G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 30M5-191

DIST. 4 REGION

W.P. No. 199-77-01 B

CONT. No. 93-89

W. O. No.

STR. SITE No.

HWY. No. 403/Q.E.W.

LOCATION Hwy 403/Q.E.W
Culvert #5

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

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FOUNDATION DESIGN SECTION

foundation investigation and design report

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 199-77-01B DIST 4

HWY 403/Q.E.W. STR SITE

Culvert #5
Hwy 403/Q.E.W. (Freeman) Interchange
CONT 93-89

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GEOCRES 30M5-191

DATE AUG 27 1992

FOUNDATION INVESTIGATION REPORT
For
Culvert #5, W.P. 199-77-01B
Hwy. 403/Q.E.W. (Freeman) Interchange
District 4, Burlington

INTRODUCTION

This report summarizes the results of a Foundation Investigation conducted in conjunction with the reinforced concrete box culvert (culvert #5) proposed at the Hwy. 403/Q.E.W. (Freeman) Interchange. The culvert will carry the Q.E.W./E - 403/W ramp, Q.E.W. WBL and Q.E.W. EBL and will transmit its waters to the constructed culvert #6 located along the same channel and immediately south of culvert #5.

SITE DESCRIPTION AND GEOLOGY

The site of the proposed culvert is located along the Q.E.W. corridor approximately 0.5 km west of Brant St. in the City of Burlington, Regional Municipality of Halton. The culvert is a component of the Hwy. 403/Q.E.W. interchange and is one of many structures proposed at the interchange. At the time of the investigation, a number of construction activities associated with the construction of the Q.E.W./Hwy. 403 EB ramp structures were in progress immediately west of the proposed culvert #5. The Q.E.W. ramp to Niagara and the existing Hwy. 403 WB is also located immediately west of the site.

The roadways present at the site are separated by grass covered medians and drainage ditches. The roadways include the Q.E.W. Westbound and Eastbound lanes, the Hwy. 403 WB-Q.E.W. EB ramp and the Brant St. S. - Hwy. 403 WB ramp. An existing hydro electric corridor is located south of the site and the existing North Service Rd. is located north of the site.

The terrain surrounding the site is generally flat and consists of grassland and a few isolated deciduous trees in the areas beyond the construction site and existing roadways.

Physiographically, the site is located in the region known as the "Iroquois Plain". The Iroquois Plain is the product of the advance and retreat of the Wisconsinan ice sheet which covered the area during the Pleistocene epoch (over 12,000 years ago). At the site, the lowland bordering Lake Ontario, when the last glacier was receding, was inundated by the glacial lake called Lake Iroquois. Conditions in the old lake plain vary greatly from site to site. At the site location, a thin veneer of a heterogeneous mixture of clayey silt, sand and gravel overlies shale bedrock of the Queenston Formation.

INVESTIGATION PROCEDURE

The field work for the investigation was carried out between 92 03 23 and 92 03 25 inclusive and consisted of three (3) sampled boreholes advanced to depths ranging from 9.1 m to 9.4 m below the existing ground surface. A track mounted Diedrich D50 drilling unit, equivalent to a CME 55, was used to advance the boreholes. Solid stem augering techniques were used to penetrate the overburden and the surficial weathered bedrock. Rock coring techniques employing BW casing and a BQ core barrel were used to retrieve up to 6.1 metres of rock core.

In view of the shallow thicknesses of overburden at the site, most of the field program consisted of rock coring. Rock core were identified in the field and physical index properties were determined by visual examination and also by measurement of rock quality designations (RQD's) and core recovery. All rock core were placed in standard rock core boxes and carefully transported to the laboratory.

The few subsoil samples that were obtained were retrieved using a standard split spoon sampler in accordance with the Standard Penetration Test (ASTM D1586). In general, subsoil samples were retrieved at 0.7 m intervals for the surficial 3 metres. Subsoil samples were identified in the field and then placed in sealed plastic jars to ensure the preservation of the in-situ natural moisture contents. Samples were subsequently transported to the laboratory for further examination and testing.

Groundwater levels were determined by monitoring the water levels in the open boreholes throughout the duration of the field investigation. All boreholes were backfilled upon completion of the field work.

The survey related to the location and elevation of the individual boreholes was provided by Central Region Surveys and Plans.

Laboratory Analyses

All subsoil samples were carefully visually examined in the laboratory in accordance with the procedures outlined in the Visual Method described in Chapter 2 of the MTO Soil Classification Manual. The behaviour, gradation and other pertinent properties of the soil were determined by conducting the appropriate laboratory tests on representative samples. These tests included:

1) Atterberg Limit Tests
2) Particle Size Analysis
3) Natural Moisture Contents
4) Bulk Unit Weights

Sample preparation and testing were conducted in accordance with the MTO Laboratory Testing Manual.

Detailed rock core logging was conducted in the laboratory by an in-house resident geologist.

Laboratory test results have been summarized below in the subsequent section of this report entitled "Subsurface Conditions" and are illustrated on the corresponding boreholes and figures included in the Appendix to this report.

Subsurface Conditions

The existing ground surface elevation at the site varies between 106 m to 107.5 m reflecting the presence of the roadways and adjoining drainage ditches. The subsurface conditions, however, are generally uniform across the site and consists of a natural thin veneer of a cohesive heterogeneous mixture of clayey silt, sand and gravel with a thickness of approximately 1.2 to 1.8 metres overlying completely to highly weathered shale ranging up to 2 metres in thickness which in turn is underlain by more competent shale bedrock with interbedded siltstone. Shallow thicknesses of brown sand and gravel fill material up to approximately 0.8 metres are also present as encountered in the boreholes advanced adjacent to the roadway.

The boundaries between the various soil types, in-situ and laboratory test results as well as groundwater levels established at the time of investigation, are shown on the attached Record of Borehole sheets in the Appendix. A plan of the site illustrating the locations and elevations of the boreholes and a subsoil stratigraphical section are provided on Dwg. 1997701B-A also included in the Appendix.

A detailed description of the subsurface conditions encountered is given below.

Sand and Gravel (Fill Material)

A brown, compact sand and gravel fill material exists as a roadway base at various locations across the site. The thickness of this fill material encountered was approximately 0.8 metres.

Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)

The surficial native deposit spread across the site consists of a heterogeneous mixture of clayey silt, sand and gravel of thickness up to approximately 1.8 metres. Boulders and cobbles, although not encountered during the investigation, are characteristic components of the deposit and hence may exist in this deposit. This deposit directly overlies the shale bedrock across the site and fragments of the parent rock are often found suspended within the lower depths of the cohesive matrix of this till deposit of glacial origin.

The deposit has been oxidized and therefore, is primarily brown in colour. In the lower metre or so of the deposit however, the deposit has a red colour characteristic of the underlying Queenston Shale.

The main component of this unsorted unstratified deposit is the clayey silt material. This material essentially binds the coarser sands and gravels within the deposit. Grain size distribution curves of representative samples of this material as determined by mechanical sieve and hydrometer analyses are given in Figure 1 in the Appendix. The curves illustrate that the fine grained portion of the deposit exceeds 50% of the deposit and hence the deposit is defined by its behaviour.

Atterberg Limit tests were carried out to define the behaviour and plasticity of the fine grained portion of the soil and the results are plotted in Figure 2. The test results reveal that the fine grained portion of the deposit is of low plasticity and hence is categorized as clayey silt. Liquid limits range from 26% to 31% and plasticity indices range from 11% to 13%. Natural moisture contents are slightly less than the plastic limit of the soil.

Standard Penetration tests conducted in this deposit produced "N" values ranging from 13 blows/0.3 m to 33 blows/0.3 m. Based on these "N" values, it can be concluded that the deposit has a consistency ranging from stiff to hard.

<u>Bedrock</u>

The overburden across the site is underlain by shale bedrock with interbedded siltstone of the Queenston Shale Formation. The depth to bedrock ranges from approximately 1.2 m to 2.6 m, equivalent to elevations ranging from 105.5 m to 104.3 m. Based on these elevations, it appears that the bedrock surface slopes slightly downward in a southerly direction. The surficial 1 to 2 metres of the bedrock were severely degraded and weathered. Solid stem augering techniques was used to penetrate this upper severely weathered zone and split spoon samples were retrieved albeit with considerable penetration resistance (typically 100 blows/0.15 m). More competent shale exists beneath the highly weathered zone and below elevations ranging from 103.5 to 103,7 m, the bedrock is unweathered.

The shale bedrock is generally greyish red and has randomly interbedded greenish grey siltstone layers ranging from approximately 25 mm to 200 mm in thickness. The rock is horizontally bedded and is an extremely friable material with a very low slaking durability. The rock contains close to extremely close spaced fractures that are generally flat, planar to undulating and smooth.

Core recoveries and Rock Quality Designations (RQD's) were determined in-situ to evaluate the competence and integrity of the rock. Core recoveries ranged from 78% to 100% but, were generally close to 100%. Rock Quality Designations (RQD's) ranged from 0% to 98% indicating a rock quality ranging from very poor to excellent. However, in general rock qualities were in the 50% to 90% indicating a rock of fair to good quality. Rock quality generally improved with depth.

Rock strength as determined by index property examination in the laboratory is generally weak to very weak.

A detailed description of the characteristics and properties of the rock as determined by the logging of the rock core in the laboratory is attached in the Appendix under the heading "Rock Core Descriptions".

GROUNDWATER CONDITIONS

Observation of the groundwater level was carried out by measuring the water level in the open boreholes throughout the duration of the field investigation. Groundwater levels determined at the time of investigation were uniform across the site and were approximately 2.5 m to 2.7 m below the ground surface (Elevation 104.4 m to 104.0 m).

Groundwater levels, in general, are subject to seasonal fluctuations and hence can vary from the values given in this report.

DISCUSSION AND RECOMMENDATIONS

As part of the Hwy. 403/QEW (Freeman) Interchange complex, it is proposed to construct a single cell reinforced concrete box culvert with cross sectional area dimensions of 5.05 m by 5.05 m and a length of 109 m. Depths of cover above the roof culvert range from approximately 1 to 3 m. Proposed low flow channel culvert invert elevations range from approximately 99.3 m at the northern end of the culvert to 99.0 m at the southern end of the culvert.

The culvert will support various roadways which are all components of the Freeman Interchange design. The roadways include from north to south, the Brant St. S. ramp to 403 W, QEW E to 403 W, QEW WBL and QEW EBL. The culvert will transmit its waters in a southerly direction to the existing culvert #6 situated just south (downstream). A common channel will connect the two culverts.

A plan illustrating the proposed culvert is given on Dwg. 1997701B-A in the Appendix. Recommendations pertaining to the following foundation and geotechnical considerations are included in the purview of this report:

1) Structure Foundations
2) Backfill to Structure
3) Construction Considerations

Structure Foundations

The proposed culvert invert at elevations ranging from 99.3 m to 99.0 m exists in competent, unweathered shale and hence the culvert foundation can be founded on conventional spread footings. For purposes of the O.H.B.D.C., it is recommended that the foundations on the shale bedrock be designed using a factored capacity at U.L.S. of 1000 kPa. The shale bedrock at the proposed invert elevation is considered to be an unyielding foundation base and consequently the bearing capacity at S.L.S. Type II will not govern the foundation design because the pressures required to produce detrimental settlement of the structure founded on bedrock will exceed the values given at

ULS.

Any loosened rock material at the founding level shall be removed and replaced with mass concrete. In addition, to protect the shale bedrock from degradation as a result of weathering and construction related activities, it is recommended that a concrete working slab be placed on the shale bedrock within 4 hours of exposure. The shale bedrock should be regarded as frost susceptible and hence adequate cover shall be provided for footings subject to frost penetration as for instance during winter construction. The frost penetration depth at the site area is 1.2 m.

Adequate provision must be made to control erosion and undermining beneath the culvert base at the culvert inlet and outlet. This can be achieved by constructing aprons and rip rap at the culvert inlet and outlet. The design of the scour protection shall be made in conjunction with applicable hydrological parameters.

BACKFILL TO STRUCTURE

Fill material in the order of 5 m will be required at the culvert sides and an additional 1 to 3 m will be required above the culvert roof elevation. Recommendations pertaining to the selection of material type, stability and settlement of the approach fills and method of construction are given below.

Material

It is recommended that to prevent hydrostatic pressure build-up on the culvert walls, backfill material against the culvert wall consist of Granular "A" or Granular "B". Design parameters of the soil are given in Table 1 below. Weep holes should also be designed in the walls to facilitate drainage.

Table 1 - Backfill Properties

	Granular "A"	Granular "B"
Unfactored Angle of Internal Friction (ϕ)	35^0	30^0
Unit Weight (kN/m^3)	22.8	21.2
Coefficient of Earth Pressure at Rest (Ko)*		
- S.L.S. Type II	0.43	0.5
- U.L.S.	0.51	0.58

* Horizontal surface backfill only.

Granular backfill geometries are illustrated on OPSD 800 series. The backfill beyond the granular wedge as illustrated on the OPSD drawings can consist of acceptable borrow material as defined in OPSS 212.05.

Stability

There are no longitudinal fill slope instabilities anticipated for slopes constructed at 2H:1V. All slopes should be protected against surface erosion using conventional methods.

Settlement

It is anticipated that approximately 25 mm of settlement will be realized within the fill itself. These settlements will be realized during or immediately following construction.

Backfill Construction

In the placement of the backfill material, all softened material should be excavated for their full depth within the plan limits prior to fill placement.

The backfill shall be constructed in 300 mm lifts on alternating sides of the rigid box structure so that the maximum differential in backfill heights at no time exceeds 300 mm. All backfill shall be constructed in accordance with OPSS 902 series and applicable OPSD 803 series. The backfill shall be compacted to achieve the target maximum dry density as outlined in OPSS 501.07.08.

CONSTRUCTION CONSIDERATIONS

General

A two stage construction scheme has been proposed to enable the construction of the reinforced concrete box culvert across the Q.E.W. and associated ramps. The present scheme includes two temporary detours and a division of construction into the following two phases:

Phase I - construction of 61 m long south portion of culvert

Phase II - construction of 48 m long north section of culvert

Considerations for the construction of the culvert are outlined below.

Roadway Protection

To facilitate the construction of the culvert according to the plan described above, a temporary shoring scheme will be required in combination with the proposed detours such that traffic can be maintained along the Q.E.W. WB during construction. It is recommended that a conventional soldier pile - timber lagging shoring wall be employed to allow vertical cut excavations up to approximately 8 metres into the existing sand and gravel fill material, native heterogeneous mixture of clayey silt, sand and gravel and shale bedrock.

The design of the shoring system shall include the appropriate earth pressures computed in accordance with Section 6.6.1.2 of the OHBDC. The loadings induced by the surcharge traffic and adjustment for any sloping surfaces shall be incorporated in the design. Soil and rock design parameters to facilitate the shoring wall design are summarized in Table 2 below.

Table 2 - Shoring Design Soil Parameters

A. SOIL

Type	Elevation (m)	Saturated Unit Weight γ_{sat}, (kN/m^3)	Effective Shear Strength Parameters (ϕ)
Sand & Gravel (Fill)	Roadway Surface - 106	20	30^0
Het. Mixture of Clayey Silt, Sand & Gravel (Glacial Till)	106 - 104.5	21	30^0

B. ROCK

Type	Elevation (m)	Saturated Unit Weight γ_{sat}, (kN/m^3)	Unconfined Compressive Strength (kPa)
Weathered Shale	104.5 - 103.5	20	1000
Unweathered Shale	<103.5	22	10000

The shoring system must be designed to satisfy earth pressure equilibrium. This equilibrium can be achieved by a cantilevered wall with soldier piles socketed into the bedrock at a required depth beneath the proposed culvert invert. Should the embedment length of the soldier piles be considered excessive contributing to an uneconomical design, consideration can be given to a raker supported shoring wall. Rakers can be installed in front of the wall with footings founded on the weathered or unweathered shale. In the weathered shale, raker footings can be designed using a bearing capacity at S.L.S Type II of 500 kPa and an ultimate capacity at U.L.S. of 1000 kPa. In the unweathered shale, the bearing capacities recommended for the culvert foundations can be applied for the design of the raker foundations. Rakers must be installed while an earth berm remains in front of the soldier pile. Slots should be cut into this berm to install rakers before the supporting berm is removed.

It is recommended that soldier piles be installed in preaugered holes and constructed with concrete soldier pile toes. Furthermore, it is recommended that the annular space surrounding the soldier pile above the concrete toe be filled with a lean mix concrete such that this material can be easily removed to facilitate the installation of the lagging boards.

The bedrock exposed along the shoring wall vertical face must be protected against deterioration as a result of exposure to wetting and drying cycles. This can be achieved by extending the lagging to the bottom of the excavation or alternatively placing shotcrete and mesh on the rock face within 24 hours of being exposed.

A step by step procedure that identifies the construction sequencing for the installation of the soldier pile-lagging wall shall be included on the contract drawings. A special provision outlining shotcrete materials, equipment and method of placement shall also be included in the contract documents.

The most economical and technically feasible shoring alternative shall be adopted for the design.

Excavations and Dewatering

Based on the results of the subsurface investigation, excavation for the culvert will extend through the surficial overburden ranging in thickness up to 2.6 metres and up to approximately 5.5 metres of bedrock. No major dewatering problems are anticipated and any groundwater seepage or surface runoff can be easily discharged employing conventional sump pumping techniques. However, it is hereby reiterated that the culvert foundation bearing surface be protected against free water by placing a concrete working slab and directing surface runoff away from open excavations.

It is recommended that side slopes within the overburden and the weathered shale be excavated at 1½H:1V perpendicular to the shoring wall and at 2H:1V parallel to the shoring wall. Excavations within the unweathered shale can be carried out at near vertical up to 4.5 metres and at 1H:1V beyond 4.5 metres. The condition of the excavation side slopes should be monitored on a regular observation basis during construction as some ravelling of the slopes will develop due to the deterioration of the exposed rock.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of T. Sangiuliano, Foundation Engineer, utilizing equipment owned and operated by Master Soils Investigation.

The project was carried out by T. Sangiuliano under the general supervision of P. Payer, Senior Foundation Engineer. The report was written by T. Sangiuliano, reviewed by P. Payer and approved by Mr. M.S. Devata, Chief Foundation Engineer.

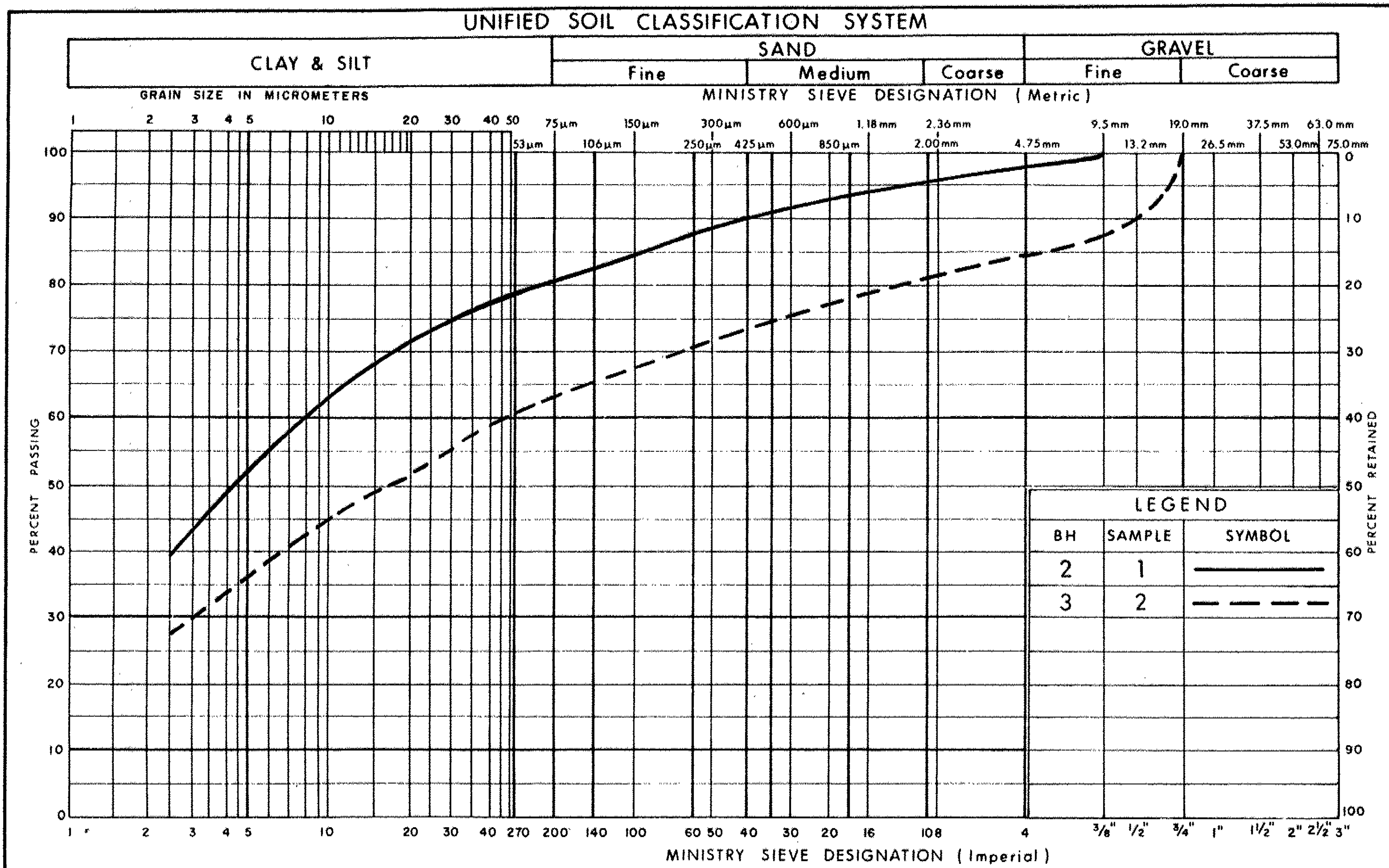
LICENSED PROFESSIONAL ENGINEER
A. J. Sangiuliano
PROVINCE OF ONTARIO

T. Sangiuliano, P. Eng.
Foundation Engineer

M.S. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX

78 12 M



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GRAIN SIZE DISTRIBUTION

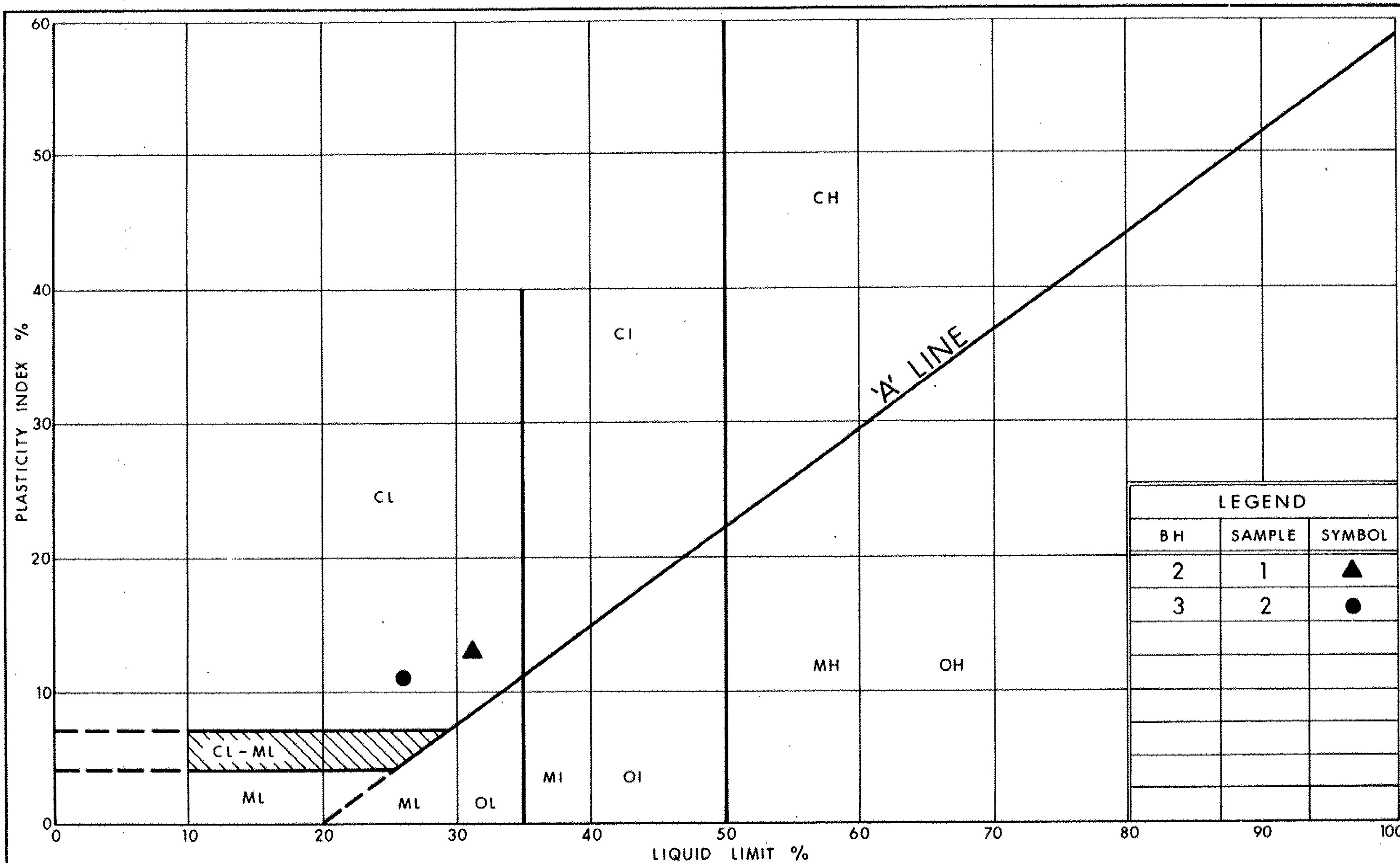
HETEROGENEOUS MIXTURE OF

CLAYEY SILT, SAND & GRAVEL (Glacial Till)

FIG No 1

W P 199 - 77 - 01 B

Oct 75, FF-S-21



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PLASTICITY CHART
HETEROGENEOUS MIXTURE OF
CLAYEY SILT, SAND & GRAVEL (Glacial Till)

FIG No 2
W P 199-77-01B

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 $\overline{N}$.

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:

RQD (%)	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

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

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'	$-^\circ$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^\circ$	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 1

METRIC

W.P. 199-77-018 LOCATION Co-ords: N 4 799 837 E 278 240 ORIGINATED BY TS

DIST 4 HWY QEW BOREHOLE TYPE SS Auger, BW Casing, BQ Rock Core COMPILED BY TS

DATUM Geodetic DATE 92 03 23 CHECKED BY PP

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.7	Ground Surface						
0.0	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till) Brown, Hard					106	
105.5		1	SS	33			
1.2		2	SS	100	/15cm		
	Weathered				▼*	104	
	Unweathered	3	SS	100	/15cm		
	Shale Bedrock with interbedded Siltstone	4	RC	REC 95%			RQD = 43%
	Red with interbedded Grey, Weak to Very Weak	5	RC	REC 100%		102	RQD = 78%
		6	RC	REC 98%		100	RQD = 64%
97.4		7	RC	REC 100%		98	RQD = 88%
9.3	End of Borehole						
	* 92 03 26						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: o UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m^3

$+^3$, $\times^5$: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\diamond}}5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W.P. 199-77-01B   LOCATION Co-ords: N 4 799 795 E 278 268   ORIGINATED BY TS

DIST 4   HWY QEW   BOREHOLE TYPE SS Auger, BW Casing, BQ Rock Core   COMPILED BY TS

DATUM Geodetic   DATE 92 03 24   CHECKED BY PP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.9	Ground Surface								
0.0	Sand and Gravel (Fill Material)								
106.1	Brown								
0.8	Heterogenous Mixture of Clayey Silt, Sand and Gravel (Glacial Till)	1	SS	13		106	plotted	22.2	2 17 44 37
	Brown / Red	2	SS	20					
104.6	Stiff to Very Stiff								
2.3	Weathered	3	SS	100	/15cm ▼*				
						104			
	Unweathered	4	SS	100	/15cm				
	Shale Bedrock with interbedded Siltstone	5	RC	REC 78%					RQD = 0%
	Red with interbedded Grey, Weak to Very Weak	6	RC	REC 98%		102			RQD = 70%
		7	RC	REC 97%		100			RQD = 88%
97.8		8	RC	REC 100%		98			RQD = 81%
9.1	End of Borehole								
	* 92 03 26								

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30.

$+^3$, $\times^5$: Numbers refer to Sensitivity $15 \overset{20}{\underset{10}{\Diamond}} 5$ (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

METRIC

W.P. 199-77-01B
LOCATION Co-ords: N 4 799 742 E 278 288
ORIGINATED BY TS
DIST 4 HWY QEW
BOREHOLE TYPE SS Auger, BW Casing, BQ Rock Core
COMPILED BY TS
DATUM Geodetic
DATE 92 03 25
CHECKED BY PP

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L (WATER CONTENT %)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
106.9	Ground Surface								
0.0	Sand and Gravel (Fill Material) Brown								
106.1 / 0.8	Heterogeneous Mixture of Clayey Silt, Sand and Gravel (Glacial Till) Stiff to Very Stiff Brown / Red	1	SS	14		106			
		2	SS	16			○├──┤	21.6	15 22 38 25
104.3 / 2.6		3	SS	80	▼*				
	Weathered / Unweathered	4	SS	60	/8cm	104			
		5	RC	REC 98%					RQD = 0%
	Shale Bedrock with interbedded Siltstone. Red with interbedded Grey, Weak to Very Weak	6	RC	REC 100%		102			RQD = 58%
		7	RC	REC 100%		100			RQD = 88%
97.5		8	RC	REC 100%		98			RQD = 98%
9.4	End of Borehole * 92 03 26								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100
WATER CONTENT (%): 10 20 30

$+^3, \times^5$: Numbers refer to Sensitivity

$15\overset{20}{\underset{10}{\diamond}}5$ (%) STRAIN AT FAILURE

ROCK CORE DESCRIPTION
WP 199-77-01B

CORE RECOVERY					CORE DESCRIPTION	
BH#	RC#	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
1	4	3.20-4.72	95	43	3.20-9.30	**SHALE,** greyish red, with interbedded greenish grey **SILTSTONE** (17%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	5	4.72-6.25	100	78		
	6	6.25-7.77	98	64		
	7	7.77-9.30	100	88		
2	5	3.20-4.72	78	0	3.20-9.09	**SHALE,** greyish red, with interbedded greenish grey **SILTSTONE** (17%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	6	4.72-6.25	98	70		
	7	6.25-7.77	97	88		
	8	7.77-9.09	100	81		
3	5	3.28-4.80	98	0	3.28-9.37	**SHALE,** greyish red, with interbedded greenish grey **SILTSTONE** (17%); very fine grained; weak to very weak; unweathered to slightly weathered; fractures moderately close to extremely close spaced, flat to near vertical, planar to undulating, smooth.
	6	4.80-6.32	100	58		
	7	6.32-7.85	100	88		
	8	7.85-9.37	100	98		

***CR** = CORE RECOVERY
***RQD** = ROCK QUALITY DESIGNATION

(NOTE: Depths are approximated where core recovery is less than 100%)

Logged by: DAW, Soils and Aggregates Section

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-207 88 10

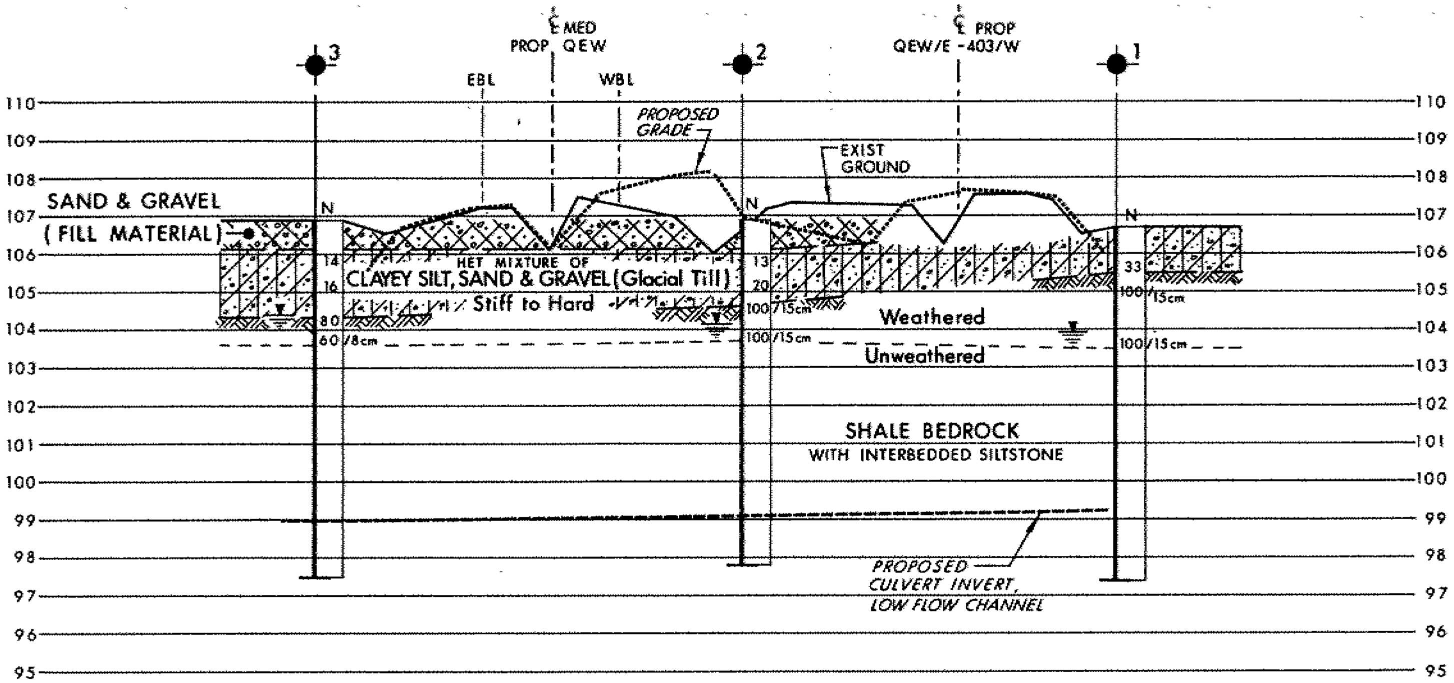
METRIC

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

CONT No
WP No 199-77-01B

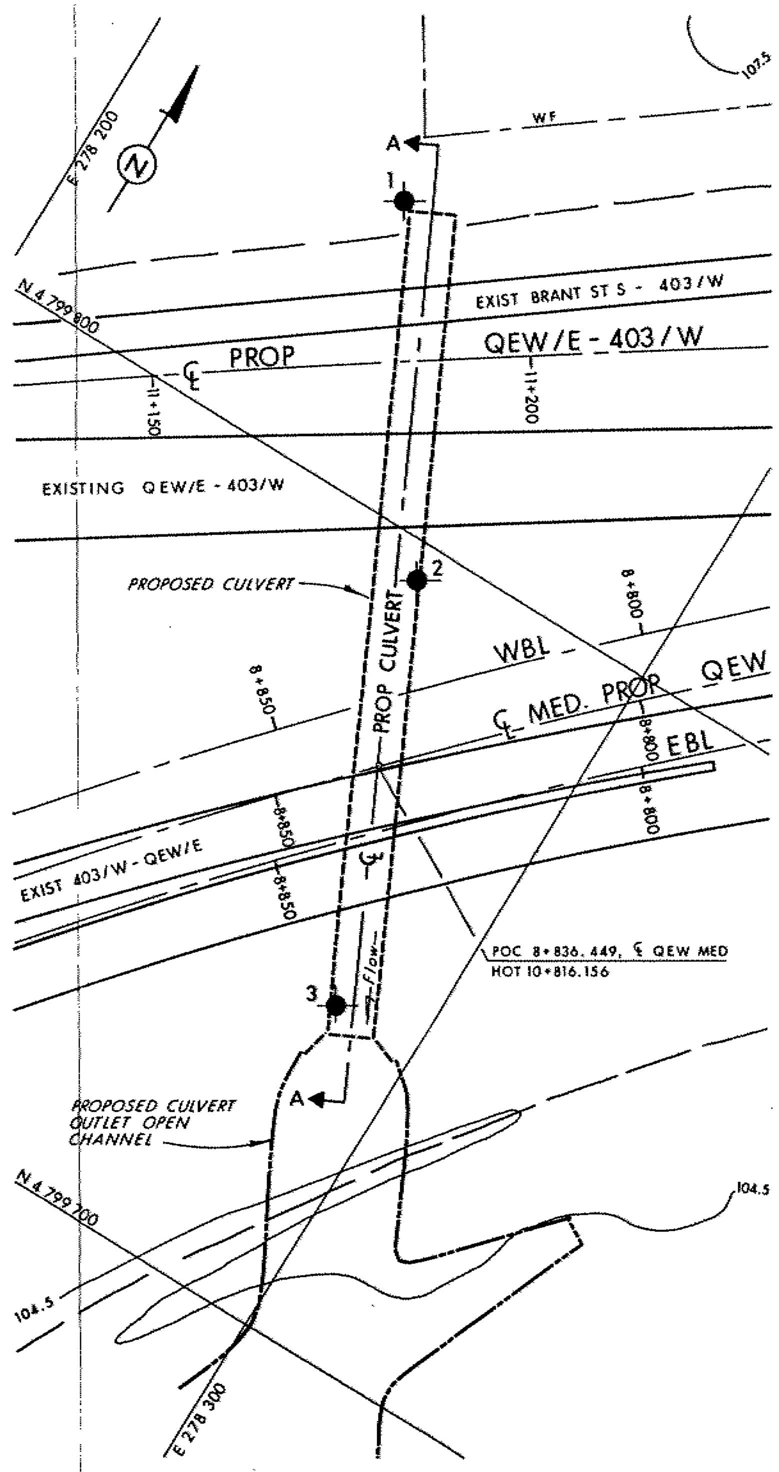
CULVERT No 5
HWY 403/QEW (FREEMAN) INT.
BORE HOLE LOCATIONS & SOIL STRATA

SHEET



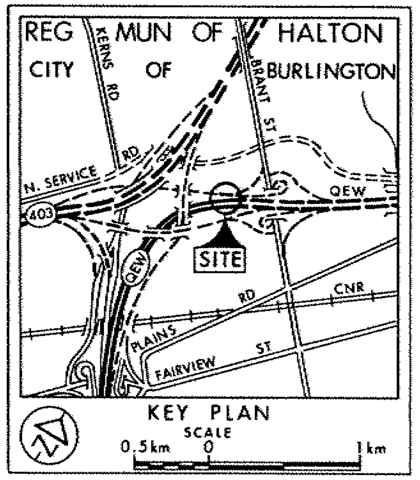
SECTION A-A

SCALES
10m 5 0 10m HOR
2m 1 0 2m VERT



PLAN

SCALE
10m 5 0 10m



KEY PLAN
SCALE
0.5km 0 1km

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W L at time of investigation 1992 03

No	ELEVATION	CO-ORDINATES NORTH	CO-ORDINATES EAST
1	106.7	4 799 837	278 240
2	106.9	4 799 795	278 268
3	106.9	4 799 742	278 288

NOTE

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.

REV	DATE	BY	DESCRIPTION

Geocres No 30M5-191

REF No Sheet 40, STA 9+180 to STA 8+850 and Sheet 43, STA 8+850 to STA 8+515 QEW, McCormick Rankin, Cons. Eng.

HWY No QEW			DIST 4
SUBM'D TS	CHECKED TS	DATE 1992 08 17	SITE
DRAWN RS	CHECKED	APPROVED	DWG 1997701B-A

memorandum

Ontario

To: V. Boehnke
Head, Structural Section

Date: 92 09 25

Attn: M. Bendayan
Senior Structural Engineer

From: Foundation Design Section
Room 315, Central Building

Re: Retaining Walls at Culvert #5
GWP 199-77-00
District 4, Burlington

Further to our telephone conversation dated 92 09 25, summarized below are the parameters and recommendations to facilitate the design of the sliding resistance of the retaining walls proposed at the inlet and outlet of culvert #5.

1. In the sound, competent bedrock at or below elevation 100 m, an unfactored angle of internal friction of **24°** can be used between the concrete footing and the bedrock.

2. Should an inadequate factor of safety against sliding exist, horizontal resistance can be augmented by incorporating a shear key in the rock or by installing dowels in the rock. Shear keys and dowels can be designed using an unconfined compressive rock strength of **10,000 kPa** or a horizontal capacity of **1000 kPa**. The horizontal capacity is applicable at the Ultimate Limit State(ULS). The bearing capacity at the Serviceability Limit State will not govern because the pressures to produce any detrimental displacement will exceed the capacity at the ULS.

Shear keys and dowels shall be a minimum of **0.5 m** in depth. Steel dowels shall be installed in drilled holes with the annular space between the dowel and drilled hole filled with a non shrinkable grout.

3. Lateral earth pressures shall be computed using the soil parameters tabulated in Table 1 of the original foundation report (pg 9). For an unyielding type of retaining wall, the active earth pressure coefficient shall be used in the computation of lateral earth pressures. The appropriate consideration shall be given to any sloped surface behind the retaining wall.

We trust the above is sufficient to enable the design of the retaining walls to proceed. If you any queries regarding the parameters or recommendations, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

P. Payer, P. Eng.
Senior Foundation Engineer

memorandum

Ontario

To:	V. Boehnke Head, Structural Section	Date:	92 09 14
Attn:	M. Bendayan Senior Structural Engineer		
From:	Foundation Design Section Room 315, Central Building		
Re:	Granular Backfill to Structure Culvert # 5, Freeman Interchange WP 199-77-00 District 4, Burlington		

Having reviewed McCormick Rankin Consulting Engineers comments regarding the backfill to the rigid frame box culvert when the frost line is above the top of the culvert as proposed at the above mentioned culvert #5, our concurrence is hereby given to the recommendation of placing the granular backfill similar to OPSD 803.01 as suggested in the Consultant's letter dated September 9, 1992.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

P. Payer, P. Eng.
Senior Foundation Engineer

McCORMICK RANKIN

CONSULTING ENGINEERS

September 9, 1992

Mr. M.D. Bendayan, P. Eng.
Structural Section
Central Region
Ministry of Transportation
1201 Wilson Avenue
4th Floor, Atrium Tower
Downsview, Ontario
M3M 1J8

RE: W.P. 199-77-01 Q.E.W./403
Freeman Interchange Phase 2
District 4 Burlington
Our File: W.O. 2131-100

Dear Sir:

In reviewing the Foundation Investigation and Design Report for Culvert #5 it was noted that this rigid frame box culvert is to be granular backfilled in accordance with OPSD 803 series standard drawings. OPSD 803.02 "Granular Backfill for Rigid Frame Box and Open Concrete Culverts" does not show a requirement for granular backfill when the frost line is above the top of culvert as is the case with Culvert #5 (depth top of pavement to top of culvert is 1.8 metres minimum).

OPSD 803.01 "Granular Backfill for Non Rigid Frame Open or Box Concrete Culverts" could be applied to Culvert #5 as it does require granular backfill 600 mm wide adjacent to the culvert walls when the frost line is above the top of culvert. Also the 600 mm width corresponds to the additional 600 mm extra excavation width to place the mass concrete. Culvert #5 is in shale and the bottom 4.5 metres can be excavated to near vertical according to the report.

Would you please review the backfill requirements of Culvert #5. We are enclosing pages 7 through 13 of the Foundation Investigation and Design Report, OPSD 803.01, OPSD 803.02 and typical excavation details associated with the culvert.

Yours very truly,

McCORMICK RANKIN

J.D. Elliott, P. Eng.

JDE:dr:67
Encl.
cc: E. Salva, Planning & Design
R.C. McCormick, Senior Project Manager
Acting for MTO
R. MacLean, Geotechnical Section

memorandum

Ontario

To:	V. Boehnke P. Eng. Head, Structural Section Central Region	Date:	1992 07 24
Attn:	M. Bendayan Senior Structural Engineer		
From:	Foundation Design Section Room 315, Central Bldg.		
Re:	Culvert #5 at the Freeman Interchange G.W.P 199-77-01 District 4, Burlington		

The NSSP prepared for the "shotcrete" and the drawing illustrating the roadway protection for stage 1 construction of the above mentioned culvert have been reviewed and are in accordance with our recommendations. We therefore have no further comments.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

P. Payer, P. Eng.
Senior Foundation Engineer

TS/nd

Report issued as
WP 199-77-01B

7540-1021

SEND TO: MR. TONY SANGIULIANO, P.Eng.
FOUNDATION SECTION
CENTRAL BLDG. - ROOM NO. 315

FROM: Morris Boudayan | DEPT.: Structural Section | DATE: 92-07-08

SUBJECT: CULVERT NO. 5 AT THE FREEMAN INTERCHANGE; G.W.P. 199-77-01

Dear Tony,

On preparing the drawings and documentation for the design and construction of subject culvert I came up with some points and questions for which I would appreciate your assistance.

1) Is the thickness of shotcrete to be 80 mm?

2) Should it not be stated in the Foundation Report that the shotcrete be applied within 24 hours of rock exposure? Does the shotcrete have to be applied on ~~both the~~ 1:1 and vertical surfaces?

3) Should the B.H. #5 shown on your plan be revised to read B.H. #1 (as noted on the Borelog sheet)?

REPLY

4) Attached is a "modified Special Provision" prepared for the shotcrete application, ~~based on~~ one used on a previous retaining wall construction project. Kindly review and comment on same.

Thank you for your valuable assistance.

REPLY FROM

REPLY DATE

memorandum

Ontario

To: V. Boehnke P. Eng.
Head, Structural Section
Central Region

Date: 1992 07 13

Attn: M. Bendayan
Senior Structural Engineer

From: Foundation Design Section
Room 315, Central Bldg.

Re: Culvert #5 at the Freeman Interchange
G.W.P 199-77-01
District 4, Burlington

Report issued under WP 199-77-01 B

The following is a response to the requests and questions concerning the proposed culvert #5 that were submitted to our office in a memo dated 92 07 08.

1) Shotcrete Thickness

It is recommended that a shotcrete thickness of a minimum of 80 mm be applied to protect the bedrock face.

2a) Time Constraint

Shotcreting shall be carried out to completion within 24 hours of initial exposure. This has already been recommended in the foundation report (see page 12), but should also be included in the special provision.

b) Application Areas

All vertical faces shall be shotcreted in accordance with the provisions given. However, the temporary 1H:1V sloped surfaces do not require any shotcrete protection.

3) Borehole Identification

The number "5" indicated in your memo is NOT an identification of the borehole. The borehole at this location (culvert outlet) is BH #1.

4) Modified Special Provision

The special provision regarding the shotcreting has been reviewed and the following comments are provided.

1) The special provision shall be revised to reflect the minimum 80 mm thickness as discussed earlier.

2) It is recommended that another item be included in the special provision entitled "Rock Face Preparation". This item shall specify that "any loosened rock be removed prior to the placement of shotcrete".

We trust the above comments answer the questions and uncertainties outlined in your memo. Should you require additional information or clarification, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

P. Payer, P. Eng.
Senior Foundation Engineer

TS/nd

memorandum

Ontario

To: V. Boehnke
Head, Structural Section

Date: 92 06 30

Attn: M. Bendayan
Senior Structural Engineer

From: Foundation Design Section
Room 315, Central Building

Re: Temporary Roadway Protection Scheme
Culvert # 5, Freeman Interchange
G WP 199-77-00
District 4, Burlington

Report issued under
WP 199-77-01 B

The sketch illustrating the proposed roadway protection scheme required to facilitate the construction of the above mentioned structure has been reviewed. The scheme is in compliance with our recommendations and hence we have no further comments.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

P. Payer, P. Eng.
Senior Foundation Engineer

memorandum

Ontario

To:	V. Boehnke, P. Eng. Head, Structural Section Central Region 4th Floor, Atrium Tower	Date:	92 04 24

Attn: M. Bendayan
Senior Structural Engineer

From: Foundation Design Section
Room 315, Central Building

Re: Rock Excavation at Culvert #5
QEW/HWY 403 (Freeman Interchange)
GWP 199-77-00
District 4, Burlington

Report issued under WP199-77-01B

This memorandum confirms our telephone discussion dated 92 04 24 regarding the excavation of unweathered shale bedrock at the above mentioned site. It is hereby submitted that any vertical cut excavation can be extended to a depth of 4.5 metres within the unweathered shale provided that the rock face be protected against deterioration by shotcreting methods for the full extent of the excavation cut as discussed in the foundation report.

We trust the above recommendation will facilitate the design of the proposed culvert at the site. Should you require any additional information regarding the foundation aspects of the design, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

M. Devata, P. Eng.
Chief Foundation Engineer