations.
- (4) The proposed grade line does not present any approach fill stability problem. Prior to the placing of embankment fill, all topsoil should be removed. Bank slopes on the upstream side of the structure should be protected by rip-rap.

A. K. Loh
A. K. Loh,
Foundation Engineer.

APPENDIX I.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 29-59 BORE HOLE NO. 1

JOB F-59-25 STATION 281/30 (50' Lt.)

DATUM Geodetic COMPILED BY B.K.

BORING DATE Apr. 16/59 CHECKED BY A.L.

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

LEGEND

1/2 UNCONFINED COMPRESSION (Qu)	---	O
VANE TEST (C) AND SENSITIVITY (S)	---	+ S
NATURAL MOISTURE AND		
LIQUIDITY INDEX	---	LI
LIQUID LIMIT	---	X
PLASTIC LIMIT	---	

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P. S. F. BLOWS/FT.	
↓	Groundlevel	305.0		10	20
	Topsoil	303.0		30	40
	Sand and gravel with some clay	300.6		7.2 BLOWS PER INCH	
	Bedrock	4.4	5	Refused @ Elev 300.6'	
			10		
			15		
			20		

[illegible]

Borehole 1

MATERIALS AND RESEARCH SECTION

W.P. 29-59

BORE HOLE NO. 2

JOB F-59-25

STATION 281/30 (50' Rt.)

DATUM Geodetic

COMPILED BY B.K.

BORING DATE Apr. 17/59.

CHECKED BY_ _ _ A.L.

2" DIA. SPLIT TUBE

2" SHELBY TUBE

2" SPLIT TUBE

2" DIA. CONE

2 SHELBY

CASING -

1/2 UNCONFINED COMPRESSION (Qu)	---	O
VANE TEST (C) AND SENSITIVITY (S)	---	+ S
NATURAL MOISTURE AND		
LIQUIDITY INDEX	---	X
LIQUID LIMIT	---	
PLASTIC LIMIT	---	

1/2 UNCONFINED COMPRESSION (Qu) _ _ O

VANE TEST(C) AND SENSITIVITY(S). -- +⁸

NATURAL MOISTURE AND LI

LIQUIDITY INDEX _____ X

ELBOLD LIMIT _____
PLASTIC LIMIT _____

PLASTIC LIMIT _____

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE	
				P.S.F.	
	↓ Groundlevel	305.0		10	20 30 40
	Alluvial topsoil	301.2		Cone Refusal at 301.2	
	Bedrock	3.8	5		
			10		
			15		
			20		

[illegible]

Borehole 2.

DEPARTMENT OF HIGHWAYS - ONTARIO

W.P. 29-59 BORE HOLE NO. 3

JOB F-59-25 STATION 281/75 (50' Rt.)

DATUM Geodetic COMPILED BY B.K.

BORING DATE Apr. 17/59. CHECKED BY A.L.

2" DIA. SPLIT TUBE _ _ _ _ _

2" SHELBY TUBE _ _ _ _ _

2" SPLIT TUBE ————— ○————○

2" DIA. CONE _____

2 SHELBY
CASINO

CASING ----- * *

LEGEND

1/2 UNCONFINED COMPRESSION (Q_u) — 0



VANE TEST (C) AND SENSITIVITY (S) -- +8

NATURAL MOISTURE AND LI

LIQUIDITY INDEX _ _ _ _ _ X

LIQUID LIMIT _____

PLASTIC LIMIT T

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				P.S.F.			
	↓ Groundlevel	306.0		10	20	30	40
	Alluvial topsoil	304.0		BLOWS/FT.			
	Sand and gravel with some clay	301.2'	5	REFUSAL AT Elev 301.2'			
	Bedrock	4.8'	10				
			15				
			20				

[illegible]

Borehole 3.

W.P. 29-59 BORE HOLE NO. 4
JOB F-59-25 STATION 281+76 (50' Lt.)
DATUM Geodetic COMPILED BY B.K.
BORING DATE Apr. 17/59 CHECKED BY A.L.

LEGEND

1/2 UNCONFINED COMPRESSION (Qu) --- O
VANE TEST (C) AND SENSITIVITY (S) --- + S
NATURAL MOISTURE AND LIQUIDITY INDEX --- LI
LIQUID LIMIT --- X
PLASTIC LIMIT ---

Borehole 4.

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE			
				P.S.F.			
	↓ Groundlevel	306.0'		10	20	30	40
	Alluvial topsoil	304.5'					
	Sand and gravel with some clay	302.3'					
	Bedrock	3.7	5				
	Limestone						
		297.2'					
	End of hole	8.8	10				
			15				
			20				

[illegible]

Appendix B
Geocres Files (Reference 2)

Golder Associates Ltd.

2390 Argentia Road
Mississauga, Ontario, Canada L5N 5Z7
Telephone: (905) 567-4444
Fax: (905) 567-6561



REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
BRIDGE STRUCTURE FOR
E-N RAMP OVER SUCKER CREEK
DETAIL DESIGN – SHORT-TERM INTERCHANGE IMPROVEMENTS
AT HIGHWAY 401 / COUNTY ROAD 41
G.W.P. 331-99-00, AGREEMENT NO. 4005-A-000206
TOWN OF GREATER NAPANEE, EASTERN REGION**

Submitted to:
Earth Tech Canada Inc.
205 Commerce Valley Drive West
Markham, Ontario
L3T 7W3

DISTRIBUTION:

- 4 Copies - The Ministry of Transportation, Ontario,
Downsview, Ontario
- 2 Copies - Earth Tech Canada Inc.
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Mississauga, Ontario



April 2003



021-1149-2

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Foundation Report titled “Hwy 401 Line ‘C’ & Selby Creek Diversion Crossing,
Lot 22, Concession II, Twp. of Richmond, Approx. 1 Mile North of Napanee”
W.P. 29-59, W.J. F-59-25, dated June 1959.

PART A

**FOUNDATION INVESTIGATION REPORT
BRIDGE STRUCTURE FOR
E-N RAMP OVER SUCKER CREEK
DETAIL DESIGN – SHORT-TERM INTERCHANGE IMPROVEMENTS
AT HIGHWAY 401 / COUNTY ROAD 41
G.W.P. 331-99-00, AGREEMENT NO. 4005-A-000206
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Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Record of Borehole Sheets (Boreholes 7 to 10)

Drawing 1

Table 1

Appendix A

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Earth Tech Canada Inc. (Earth Tech) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services associated with the short-term improvements to the Highway 401 and County Road 41 interchange within the Town of Greater Napanee. The project includes the replacement of the existing County Road 41 bridge structure over Sucker Creek and the extension of the existing Highway 401 bridge structure over Sucker Creek. The limits of the project include a 0.7 km section of County Road 41 between Richmond Boulevard and Community Road.

This report addresses the proposed bridge structure over Sucker Creek from Westbound Highway 401 to Northbound County Road 41 (E-N Ramp). A foundation investigation was carried out to determine the subsurface conditions at the location of the proposed bridge structure by drilling a limited number of boreholes, and carrying out in-situ testing and laboratory testing on selected samples.

The work was carried out in accordance with Golder Associates' Quality Control Plan for Foundation Design Services, dated September 2002

The subsurface information contained in the following reports prepared by the MTO was reviewed in the preparation of this report:

- Foundation Report titled "Hwy 401 Line 'C' & Hwy 41 Crossing, Lots 21 & 22, Concession II, Twp. of Richmond, Approx. 1 Mile North of Napanee" W.P. 28-59, W.J. F-59-28, dated June 1959.
- Foundation Report titled "Hwy 401 Line 'C' & Selby Creek Diversion Crossing, Lot 22, Concession II, Twp. of Richmond, Approx. 1 Mile North of Napanee" W.P. 29-59, W.J. F-59-25, dated June 1959.

It should be noted that Selby Creek is also known as Sucker Creek within the project limits.

2.0 SITE DESCRIPTION

The site is located in the Town of Greater Napanee, Geographical Township of Richmond, County of Lennox and Addington. Kings Highway 41 was transferred to the County of Lennox and Addington in 1998 and it is now known as County Road 41. An interchange exists at the location of where County Road 41 and Highway 401 cross, where Highway 401 is carried over County Road 41 via a single span bridge structure. Within this area, Highway 401 is a four-lane, divided highway with a posted speed limit of 100 km/hr. County Road 41 is currently a two-lane, undivided roadway with a posted speed limit of 70 km/hr.

The overall trend of the ground surface in this general area is one of dipping gently to the south towards Lake Ontario. Several watercourses, including the Napanee and Salmon Rivers, traverse the area as they follow this downward trend to eventually drain into the lake. Sucker Creek is the predominant watercourse within the project limits, where it crosses Highway 401 and County Road 41 at about 250 m to the east and 200 m to the south of the interchange, respectively, flowing north to south. Sucker Creek flows in a well-developed sinuous channel, and is characterized by generally shallow banks covered in vegetation consisting of long grass. The creek is up to about 0.3 m deep and 7 m wide at the existing Highway 401 bridge location. The immediate creek banks are up to about 1 m high in this area. Within the project limits, drainage of the pavement and immediate underlying subgrade material for the roadways is provided by a raised grade line in fill areas and side ditches within the cut sections. Culverts provide cross drainage of the roadways.

Based on the existing construction drawings, Highway 401 is carried over Sucker Creek via a 32.2 m long, single span bridge structure constructed circa 1953. The site plan on one construction drawing indicates that Sucker Creek was diverted to the west of its original alignment to a more direct alignment at the location of the bridge. The normal creek level at the bridge is about Elevation 92.6 m, giving the existing bridge a freeboard equal to about 2.9 m.

Bedrock was noted exposed at the base of Sucker Creek where it crosses Highway 401. Bedrock outcrops are also present in the general site area.

Apart from the commercial and some residential developments, the land within the project limits is mainly used for agricultural purposes. Occasional stands of young and mature trees, along with bushes, shrubs, and long grass, appear within the open spaces.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out on December 12, 2002. At that time, a total of four (4) boreholes, numbered Boreholes 7 to 10, were advanced on the north side of the existing Highway 401 bridge over Sucker Creek. Boreholes 8 and 9 were advanced near the location of the proposed west and east abutments, respectively. Boreholes 7 and 10 were put down at the location of the proposed west and east approaches, respectively. The information from the current boreholes was supplemented with the information from the previous MTO boreholes put down in the area of the existing Highway 401 bridge over Sucker Creek in 1959. These boreholes are identified as Boreholes 59-1 to 59-4, inclusive.

All of the current boreholes were extended to the top of the bedrock surface (inferred based on auger resistance / grinding) with a track-mounted CME-55 drill rig using 114 mm diameter solid stem augers supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. In the boreholes, overburden samples were obtained at continuous intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. Boreholes 8 and 9 were extended about 3 m into the bedrock by coring in NQ core size. The groundwater conditions in the open boreholes were observed throughout the drilling operations, and a piezometer was installed in Borehole 8 to permit monitoring of the groundwater level at this location. The piezometer consists of a 25 mm outside diameter pipe with 0.3 m long slotted tip sealed at a selected depth within the borehole. The boreholes were backfilled to ground surface with bentonite mixed with soil cuttings.

The field work was supervised on a full-time basis by a member of our engineering staff who cleared the area of buried utilities, located the boreholes in the field, directed the drilling, sampling, and in-situ testing operations, and logged the boreholes. The soil samples and bedrock core were identified in the field, placed in labelled containers (split-spoon samples), and boxes (bedrock core), and transported to our laboratory in Mississauga for further examination. Point load tests were carried out on selected sections of the recovered core.

The borehole locations were established relative to the proposed bridge structure as shown on the General Arrangement drawing provided in the Preliminary Design report dated April 2001. The as-drilled borehole locations and elevations were surveyed by J.D. Barnes and Associates Ltd. of Markham, Ontario. It is understood that the northing and easting coordinates are referenced to the MTM coordinate system and that the elevations are referenced to Geodetic Datum. The current and previous borehole locations are shown on Drawing 1.

4.0 GENERAL SITE GEOLOGY AND STRATIGRAPHY

4.1 Site Geology

From published literature, the site is located within the physiographic region known as the Napanee Plain ("The Physiography of Southern Ontario", 3rd Edition, Chapman and Putnam, 1984). This region is characterized by a flat to undulating plain of limestone from which most of the overburden was stripped as a result of glacial action. Geologic mapping (Map 2544, Ministry of Northern Development and Mines, 1991) indicates the bedrock at the site consists of Phanerozoic rock of the middle Ordovician age. The limestone is derived mainly from the Gull River and Bobcaygeon Formations. Shallow soil cover blankets much of the plain; however, some deeper glacial till is present in the stream valleys and in the north portion of the region. In the south, low areas have thin deposits of stratified clay. Well records indicate that the average depth to limestone within the area is approximately 2 m but ranges from exposed bedrock to a maximum depth of about 7 m.

The greatest forms of relief within the Plain are found within the valleys of the Salmon and Napanee Rivers, which have cut the plain to depths of about 15 m to 30 m. Alluvial deposits cover the valley floor of these rivers.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock, and groundwater conditions encountered in the boreholes, together with the results of the laboratory tests carried out on selected soil and bedrock samples, are given on the attached Record of Borehole and Drillhole sheets and on Table 1 following the text of this report. Copies of the previous MTO Record of Boreholes for the boreholes put down in the area of the existing bridge are found in the attached Appendix A. The stratigraphic boundaries shown on these records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. Subsoil and bedrock conditions will vary between and beyond the borehole / corehole locations.

In summary, the subsurface conditions at the site consist of a shallow cover of silty clay till and silty clay with topsoil that overlie limestone bedrock. Topsoil and sand and gravel with clay were encountered overlying the limestone in the previous MTO boreholes put down in the area of the existing bridge. The groundwater level is likely about coincident with the water level in the adjacent Sucker Creek. These conditions are similar to those encountered during the previous MTO investigations at the existing Highway 401 / County Road 41 Interchange (about 200 m north of the subject site) and at the existing crossing of Highway 401 over Sucker Creek.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. The locations and elevations of the current and previous boreholes are shown on the attached Drawing 1.

4.2.1 Topsoil

A surficial layer of topsoil was encountered in the previous MTO boreholes. The topsoil ranges in thickness from 0.45 m to 1.2 m at the borehole locations. At the location of Borehole 59-2, the topsoil overlies limestone bedrock.

4.2.2 Sand and Gravel

An alluvium deposit of sand and gravel, some clay was encountered below the topsoil in the remaining previous MTO boreholes. The sand and gravel ranges in thickness from 0.7 m to 0.9 m in the boreholes where the deposit overlies limestone bedrock at these locations.

4.2.3 Silty Clay Till

A surficial layer of brown and grey silty clay till with trace to some sand and gravel was encountered at the location of Boreholes 7 and 8 (west side of Sucker Creek). Standard Penetration Testing (SPT) carried out within the silty clay till measured 'N' values of 7 blows and 9 blows per 0.3 m of penetration, which indicates a firm consistency.

The silty clay till is 0.6 m and 0.3 m thick in Boreholes 7 and 8, respectively, and extends to the bedrock surface in each of the boreholes.

4.2.4 Silty Clay with Topsoil

A 0.8 m thick deposit of brown silty clay with topsoil was encountered surficially in Boreholes 9 and 10 (east side of Sucker Creek). Standard Penetration Testing (SPT) carried out within the silty clay measured 'N' values of 5 blows to greater than 100 blows per 0.3 m of penetration. The high 'N' values reflect the close proximity of the bedrock surface, which immediately underlies the deposit. It is therefore considered that the silty clay is of firm consistency.

4.2.5 Bedrock

Bedrock was encountered at Elevations 92.7 m and 92.5 m in Boreholes 8 and 9, respectively (about 0.3 m and 0.8 m, respectively, below the ground surface). Bedrock was inferred at 0.6 m depth (Elevation 92.7 m) and at 0.8 m depth (Elevation 92.7 m) in Boreholes 7 and 10, respectively, based on auger grinding / refusal. In the previous MTO boreholes, bedrock was

encountered at 1.3 m depth (Elevation 91.6 m), 1.2 m depth (Elevation 91.8 m), 1.5 m depth (Elevation 91.8 m), and 1.1 m depth (Elevation 92.1 m) in Boreholes 59-1, 59-2, 59-3, and 59-4, respectively. Boreholes 8 to 9 were advanced about 3 m into the bedrock by coring in NQ size. The rock core samples consist of fresh, grey, medium to fine crystalline, strong to very strong, massive limestone with interbeds of finely laminated, dark grey to black shaley limestone. In general, very few fractures exist within the recovered core and the fracture index is generally between 1 and 3 fractures per 0.3 m. At the location of Borehole 8 and 9, a 0.3 m thick zone of highly fractured rock was encountered at about 1.7 m below and at the bedrock surface, respectively. Apart from these fractured zones, the Rock Quality Designation (RQD) measured on the core samples range from about 42 percent to 85 percent, which indicates that the rock mass is of poor to good quality based on the guidelines provided in the Canadian Foundation Engineering Manual (CFEM, 3rd Edition, 1992).

Strength testing carried out on selected samples of the recovered core gave diametral point load indices of about 4.5 MPa to 5.5 MPa, with a general trend of increasing strength with depth. The corresponding range in interpreted unconfined compressive strength of the rock mass is about 105 MPa to 125 MPa. The results of the point load testing are summarized in Table 1 following the text of this report, which includes the results of the testing carried out on the bedrock core samples from Borehole 2 to 5 that were put down at the County Road 41 Bridge over Sucker Creek.

4.2.6 Groundwater Conditions

Based on the GA drawing provided, the normal and high water levels at the bridge are 92.64 m and 94.00 m, respectively. The groundwater conditions were observed in the open boreholes upon completion of drilling and immediately prior to introducing water into the holes for the purpose of coring. The open boreholes were dry upon completion of drilling, except in Borehole 8 where the base of the borehole was wet.

A piezometer was sealed within the bedrock in Borehole 8. Details of the piezometer installation are shown on the attached Record of Borehole Sheet. Water was frozen in the piezometer at 0.2 m above the ground surface (Elevation 93.2 m) about 5 weeks after its installation. Water was frozen in the piezometer at the ground surface (Elevation 93 m) about 3 months after its installation.

The information from the previous MTO investigation at the site (W.P. 29-59, dated June 1959) indicates that the groundwater level is about coincident with the adjacent creek level. The water level in the piezometers sealed in bedrock in Boreholes 2 and 5 that were put down about 300 m downstream of the site as part of this project was measured at 3.7 m (Elevation 88.9 m) and 1.3 m depth (Elevation 92.1 m) below and at the ground surface, respectively. The water level in

the boreholes put down as part of the previous MTO investigation at Highway 401 / County Road 41 (W.P. 28-59, dated June 1959) was noted to be at or just below the original ground surface, where the ground surface is about 2 m higher than that in the area of the creek.

Based on the above water level measurements, it is anticipated that the groundwater table is at or close to the original ground surface and is influenced by the creek level. It should be noted that the groundwater level will be subject to seasonal variations.

GOLDER ASSOCIATES LTD.

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Principal

DKB/ASP/FJH/vpc/mmh

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PART B

**FOUNDATION DESIGN REPORT
BRIDGE STRUCTURE FOR
E-N RAMP OVER SUCKER CREEK
DETAIL DESIGN – SHORT-TERM INTERCHANGE IMPROVEMENTS
AT HIGHWAY 401 / COUNTY ROAD 41
G.W.P. 331-99-00, AGREEMENT NO. 4005-A-000206
TOWN OF GREATER NAPANEE, EASTERN REGION**

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed bridge over Sucker Creek immediately to the north of the existing Highway 401 bridge as part of the proposed East to North (E-N) ramp. The recommendations are based on interpretation of the factual data obtained from a limited number of boreholes advanced during the investigation at this site. The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives for design of the proposed bridge structure and staging of this construction. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the planning of the project.

The project is located within the Town of Greater Napanee, the Geographical Township of Richmond and in the County of Lennox and Addington. The project limits include a section of Highway 401 within the limits of the existing Highway 401 / County Road 41 interchange, as well as a 0.7 km section of County Road 41 between Richmond Boulevard and Community Road. Within these limits, it is understood that improvements to the existing interchange will be carried out in the short, mid, and long-terms. The short-term improvements include the construction of bridge structure to carry the proposed E-N ramp over Sucker Creek immediately to the north of the existing Highway 401 bridge over Sucker Creek.

Based on the GA drawings of the existing Highway 401 bridge over Sucker Creek, dated December 1959, the existing abutment foundations are about 0.8 m thick with the base of the footings at about Elevation 91.3. A note on the drawings indicates that the footings were to be recessed a minimum of 0.3 m into sound bedrock.

5.2 Bridge Foundations

It is understood that the proposed bridge over Sucker Creek as part of the proposed E-N ramp will be a single-span structure that will accommodate one lane of traffic and a shoulder. Based on the information provided, the bridge will be about 17.2 m long and 15 m wide, with the elevation of top of the bridge deck at about Elevation 97 m, which will result in embankments up to 3 m in height.

Boreholes 8 and 9 were advanced as close to the proposed foundation units as possible, given the location of the embankment slope and creek alignment.

In summary, the subsurface conditions at the site consist of shallow overburden (0.3 m to 0.8 m thick) consisting of firm silty clay till, and silty clay with topsoil materials overlying limestone

bedrock. A surficial layer of topsoil underlain by a 0.7 m to 0.9 m thick layer of sand and gravel was encountered in the previous MTO boreholes put down in the area of the existing bridge. The bedrock surface established at the current and previous borehole locations is as follows:

<i>Location</i>	<i>Relevant Borehole(s)</i>	<i>Elevation of Top of Bedrock (m)</i>
Proposed West Abutment	59-4	92.1
	8	92.7
Proposed East Abutment	9	92.5

The above variation in bedrock surface elevation may have been influenced by the channel configuration prior to the construction of the existing bridge.

The groundwater level is expected to be at or close to the ground surface and will be subject to fluctuations of the water level in the adjacent creek.

Consideration could be given to supporting the proposed structure with semi-integral abutments, false abutments, or conventional abutments. It is understood that an open abutment design is being considered and that retaining walls are not required. Based on the subsurface information available at the proposed bridge structure location, it is recommended that the proposed structure be founded on spread footings placed on the bedrock surface.

5.2.1 Shallow Spread Footings

Footings may be placed within the bedrock or may be placed on the bedrock surface / mass concrete after excavation of the overburden and cleaning of the bedrock surface. Further details of these two alternatives are given below.

The design founding level could be based on the lowest bedrock surface elevation encountered in the boreholes and include a nominal embedment into the bedrock. This option results in a design founding level of Elevation 91.8 for the proposed abutment footings. Bedrock excavation of variable depth will be required depending on the variability of the bedrock surface, particularly due to the previous creek channel, and is potentially up to about 1 m based on the borehole data to date.

Alternatively, in order to minimize the bedrock excavation requirements, a design founding level at or above the highest bedrock surface could be adopted. Based on the results of Borehole 9 and

the previous Borehole 59-4, which are the two holes closest to the proposed footings, a founding level of Elevation 92.5 m may be assumed for both abutments. This would then involve excavation of the overburden to expose the bedrock surface and placement of mass concrete to raise the founding level where required. Some bedrock excavation may be required depending on the variability of the bedrock surface. All loose or fractured rock exposed at the surface should be removed prior to placing the mass concrete. Provision should be made in the contract for bedrock excavation and / or mass concrete placement to accommodate the variations in the bedrock surface. It should be noted that with this alternative for founding level, the base of the footings will be close to the normal creek water level.

Depending on the chosen founding level, bedrock excavation may be required for the footing construction. The bedrock is classified as strong to very strong and the level of fracturing in the upper portions of the rock is variable. Excavation, particularly where only small depths and narrow areas are needed, will potentially be difficult. The excavation could be carried out using drilling and hoe ramming techniques where a relatively shallow depth of cut into the bedrock is required. This procedure will tend to result in a very uneven founding surface and some over excavation is likely. Line drilling and pre-shearing techniques will provide better control over the configuration of the founding surface, and this procedure would be the preferred approach where deeper excavation into the bedrock is required for footing construction.

The simplest option for the bridge footings from a foundation standpoint is spread footings placed on the bedrock surface or on mass concrete placed on the bedrock which should minimize the bedrock excavation difficulties. The cost effectiveness of each of the foundation alternatives should be considered in the design.

5.2.2 Geotechnical Resistance

The factored geotechnical resistance at Ultimate Limit States (ULS) that may be used for design of the footings placed on / within the limestone bedrock is 3,000 kPa. For footings placed on a mass concrete pad, the factored geotechnical resistance at Ultimate Limit States (ULS) is as given above for the bedrock assuming that the strength of the concrete used to form the pad is at least 25 MPa. Serviceability Limit States (SLS) conditions do not apply to footings placed on the fresh bedrock or mass concrete pad.

All loose, shattered and / or fractured rock within the footprint of the footings and at the footing level should be removed and replaced with concrete. A NSSP should be included in the contract documents to the requirement for field inspection; MTO Special Provision 902S01 – Excavation and Backfilling – should be included in the contract documents.

The geotechnical resistance provided is given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Sections 6.7.2 and 6.7.4 of the *Canadian Highway Bridge Design Code (CHBDC)* and its *Commentary*.

5.2.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the base of the concrete footings and the founding stratum should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, $\tan \delta$, may be taken as 0.7 between the base of the concrete footings and / or mass concrete and the bedrock. This represents an unfactored value; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

If necessary, the sliding resistance can be supplemented by dowelling into the bedrock. The horizontal resistance of the dowels is dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. This assumes that the unconfined compressive strength of the grout will be similar to that of the concrete. The dowels should have a minimum embedded length within the bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

5.2.4 Frost Protection

No minimum frost protection is required for footings placed directly on mass concrete or bedrock.

5.3 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the abutments, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.4 m behind the back of the walls (Case I in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case II in Figure C6.9.1(l) of the *Commentary to the CHBDC*).
- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be assumed based on the use of Select Subgrade Material:

Soil unit weight:	21 kN/m ³
Coefficients of lateral earth	
"Active", K_a	0.35
"At rest", K_o	0.5

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	Granular 'B'
		Type II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of lateral earth pressure:		
"Active", K_a	0.27	0.31
"At rest", K_o	0.43	0.47

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.
- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 2. The site-specific zonal acceleration ratio for Napanee is 0.10. Based on experience, for the subsurface conditions at this site, a 10 to 20 per cent

amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.10g to between 0.11g and 0.12g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.12$.

- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC* and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.06$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.18$). The following seismic active pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.33	0.29	0.31
Non-yielding wall	0.47	0.38	0.45

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to $250A$ (mm), where A is the design zonal acceleration ratio of 0.12. This corresponds to outward displacements of up to 30 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$K_a \gamma' d + (K_{AE} - K_a) \gamma' H$$

- where K_a is the static active earth pressure coefficient;
 K_{AE} is the seismic active earth pressure coefficient;
 γ' is the effective unit weight of the soils (kN/m^3)
- taken as soil unit weights given above for the fill materials.
 - taken as 20 kN/m^3 above Elevation 93 m and 10 kN/m^3 below Elevation 93 m for the native materials.
- d is the depth below the top of the wall (m); and
 H is the height of the wall above the toe (m).

5.4 Excavations

5.4.1 Groundwater and Surface Water Control

The existing roadway grade at the bridge is at about Elevation 97 m and therefore excavations made through the fill and native materials to the bedrock surface could be as much as 5 m below the existing road grade.

The groundwater level is considered to be about coincident with the ground surface and will be influenced by the water level in the adjacent creek, which is normally at about Elevation 92.6 m, with a high water level of about Elevation 94.0 m. Perched water conditions may exist in the adjacent embankment fill. A deposit of sand and gravel (indicated to be stream deposits) was encountered overlying the bedrock in Boreholes 59-3 and 59-4. Although not encountered in the two recent boreholes, this permeable material should be expected and significant water inflow to the excavation will occur through this deposit and the underlying fractured bedrock. Depending on the founding level adopted, the base of the footing excavations could be as much as 1 m below the normal creek water level and up to about 1.5 m below the groundwater table.

Some form of groundwater control will be required in order to place the concrete for the footings on the bedrock in the “dry” condition. In this regard, reference should be made to special Provision 902S01. A creek diversion and/or cofferdam will be required in order to minimize the amount of water entering the excavation from the adjacent creek. This may require trench excavation through the overburden (silty clay / silty clay till and sand and gravel deposits) and placement of suitable cut-off measures. Water will also flow upward from the excavation base and, in this regard, pumping from properly filtered sumps placed within the excavation but outside the footing areas will also be required to control the water flow. Pumping would have to be carried out from sumps formed within the bedrock below the founding level; the number of sumps required will depend on the size and duration of the excavations.

Sediment control such as silt fences and / or erosion control blankets may be required during the diversion of the creek and construction activity. All surface water should be directed away from the excavations.

5.4.2 Temporary Excavations

If space permits, temporary excavations (i.e. those that are open only for a relatively short period) through the fill and native materials above the groundwater table (Elevation 92.6 m) may be made with side slopes no steeper than 1.5 horizontal to 1 vertical (1.5H:1V) for the fills and 1H:1V in the native clay materials. Some sloughing of the side slopes at these inclinations is possible, particularly if perched conditions are present within the soils exposed along the cut face. Below

the groundwater table (Elevation 98 m), shallower side slopes (3H:1V) will be required. All excavations should be carried out in accordance with the current Occupational Health and Safety Act.

If space and / or staging restrict the use of open cuts, a temporary support system could be constructed to support the excavations and the adjacent road embankment in the area of the bridge structure. The temporary excavation support system should be in accordance with Special Provision 539S01. The temporary support system should be designed to Performance Level 2 as defined in SP 539S01.

Roadway protection should be as per current MTO Special Provision 539S01.

5.5 Approach Embankment Design

The construction of the approach embankments for the proposed bridge as part of the N-E ramp will require placement of up to about 3m of fill material. Based on the borehole results, the embankment subgrade soils consist of a shallow cover of firm silty clay and silty clay till over bedrock. All topsoil / organic matter, and softened / loosened soils should be stripped from beneath the proposed approach embankment envelopes, and all subgrade soils should be proof-rolled prior to fill placement. Construction of the embankment above the prepared subgrade may be carried out using clean earth fill meeting specifications of OPSS 212 or Selected Subgrade Material meeting specifications with of OPSS 1010, depending on material availability. All embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Where appropriate, the new embankment fill must be keyed into the existing embankment slopes in accordance with OPSD 208-01 in order to reduce the impact of differential settlement.

A slope stability analysis was carried out to assess the stability of the proposed embankments. The commercially available program SLOPE/W was used to carry out the analysis following the general limit equilibrium method of analysis (Morgenstern-Price). The factor of safety of numerous potential failure surfaces were computed in order to determine minimum factors of safety. The factor of safety is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause.

With appropriate subgrade preparation and proper placement and compaction of embankment fill materials, embankments up to about 3 m in height with side slopes maintained at 2 horizontal to 1 vertical (2H:1V) will have an adequate Factor of Safety (F.S.) against deep-seated slope instability (i.e. F.S. = 1.3). To reduce surface water erosion, placement of topsoil and seeding or pegged sod is recommended as per OPSS 572.

The soils beneath the embankment (i.e. silty clay native materials) are considered to be non-susceptible to liquefaction.

Scour protection of the embankments in the vicinity of the bridge should be provided by means of rip-rap to an elevation that is 0.5 m above the high water level, i.e. the flood level.

The settlement related to the compression of the subgrade and fill materials comprising the new embankment, properly placed and compacted, will be less than 50 mm and the majority will occur during construction.

6.0 CLOSURE

This report was prepared by Dan Breeze, P.Eng., Project Engineer, under the supervision of Anne Poschmann, P.Eng., Project Manager. The overall review and quality control of this project was carried out by Fin Heffernan, P.Eng., Designated MTO Contact.

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

(b) Cohesive Soils

Consistency	c_u, s_u	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (L.V-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
in x,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index = $(w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index = $(w - w_p)/I_p$
I_C	consistency index = $(w_l - w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = $\tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

- Notes: 1 $\tau = c' + \sigma' \tan \phi'$
 2 shear strength = (compressive strength)/2
 * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains > 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

PROJECT <u>021-1149</u>		RECORD OF BOREHOLE No 7		1 OF 1	METRIC
W.P. <u>331-99-00</u>	LOCATION <u>N 4903019.0; E 267945.9</u>	ORIGINATED BY <u>PKS</u>			
DIST <u>6</u> HWY <u>401</u>	BOREHOLE TYPE <u>108mm I.D. Hollow Stem Augers</u>	COMPILED BY <u>DKB</u>			
DATUM <u>Geodetic</u>	DATE <u>December 12, 2002</u>	CHECKED BY <u>ASP</u>			

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT <div><div></div><div>20406080100</div></div>	<div>PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT</div> <div><div>W_p</div><div>W</div><div>W_L</div></div>	UNIT WEIGHT <div>γ</div> kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES							
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED <div>20406080100</div>												
93.4 0.0	GROUND SURFACE Silty Clay, trace sand and gravel (TILL) Firm Brown/grey Moist	<div></div>	1	SS	9			93				
92.7 0.6	END OF BOREHOLE Refusal on probable bedrock											
Note: 1. Open borehole dry upon completion of drilling.												

MISS_MTO 021-1149.GPJ ON_MOT.GDT 22/8/03

PROJECT 021-1149		RECORD OF BOREHOLE No 8				1 OF 1		METRIC								
W.P. 331-99-00		LOCATION N 4903023.0; E 267964.4				ORIGINATED BY PKS										
DIST 6 HWY 401		BOREHOLE TYPE 108mm I.D. Hollow Stem Augers				COMPILED BY DKB										
DATUM Geodetic		DATE December 12, 2002				CHECKED BY ASP										
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								
93.0	GROUND SURFACE						20	40	60	80	100					
0.0	Silty Clay, some sand and gravel (Till)		1	SS	7											
0.3	Firm Brown Fresh, massive, medium to fine crystalline, grey, strong to very strong LIMESTONE, with interbeds of finely laminated dark grey to black shaley limestone. Styolitic. Fracturing is bedding -parallel with fresh to slightly altered surfaces. For borehole coring details refer to Record of Drillhole 8.															
92.7																
92.0																
91.0																
90.0																
89.5																
3.5	END OF BOREHOLE Refusal on probable bedrock Notes: 1. Base of open borehole wet prior to coring bedrock. 2. Water frozen in piezometer at 0.2m above ground surface (El. 93.2m) on January 22, 2003. 3. Water frozen in piezometer at the ground surface (El. 93.0m) on March 18, 2003.															

MISS_MTO 021-1149.GPJ ON_MCT.GDT 22/8/03

PROJECT: 021-1149

RECORD OF DRILLHOLE: 8

SHEET 2 OF 2

LOCATION: N 4903023.0; E 267964.4

DRILLING DATE: December 12, 2002

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55 Track Mount

DRILLING CONTRACTOR: Marathon Drilling Ltd.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	CORRECTION										CORRECTION										HYDRAULIC CONDUCTIVITY K _f cm ² /sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
						PENETRATION RATE (mm/min)		FLUSH		COLOUR		% RETURN		FR/FX-FRACTURE F-FAULT		J-JOINT		SM-SMOOTH		FL-FLEXURED		BC-BROKEN CORE						
						CL-CLEAVAGE		SH-SHEAR		VN-VEIN		S-SLICKENSIDED		PL-PLANAR		R-ROUGH		UE-UNEVEN		MB-MECH. BREAK								
						P-POLISHED		ST-STEPPED		C-CURVED		W-WAVY		B-BEDDING														
RECOVERY		R.Q.D. %		FRACT. INDEX PER 0.3		DISCONTINUITY DATA		TYPE AND SURFACE DESCRIPTION		CORRECTION		CORRECTION		CORRECTION		CORRECTION												
TOTAL CORE %		SOLID CORE %		DIP w/1 CORE AXIS		CORRECTION		CORRECTION		CORRECTION		CORRECTION		CORRECTION		CORRECTION												
1		2		3		4		5		6		7		8		9												
1		2		3		4		5		6		7		8		9												
1		2		3		4		5		6		7		8		9												
1		2		3		4		5		6		7		8		9												
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1		2		3		4		5		6		7		8		9												
1		2																										

PROJECT 021-1149			RECORD OF BOREHOLE No 9			1 OF 1 METRIC		
W.P. 331-99-00			LOCATION N 4903028.9; E 267989.6			ORIGINATED BY PKS		
DIST 6 HWY 401			BOREHOLE TYPE 108mm I.D. Hollow Stem Augers			COMPILED BY DKB		
DATUM Geodetic			DATE December 12, 2002			CHECKED BY ASP		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x REMOULDED 20 40 60 80 100
93.3	GROUND SURFACE							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%) 10 20 30
0.0	Silty Clay with topsoil Firm Brown Moist		1	SS	9		93	
92.5			2	SS	100/15			
0.8	Fresh, massive, medium to fine crystalline, grey, strong to very strong LIMESTONE, with interbeds of finely laminated dark grey to black shaley limestone. Styolitic. Fracturing is bedding-parallel with fresh to slightly altered surfaces. For borehole coring details refer to Record of Drillhole 9.						92	
							91	
							90	
89.5								
3.8	END OF HOLE Note: 1. Open borehole dry prior to coring bedrock.							

MISS_MTO 021-1149.GPJ ON_MOT.GDT 22/8/03

SHEET 2 OF 2

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling Ltd.

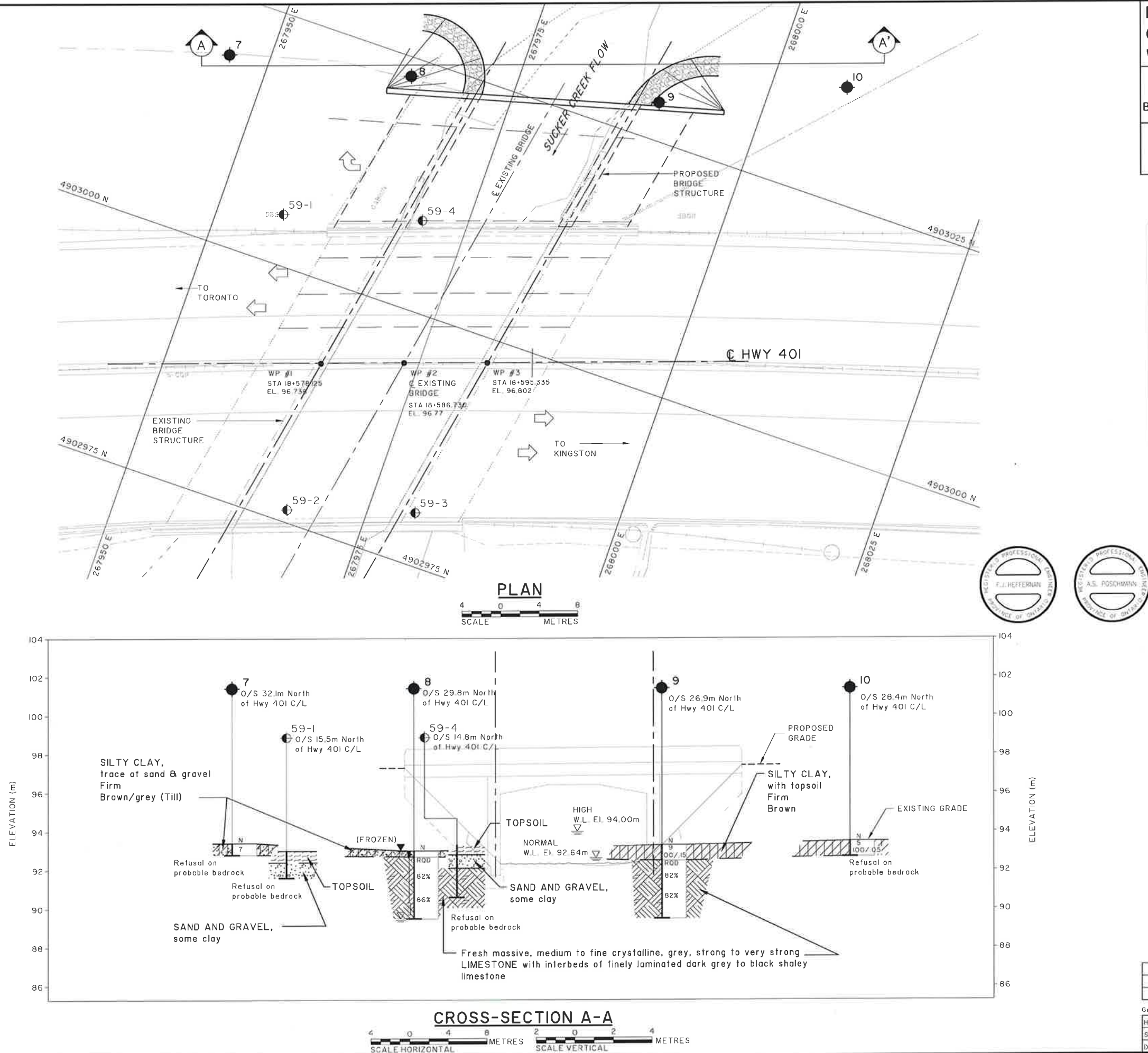
[illegible]

CHECKED: DKB



PROJECT 021-1149		RECORD OF BOREHOLE No 10				1 OF 1		METRIC							
W.P. 331-99-00		LOCATION N 4903036.7; E 268007.5		ORIGINATED BY PKS											
DIST 6 HWY 401		BOREHOLE TYPE 108mm I.D. Hollow Stem Augers		COMPILED BY DKB											
DATUM Geodetic		DATE December 12, 2002		CHECKED BY ASP											
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100	20 40 60 80 100	W _p W W _L	WATER CONTENT (%)	10 20 30	γ	GR SA SI CL	
93.5	GROUND SURFACE														
0.0	Silty Clay with topsoil Firm Brown Moist		1	SS	5		93								
92.7			2	SS	100/05										
0.8	END OF BOREHOLE Refusal on probable bedrock Note: 1. Open borehole dry upon completion of drilling.														

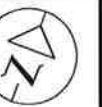
MISS_MTO 021-1149.GPJ ON_MOT GDT 22/8/03



DIST. HWY. 401
 CONT No. 99-232
 WP No. 331-99-00

HIGHWAY 401
 SUCKER CREEK BRIDGE

BOREHOLE LOCATIONS AND SOIL STRATA

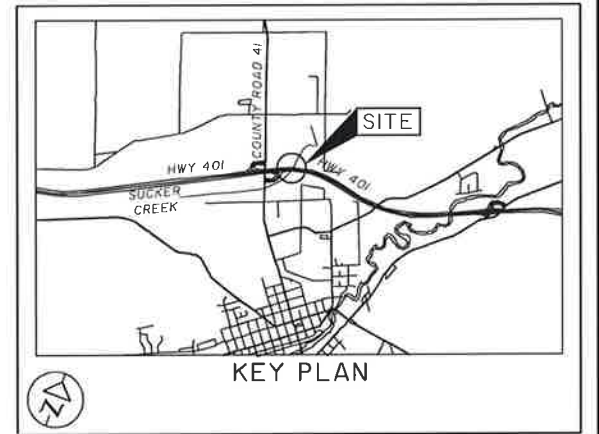


SHEET



Golder Associates Ltd.
 MISSISSAUGA, ONTARIO, CANADA

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

**LEGEND**

- Borehole - Current Golder Associates Ltd. Investigation
- Borehole - Previous MTO Investigation (W.P. 29-59), dated June 1959
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer, on March 18, 2003
- WL in borehole upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
7	93.4	4903019.0	267945.9
8	93.0	4903023.0	267964.4
9	93.3	4903028.9	267989.6
10	93.5	4903036.7	268007.5
59-1 *	93.0	4903005.1	267956.6
59-2 *	93.0	4902976.3	267967.1
59-3 *	93.3	4902980.4	267979.7
59-4 *	93.3	4903009.2	267970.5

* Northing and Easting coordinates obtained from approximate location of previous boreholes shown on the Control Drawing D-4427-1 (W.P. 29-59), dated December 1959.

NOTES

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview, Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen Cond.

REFERENCE

Digital file provided by EARTH TECH (CANADA INC) LONDON, ONTARIO, Titled "HWY 401 SUCKER CREEK BRIDGE GENERAL ARRANGEMENT" Dated Feb. 2003, received Mar. 5, 2003.

NO.	DATE	BY	REVISION

Geocres No.	HWY 401	PROJECT NO.	02I-1149-2	DIST.	
SUBM'D.	DKB	CHKD.		DATE:	MAR. 2003
DRAWN:	JFC	CHKD.		APPD.	
					DWG. 1

TABLE 1 - SUMMARY OF POINT LOAD TESTS ON ROCK CORE SAMPLES

PROJECT NO. 021-1149-2

LOCATION: County Road 41 Bridge Structure over Sucker Creek

Ram Area: 1,355mm²

DATE: December 30, 2002

Borehole Number	Sample Number	Sample Depth (m)	Test Type	Core Length (mm)	Core Diameter (mm)	Equivalent Diameter (mm)	Ram Pressure (kPa)	Load (P) (kN)	Is Axial (MPa)	Is Diametral (MPa)	Is (MPa)	Approx. (1) UCS (MPa)
2	Shaley and Styolitic LST	3.9	D		47.60		4,000	5.42		2,392	2,340	54
2	Shaley and Styolitic LST	4.0	D		47.60		4,500	6.10		2,691	2,632	61
2	Shaley and Styolitic LST	4.1	A	40.00	47.60	49.24	7,000	9.49	3,913	4,186	3,827	88
2	Massive, Crystalline LST	4.2	D		47.60		8,000	10.84		4,784	4,680	108
2	Massive, Crystalline LST	4.3	A	58.00	47.60	59.29	12,000	16.26	4,626		4,525	104
3	Massive, Crystalline LST	4.7	D		47.60		8,000	10.84		4,784	4,680	108
3	Massive, Crystalline LST	4.9	A	33.00	47.60	44.72	12,000	16.26	8,130		7,952	183
5	Shaley and Styolitic LST	5.5	D		47.60		4,000	5.42		2,392	2,340	54
5	Shaley and Styolitic LST	5.6	A	34.00	47.60	45.39	9,000	12.20	5,918		5,789	133
5	Massive, Crystalline LST	5.7	D		47.60		9,000	12.20		5,382	5,264	121
5	Massive, Crystalline LST	5.8	A	36.00	47.60	46.71	9,000	12.20	5,590		5,467	126
8*	Shaley and Styolitic LST	0.6	D		47.60		8,500	11.52		5,083	4,972	114
8*	Shaley and Styolitic LST	0.8	A	48.00	47.60	53.94	10,000	13.55	4,658		4,556	105
8*	Massive, Crystalline LST	1.2	D		47.60		8,000	10.84		4,784	4,680	108
8*	Massive, Crystalline LST	1.5	A	48.00	47.60	53.94	12,000	16.26	5,590		5,467	126

SUMMARY OF TEST RESULTS

Shaley and Styolitic Limestone		Mean Is(50)
Axial	4.7	Very Strong Rock
Diametral	3.1	Strong Rock
Anisotropy ratio (A/D) =	1.5	

Massive, Crystalline Limestone		Mean Is(50)
Axial	5.9	Very Strong Rock
Diametral	4.8	Very Strong Rock
Anisotropy ratio (A/D) =	1.2	

Note that Axial tests were oriented perpendicular to bedding planes. Test results indicate that the core is weaker parallel to bedding and that the massive crystalline limestone intervals are stronger than the shaley and styolitic limestone interval

* Boreholes 2, 3, and 5 located within project limits (County Road 41 crossing at Sucker Creek - Golder Report No. 021-1149-1, dated April 2003). Data included for completeness of testing

(1) Is₅₀ x 23 (actual value will have to be confirmed by UCS testing), from ISRM ("Suggested Methods for Determining Point Load Strength", International Society for Rock Mechanics Commission on Testing Methods, Int. J. Rock. Mech. Min. Sci. and Geomechanical Abstr., Vol 22, No. 2 1985, pp. 51-60.
(2) Actual distance between point load cones at time of failure.

APPENDIX A
PREVIOUS MTO RECORD OF BOREHOLES

**FOUNDATION REPORT TITLED "HWY 401 LINE 'C' & SELBY CREEK
DIVERSION CROSSING, LOT 22, CONCESSION II, TWP. OF
RICHMOND, APPROX. 1 MILE NORTH OF NAPANEE" W.P. 29-59, W.J.
F-59-25, DATED JUNE 1959.**

59-1

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 29-59 BORE HOLE NO. 1JOB F-59-25 STATION 261+30 (50' Lt.)DATUM Geodetic COMPILED BY B.K.BORING DATE Apr. 16/59 CHECKED BY A.L.**LEGEND**

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE		CONSISTENCY	
				P.S.F.		MOIST. CONTENT - % DRY WT.	
	↓ Groundlevel	305.0		10 20 30 40 BLOWS/FT.			
	Topsoil	303.0		<p>Refused to blow 300.6</p>			
	Sand and gravel with some clay	300.6					
	Bedrock	4.4	5				
			10				
			15				
			20				

59-2

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS AND RESEARCH SECTION

W.P. 29-59 BORE HOLE NO. 2

JOB F-59-25 STATION 281/30 (50' Rt.)

DATUM Geodetic COMPILED BY B.K.

BORING DATE Apr. 17/59. CHECKED BY A.L.

LEGEND

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

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

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE				CONSISTENCY	
				P.S.F.				MOIST. CONTENT - % DRY WT.	
	↓ Groundlevel	305.0		10	20	30	40		
	Alluvial topsoil	301.2		Cone Refusal at 301.2'					
	Bedrock	3.8	5						
			10						
			15						
			20						

59-3

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

1/2 UNCONFINED COMPRESSION (Q
VANE TEST (C) AND SENSITIVITY (I:
NATURAL MOISTURE AND
LIQUIDITY INDEX _ _ _ _
LIQUID LIMIT _ _ _ _ _
PLASTIC LIMIT _ _ _ _ _

SYMBOL	DESCRIPTION	ELEV. FEET	DEPTH FEET	STRENGTH AND PENETRATION RESISTANCE		CONSISTENCY		
				P. S. F.			MOIST. CONTENT - % DRY WT.	
				BLOWS/FT.				
				10	20	30	40	
	↓ Groundlevel	306.0						
	Alluvial topsoil	304.0						
	Sand and gravel with some clay	301.2'		REFUSAL AT Elev 301.2'				
	Bedrock	4.8'	5					
			10					
			15					
			20					

Appendix C
Site Photographs



Photo 1 - View of South Side of Sucker Creek Bridge



Photo 2 - Erosion at South East Approach Fill Due to Burrowing Animals



Photo 3 - View of North Side of Sucker Creek Bridge 2004 Widening



Photo 4 – Looking West over Sucker Creek Structure Towards County Road 41



Photo 5 - Embankment Slopes on E-N Ramp



Photo 6 - Rock Lined Ditch at Toe of Slope, E-N Ramp