

FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
CNR BRIDGE  
QEW/HIGHWAY 420 INTERCHANGE  
W.P. 123-00-01  
NIAGARA FALLS, ONTARIO

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May, 2001

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## FOUNDATION INVESTIGATION REPORT

For  
CNR Bridge  
QEW/Highway 420 Interchange  
W.P. 123-00-01  
Niagara Falls, Ontario

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### INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed replacement of the existing single-span structure that carries the southbound lanes of the QEW over the CN railway at the QEW/Highway 420 interchange in Niagara Falls, Ontario. The investigation was conducted for Philips Engineering Ltd. on behalf of the Ontario Ministry of Transportation.

The proposed overpass structure will comprise a single span. The QEW southbound lanes and exit ramp to Lundy's Lane will cross the CN railway at approximate Station 15+666, QEW chainage.

Road grade over the structure will be raised approximately 0.6 m above the existing grade. The CN railway is near elevation 194.4 under the existing bridge.

The report pertains to the proposed bridge structure and approaches within about 20 m of the abutments.

### SITE DESCRIPTION

The existing structure carries the southbound lanes of the QEW and the N(QEW) – W(Lundy's Lane) ramp over the CN railway. The QEW is constructed on an approach embankment raised above the grade of the surrounding lands.



The surrounding lands are developed for commercial, light industrial, and residential purposes.

The site is located in the broad physiographic region known as the Haldimand Clay Plain. In general, the topography on the plain is relatively flat to undulating. The overburden is typically some 10 to 15 m thick and typically comprises deposits of glaciolacustrine clay, silt and sand. Bedrock consists of dolostone of the Lockport Formation.

### **INVESTIGATION PROCEDURES**

The fieldwork was carried out during the period January 17 to 19, 2001 and comprised six boreholes drilled at the locations indicated on Drawing 1, appended. Four boreholes were extended to bedrock at the corners of the abutments, two of which were advanced approximately 3.0 m and 4.3 m into bedrock. Two boreholes were drilled to depths of about 8.1 and 9.6 m in the approaches some 20 m beyond the abutments.

The borehole locations were selected by Peto MacCallum Ltd., subject to access limitations in the field. The MTO co-ordinates and ground surface elevations at the boreholes were interpolated from untitled and undated drawings provided by Philips Engineering Ltd.

The boreholes were advanced using continuous flight solid stem augers and NXL rock coring equipment, powered by a truck-mounted CME-75 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

Representative samples of the overburden were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata.

In the deep boreholes drilled at opposite corners of the north and south abutments, casing was extended to the bedrock surface and an approximate 3.0 and 4.3 m length of rock core was recovered using NXL rock coring equipment.

The groundwater conditions in the boreholes were closely monitored during the course of the fieldwork.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. Grain size distribution analyses and Atterberg Limits tests were carried out on selected samples.

### **SUMMARIZED SUBSURFACE CONDITIONS**

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, elevation of the boundary between stratigraphic units, standard penetration test "N" values, and groundwater observations. The results of laboratory grain size distribution analyses, Atterberg Limits tests, and moisture content determinations are also shown.

The borehole locations and a stratigraphic profile prepared from the borehole data are presented on Drawing 1.

The subsurface stratigraphy revealed at the CN bridge site generally comprised a surficial pavement structure overlying approach fill over a topsoil layer. The fill/topsoil was underlain by native clay, silt and/or sand deposits mantling bedrock. The strata encountered are summarized below:



### Pavement Structure

The surficial pavement structure was encountered in boreholes 1 and 3 to 5, and varied in thickness from 330 to 1200 mm, typically 330 mm to 460 mm. The pavement structure comprised 145 to 255 mm, locally 25 mm, of asphalt over crushed limestone and/or concrete. The concrete thickness ranged from 195 to 305 mm.

### Fill

Fill was revealed surficially in boreholes 2 and 6 and below the pavement structure in the remaining boreholes. The fill typically comprised loose to compact, locally dense, non-cohesive silt to silty fine sand and locally stiff cohesive silty clay. The moisture content of the fill ranged from 7 to 25%, typically 15 to 20%. The fill layer ranged from 6.7 to 7.9 m in thickness and was penetrated at depths of 7.2 to 7.9 m (elevation 194.0 to 194.9). Drilling was terminated within the fill in borehole 5 at a depth of 8.1 m. The results of particle size distribution analyses conducted on the fill are presented on Figure 1.

### Topsoil

A 0.7 to 1.6 m thick topsoil layer was encountered beneath the fill in three of the abutment boreholes (1, 2 and 3) and one of the approach holes (borehole 6). The topsoil consisted of non-cohesive silt or sandy silt and was loose to very loose. The moisture content of the topsoil ranged from 18 to 27%. The topsoil was contacted at depths ranging from 7.2 to 7.9 m, typically 7.9 m (elevation 194.0 to 194.9) and was penetrated in all four boreholes at depths of 8.6 to 9.5 m (elevation 192.5 to 193.5).

### Clay

A layer of cohesive silty clay was encountered below the topsoil in boreholes 1, 2, 3 and 6. The consistency of the clay ranged from very stiff to hard with a moisture content range of 20 to 22%. The clay layer was 0.8 and 1.5 m thick in boreholes 2 and 3 and was penetrated at depths of 10.2 and 10.4 m (elevation 191.7 and 191.9). In situ testing/sampling in borehole 1 was terminated in clay at 9.6 m depth and the hole was extended to bedrock by power augering; drilling was terminated within the clay in borehole 6 at a depth of 9.6 m.

The results of particle size distribution analyses conducted on the clay are presented on Figure 2. Liquid limits of 48 and 53 and plasticity indices of 27 and 30 were determined on two selected samples of the clay, indicating a medium to high plastic clay (refer to Figure 4).

### Silt to Fine Sand

A major non-cohesive deposit of silt to fine sand with silt was contacted at 7.3 to 10.4 m depth (elevation 191.7 to 194.6) in boreholes 2, 3 and 4. The relative density of the silt/sand was dense to very dense, locally compact in borehole 4. The silt/sand deposit was 9.0 and 9.7 m thick in boreholes 2 and 3. In situ testing/sampling in borehole 4 was terminated 0.8 m into the silt/sand at 8.1 m depth and the hole was extended to bedrock by power augering.

The results of the grain size distribution analyses conducted on the silt/sand are presented on Figure 3. In general, the silt/sand was non-plastic with a trace of clay, locally with lenses/layers of clay. Moisture contents ranged from 11 to 21%. The silt mantled bedrock in boreholes 2 and 3.



### Bedrock

Bedrock or probable bedrock was contacted below the overburden in boreholes 1 to 4 at the following depths and elevations:

Location	Depth to Rock (m)	Bedrock Elevation
North Abutment, East End	19.5	182.5
North Abutment, West End	20.0	182.2
South Abutment, East End	19.2	182.7
South Abutment, West End	18.6	183.3

The bedrock consists of dolostone. A geologic description of the rock cores recovered from boreholes 2 and 3 is provided in Table I. Core recovery was typically 100%, 95% in one run.

The RQD determined from the rock cores ranged from 18 to 30% (very poor to poor quality) in the upper 2.7 m at the east end of the south abutment, increasing to 80% (good quality) in the bottom 1.5 m. The RQD determined from core recovered from the west end of the north abutment ranged from 77 to 85%, indicating good quality rock.

The unconfined compressive strengths of selected rock core samples were as follows:

Borehole	Depth (m)	Unconfined Compressive Strength (MPa)
2	21.9	58.4
2	23.4	17.2
3	20.1	45.2
3	21.7	37.6

Groundwater

Water was observed upon completion of augering in two boreholes at the following depths/elevations:

Borehole	Depth to Water (m)	Elevation
1	16.2	185.8
3	15.3	186.9

Below a depth of 7.6 m, the lower 0.3 m of the fill in borehole 3 was saturated. Water was not detected in the remaining boreholes during the fieldwork.

Observed groundwater levels are subject to seasonal fluctuations and rainfall patterns.

## CLOSURE


The fieldwork was carried out under the supervision of Mr. M. Rapsey. Direction of the fieldwork was provided by Mr. M.R. Anderson, P.Eng. and Mr. P. Cullen, B.Eng. The equipment was supplied by Elite Drilling.

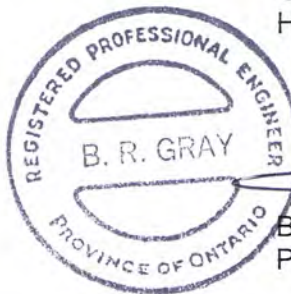
The report was prepared by Mr. P. Cullen, B.Eng. and Mr. M.R. Anderson, P.Eng., Senior Project Engineer, and reviewed by Mr. D.W. Kerr, P.Eng., Manager of Geotechnical and Geo-Environmental Services, Hamilton.

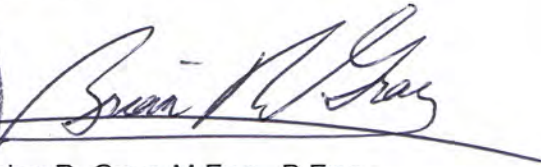
Yours very truly

**Peto MacCallum Ltd.**



  
Dennis W. Kerr, M.Eng., P.Eng.  
Manager Geotechnical and  
Geo-Environmental Services  
Hamilton



  
Brian R. Gray, M.Eng., P.Eng.  
President

MRA:lh



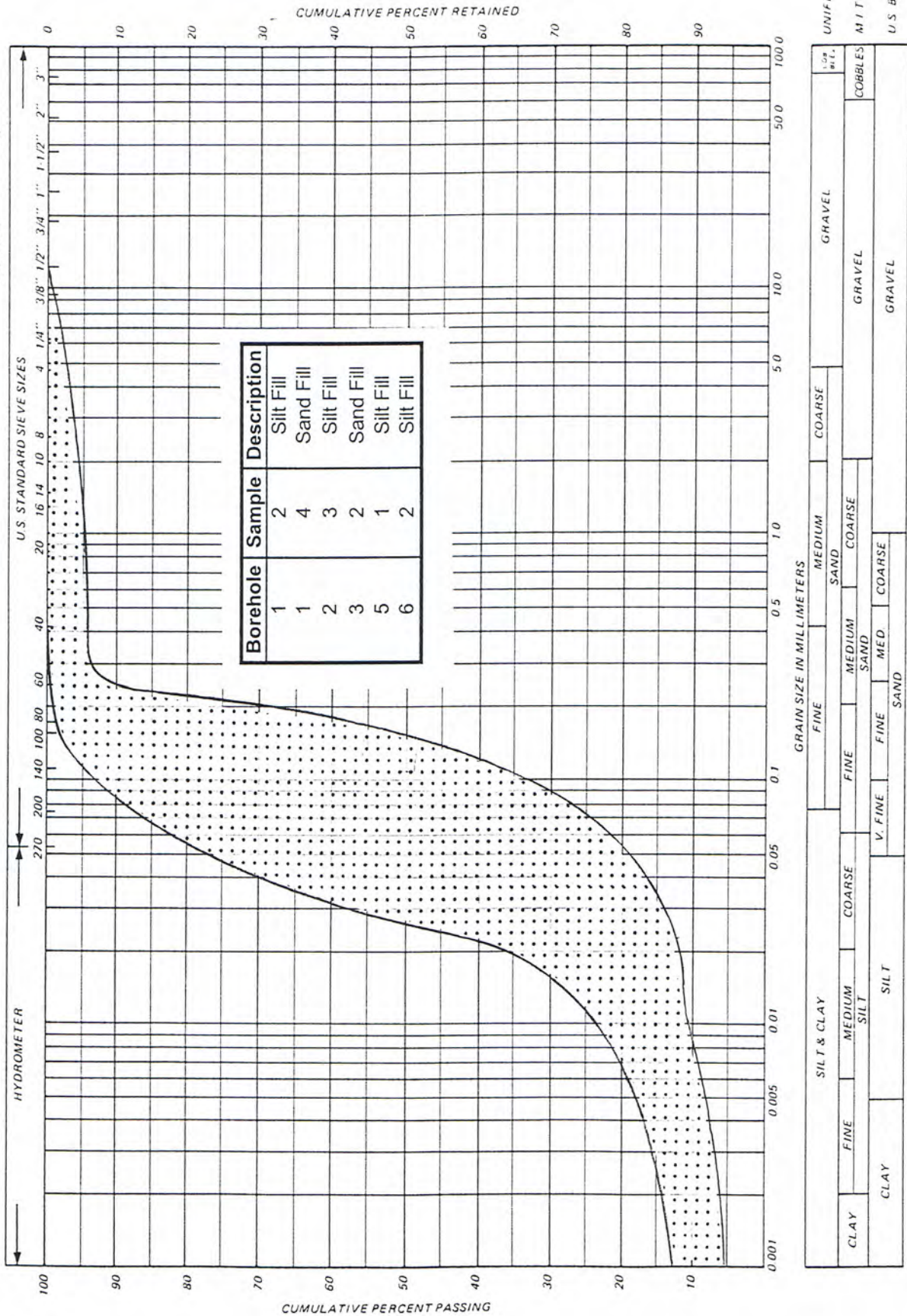
**TABLE I**

**ROCK CORE DESCRIPTION  
CNR BRIDGE  
QEW/HIGHWAY 420 INTERCHANGE  
W.P. 123-00-01  
NIAGARA FALLS, ONTARIO**

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	CORE NO.	RUN (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
2	11	19.20 – 20.40	100	30	19.20 – 23.45	<b>DOLOSTONE:</b> Dark grey to grey aphanitic to fine crystalline; medium to high strength; unweathered to moderately weathered; occ black shaly/petroliferous partings, occ vugs with calcite, quartz, sphalerite, very porous with occ solution cavities, occ stylolitic partings; very close to wide spaced flat to dipping partings, smooth planar, tight/slightly altered with red encrustation or clay filling; poor to good quality.
	12	20.40 – 21.93	100	18		
	13	21.93 – 23.45	95	80		
3	11	20.00 – 21.53	100	77	20.00 – 23.05	<b>DOLOSTONE:</b> Grey fine crystalline; high strength; slightly weathered; occ vugs with calcite, quartz, occ stylolitic partings; very close to moderate spaced flat partings, smooth planar, tight/slightly altered with red or white encrustation or clay filling; good to excellent quality.
	12	21.53 – 23.05	100	85		

PML REF. 00HF119A  
W.P. No. 123-00-01  
FIGURE 1

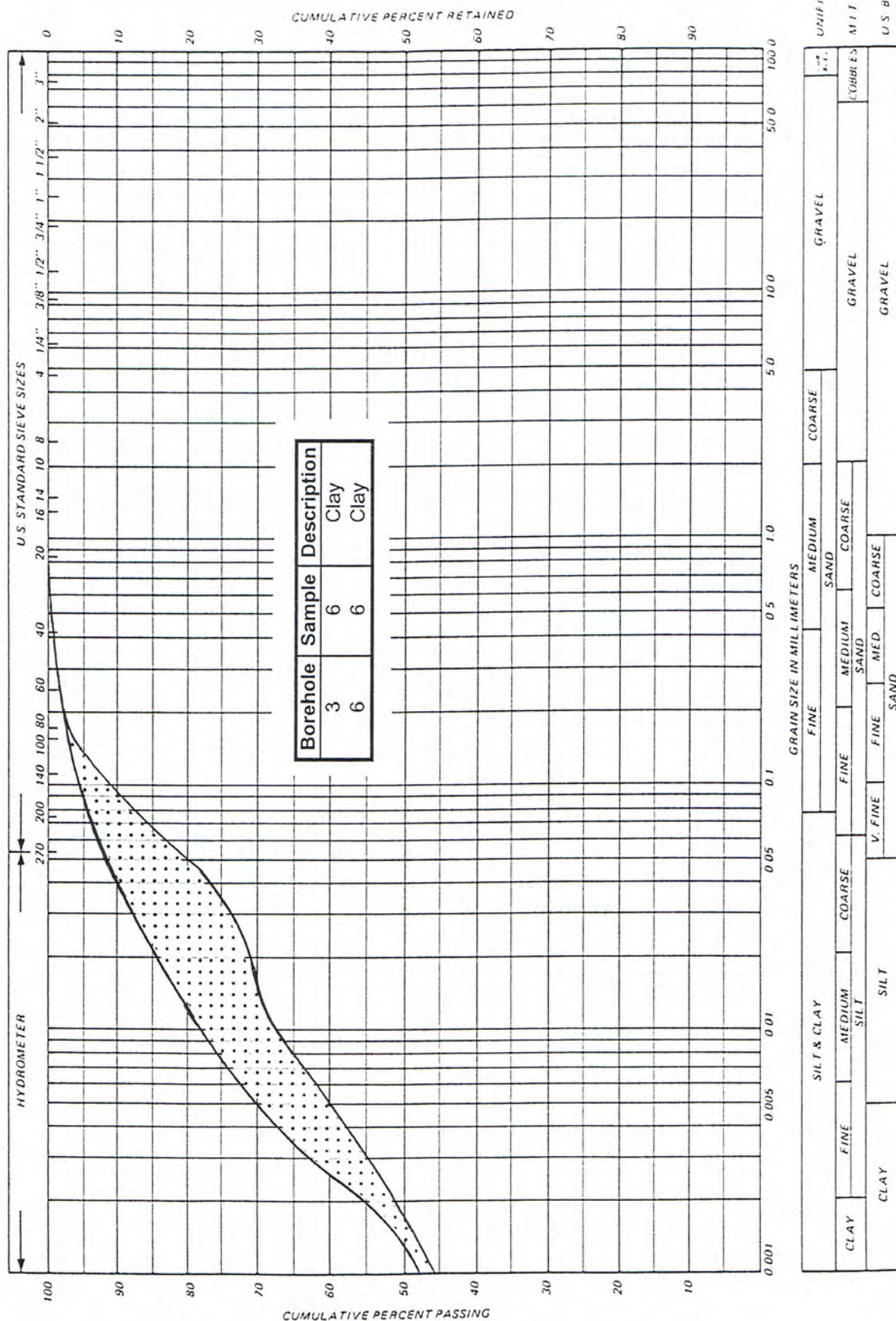
# PARTICLE SIZE DISTRIBUTION CHART



REMARKS Silt/Sand Fill



## PARTICLE SIZE DISTRIBUTION CHART

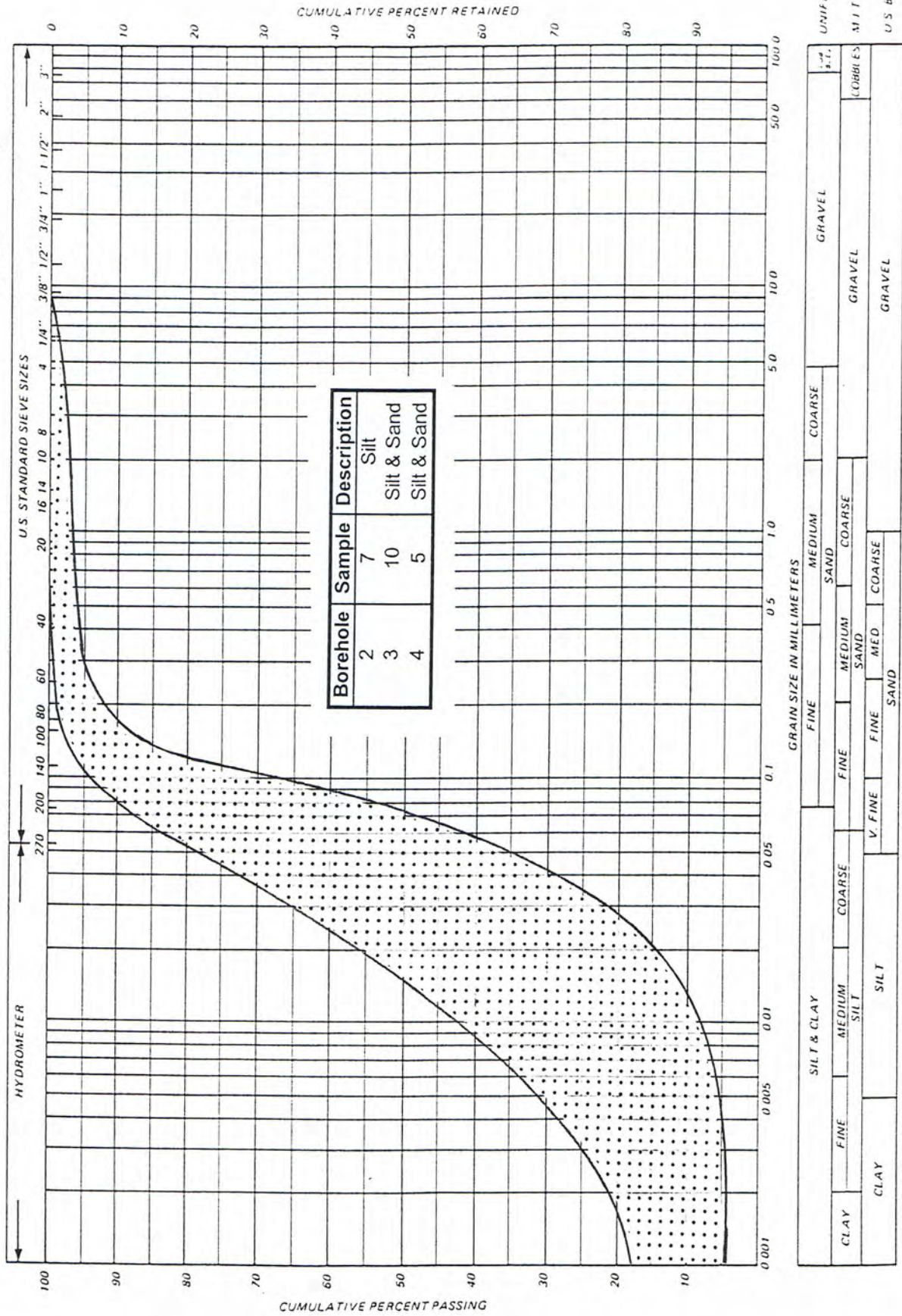


REMARKS Clay



PML REF. 00HF119A  
W.P. No. 123-00-01  
FIGURE 3

# PARTICLE SIZE DISTRIBUTION CHART



REMARKS Silt/Sand





## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL, DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

### DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>	
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4	
SOFT	2 - 4	12 - 25	LOOSE	4 - 10	
FIRM	4 - 8	25 - 50	COMPACT	10 - 30	
STIFF	8 - 15	50 - 100	DENSE	30 - 50	
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50	
HARD	> 30	> 200			
W.T.P.L.	WETTER THAN PLASTIC LIMIT		D.T.P.L.	DRIER THAN PLASTIC LIMIT	
	A.P.L. ABOUT PLASTIC LIMIT				

### TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W	THINWALL OPEN
W.S	WASHED SAMPLE	T.P	THINWALL PISTON
S.B	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C	ROCK CORE
S.T	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

### SOIL TESTS

Q <sub>u</sub>	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Q <sub>cu</sub>	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Q <sub>d</sub>	DRAINED TRIAXIAL		

▲, Δ - Undisturbed and remoulded shear strength determined from in situ vane test.

■ - Undrained shear strength determined from pocket penetrometer test.



## RECORD OF BOREHOLE No 1

ORIGINATED BY M.R.  
COMPILED BY P.C.  
CHECKED BY M.R.A.

[illegible]

## RECORD OF BOREHOLE No 1 Cont'd

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario  
DIST. HWY. QEW BORING DATE Jan. 19, 2001  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers

ORIGINATED BY M.R.  
COMPILED BY P.C.  
CHECKED BY M.R.A.

[illegible]



N 4 773 144  
E 335 677

## RECORD OF BOREHOLE No 2

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario ORIGINATED BY M.R.  
DIST. HWY. QEW BORING DATE Jan. 17, 2001 COMPILED BY P.C.  
DATUM Geodetic BOREHOLE TYPE Cont. Flight Solid Stem Augers/NXL Rock Coring CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION * RESISTANCE PLOT STANDARD PENETRATION TEST *				LIQUID LIMIT <u>W<sub>L</sub></u> PLASTIC LIMIT <u>W<sub>P</sub></u> WATER CONTENT <u>W</u>			UNIT WEIGHT <u>γ</u> kN/m <sup>3</sup>	REMARKS			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N - VALUES		20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>				
201.9	Ground Level																	
0.00	Fill, sandy silt,																	
0.30	Dark Brown																	
201.6						201												
1.5	Fill, Silt, some sand, trace of clay,		1	SS	6	200												
	Loose Brown																	
3.0			2	SS	7	199												
						198												
4.5			3	SS	7	197												
						196												
5.65	Compact with occ. dark brown mottling		4	SS	13	195												
						194												
7.90	Topsoil, Silt, trace of sand and gravel, low organic		5	SS	12	193												
	Dark Brown					192												
9.45	Silty clay, trace of sand,		6	SS	13	191												
	Brown					190												
10.20	Silt, some clay and fine sand, occ. lenses of brown silty clay		7	SS	66	275mm*												
	Very Dense Brown					189												
			8	SS	70	300mm*												
13.10	fine sandy, damp		9	SS	83	300mm*												
						187												
14.65	trace of fine sand		10	SS	48	186												
						185												

0 16 75 9

0 11 69 20  
\* 50/LAST 125mm

\* 50/LAST 150mm

\* 50/LAST 100mm



## RECORD OF BOREHOLE No2 Cont'd

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario ORIGINATED BY M.R.  
 DIST. HWY. QEW BORING DATE Jan. 17, 2001 COMPILED BY P.C.  
 DATUM Geodetic BOREHOLE TYPE Cont. Flight Solid Stem Augers/NXL Rock Coring CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION * RESISTANCE PLOT STANDARD PENETRATION TEST *				LIQUID LIMIT <u>W<sub>L</sub></u> PLASTIC LIMIT <u>W<sub>P</sub></u> WATER CONTENT <u>W</u>				UNIT WEIGHT <u>γ</u> kN/m <sup>3</sup>	REMARKS % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	N - VALUES	GROUND WATER ELEV.	20	40	60	80	100	W <sub>p</sub>	W	W <sub>L</sub>	
16.5	Ground Level														
18.0						185									
18.3.2 18.75	till-like					184									
182.7 19.20	Bedrock, Dolostone		11	RC		183	RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)					
						182	1200	100	30	0*					
21.0			12	RC		181	1525	100	18	0					
22.5			13	RC		180	1525	95	80	0					
178.5 23.45	End of Borehole					179									
24.0						178									
25.5															
27.0															
28.5															
30.0															
32.5															
34.0															

\* LOST DRILL  
WATER AT 19.5m  
DEPTH.

\* NO RECOVERY



N	4	773	175
E		335	647

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario ORIGINATED BY M.R.  
DIST. HWY. QEW BORING DATE Jan. 18, 2001 COMPILED BY P.C.  
DATUM Geodetic BOREHOLE TYPE Cont. Flight Solid Stem Augers/NXL Rock Coring CHECKED BY M.R.A.

[illegible]



## RECORD OF BOREHOLE No 4

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario  
DIST. HWY. QEW BORING DATE Jan. 17, 2001  
DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers

ORIGINATED BY M.R.  
COMPILED BY P.C.  
CHECKED BY M.R.A.

[illegible]

## RECORD OF BOREHOLE No 4 Cont'd

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario ORIGINATED BY M.R.  
 DIST. HWY. QEW BORING DATE Jan. 17, 2001 COMPILED BY P.C.  
 DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers CHECKED BY M.R.A.

SOIL PROFILE		SAMPLES			GROUND WATER ELEV.	DYNAMIC CONE PENETRATION * RESISTANCE PLOT STANDARD PENETRATION TEST *				LIQUID LIMIT — W <sub>L</sub> PLASTIC LIMIT — W <sub>P</sub> WATER CONTENT — W				UNIT WEIGHT Y kN/m <sup>3</sup>	REMARKS % GR. SA. SI. CL.
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		N - VALUES	SHEAR STRENGTH (kPa) O UNCONFINED * FIELD VANE ■ PENETROMETER * LAB VANE				WATER CONTENT % W <sub>p</sub> — W — W <sub>L</sub> 20 40 60				
16.5	Ground Level														
						185									
18.0						184									
183.3															
18.60	End of Borehole Probable Bedrock Refusal to Auger					183									
19.5															
21.0															
22.5															
24.0															
25.5															
27.0															
28.5															
30.0															
32.5															
34.0															

UPON COMPLETION  
OF AUGERING, NO  
WATER, NO CAVE.



N	4	773	208
E		335	668

ORIGINATED BY M.R.  
COMPILED BY P.C.  
CHECKED BY M.R.A.

UPON COMPLETION  
OF AUGERING, NO  
WATER, NO  
CAVE-IN.



## RECORD OF BOREHOLE № 6

N 4 773 119

E 335 659

W.P. 123-00-01 LOCATION Hwy. 420/QEW CNR Bridge, Niagara Falls, Ontario

ORIGINATED BY M.R.

DIST. \_\_\_\_\_ HWY. QEW BORING DATE Jan. 19, 2001

COMPILED BY P.C.

DATUM Geodetic BOREHOLE TYPE Continuous Flight Solid Stem Augers

CHECKED BY M.R.A.

[illegible]



FOUNDATION DESIGN REPORT  
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May, 2001



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## FOUNDATION DESIGN REPORT

For  
CNR Bridge  
QEW/Highway 420 Interchange  
W.P. 123-00-01  
Niagara Falls, Ontario

---

### INTRODUCTION

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches for the proposed replacement of the QEW structure over the CN railway at the QEW/Highway 420 interchange in Niagara Falls, Ontario. The investigation was conducted for Philips Engineering Ltd. on behalf of the Ontario Ministry of Transportation.

The proposed overpass structure will comprise a single span of about 16 m length and 32 m width. The QEW southbound lanes and exit ramp to Lundy's Lane will cross the CN railway at approximate Station 15+666, QEW chainage.

Road grade over the structure will be raised approximately 0.6 m above the existing grade. The CN railway is near elevation 194.4 under the existing bridge.

The subsurface stratigraphy revealed at the site generally comprised a surficial pavement structure overlying approach fill over a topsoil layer. The fill/topsoil was underlain by native clay, silt and/or sand deposits mantling bedrock. Dolostone bedrock was contacted below the overburden at the following depths/elevations:



Location	Depth to Rock (m)	Bedrock Elevation
North Abutment, East End	19.5	182.5
North Abutment, West End	20.0	182.2
South Abutment, East End	19.2	182.7
South Abutment, West End	18.6	183.3

## **FOUNDATIONS**

### **Driven Piles**

Driven piles may be employed to support the abutments of the proposed structure.

Construction of integral abutments supported on steel H-piles is considered feasible. The piles should be equipped with driving shoes and driven to refusal on bedrock anticipated at depths of 18.6 to 20.0 m (elevation 182.2 to 183.3). Recommended values for factored axial resistance at ULS for two pile sections are presented below:

H-Pile Section	Factored Resistance at ULS (kN)
HP 310 x 79	1160
HP 310 x 110	1600

It is noted that the recommended ULS resistances are conservative to reflect the relatively poor quality rock identified in borehole 2 (south abutment, east end).

The resistance at serviceability limit states normally allows for 25 mm of compression of the pile and founding medium. Considering the bedrock to be non-yielding and the pile length required, the design is not expected to be governed by settlement since the loading required to produce deformation of the pile will be much larger than the factored resistance at ULS.

Considering the quality of the upper portion of the rock encountered at the east end of the south abutment, as well as that in holes drilled for other projects in the vicinity of this site, it is recommended that the upper 3.0 m of the bedrock be grouted to fill potential voids prior to driving piles. The grout should be injected into the rock by drilling small diameter holes through the overburden and at least 3 m into rock. The holes should be drilled on a 2 m grid commencing at the centre of the foundation unit and extend beyond the outside limits of the pile tips at least 2 m. A non-shrink cementitious grout with a compressive strength of 25 MPa should be used. A specialist grouting contractor must be retained for this work.

Additional boreholes should be drilled at each foundation, to confirm the quality of the rock, delineate the extent of poor quality rock, investigate the presence of voids and further assess requirements for deep foundations, if employed.

The soil adjacent to the upper portion of the piles is expected to comprise dense to very dense silt. To accommodate movement of the integral abutment, two concentric CSPs that extend 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with cement-bentonite grout or loose sand meeting the gradation requirements shown on Table I. Refer to MTO Report SO-96-01 for further details.

The type of equipment required to drive the piles will be somewhat dictated by the design capacity. In general, the piles should be driven to practical refusal using a hammer which transfers at least 40 KJ of energy to the pile. Since the piles will set on hard rock, a specific set for this project is not provided.

The installation operations should be inspected on a full-time basis by qualified geotechnical personnel to confirm the founding elevation, alignment, plumbness, uniformity of set, and quality of splices.



Driving shoes should be provided (OPSD 3301) to minimize the potential for damage when driving through dense zones and setting into bedrock.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The coefficient of horizontal subgrade reaction,  $k_s$ , for native overburden may be computed using the following equation to evaluate the point of contraflexure:

$$k_s = n_h z/b$$

where  $z$  = depth  
 $b$  = pile width (m)

The recommended values for  $n_h$  are as follows:

Compact to Very Dense Silt/Sand	10,000 kN/m <sup>3</sup>
Silt/sand below 15 m depth	6,000 kN/m <sup>3</sup>

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile below the annular space. The lateral resistance recommended for the native dense to very dense non-cohesive silt/sand for two pile sections is:

	<u>HP 310 x 79</u>	<u>HP 310 x 110</u>
Factored Lateral Resistance at ULS =	110 kN	120 kN
Lateral Resistance at SLS =	40 kN	50 kN

### Spread Footings

Supporting the structure on conventional spread footings founded on the native overburden is considered feasible. The footings should be founded on the native dense to very dense silt at the following depths/elevations:

Foundation Unit	Depth (m)	Founding Elevation
North Abutment	10.4	191.9
South Abutment	10.2	191.7

Footings constructed on the native silt at the above noted levels should be designed using the following:

Factored Bearing Resistance at ULS	=	675 kPa
Bearing Resistance at SLS	=	450 kPa

Spread footings could also be constructed on structural fill placed in the approaches. Construction of structural fill supporting foundations should include excavation and replacement of the existing approach fill, topsoil and otherwise soft/loose material with engineered fill. The engineered fill should comprise Granular "A" material placed in maximum 200 mm thick lifts, compacted to 100% standard Proctor maximum dry density, and extended laterally to a line inclined outwards at 1:1 (H:V) originating at least 1 m from the top of footing. This scheme is illustrated on Figure 1.

The bearing resistance for a minimum 2.5 m wide footing constructed on a minimum 1.5 m thick pad of structural fill is:

Factored Bearing Resistance at ULS	=	900 kPa
Bearing Resistance at SLS	=	350 kPa



The recommended resistance at SLS allows for 25 mm of total settlement; differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistances.

Sliding will be resisted in part by the friction force developed between the underside of the footing and the native silt/sand or granular fill. Unfactored friction factors of 0.35 and 0.45 are recommended for footings on silt and granular fill, respectively.

All footings subject to frost action should be provided with the normal 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Prior to placement of structural concrete, all foundation excavations should be examined by qualified geotechnical personnel to verify the competency of the founding surface.

### Caissons

Supporting the structure on augered caissons socketed into bedrock is also considered feasible. The caissons should be designed using a factored end-bearing resistance at ULS of 3,000 kPa and a factored bond stress at ULS of 500 kPa in sound bedrock, 250 kPa in the poor quality rock. The factored axial resistance of four caissons of selected diameter socketed 2.8 m plus half the diameter of the caisson into rock is as follows:

Caisson Diameter (m)	Factored Axial Resistance at ULS (kN)
0.76	3000
0.91	4100
1.07	5400
1.22	6700

It should be noted that the additional socket length of 2.8 m is included in the resistance computations to ensure the caisson is founded on the sound bedrock below the poor quality rock revealed in borehole 2. The socket length may be reduced if sound bedrock is encountered above the assumed depth during construction.

Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement since the loading required to produce deformation will be much larger than the factored resistance at ULS.

The caissons should be installed and inspected in accordance with Special Provision 903S01 (April 2000).

It is anticipated that augering will be feasible to advance the caissons through the overburden. Groundwater may present some problems with the installation of the caissons; the caisson liner should be sufficient to control groundwater seepage from the overburden. Placement of concrete by tremie method will probably be necessary.

### **ABUTMENT WALLS**

The abutment walls should be designed to resist the unbalanced horizontal earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure,  $p$ , may be computed using the equivalent fluid pressures presented in Section 6-7.4 of the Ontario Bridge Design Code (OHBDC, 3rd Edition, 1991) or employing the following equation, assuming a triangular pressure distribution:



$$p = K (\gamma h + q)$$

where  $K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of free-draining  
granular material ( $\text{kN/m}^3$ )

$h$  = depth below final grade (m)

$q$  = surcharge load (kPa), if present

Free-draining granular material should be used as backfill behind the wall. The following parameters are recommended for design:

	Granular "A"	Granular "B"
Angle of Internal Friction (degrees)	35	32
Unit weight ( $\text{kN/m}^3$ )	22.8	21.2
Active Earth Pressure Coefficient ( $K_a$ )	0.27	0.31
At Rest Earth Pressure Coefficient ( $K_o$ )	0.43	0.47
Passive Earth Pressure Coefficient ( $K_p$ )	3.69	3.25

Refer to MTO Report SO-96-01 for procedures to determine the earth pressure coefficient to be employed to design integral abutments. The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures.

A weeping tile system and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

A retained soil system could also be employed. The founding material is expected to comprise granular engineered fill or silt overburden. The following parameters should be employed for design of the system foundation:

	<u>Granular "A"</u>	<u>Native Overburden</u>
Friction Angle (degrees)	35	32
Cohesion (kPa)	0	0
Unit weight (kN/m <sup>3</sup> )	22.8	20.4

The bearing resistances recommended previously for spread footings constructed on the native overburden should be employed for design of the RSS wall.

The supplier of the retained soil system should be responsible for design of the structure (backfill, reinforcement, internal and external stability) and provide drawings to show pertinent information such as location, length, height, elevations, performance level, appearance etc.

## **APPROACHES**

Backfilling adjacent to the structure should be carried out in conformance with Ontario Provincial Standards specifications for granular backfill (OPSD 3501.00).

The embankments should be constructed in accordance with OPSD 200.01, 202.01 and 208.01. Embankment slopes up to 10 m high inclined at 2 horizontal to 1 vertical should be stable. These recommendations should be reviewed if fill heights exceed 10 m.

No settlement or bearing capacity problems due to placing fill on the inorganic native overburden are anticipated. The existing fill layer, topsoil, and other deleterious material should be stripped prior to placement of the approach embankment.



## **EXCAVATION AND GROUNDWATER CONTROL**

Excavation for construction of footings and/or pile caps is expected to extend some 10.4 m below the QEW road grade, and approximately 2.4 m below the top of rail of the railway track. It is anticipated that excavation will be carried out primarily within the sloping fill in front of the existing abutments and extend into the underlying native clay and silt. Excavation within the existing approach fill (loose silt) behind the abutment wall may be required contingent upon the excavation geometry.

Excavation through the fill, clay and silt to the anticipated depth is expected to be relatively straightforward using conventional equipment. The fill and overburden are classified as Type 3 soil according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes inclined at 1 horizontal to 1 vertical should generally be stable. A 2.0 m wide mid-height berm should be incorporated for cut depths exceeding 6.0 m, in accordance with MTO policy.

Flatter sideslopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered.

We understand that construction of the bridge will be carried out in two sections to maintain traffic on the southbound QEW. Shoring will be required to support the traffic lanes adjacent to the excavation, permit backfilling of the new bridge approaches during the first stage of construction, and possibly to limit the longitudinal extent of excavation along the QEW approaches.

The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence. The recommended design earth pressure distribution for multiple and singly braced walls, for the conditions which exist at the site, are presented on Figures 2 and 3, respectively. Recommendations concerning design and construction of the braced excavation support systems are also presented on the figures.

A soldier pile and lagging system may be considered. Provided the spacing between soldier piles is at least five pile diameters, the unfactored lateral passive resistance developed on the face of the soldier pile below the base of the excavation may be taken as the passive earth pressure developed over an equivalent wall area of width three times the pile diameter and depth of six times the pile diameter. A passive earth pressure coefficient,  $K_p$ , of 3.0 is recommended for this computation.

Additional lateral resistance could be provided by installing tiebacks anchored in the dense native silt or underlying bedrock. The factored pull-out resistance at ULS of soil anchors in the dense silt below elevation 192 may be computed as follows:

$$r_{su} = 0.3 K_f \gamma' h$$

where  $K_f = 1.0$  for dense silt

$$\begin{aligned}\gamma' &= 20.4 \text{ kN/m}^3 \text{ above water table} \\ &= 10.6 \text{ kN/m}^3 \text{ below water table (assume elevation 188.0)}\end{aligned}$$

$h$  = depth of soil above the midpoint of anchored length

A factored rock-grout bond stress of 900 kPa at the ultimate limit state (minimum 30 MPa grout) is recommended for design of rock anchors.

The ground surface adjacent to the excavation is expected to experience some inward movement and vertical settlement. The magnitude of movements adjacent to a braced cut can be limited by selection of an appropriate lateral earth pressure coefficient (see Figures 2 and 3) provided good quality workmanship and construction practice is employed. The anticipated magnitude of movements are as follows:



Movement (% of Excavation Depth)

Lateral Movement

Braced Excavation	0.2%
Anchored Wall	0.1%

Vertical Movement	0.05%
-------------------	-------

Construction procedures should be specifically suited to prevent erosion of fine silt materials into the cut by water flow, minimize inadvertent densification of the existing loose approach fill, and limit any consequent settlement of the pavement subgrade behind the excavation face.

Foundations of heavily loaded/settlement sensitive structures and/or utilities located within close proximity to the excavation may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are presented in Figure 4.

The preliminary General Arrangement Drawing (Drawing S1, Sheet S-7 dated March 2001) prepared by Philips Engineering Ltd. for the proposed structure indicates the foundation excavation will extend approximately 2.4 m below the base of the railway ballast at a distance of some 4.0 m from the end of the existing railway tie. Open cut excavation with the excavation sidewall inclined at 1:1 in the ballast and underlying clay/silt subgrade is considered to be feasible and is not expected to impact the railway alignment. The crest of the cut slope adjacent to the track will be about 1.5 m from the end of tie.

In general, it appears the excavation will be carried out behind the inner row of existing bridge columns. The below-grade portion of the existing bridge foundations should be left in place to minimize the potential for disturbance of the railway ballast.

Daily monitoring of the vertical and horizontal alignment of the south rail is recommended during excavation and construction of the south abutment foundation to confirm negligible movement and/or permit prompt remedial action should unforeseen movements occur.

A zone of saturated material was identified at the base of the approach fill in one borehole. Seepage or surface water which enters the excavation should be readily handled by conventional sump pumping techniques. Surface runoff should be directed away from the excavation.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

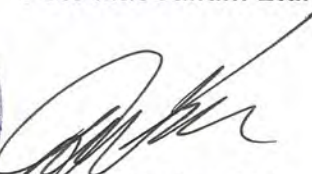
#### **CLOSURE**

The report was prepared by Mr. P. Cullen, B.Eng. and Mr. M.R. Anderson, P.Eng., Senior Project Engineer and reviewed by Mr. D.W. Kerr, P.Eng., Manager of Geotechnical and Geo-Environmental Services, Hamilton.

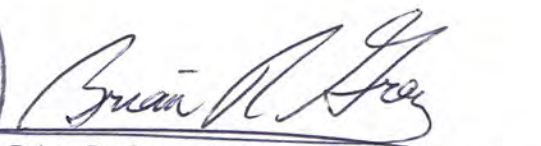
Yours very truly

**Peto MacCallum Ltd.**



  
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MRA:lh



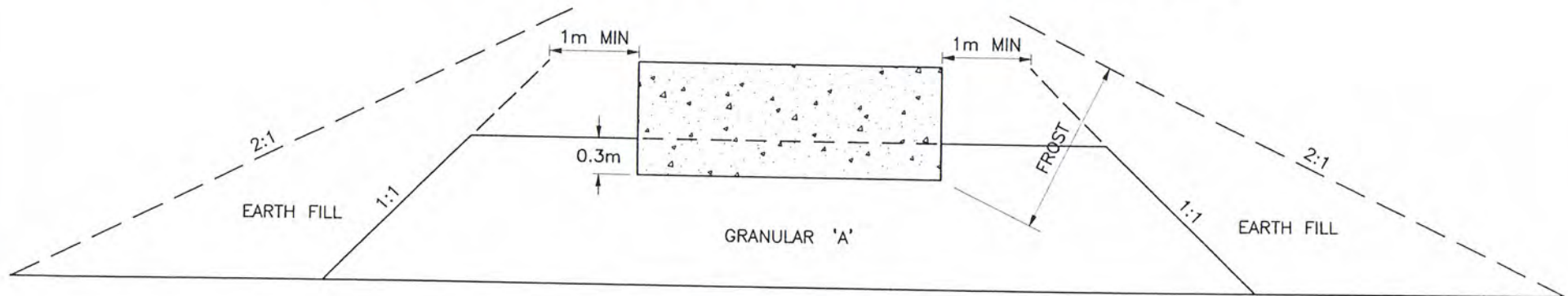
TABLE I

**Gradation Specification for Sand Fill in  
Pre-Augered Holes at Integral Abutments**

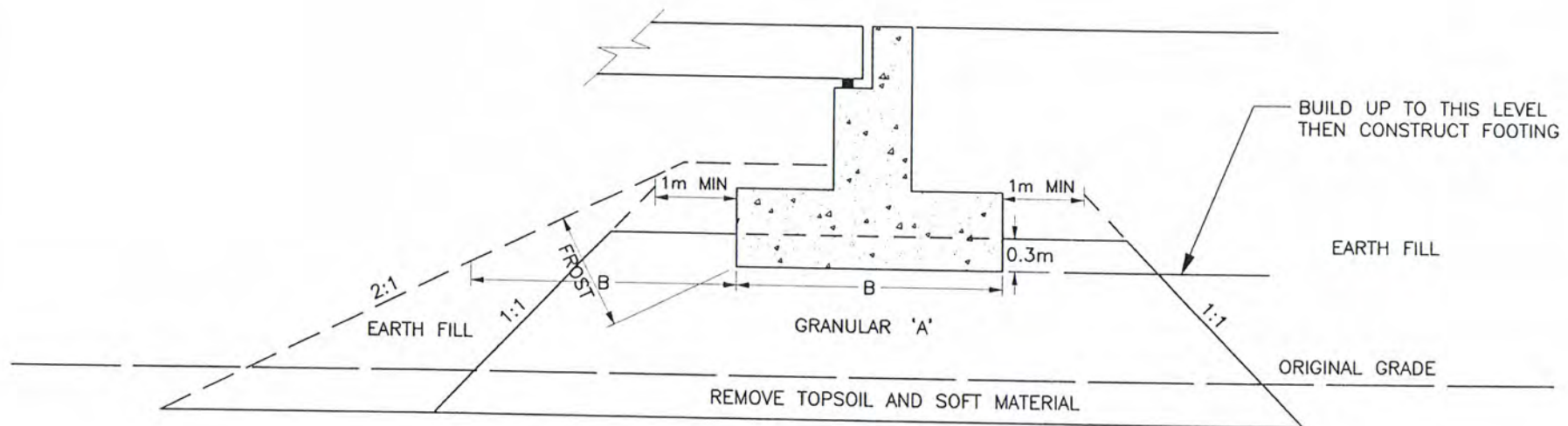
MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100
600 $\mu\text{m}$	#30	80 – 100
425 $\mu\text{m}$	#40	40 – 80
250 $\mu\text{m}$	#60	5 – 25
150 $\mu\text{m}$	#100	0 – 6

From MTO Report S0-96-01, Revision 1 – July, 1996.

# ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE



CROSS SECTION



LONGITUDINAL SECTION

## NOTES

1. REMOVE TOPSOIL AND/OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED
5. REFER TO TEXT OF REPORT FOR FROST DEPTH

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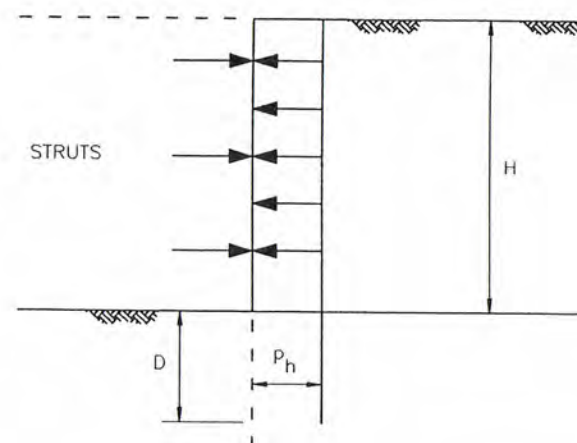
DATE	SCALE	JOB NO.	FIGURE NO.
JAN. 2001	NTS	00HF119A	1



## NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

## EARTH PRESSURE DIAGRAM



$$P_h = \text{design lateral earth pressure} \\ = 0.65 \ K \gamma H$$

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of soil

$H$  = depth of excavation

$D$  = depth of embedment of soldier piles (if used).

## RECOMMENDED DESIGN PARAMETERS

$$\gamma = 21 \text{ kN/m}^3$$

$K = 0.3$  (slight movement of retained soil acceptable)

0.5 (movement of adjacent facilities unacceptable)

DATE	JOB NUMBER	FIGURE NO.
JAN. 2001	00HF119A	3



## NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

### ZONE A:

Foundations of relatively heavy and/or settlement sensitive structures/utilities located in Zone A generally require underpinning.

### ZONE B:

Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

### ZONE C:

Utilities and foundations located within Zone C do not normally require underpinning.

Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non-underpinned foundations must be considered in the design of the support system.
3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.
4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
5. This sheet is to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

