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W. O. No.

STR. SITE No. 10-335

HWY. No. 403

LOCATION Ramp Q.E.W./E-403/W over
Ramp 403/E-Q.E.W./S (Bridge #33)

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I.-30 SEPT. 1976



Golder Associates Ltd.

CONSULTING ENGINEERS

REPORT

TO

MINISTRY OF TRANSPORTATION ONTARIO

CONT 93-89
GEOTECHNICAL INVESTIGATION

PROPOSED BRIDGE 33

SITE 10-335

FREEMAN INTERCHANGE

BURLINGTON, ONTARIO

W.P. 199-77-05

DISTRICT 4

GEOCRES # 30M5-177

Distribution:

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Appendix A

Explanation of Terms used in Report, Abbreviations and Symbols

Record of Borehole Sheets

Record of Test Pit Sheets

Drawing 1997705-A

1. INTRODUCTION

Golder Associates Ltd. has been retained by the Ministry of Transportation of Ontario (MTO) to carry out a series of site specific subsurface investigations for the design of structures for the proposed reconstruction of the Freeman Interchange in Burlington, Ontario. This report presents the results of a subsurface investigation carried out at the site of the proposed Bridge No. 33. As presently proposed, Bridge 33 will carry the QEW east/403 west ramp traffic over the 403 east/QEW south ramp traffic. The proposed high level Bridges 34 and 35 will carry the 403 westbound and eastbound traffic over bridge 33. The location of the site is shown on the Key Plan, Drawing 1997705-A.

The purpose of the investigation was to determine the subsurface conditions at the site and to provide geotechnical engineering recommendations for the design of the rigid frame bridge. A proposal for carrying out the work was provided in our letter to the Ministry of Transportation Ontario (MTO) dated July 27, 1990.

2. SITE AND PROJECT DESCRIPTION

The site of the proposed Bridge 33 is located in a relatively flat, grassed area, situated south of the existing QEW/403 west ramp and north of the existing QEW southbound lanes and the QEW east/403 west ramp. The location of the site is shown on the Key Plan, Drawing 1997705-A.

The site is situated within the physiographic region of Southwestern Ontario known as the Iroquois Plain. Available geologic information indicates that the overburden in the

general area of the site consists of a thin veneer of sands, glacial till and/or residual soil derived from the weathering of the underlying shale bedrock. The Queenston shale formation which comprises the bedrock in the area generally consists of thinly bedded red shale with occasional bands of grey limestone.

The bridge will carry QEW east/403 west ramp traffic over the 403 east/QEW south ramp. The proposed high level bridges (34 and 35) will carry the 403 west and east bound traffic over Bridge 33.

The proposed bridge will consist of a 19 metre span rigid frame, some 17 metres in length. Approach fills of some 7 metres in height will be required.

3. INVESTIGATION PROCEDURE

The field work for this investigation was carried out on August 16 and 20, 1990 at which time one test pit (numbered 3) was excavated and two boreholes (numbered 6 and 7) were drilled. The locations of the boreholes and test pit are shown on Drawing 1997705-A.

In addition, the results of borehole 16 drilled as part of the investigation programme for Bridge No. 35 (Site 10-480 WP 199-77-07) have been included in this report.

The initial stage of the field work consisted of excavating one test pit (numbered 3) some 20 metres west of the west abutment of the proposed Bridge 33. The test pit was excavated to a depth of about 3.7 metres using a "John Deere 690" hydraulic backhoe supplied and operated by a local

contractor. Chunk samples were obtained from the predominant soil strata exposed in the test pit and the test pit was loosely backfilled following sampling and logging.

The boreholes were drilled using track mounted power auger drillrigs equipped for rotary drilling which was supplied and operated by a specialist drilling contractor. The boreholes were drilled to practical auger refusal using both nominal 150 millimetre outside diameter hollow stem augers and nominal 100 millimetre diameter solid stem augers. Standard penetration testing and sampling was carried out within the overburden encountered in the boreholes using 35 millimetre inside diameter split spoon sampling equipment.

Samples of the overburden recovered from the test pit and boreholes, were taken to our Hamilton laboratory for examination and water content determinations.

The soil and rock stratigraphy encountered in the boreholes and test pit are shown in detail on the Records of Boreholes and Record of Test Pit following the text of this report and on Drawing 1997705-A. The results of the field and laboratory testing are also shown on the Record of Borehole and Record of Test Pit sheets.

Groundwater levels were observed in the open boreholes during drilling and in the test pit during and after excavation. Notes pertaining to the groundwater conditions encountered in the boreholes and test pit are shown on the Records of Boreholes and Test Pit and on Drawing 1997705-A.

The locations and ground surface elevations at the borehole and test pit locations have been determined by Golder

Associates staff with reference to site specific points and temporary bench marks provided by McCormick Rankin & Associates Limited. The final locations and ground surface elevations at the boreholes were subsequently verified by McCormick Rankin. The elevations provided are understood to be referred to geodetic datum.

4. SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions encountered in the boreholes and test pits put down at the site are shown in detail on the Record of Borehole and Record of Test Pit sheets, and in summary form on Drawing 1997705-A. The soil boundaries and rock stratigraphy indicated, particularly for the boreholes, are inferred from non-continuous sampling and resistance to drilling advance. These boundaries typically represent a transition between one soil or rock type to another and are not intended to define an exact plane of geological change. Conditions will vary between and beyond the borehole and test pit locations.

The subsurface conditions encountered at the site generally consisted of fill and topsoil overlying glacial till, and completely to highly weathered shale.

The following discussion has been simplified in terms of major soil and rock strata for the purposes of geotechnical design.

It should be noted that due to the relatively soft and weathered nature of the Queenston shale formation,

particularly the upper zone, together with the effects of glacial overriding at the bedrock surface, it is difficult to accurately define the bedrock surface both from a geological and a contractual standpoint.

During the course of this investigation, a stratigraphic unit directly overlying the shale bedrock, which in strictly geological terms is described as deformation till, has been encountered. This stratigraphic unit consists of an imbricate embedment of fragments of bedrock in a till matrix which has been formed from glacial overriding of the parent bedrock. This till is characterized by the presence of rounded and sub-angular clasts of the parent rock with non horizontal bedding. Based on the consistency, the relatively high penetration resistance encountered in the boreholes and the difficulty of excavation experienced in the test pits, this stratum could be contractually interpreted as a zone of the bedrock and for this reason has been referred to in this report as an upper zone of the bedrock formation.

4.2 Topsoil

Layers of topsoil were encountered at the ground surface depths in borehole 16 and test pit 3 and beneath the fill in borehole 7 and test pit 3.

4.3 Silty Clay, trace sand, trace to some topsoil, occasional gravel (Fill)

Layers of silty clay fill were encountered to depths of about 1 metre in boreholes 6 and 7 and in test pit 3 and to a depth of about 1.7 metres in borehole 16. The silt clay fill in borehole 16 had an in-situ water content of about 20 per cent

and an N value as determined in the standard penetration test of 16 blows per 0.3 metres.

4.4 Clayey Silt, trace to some sand,
occasional gravel (Till)

Glacial till was encountered beneath the fill and topsoil and generally consisted of an upper zone of stiff to hard brown clayey silt and a lower zone of reddish brown clayey silt which is somewhat harder than the overlying till.

The N values determined in the upper till zone ranged from 13 to 43 blows per 0.3 metres. The natural water content of samples of the upper till recovered from the boreholes and test pit ranges from about 11 to 20 per cent. The upper till deposit was fully penetrated in the boreholes and test pit at depths of about 2 to 2.5 metres below the existing ground surface, or at about elevation 103.6 metres.

Standard penetration testing was carried out in the lower till zone, but due to its hard consistency it was not practical to advance the sampler the entire 450 millimetres required to establish an N value. However, N values of the order of greater than 100 blows per 0.3 metres can generally be inferred from the penetration testing. The natural water content of the lower till typically ranges from about 5 to 14 per cent.

Both of the till layers encountered were generally fine grained in nature and no major concentrations of coarse particles such as boulders were encountered during this investigation. This does not necessarily mean that the coarser particle sizes are not present at random or in

concentrations within the deposit, since till is an inherently variable material.

4.5 Shale, completely to slightly weathered (Bedrock)

At the borehole locations, the overburden materials are underlain by shale bedrock of the Queenston Formation. Bedrock was encountered between about elevations 102.1 and 102.8 metres, or at depths of from about 3.1 to 3.7 metres below the existing ground surface.

The upper zone of the bedrock is generally highly weathered and has been described as deformation till in borehole 7 and test pit 3. More competent shale was encountered in borehole 16, at about elevation 101 metres.

4.6 Groundwater Conditions

Groundwater was not encountered during the field drilling operations in boreholes 6 and 7 which were terminated at auger refusal. Minor groundwater seepage into test pit 3 was observed at about elevation 104.9 metres or at a depth of about 1.2 metres below the existing ground surface.

It should be noted that the piezometric groundwater level within the subsoil and underlying bedrock is subject to fluctuation not only due to precipitation conditions, but also due to seasonal variations. The groundwater conditions reported above may therefore not necessarily represent stabilized conditions or conditions which may be encountered during construction.

5. DISCUSSION AND DESIGN RECOMMENDATIONS

5.1 General

This section of the report provides our interpretation of the factual geotechnical data obtained during the investigation. The geotechnical engineering parameters given in the following discussion are intended for design purposes only. Where comments are made on construction, they are provided only in order to highlight those aspects which could affect the design of the project. Contractors bidding on or undertaking the works should make their own interpretation of the subsurface information provided as it affects their proposed construction methods, equipment selection, scheduling and the like.

5.2 Foundations

It is understood that the proposed Bridge No. 33 will consist of a 19 metre wide single span rigid frame some 17 metres in length. The proposed finished deck grade will be at about elevation 111 metres. The proposed finished grade beneath the bridge will range from about elevation 103.7 metres to 104.2 metres. Approach fills of about 7 metres in height will be required.

It is understood that the pier and abutment foundations for Bridges 33, 34, and 35 which are in close proximity to each other will be built during the same construction phase. This will minimize the impact of subsequent construction on completed foundation units and may minimize the disruption to traffic.

Based on the results of the present investigation it is considered that the bridge abutments may be founded on conventional spread or strip footings bearing in the lower till or weathered bedrock at or below about elevation 103.2 metres. Footings founding as outlined above may be designed using bearing pressures of 750 kilopascals at Ultimate Limit States and 500 kilopascals at Serviceability Limit States.

The coefficient of friction between the concrete and the glacial till may be computed by taking ϕ as 26° for the till (unfactored).

In order to achieve the design bearing pressures, and minimize post construction settlement, it is essential that all material at the founding level which is loosened, softened or disturbed during construction excavation be removed from the base of the excavation. Effective control of groundwater during construction is a critical aspect to preserving the integrity of the bearing strata.

For all spread footings it is recommended that a minimum 150 millimetre thick layer of lean concrete be placed within 4 hours of final excavation. To be of optimal benefit, the removal of loosened and disturbed material and the placement of the protective mat should be carried out on a simultaneous basis.

It is recommended that the foundation excavations be inspected by experienced geotechnical personnel prior to any concrete placement to ensure that the base has been adequately prepared, is free of any softened or disturbed zones, and is within the natural till deposit.

Care and adequate protection during winter construction should be provided to prevent any freezing of the foundation subgrade.

For design purposes, a minimum soil cover of 1.2 metres below final grade should be provided to the footing base for frost protection.

5.3 Earth Pressures for Abutments

The lateral earth loads acting on the abutments will depend on the rigidity of the structure, on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill and on the subsequent lateral movement of the structure. The following recommendations are made concerning the design of the abutments.

- Select free draining granular fill in accordance with OPSS Granular "A" or "B" should be used as backfill immediately behind the abutments and retaining walls. The minimum limits of the granular backfill are as outlined in section 6-9.6.1 of the Ontario Highway Bridge Design Code.
- All granular fill should be placed and compacted in accordance with the current OPSS practices and MTO directives.
- Longitudinal drains should be installed to provide positive drainage of the granular backfill.
- If the abutment support allows lateral yielding at its top equal to not less than 0.05 per cent of the retained

height, 'active' earth pressure conditions should apply. If, however, the structure is not permitted to yield by this amount, 'at rest' pressure conditions should be used. The following earth pressure parameters may be used for the calculation of lateral earth pressures in accordance with the Ontario Highway Bridge Design code:

Granular "A"	
Unit weight	22.8 kN/m ³
ϕ	35°
Granular "B"	
Unit weight	21.2 kN/m ³
ϕ	30°

5.4 Excavations

Based on the results of the current investigation, excavations for the foundations will encounter fill and topsoil, glacial till, and possibly weathered bedrock.

Groundwater seepage from the fill and from permeable layers within the till should be anticipated.

As discussed previously, adequate control of the groundwater during construction is essential to minimize deterioration of the bearing strata during final excavation and preparation of the bearing surface prior to mass foundation concreting operations. In addition, the stability of the excavation side slopes may be influenced by the adequacy of the groundwater control measures.

All excavations must conform to the current Occupational Health and Safety Act and care should be taken to direct

surface runoff away from open excavations. Excavation side slopes within the overburden should not exceed an inclination of 1 horizontal to 1 vertical unless adequate temporary support is provided. Some sloughing may occur in localized zones due to groundwater seepage from relatively permeable strata within the overburden slopes.

5.5 Embankments

It is understood that approach fills of about 7 metres in height will be required at the abutment locations. The subgrade encountered in the boreholes and test pit at the proposed abutment locations generally consists of about 1 metre of silty clay fill. The fill was mixed with and underlain by topsoil in some locations and is therefore considered to be poor quality earth fill.

In the event that the embankment fill can be constructed well in advance of the pavements and some post construction deformation can be tolerated, consideration could be given to leaving some of the existing fill in place. As a minimum, however, the toe of the embankment side slopes should be keyed into the native subgrade. The fill subgrade should be stripped of any topsoil and adequately proofrolled under the direction of the geotechnical engineer. Remedial work should be carried out on any soft zones encountered.

Following completion of the preparatory work as outlined above, the embankment fill should be constructed consistent with the current OPSS practices.

Settlement of the embankments constructed as outlined above is estimated to be in the range of about 80 to 120

millimetres. Differential settlements of about 50 per cent of the above range should be anticipated due to consolidation of the existing fill and topsoil beneath the proposed embankments. If settlements of this magnitude are not tolerable, consideration should be given to removing all of the existing fill and topsoil prior to constructing the embankments as a means of reducing the differential settlement. Much of the existing earth fill is not considered suitable for reuse in the construction of earth embankment fill.

For the height of fills required and provided the embankments are constructed as outlined above, side slope inclinations of 2 horizontal to 1 vertical or flatter may be used. The completed embankment side slopes should be blanketed with an appropriate vegetation cover.

The potential for differential settlement (although relatively small), between the abutment footings founded on the till below the approach embankments exists. For design purposes, differential settlements in the order of about 20 millimetres should be considered. Poor construction practices would however result in larger settlement. If the bridge structure is unable to tolerate such differential movements, consideration will have to be given to founding the abutments on bedrock or on caissons taken into the bedrock.

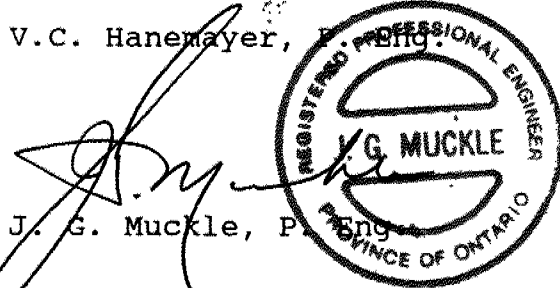
It should be brought to the attention of the contractor by way of a special provision in the contract, that the test pit put down in this investigation and test pits excavated as part of any other preconstruction investigations, are to be located in the field by the contractor to determine whether

they are within the bridge or embankment alignments. Test pits within the embankment areas should be re-excavated and backfilled with appropriate material placed and compacted as outlined above.

GOLDER ASSOCIATES LTD.

Vin Hanemayer

V.C. Hanemayer, P. Eng.



J. G. Muckle, P. Eng.

J. L. Seychuk

J. L. Seychuk, P. Eng.
att.

APPENDIX "A"

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kn/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m^3	SEEPAGE FORCE
γ'	kn/m^3	UNIT WEIGHT OF SUBMERGED SOIL						



METRIC

W P 199-77-05 LOCATION Co-ordinates N4,799,544.5 E277,853.4 ORIGINATED BY CB
DIST 4 HWY QEW/403 BOREHOLE TYPE Solid Stem Auger COMPILED BY MHW
DATUM Geodetic DATE August 20, 1990 CHECKED BY 104

[illegible]

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 7

METRIC

W P 199-77-05 LOCATION Co-ordinates N4,799,553.2 E277,868.4 ORIGINATED BY VCH
 DIST 4 HWY QEW/403 BOREHOLE TYPE Solid Stem Auger COMPILED BY MHW
 DATUM Geodetic DATE August 20, 1990 CHECKED BY VCH

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
105.9	Ground Surface																
0.0	Fill - Silty clay																
105.2	Reddish Brown																
0.7	Topsoil																
0.9	Clayey Silt (Till) Trace to some sand		1	SS	14												
			2	SS	43												
	Stiff to Hard Brown becoming reddish brown at about elev. 103.8 m		3	SS	65/75mm		104										
102.8			4	SS	50/50mm												
3.1	Shale (Deformation Till completely to highly weathered)																
101.9	Reddish Brown		5	SS	70/75mm		102										
4.0	Shale Bedrock Highly to moderately weathered																
101.3			6	SS	70/100mm												
4.6	End of Borehole																
							100										

OFFICE REPORT ON SOIL EXPLORATION

METRIC









W P 199-77-08 LOCATION Co-ordinates N4799544.2 E277872.3 ORIGINATED BY VCH
DIST 4 HWY 403/QEW BOREHOLE TYPE Hollow Stem Auger; NQ Rock Core COMPILED BY MHW
DATUM Geodetic DATE August 29, 1990 CHECKED BY _____

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 (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20						40	60	80	100
								SHEAR STRENGTH kPa						WATER CONTENT (%)			
							○ UNCONFINED + FIELD VANE										
							● QUICK TRIAXIAL x LAB VANE										
105.5	Ground Surface																
0.0	Topsoil																
0.2	Silty Clay trace sand, tr topsoil occ. gravel (Fill)		1	SS	16												
103.8	Stiff to Very Stiff Brown					104											
1.7	Clayey Silt trace to some sand occasional gravel (Till)		2	SS	14												
			3	SS	65/ 100mm												
			4	SS	90/150mm												
101.8	Hard Reddish Brown					102											
3.7	Shale Bedrock highly weathered		5	SS	50/50mm												
101.1	Reddish Brown																
4.4	Shale Bedrock moderately to slightly weathered thinly to medium bedded Reddish Brown interbedded with fine grained argillaceous Limestone		6	NQ RC	*TCR =93% SCR =57% RQD = 8%	100											
			7	NQ RC	TCR =100% SCR =78% RQD =42%												
98.0	End of Borehole					98											
7.5	End of Borehole					96											
* TCR: Total Core Recovery SCR: Solid Core Recovery RQD: Rock Quality Designation																	



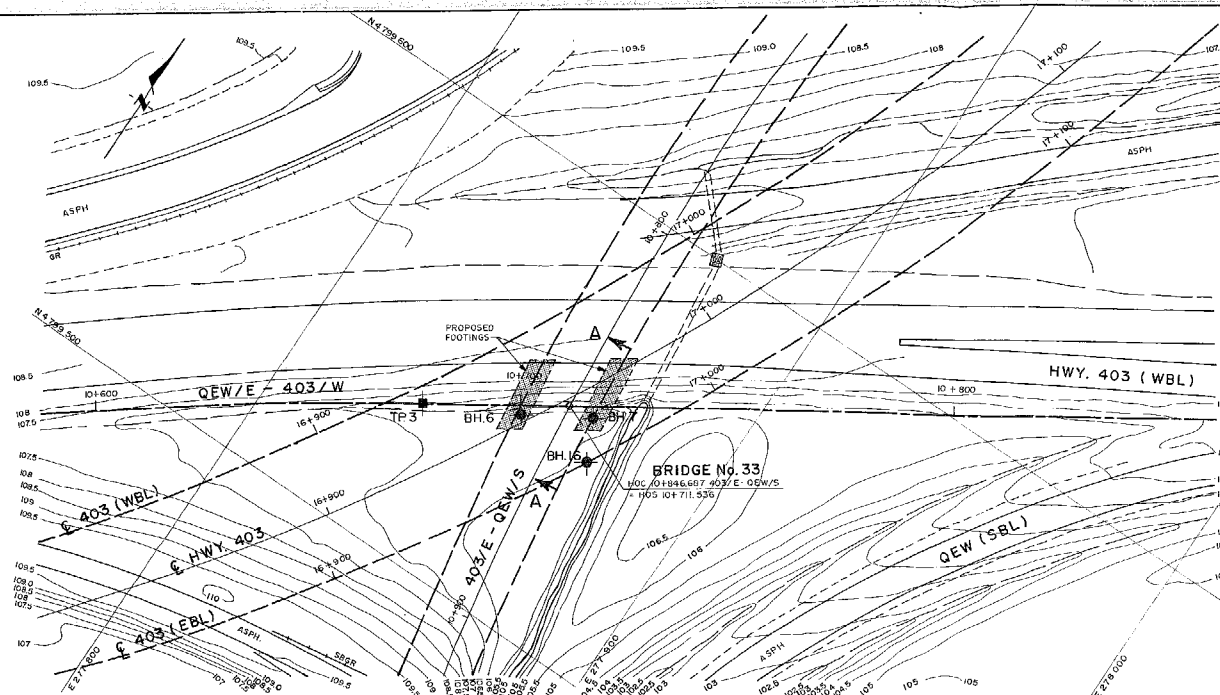
METRIC

W P 199-77-05 LOCATION Co-ordinates N4,799,534 E277,833 ORIGINATED BY VCH
DIST 4 HWY 403/QEW BOREHOLE TYPE Backhoe Dug; John Deere 690 COMPILED BY MW
DATUM Geodetic DATE August 16, 1990 CHECKED BY VCH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	SHEAR STRENGTH kPa	W _p	W			W _L
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE	WATER CONTENT (%)				
106.1	Ground Surface													
0.0	Topsoil						106							
0.3	Silty Clay (Fill)													
105.2	trace sand occ. gravel Brown													
0.9	Topsoil													
1.1	Clayey Silt (Till) trace sand, occ. gravel		1	CS				Water Seepage at elevation 104.9 metres during excavation						
	Brown to Red Brown						104							
102.7														
3.4	Shale (Deformation Till)													
3.7	Bottom of Pit						102							

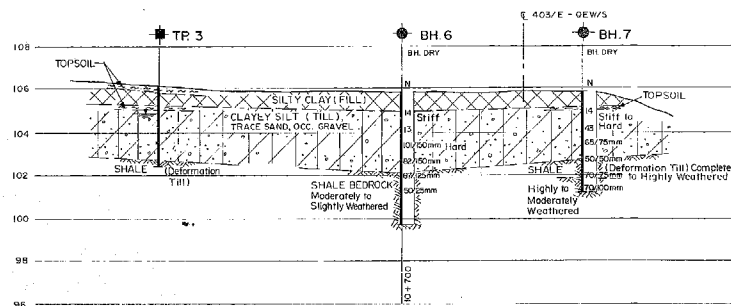
+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION



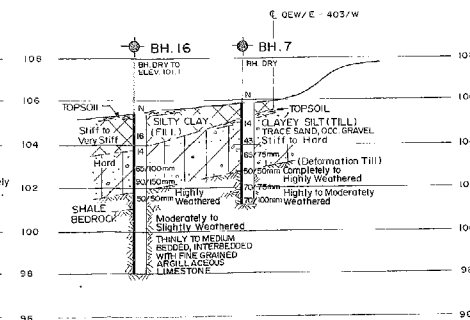
PLAN

0 5 10 20 30 40 50 Metres



Q PROFILE QEW/E - 403/W

HOR. 0 5 10 20 Metres
VERT. 0 2.5 5 10 Metres



SECTION A-A

REF. No. E-83-403-11.00.05

METRIC

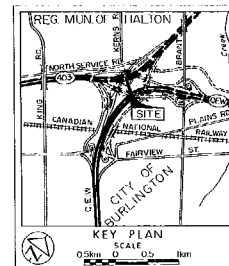
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES - METRES.

CONT No
WP No 199-77-05

BRIDGE No. 33
(QEW/E - 403/W)

BORE HOLE LOCATIONS & SOIL STRATA

GOLDER ASSOCIATES LTD.
CONSULTING ENGINEERS



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 lbf/blow)
- CONE Blows/0.3m (60° Cone, 475 lbf/blow)
- Wt at time of investigation August, 1990
- ⊕ Test Pit

No.	ELEVATION	COORDINATES
		NORTHING EASTING
BH.6	105.3	4,799,541.5 277,853.4
BH.7	105.5	4,799,303.2 277,866.4
TP.3	106.1	4,799,534 277,833
BH.16	105.5	4,799,544.2 277,872.3

NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview, information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

DATE	BY	DESCRIPTION
30/05/97	17	Geotechnical
10/06/97	33	QEW/E-403/W, BRIDGE 33
10/06/97	33	SUBMIT, AM CHECKED, DATE JAN 9, 1991, SILENCE
10/06/97	33	DRAWN BY CHECKED
10/06/97	33	DWG 1997705-A