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W.P. No. 309-94-00

CONT. No.

W. O. No.

STR. SITE No.

HWY. No. 420

LOCATION DRUMMOND RD. STRUCTURE

QEW TO RAINBOW BRIDGE

No of PAGES - 1

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**FOUNDATION INVESTIGATION REPORT  
FOR  
DRUMMOND ROAD STRUCTURE  
HIGHWAY 420 RECONSTRUCTION  
QEW TO RAINBOW BRIDGE  
G.W.P. 309-94-00  
NIAGARA FALLS, ONTARIO**

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Job No. 98HF100A

January, 2000

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## **FOUNDATION INVESTIGATION REPORT**

For  
Drummond Road Structure  
Highway 420 Reconstruction  
QEW to Rainbow Bridge  
G.W.P. 309-94-00  
Niagara Falls, Ontario

---

### **INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for replacement of the structure at Drummond Road along Highway 420 in Niagara Falls, Ontario. The investigation was conducted for Philips Engineering Ltd. on behalf of The Ministry of Transportation Pavements and Foundations Section.

The report pertains to the proposed bridge structure and the approaches within about 20 m of the abutments. The approximate limits are as follows:

Drummond Road	Station 9+965 – 10+035
Highway 420	Station 14+465 – 14+485

### **SITE DESCRIPTION**

The site is located at the existing crossing of Highway 420 and Drummond Road, approximately 1.5 km east of the QEW in the City of Niagara Falls. The proposed structure will carry Drummond Road traffic over the four-lane section of Highway 420. At the structure location, Drummond Road runs roughly north-south.

The lands adjacent to the site are primarily residential with occasional commercial/industrial properties.

The site is located in the physiographic region known as the Haldimand Clay Plain. In general, the topography on the plain is relatively flat with slight undulations. The overburden is some 20 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 July 20 to August 14, 1999 and comprised three boreholes extended to bedrock at depths of 20.1 to 20.3 m and five boreholes drilled to depths of 6.6 to 8.1 m. The three deep boreholes were extended a further 3.0 to 3.1 m into bedrock by coring. The borehole locations are shown on Drawing 1. The boreholes are designated by a "D" series (D1 to D8) to differentiate from other sites drilled during the current investigation.

The borehole locations were selected by Peto MacCallum Ltd., subject to access limitations in the field. The locations of and ground surface elevations at the boreholes were subsequently determined by Philips Engineering Ltd.

The boreholes were advanced using continuous flight solid and hollow 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. 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 and moisture content determinations are also shown. Stratigraphic profiles prepared from the borehole data are presented on Drawing 1.

The subsurface stratigraphy revealed at the bridge site generally comprised a surficial pavement structure and/or sand/silt fill layer overlying native loose to compact silt contacted at depths of 5.5 m at the bridge approaches and 0.3 to 1.6 m in the holes drilled on Highway 420. This was underlain by very dense silt and sand unit that extended to an approximate depth of 14.1 to 14.6 m. A dense silt and sand till layer that mantles dolostone bedrock at 20.1 to 20.3 m depth was encountered below the silt and sand deposit. The typical thicknesses and elevations of the strata were as follows:

<u>Unit</u>	<u>Typical Thickness (m)</u>	<u>Elevation of Upper Boundary*</u>
Pavement Structure	0.5 to 1.6	189.0 to 189.1
Sand/Silt Fill	0.3 to 0.8	188.9 to 189.3
Native Silt	13.0 to 14.3	187.5 to 188.7
Silt and Sand Till	5.4 to 6.2	174.3 to 175.0
Bedrock	-	168.8 to 168.9
Groundwater	-	184.2 to 185.4

\*Boreholes drilled on Highway 420

Further description of the strata encountered is provided below:

#### Pavement Structure

A pavement structure was encountered in five of the eight boreholes. On Drummond Road, the pavement structure comprised 155 mm asphaltic concrete over 595 mm crushed limestone in borehole D1 and 245 mm asphalt over 80 mm crushed limestone in borehole D8.

In boreholes D4, D5 and D7 drilled in the paved shoulders of Highway 420, the pavement structure consisted of 80 to 100 mm asphaltic concrete over 350 to 1520 mm crushed limestone and sand and gravel.

#### Fill

The approach fill revealed below the pavement structure in boreholes D1 and D8 generally comprised loose to compact non-cohesive silt. The fill became predominantly silt topsoil between 2.9 to 4.9 m depth in borehole D1. A 0.6 m thick layer of non-cohesive sand and gravel and a 1.0 m thick layer of non-cohesive silty sand were encountered below the silt fill in boreholes D1 and D8, respectively. The fill was penetrated at 5.5 m depth (elevation 188.4 and 188.7) in both boreholes.

A 300 mm thick layer of sand fill was encountered surficially in borehole D2. Surficial silt fill was penetrated at depths of 1000 and 750 mm (elevation 187.8 and 188.6) in boreholes D3 and D6, respectively.

#### Topsoil

A 250 mm thick layer of clayey silt topsoil was encountered below the fill in borehole D6. The topsoil was judged to have a low organic content. The topsoil was penetrated at a depth of 1.0 m (elevation 188.3).

### Silt

A major deposit of non-cohesive silt of glaciolacustrine origin was encountered below the silt/sand fill, topsoil and pavement structure in boreholes D2 to D7, and below the fill in the bridge approach holes D1 and D8.

The silt was generally loose to compact with occasional dense to very dense zones. A zone of silt and sand was revealed within the silt in boreholes D2 and D5, and a 300 mm thick cohesive clay layer was encountered in borehole D5. Moisture contents ranged from 13 to 27%, typically 18 to 24%.

The results of grain size distribution analyses conducted on samples of the silt are presented on Figure 1. The results of Atterberg Limits tests conducted on samples exhibiting plasticity are presented on Figure 2. The deposit is essentially non-plastic with zones of slight plasticity.

Testholes D1, D3, D4, D6 and D8 were terminated in the silt layer at depths of 6.6 to 8.1 m. The silt was penetrated at depths of 14.1 to 14.6 m (elevation 174.3 to 175.0) in the remaining boreholes.

### Silt and Sand Till

A silt and sand till unit of glacial origin was encountered below the major silt deposit in the three deep testholes and ranged in thickness from 5.4 to 6.2 m. The silt and sand till layer was very dense (non-cohesive) and contained a trace to some gravel. The moisture content ranged from 7 to 9%. The till unit mantled bedrock.

### Bedrock

Bedrock was contacted below the silt and sand till in testholes D2, D5, and D7 at the following depths/elevations; the bedrock surface appears to be flat.



<u>Borehole</u>	<u>Depth to Bedrock (m)</u>	<u>Elevation</u>
D2	20.1	168.8
D5	20.3	168.8
D7	20.2	168.9

Drilling was terminated after coring 3.0 to 3.1 into bedrock. A description of the rock cores recovered from testholes D2, D5 and D7 is provided in Table I. The bedrock consists of Dolostone. The core recovery ranged from 98 to 100%. The RQD ranged from 25 to 73% (poor to fair quality) in the upper 1.5 m, 82 to 98% (good to excellent quality) below this depth. A 25 to 75 mm thick void was detected within the bedrock in all three cores, at approximately 20.9 m depth (elevation 168.0 to 168.2).

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

<u>Borehole</u>	<u>Depth (m)</u>	<u>Unconfined Compressive Strength (MPa)</u>
D2	20.9	85.2
D2	21.9	100.2
D5	20.7	81.4
D5	21.3	94.2
D7	20.6	55.5
D7	21.8	136.7

#### Groundwater

Free water was observed in three boreholes at the following depths/elevations:

<u>Borehole</u>	<u>Depth to Free Water (m)</u>	<u>Elevation</u>
D2 (during drilling)	3.5	185.4
D3 (upon completion)	4.6	184.2
D5 (during drilling)	6.5	182.6

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

### CLOSURE

The fieldwork was carried out under the supervision of Mr. M. Rapsey and direction of Mr. M.R. Anderson, P.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., Project Engineer. It was 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



Brian R. Gray, M.Eng., P.Eng.  
Vice-President  
Geotechnical and  
Geo-Environmental Services

MRA:ld

**TABLE I**

**ROCK CORE DESCRIPTION  
G.W.P. 309-94-00  
DRUMMOND ROAD STRUCTURE  
NIAGARA FALLS, ONTARIO**

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	CORE NO.	RUN (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
D2	13	20.10 - 21.65	100	25	20.10 - 21.65	<b>DOLOSTONE:</b> Buff, very fine crystalline, with vugs, calcite inclusions; high strength; unweathered; close spaced flat partings, rough undulating, tight; poor quality.
	14	21.65 - 23.15	100	98	21.65 - 23.15	A 50 mm thick void was detected at a depth of 20.9 m. <b>DOLOSTONE:</b> Grey, fine crystalline, with stylitic partings; high strength, unweathered; close to moderate spaced flat partings, rough undulating, tight; excellent quality.
D5	13	20.30 - 21.80	100	73	20.30 - 21.80	<b>DOLOSTONE:</b> Buff, very fine crystalline, occ. stylitic partings, with vugs, calcite, gypsum; close spaced flat partings, rough undulating, tight with wide spaced crossjoints, dipping, smooth undulating, tight; fair quality.
	14	21.80 - 23.35	98	82	21.80 - 23.35	A 75 mm thick void was detected at a depth of 20.9 m. <b>DOLOSTONE:</b> Grey, fine crystalline, occ. stylitic partings; high strength; unweathered; close spaced flat partings, rough undulating to planar, tight; good quality.

**TABLE I (Cont'd)**

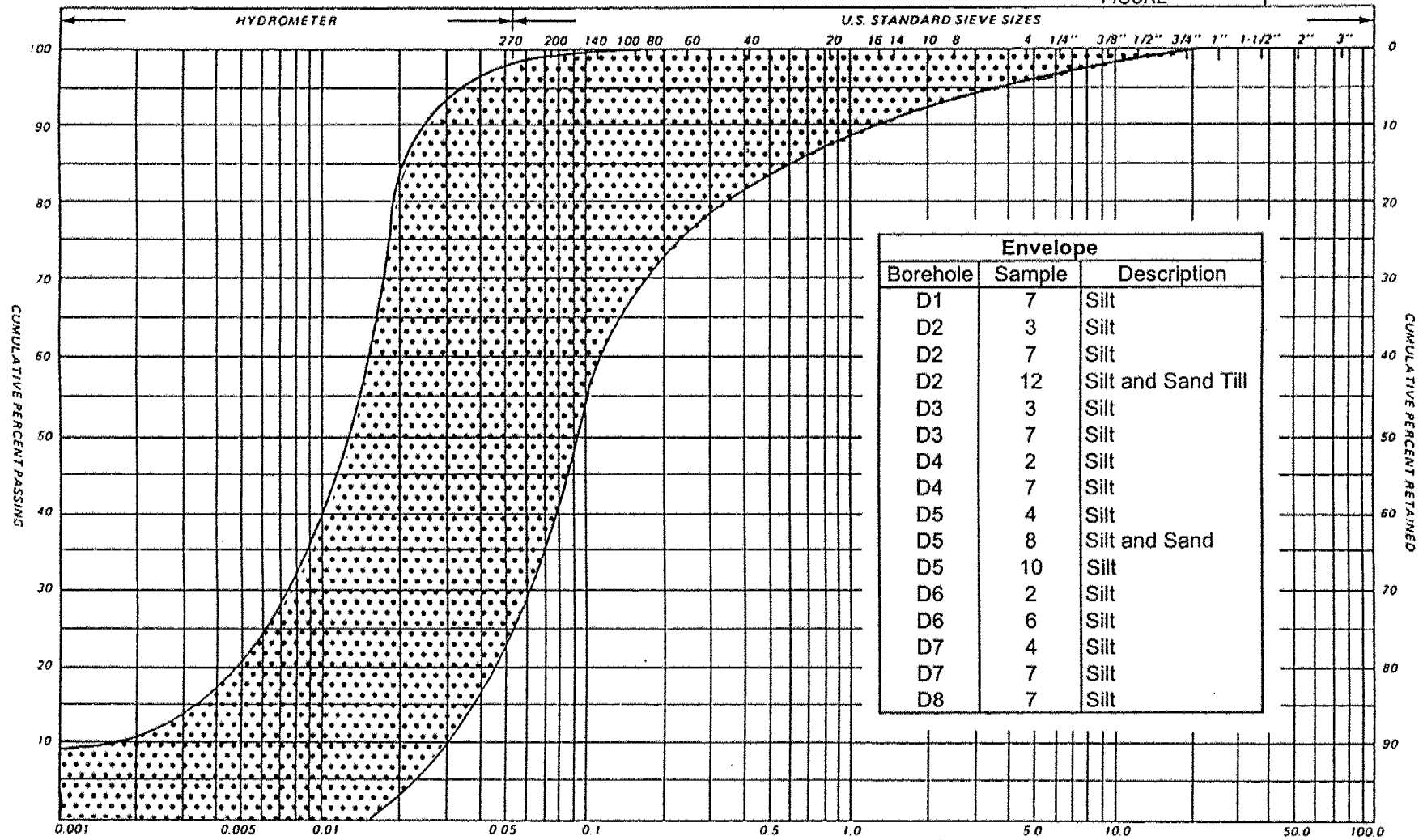
**ROCK CORE DESCRIPTION  
G.W.P. 309-94-00  
DRUMMOND ROAD STRUCTURE  
NIAGARA FALLS, ONTARIO**

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	CORE NO.	RUN (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
D7	11	20.25 - 21.80	100	60	20.25 - 22.10	<b>DOLOSTONE:</b> Buff, very fine crystalline, with vugs, gypsum; high strength; unweathered; close spaced flat partings, rough undulating, tight with close spaced crossjoints, dipping, rough planar, tight; fair quality.  A 25 mm thick void was detected at a depth of 20.9 m.
	12	21.80 - 23.30	98	82	22.10 - 23.30	<b>DOLOSTONE:</b> Grey fine crystalline, with stylitic partings; with calcite, sphalerite inclusions; high strength; unweathered; with close spaced flat partings, rough undulating, tight; good quality.



# PARTICLE SIZE DISTRIBUTION CHART

PML REF. 98HF100A  
REPORT NO. 1  
FIGURE 1



GRAIN SIZE IN MILLIMETERS										COB- BLES	UNIFIED					
SILT & CLAY			FINE		MEDIUM SAND		COARSE		GRAVEL							
CLAY	FINE		MEDIUM SILT		COARSE		FINE		MEDIUM SAND		COARSE		GRAVEL			
CLAY		SILT		V. FINE		FINE		MED.			COARSE		SAND		GRAVEL	
																M. I. T.
																U. S. BUREAU

REMARKS \_\_\_\_\_

## LIST OF ABBREVIATIONS

### PENETRATION RESISTANCE

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

DYNAMIC PENETRATION RESISTANCE: - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51 mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS. 0.3 m 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

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

▲, Δ - UNDISTURBED AND REMOULDED SHEAR STRENGTH DETERMINED FROM IN SITU VANE TEST.

■ - UNDRAINED SHEAR STRENGTH DETERMINED FROM POCKET PENETROMETER TEST.

## LOG OF BOREHOLE NO. D1

N 4 772 812  
E 337 509

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

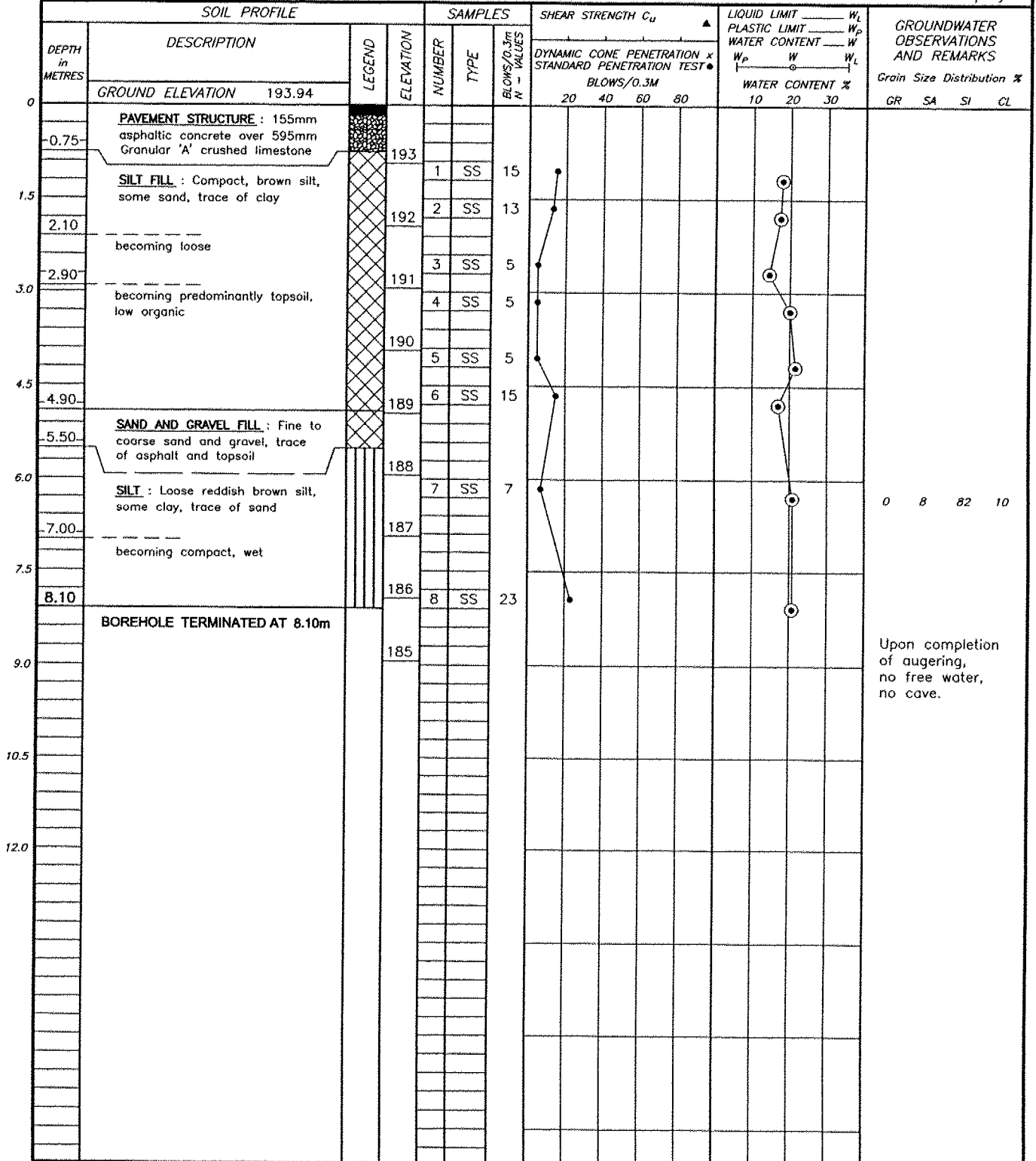
LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE August 14, 1999

ENGINEER P. Cullen

BORING METHOD Continuous Flight Solid Stem Augers

TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: 



## LOG OF BOREHOLE NO. D2

N 4 772 788  
E 337 494

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE July 22, 1999

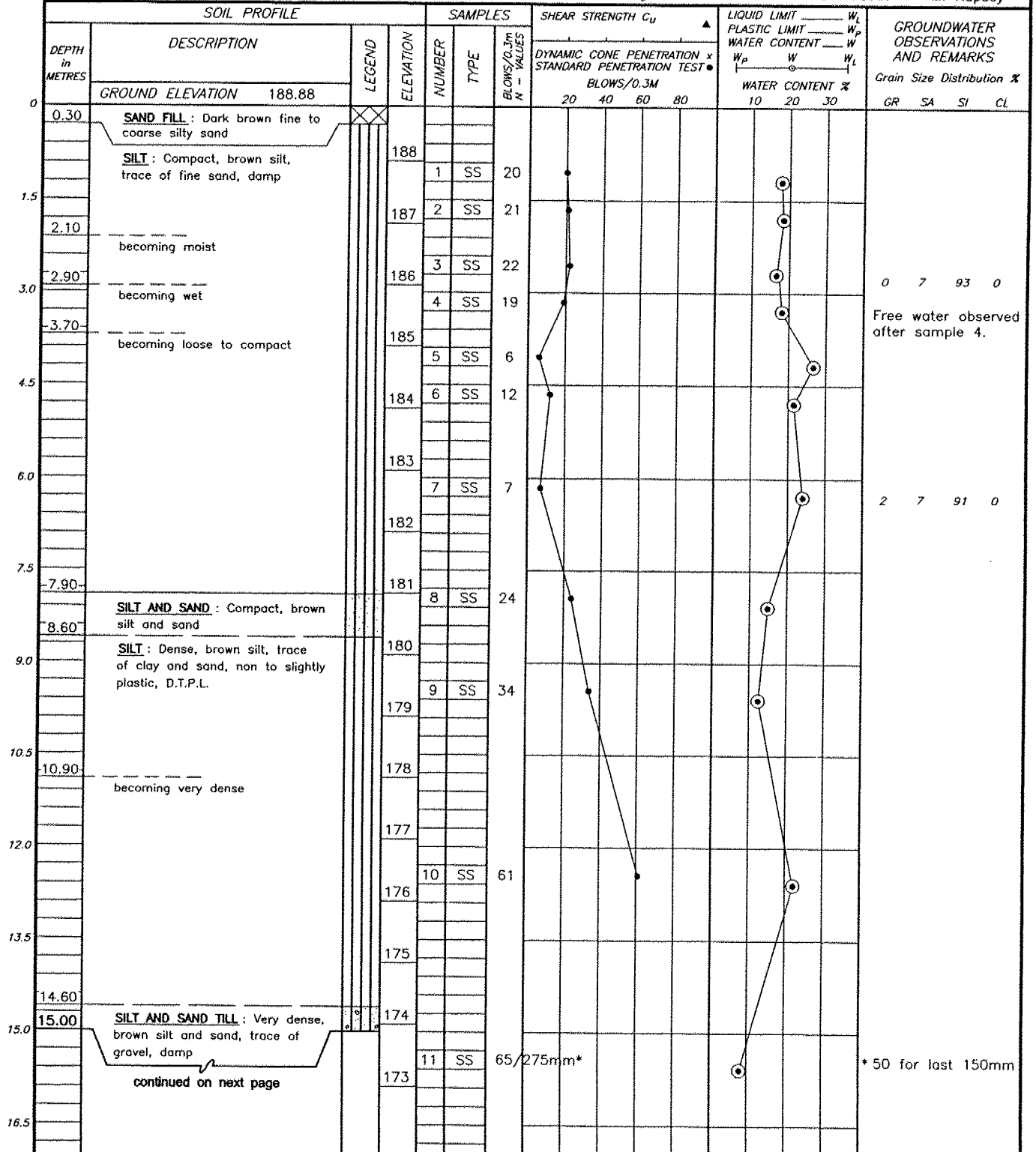
ENGINEER

P. Cullen

BORING METHOD Continuous Flight Hollow Stem Augers & NXL Rock Coring

TECHNICIAN

M. Rapsey



NOTES:

CHECKED BY: *[Signature]*

## LOG OF BOREHOLE NO. D2 (con't)

N 4 772 788  
E 337 494

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

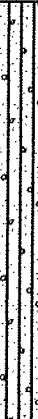

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE July 22, 1999

ENGINEER P. Cullen

BORING METHOD Continuous Flight Hollow Stem Augers & NXL Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$				LIQUID LIMIT $W_L$ PLASTIC LIMIT $W_P$ WATER CONTENT $W$			GROUNDWATER OBSERVATIONS AND REMARKS  Grain Size Distribution % GR SA SI CL		
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •  BLOWS/0.3M 20 40 60 80				WATER CONTENT % $W_P$ $W$ $W_L$				
15.0	GROUND ELEVATION														
	<u>SILT AND SAND TILL</u> : Very dense, brown silt and sand, trace of gravel, damp		173	11	SS	65/275mm*								*50 for last 150mm	
16.5															
			172												
18.0															
			171												
19.5			170	12	SS	80								4 46 47 3	
20.10			169												
21.0	<u>BEDROCK</u> : Dolostone		168	13	RC	1525 100 25 *								* lost drill water in 50mm void at 20.90m.	
			167				End of Run Start of Run								
22.5			166	14	RC	1525 100 98 100									
23.15	BOREHOLE TERMINATED AT 23.15m														
24.0															

NOTES:

CHECKED BY: 

## LOG OF BOREHOLE NO. D3

N 4 772 796  
E 337 525

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE August 13, 1999

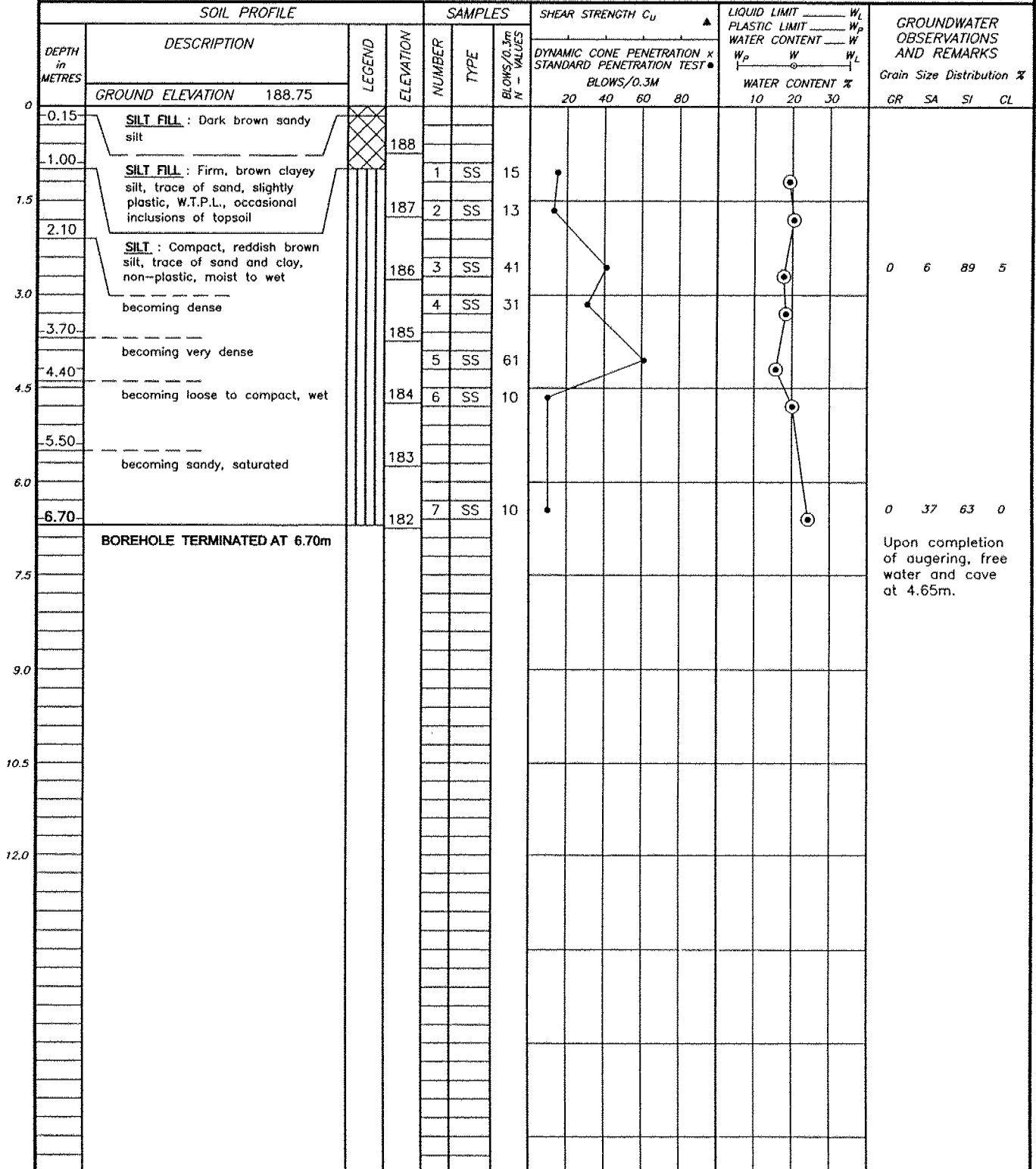
ENGINEER

P. Cullen

BORING METHOD Continuous Flight Solid Stem Augers

TECHNICIAN

M. Rapsey



NOTES:

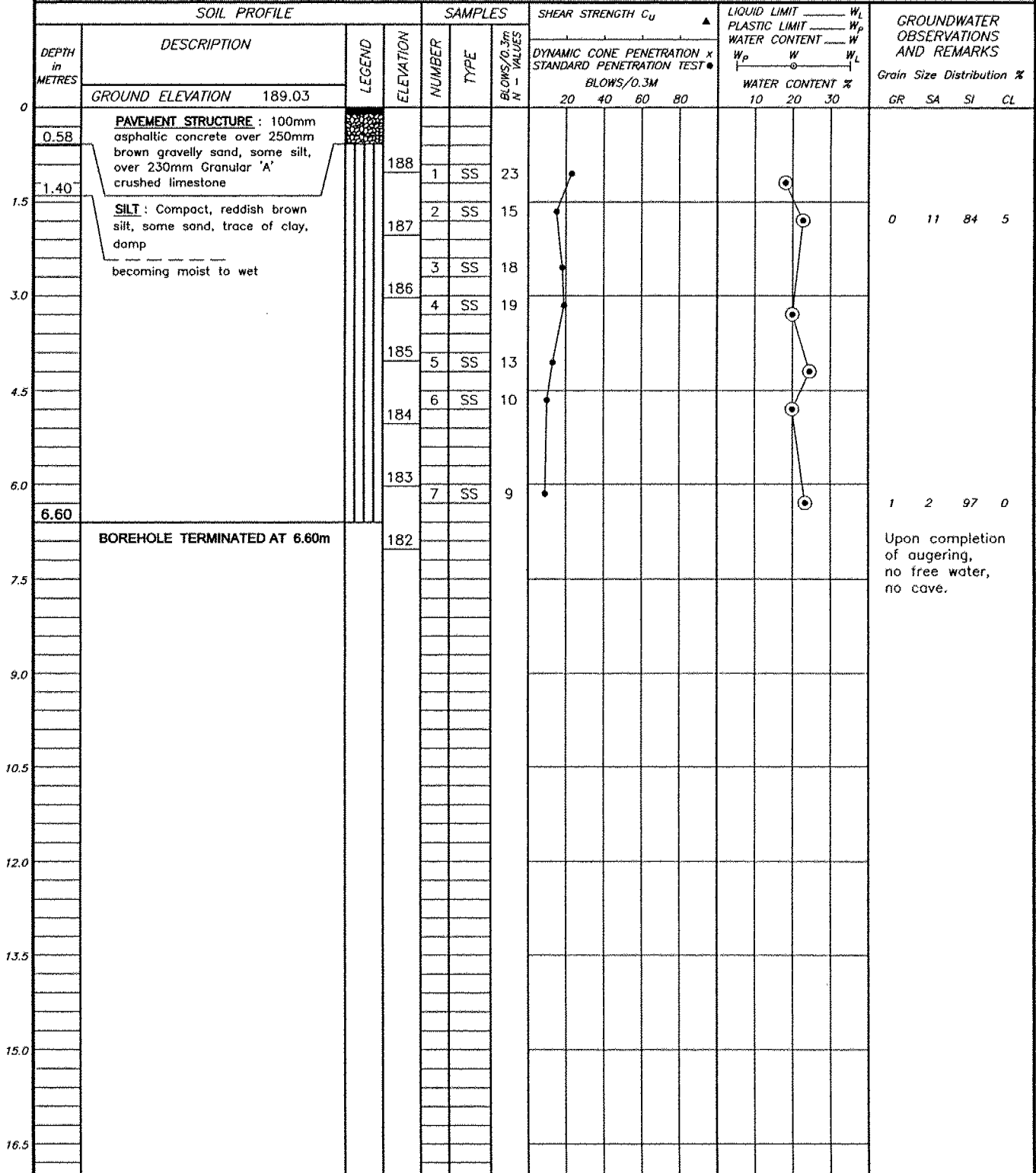
CHECKED BY: *[Signature]*

## LOG OF BOREHOLE NO. D4


N 4 772 767  
E 337 499

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE  
LOCATION Drummond Structure, Niagara Falls, Ontario  
BORING METHOD Continuous Flight Solid Stem Augers

OUR PROJECT 98HF100A  
ENGINEER P. Cullen  
TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: 

## LOG OF BOREHOLE NO. D5

N 4 772 773  
E 337 517

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION -QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE July 21, 1999

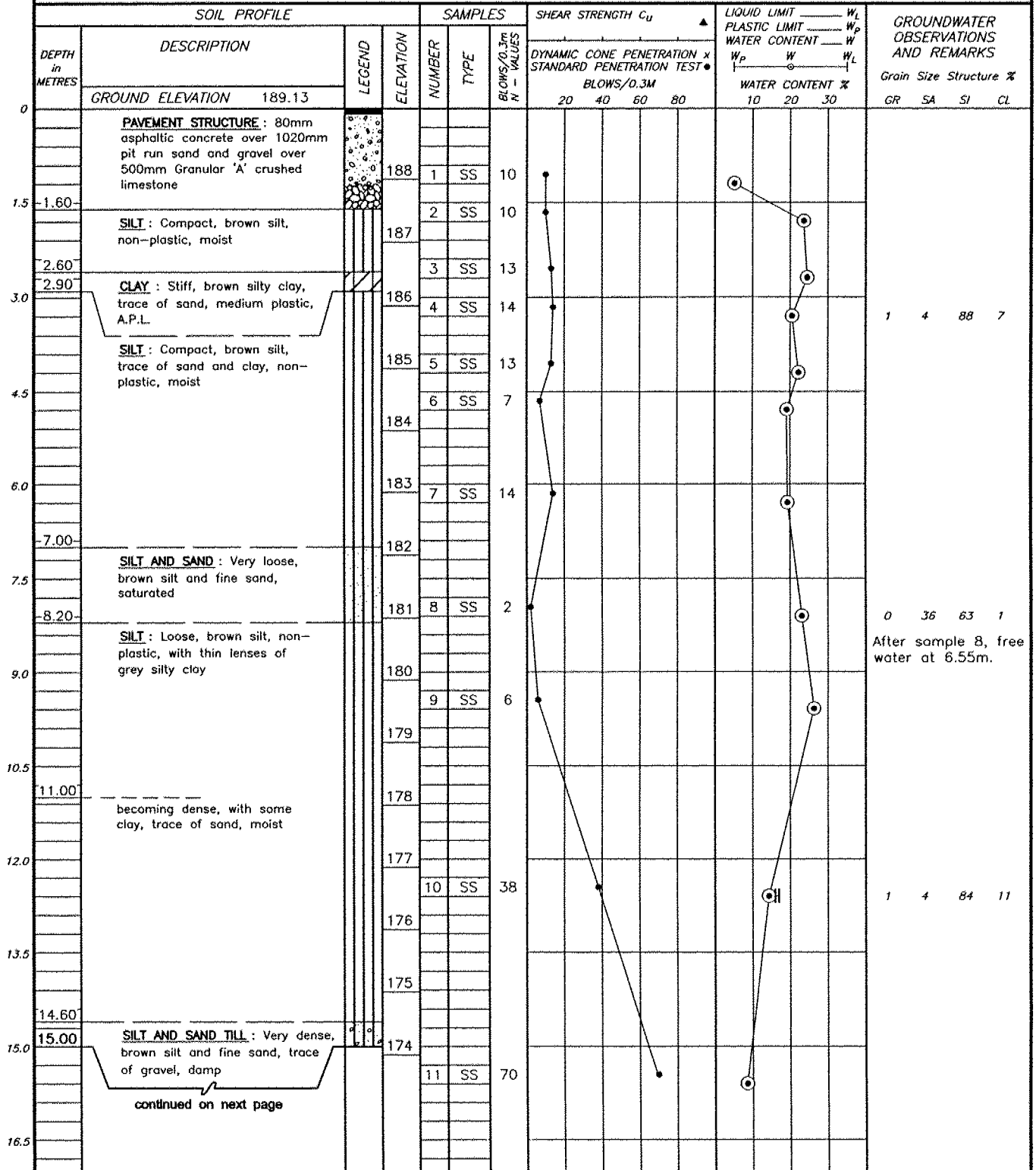
ENGINEER

P. Cullen


BORING METHOD Continuous Flight Hollow Stem Augers & NXL Rock Coring

TECHNICIAN

M. Rapsey



NOTES:

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## LOG OF BOREHOLE NO. D5 (con't)

N 4 772 773  
E 337 517

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE July 19, 1999

ENGINEER P. Cullen

BORING METHOD Continuous Flight Hollow Stem Augers & NXL Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$				LIQUID LIMIT $W_L$			GROUNDWATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST *				WATER CONTENT %		
							BLOWS/0.3M				WATER CONTENT %		
							20	40	60	80	10	20	
15.0	GROUND ELEVATION												Grain Size Distribution % GR SA SI CL
	<u>SILT AND SAND TILL</u> : Very dense, brown silt and fine sand, trace of gravel, damp			11	SS	70							
			173										
16.5			172										
			171										
18.0				12	SS	98/275mm							
			170										
19.5			169										
20.30													
	<u>BEDROCK</u> : Dolostone												
21.0			168	13	RC	1525	100	73	*				
			167										
22.5				14	RC	1525	98	82	100				
23.35			166										
	BOREHOLE TERMINATED AT 23.35m												
24.0													

NOTES:

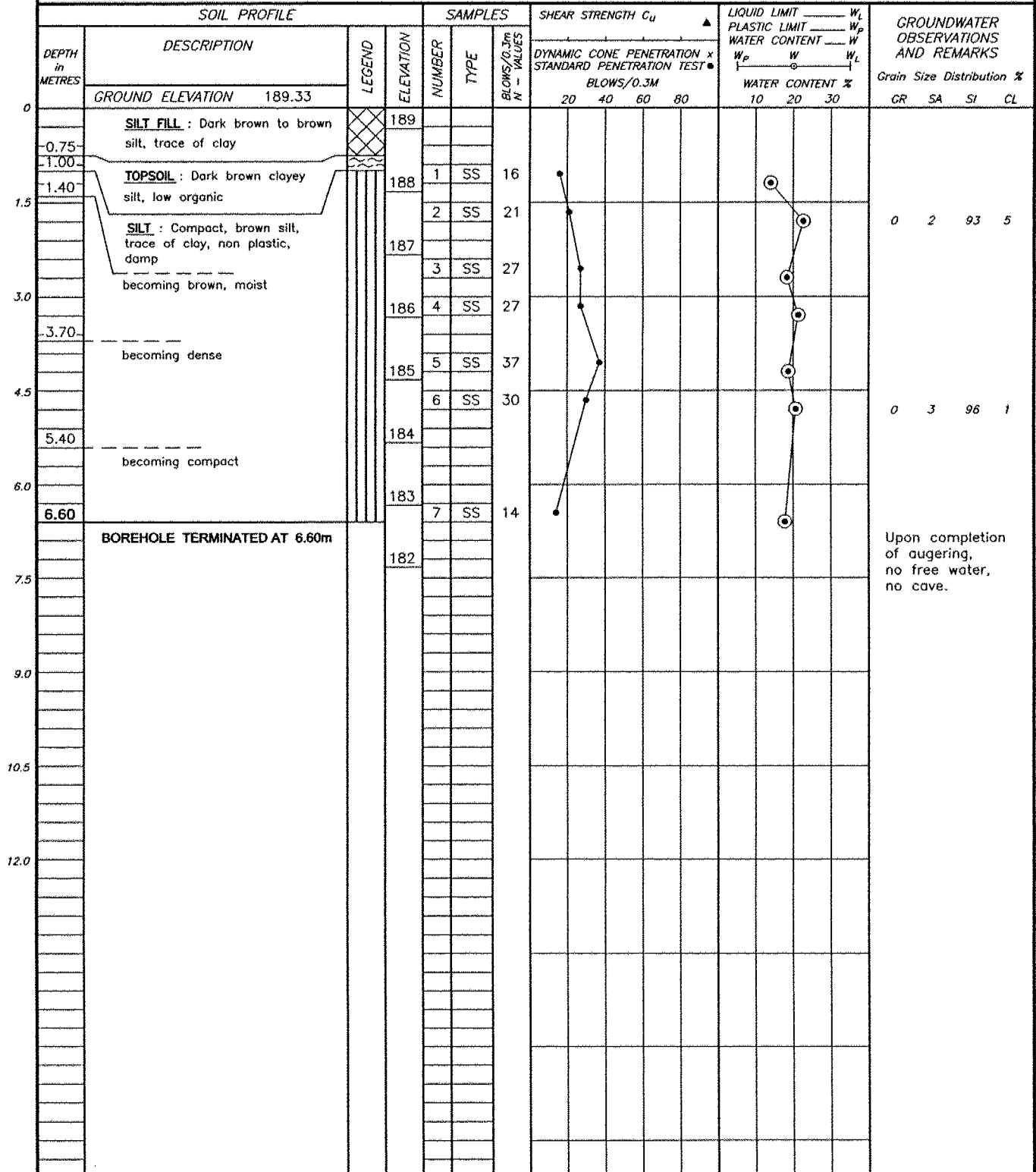
CHECKED BY: *[Signature]*

## LOG OF BOREHOLE NO. D6


N 4 772 744  
E 337 489

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE  
LOCATION Drummond Road Structure, Niagara Falls, Ontario  
BORING METHOD Continuous Flight Solid Stem Augers

OUR PROJECT 98HF100A  
ENGINEER P. Cullen  
TECHNICIAN M. Rapsey



NOTES:

CHECKED BY: 

## LOG OF BOREHOLE NO. D7

N 4 772 754  
E 337 521

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE July 23, 1999

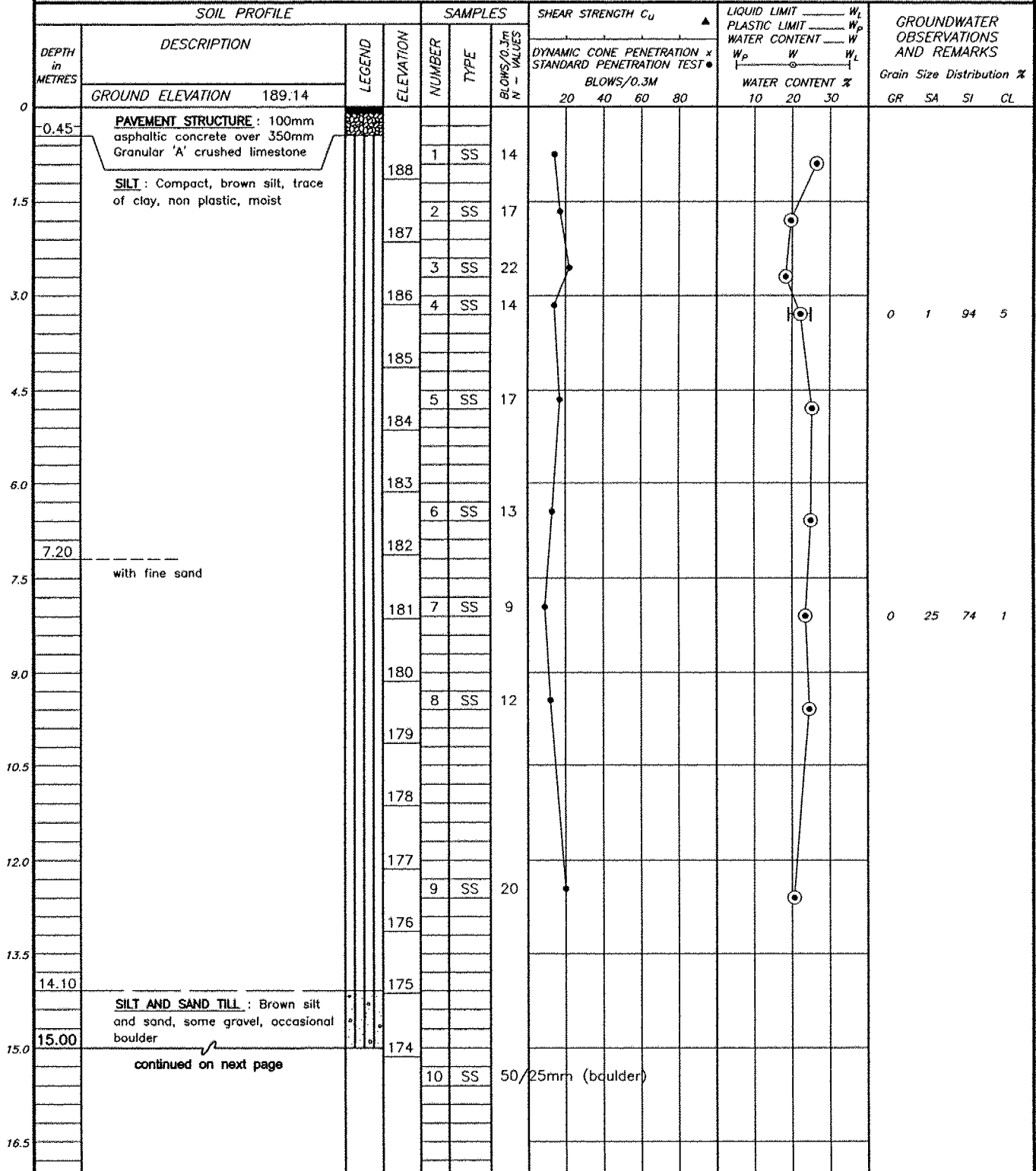
ENGINEER

P. Cullen

BORING METHOD Continuous Flight Hollow Stem Augers & NXL Rock Coring

TECHNICIAN

M. Rapsey



NOTES:

CHECKED BY: *f*



## LOG OF BOREHOLE NO. D7 (con't)

N 4 772 754  
E 337 521

PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE

OUR PROJECT 98HF100A

LOCATION Drummond Road Structure, Niagara Falls, Ontario

BORING DATE July 23, 1999

ENGINEER P. Cullen

BORING METHOD Continuous Flight Hollow Stem Augers & NXL Rock Coring

TECHNICIAN M. Rapsey

SOIL PROFILE				SAMPLES		SHEAR STRENGTH $C_u$				LIQUID LIMIT $W_L$				GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST				WATER CONTENT %				
							BLOWS/0.3M				WATER CONTENT %				
							BLOWS/0.3M				WATER CONTENT %				
							BLOWS/0.3M				WATER CONTENT %				
15.0	GROUND ELEVATION						20	40	60	80	10	20	30	GR SA SI CL	
	<u>SILT AND SAND TILL</u> : Brown silt and sand, some gravel, occasional boulder			10	SS	50/25mm (boulder)								Cored through boulder from 17.30 to 17.90m.	
				173											
16.5															
				172											
18.0															
	<u>BEDROCK</u> : Dolostone													Upon completion of augering, no free water, no cave.  * lost drill water in 25mm void at 20.90m.	
19.5															
20.25				169											
				168	11	RC	1525	100	60	*					
21.0															
	<u>BOREHOLE TERMINATED AT 23.30m</u>														
22.5															
23.30															
				166											
24.0															

NOTES:

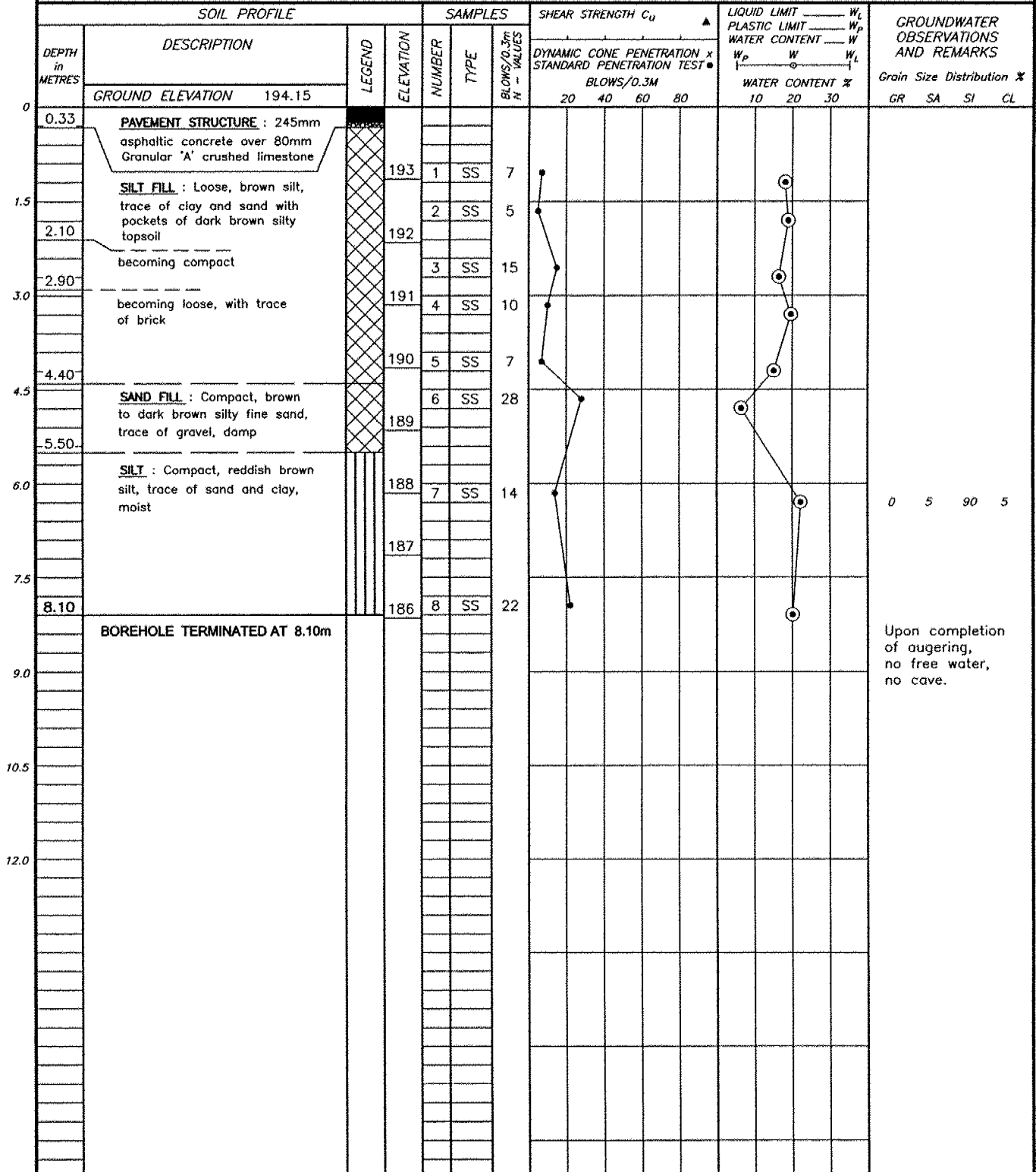
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## LOG OF BOREHOLE NO. D8

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E 337 508

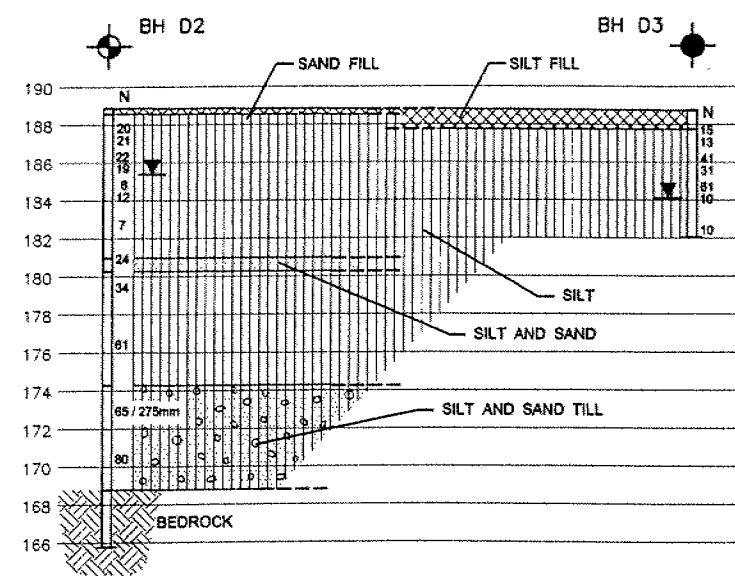
PROJECT GWP 309-94-00 HIGHWAY 420 RECONSTRUCTION - QEW to RAINBOW BRIDGE  
LOCATION Drummond Road Structure, Niagara Falls, Ontario  
BORING METHOD Continuous Flight Solid Stem Augers

OUR PROJECT 98HF100A  
ENGINEER P. Cullen  
TECHNICIAN M. Rapsey

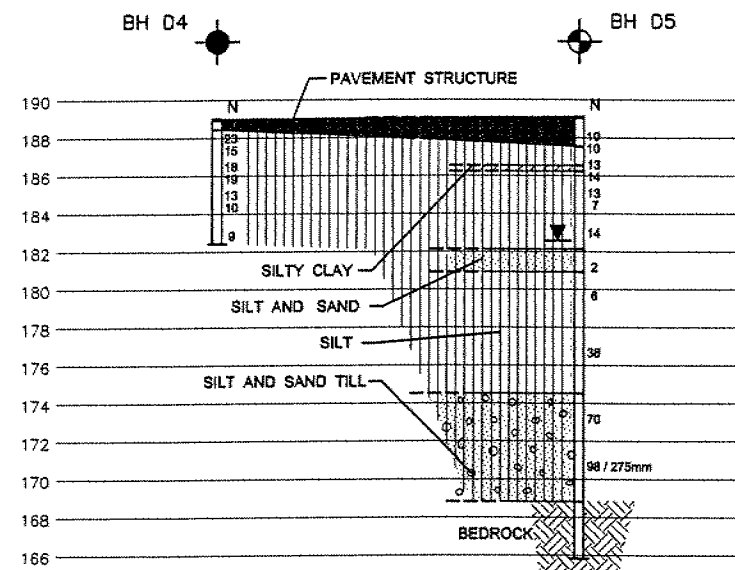


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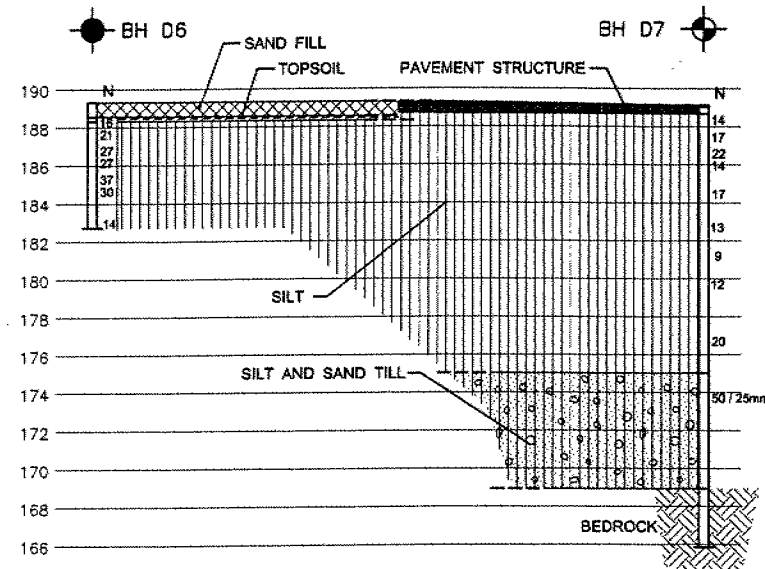
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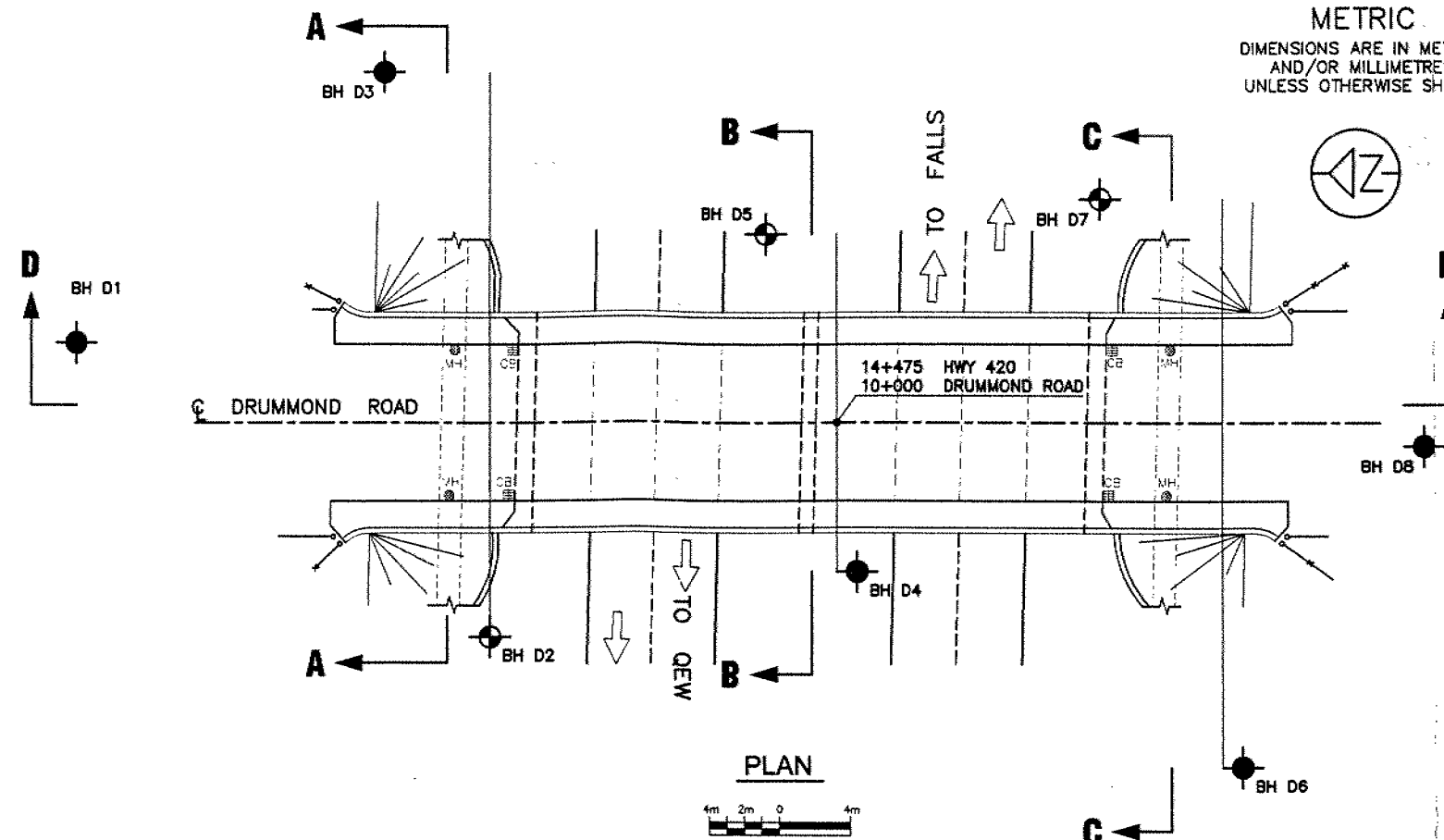
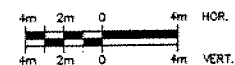
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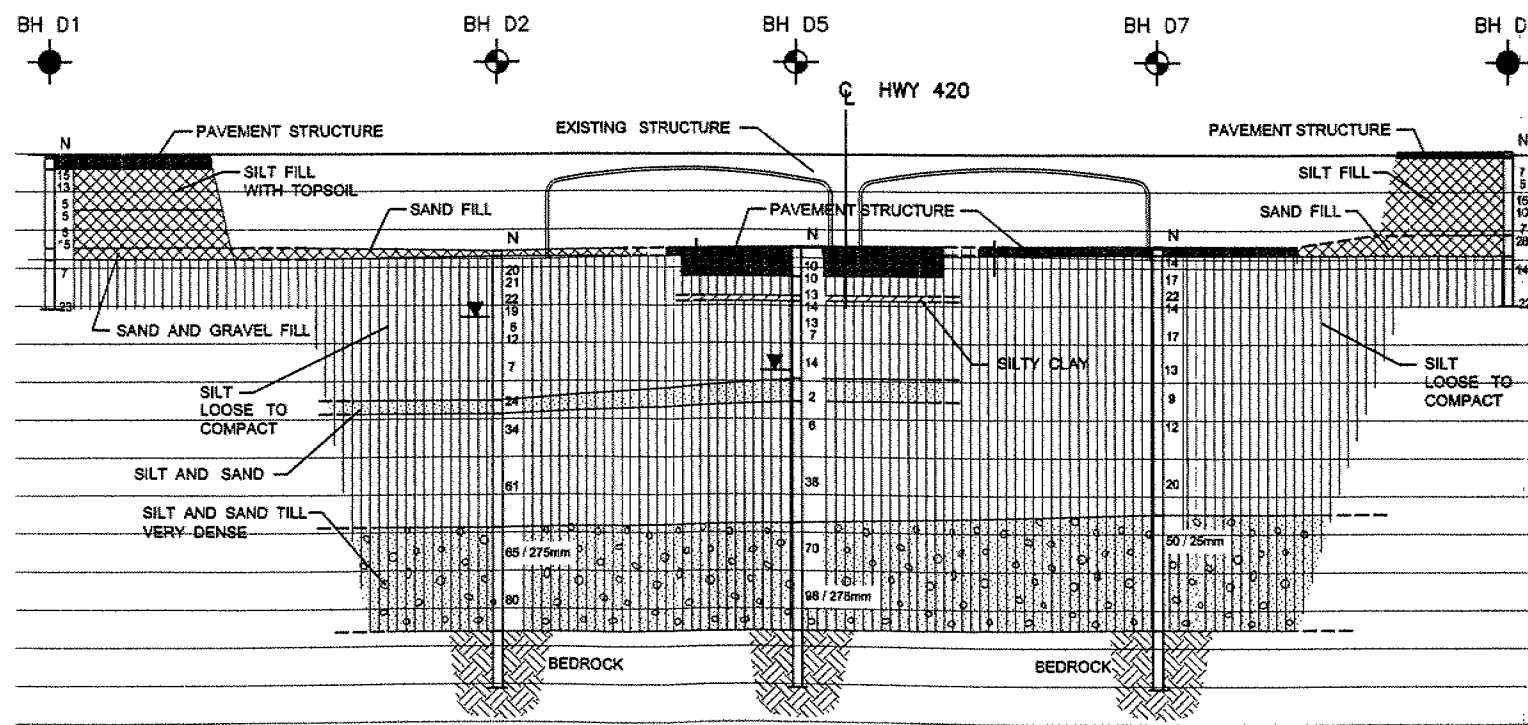
SECTION B-B



SECTION C-C



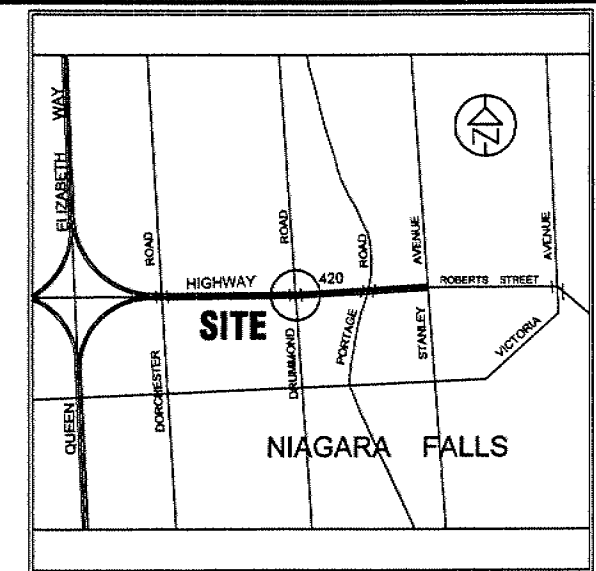
PLAN



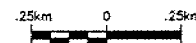
SECTION D-D



G.W.P. 309-94-00  
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN



KEY PLAN



BOREHOLE	NORTHING	EASTING	ELEVATION
BH D1	N 4 772 812	E 337 509	193.94
BH D2	N 4 772 788	E 337 494	188.88
BH D3	N 4 772 796	E 337 525	188.75
BH D4	N 4 772 767	E 337 499	189.03
BH D5	N 4 772 773	E 337 517	189.13
BH D6	N 4 772 744	E 337 489	189.33
BH D7	N 4 772 754	E 337 521	189.14
BH D8	N 4 772 735	E 337 508	194.15

LEGEND

- BOREHOLE
- BOREHOLE & ROCK CORE
- OBSERVED WATER LEVEL  
( DURING OR UPON COMPLETION OF DRILLING )

NOTE

- REFER TO LOG OF BOREHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
- THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES, THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.
- PLAN REPRODUCED FROM UPDATED GENERAL ARRANGEMENT DRAWINGS OF DRUMMOND ROAD UNDERPASS PROVIDED BY PHILIPS PLANNING AND ENGINEERING LIMITED.

DRUMMOND ROAD STRUCTURE  
AT  
HIGHWAY 420  
REGIONAL MUNICIPALITY OF NIAGARA

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS  
45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3G6

DRAWN	CB	DATE	SCALE	JOB NO.	DRAWING NO.
CHECKED	PC	SEPT. 1999	AS SHOWN	98HF100A	1
APPROVED	DWK				

BOREHOLE LOCATION PLAN  
AND SOIL PROFILES

**FOUNDATION DESIGN REPORT  
FOR  
DRUMMOND ROAD STRUCTURE  
HIGHWAY 420 RECONSTRUCTION  
QEW TO RAINBOW BRIDGE  
G.W.P. 309-94-00  
NIAGARA FALLS, ONTARIO**

Distribution:

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1 cc: Philips Engineering Ltd.  
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Job No. 98HF100A

January, 2000

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## **FOUNDATION DESIGN REPORT**

For  
Drummond Road Structure  
Highway 420 Reconstruction  
QEW to Rainbow Bridge  
G.W.P. 309-94-00  
Niagara Falls, Ontario

---

### **INTRODUCTION**

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches for the proposed Highway 420 underpass at Drummond Road in Niagara Falls, Ontario. The report was prepared for Philips Engineering Ltd. on behalf of The Ministry of Transportation Pavements and Foundations Section.

Replacement of the existing bridge structure at Drummond Road is planned. The existing bridge is a two span reinforced concrete structure supported by spread footings. It has a span length of approximately 15.9 m, an approximate width of 12.5 m, and was constructed in 1941. Road grade on Drummond Road over the structure is near elevation 194.0, some 5 to 6 m above existing grade on Highway 420 (based on undated general arrangement drawings of the Drummond Road underpass provided by Philips Engineering Ltd. and existing ground surface elevations determined at the testhole locations).

It is anticipated the new bridge at Drummond Road will comprise a two span structure with span lengths similar to or slightly larger than the existing bridge. The grade on Drummond Road will be raised approximately 800 mm above its present level.

The subsurface stratigraphy revealed at the bridge site generally comprised a surficial pavement structure and/or sand/silt fill layer penetrated at depths of 0.3 to 1.6 m (elevation 187.5 to 188.7) overlying native loose to compact, non cohesive silt to depths of 14.1 to 14.6 m (elevation 174.3 to 175.0). The silt was underlain by very dense non cohesive silt and sand till mantling dolostone bedrock contacted at depths of 20.1 to 20.3 m, elevation 168.8 to 168.9.

## **FOUNDATIONS**

### **Integral Abutments on Piles**

Construction of integral abutments supported on steel H-piles is considered feasible. The piles should be driven to refusal on bedrock at depths anticipated at 20.1 to 20.3 m (elevation 168.8 to 168.9). The recommended axial resistances for selected pile sections are as follows:

<u>H-Pile Section</u>	<u>Factored Resistance at ULS (kN)</u>
HP 310 x 79	1450
HP 310 x 110	2000

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 capacity at ULS.

The soil adjacent to the upper portion of the piles is expected to comprise well compacted approach fill placed on loose to compact silt.

To accommodate movement of the integral abutment, it is recommended that a 600 mm diameter pre-augered hole be provided around the pile to a depth of 3 m below the bottom of the abutment. The pre-augered hole should be filled with loose sand meeting the gradation requirements shown on Table I. Alternately, two concentric CSPs may be placed around the pile to create an annular space; the inner CSP should be filled with cement-bentonite grout. Refer to MTO Report SO-96-01 for further details.

A 25 to 75 mm void was detected in the rock core of the three deep boreholes drilled at the abutment/piers. It is imperative that the surficial 3 m of rock is grouted to fill the voids prior to driving piles, if this option is to be implemented. 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 entered beyond the outside limits of the foundation at least 2 m. A specialist grouting contractor must be retained for this work.

The type of equipment employed 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 the very dense silt and sand till 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 Granular "A" backfill or native overburden may be computed using the following equation to evaluate the point of contraflexure:

$$\begin{aligned} k_s &= n_h z/b \\ \text{where } z &= \text{depth (m)} \\ b &= \text{pile width (m)} \end{aligned}$$

The recommended values for  $n_h$  are as follows:

	<u><math>n_h</math> (kN/m<sup>3</sup>)</u>
Granular "A" Backfill	14,000
Loose/Compact Silt	4,000

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile below the annular space.

The lateral resistances recommended for HP 310 x 110 piles are as follows:

Factored Lateral Resistance at ULS	=	120 kN
Lateral Resistance at SLS	=	50 kN

### Spread Footings

Supporting the structure on conventional spread footings founded in the native loose to compact silt at approximate 1.2 m depth below existing grade (1.6 m at borehole D5) may be considered. The bearing resistances at ultimate and serviceability limit states of footings constructed in the native silt at elevation 187.5 are as follows:

<u>Location</u>	<u>Footing Width (m)</u>	<u>Bearing Resistance at SLS (kPa)</u>	<u>Factored Bearing Resistance at ULS (kPa)</u>
North Abutment	3	200	600
	4	175	650
	5	150	700
Centre Pier	3	150	450
	4	150	550
South Abutment	5	150	650
	3	200	600
	4	200	700
	5	175	750

Alternatively, spread footings could be constructed on structural fill placed in the approaches. 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 existing approach fill should be removed prior to placement of the structural fill.

The bearing resistances for a minimum 2.5 m wide footing constructed on a minimum 2.5 m thick pad of structural fill are:

Factored Bearing Resistance at ULS	=	1200 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 silt or granular fill. Unfactored friction factors of 0.35 and 0.45 are recommended for footings on silt and granular fill, respectively.

The design capacities should be reviewed when further details regarding founding levels and abutment locations are established.

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 caissons bearing on the very dense till or socketed at least 1 m into bedrock (0.5 to 0.9 m below the voids identified in the rock core) is considered to be feasible. The depths/elevations of the very dense till and bedrock are as follows:

<u>Location</u>	<u>Very Dense Till</u>		<u>Bedrock</u>	
	<u>Depth (m)</u>	<u>Elevation</u>	<u>Depth (m)</u>	<u>Elevation</u>
North Abutment	14.6	174.3	20.1	168.8
Centre Pier	14.6	174.5	20.3	168.8
South Abutment	14.1	175.0	20.2	168.9

Due to the relatively wide variation in relative density/strength of the glaciolacustrine overburden deposits, shaft resistance developed in the sand and silt unit should not be used to compute the axial resistance of the caissons.

The caissons should be designed using the following resistances:

Factored End-Bearing Resistance at ULS

Very Dense Till (at till surface)	= 1000 kPa
Very Dense Till (minimum three diameters into very dense till)	= 2000 kPa
Bedrock (minimum 1 m into rock)	= 3000 kPa

Factored Bond Stress at ULS

Very Dense Till	= 50 kPa
Bedrock	= 500 kPa

Based on these values, the factored axial resistance (kN) for selected caisson diameters are presented below:

Caisson Diameter (m)	0.76	0.91	1.07	1.22
Factored Axial Resistance at ULS (kN)				
Very dense till (at till surface)	450	650	900	1150
Very dense till (3 diameters into till)	1150	1650	2300	3000
Bedrock (socketed 1 m into the rock)	2550	3350	4350	5400

The resistance at serviceability limit states normally allows for 25 mm of compression of the founding medium. Considering the very dense till and 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.

All caisson bases should be hand cleaned then inspected by qualified geotechnical personnel to verify the competency of the founding surface. A steel liner and air quality monitoring must be provided by the contractor for down-hole-entry for inspection of the caisson bases.

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; a caisson liner should be installed to minimize the potential for sloughing and groundwater inflow.

### **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:

	<u>Granular "A"</u>	<u>Granular "B"</u>
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. The following parameters should be employed for design of the system foundation:

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

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.

At the proposed underpass location, the road grade on Drummond Road will be raised about 800 mm above existing grade. Prior to placement of fill on the existing embankment slope to accommodate the grade raise, the embankment slopes should be benched in accordance with OPSD 208.01.

Approach embankments should be constructed in accordance with OPSD 200.01, 202.01 and 208.01. Embankment slopes 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. If footings are placed on structural fill, the existing fill/topsoil and other deleterious material should be stripped prior to placement of the approach fill.

### **EXCAVATION AND GROUNDWATER CONTROL**

Excavation for construction of footings, if employed, is expected to be carried out primarily within the approach fill and/or native silt. Excavation is expected to be relatively straightforward. The fill/silt is classified as a 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. Flatter sideslopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered.

Free water was observed in three of the eight boreholes at depths of 3.5 to 6.5 m during the course of the fieldwork. Seepage or surface water which enters the excavation should be readily handled by conventional sump pumping techniques.

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., Project Engineer. It was reviewed by Mr. D.W. Kerr, P.Eng., Manager of Geotechnical and Geo-Environmental Services, Hamilton.



Yours very truly

**Peto MacCallum Ltd.**

A handwritten signature in black ink, appearing to read "D. W. Kerr", written over a horizontal line.

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



A handwritten signature in black ink, appearing to read "Brian R. Gray", written over a horizontal line.

Brian R. Gray, M.Eng., P.Eng.  
Vice-President  
Geotechnical and  
Geo-Environmental Services

DWK:dI



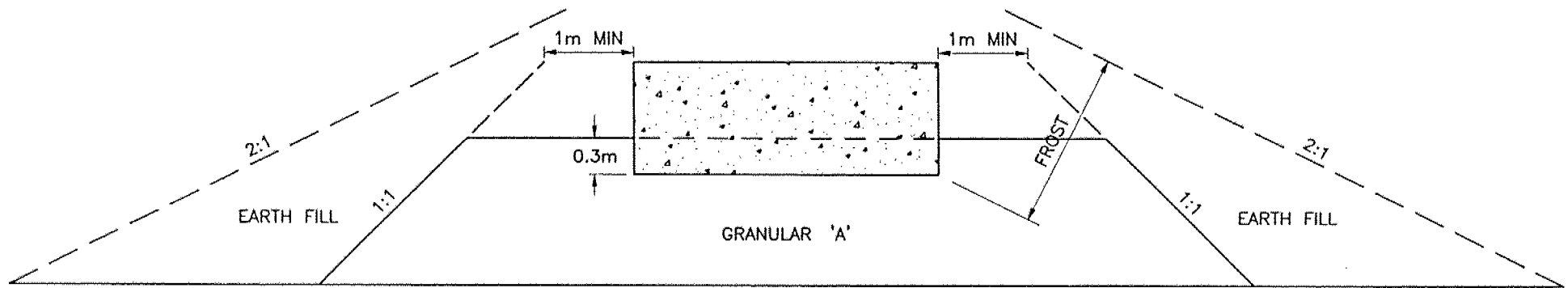
TABLE I

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

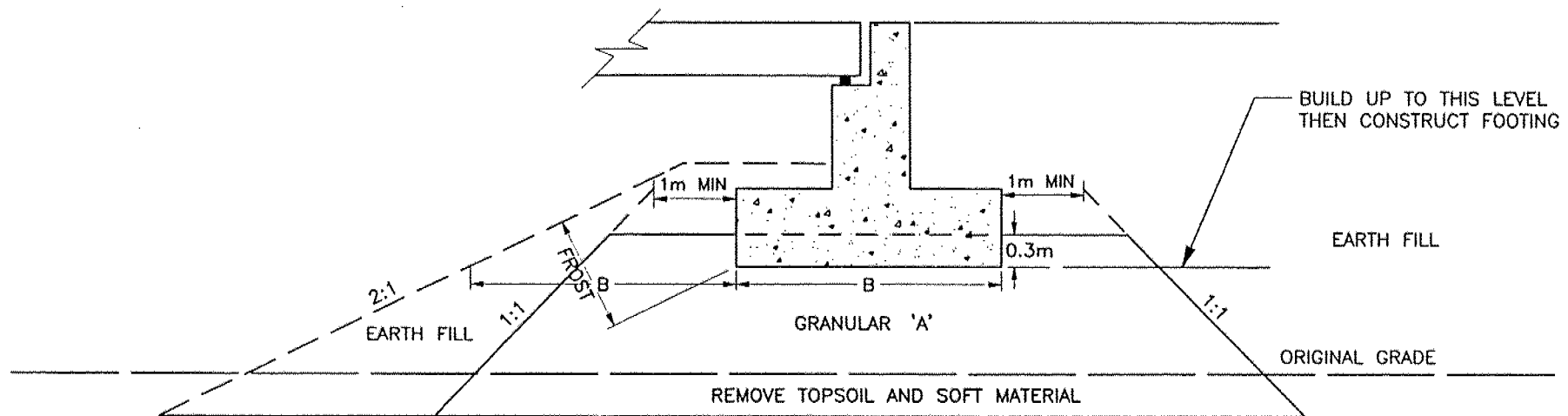
MTO Sieve Designation		Percentage Passing by Mass
2 mm	#10	100
600 µm	#30	80 - 100
425 µm	#40	40 - 80
250 µm	#60	5 - 25
150 µm	#100	0 - 6

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

**Peto MacCallum Ltd.**  
CONSULTING ENGINEERS

45 BURFORD ROAD, HAMILTON, ONTARIO L8E 3C8  
Tel: (905) 561-2231 Fax: (905) 561-8383

DATE	SCALE	JOB NO.	FIGURE NO.
Jan. 2000	NTS	98HF100A	1

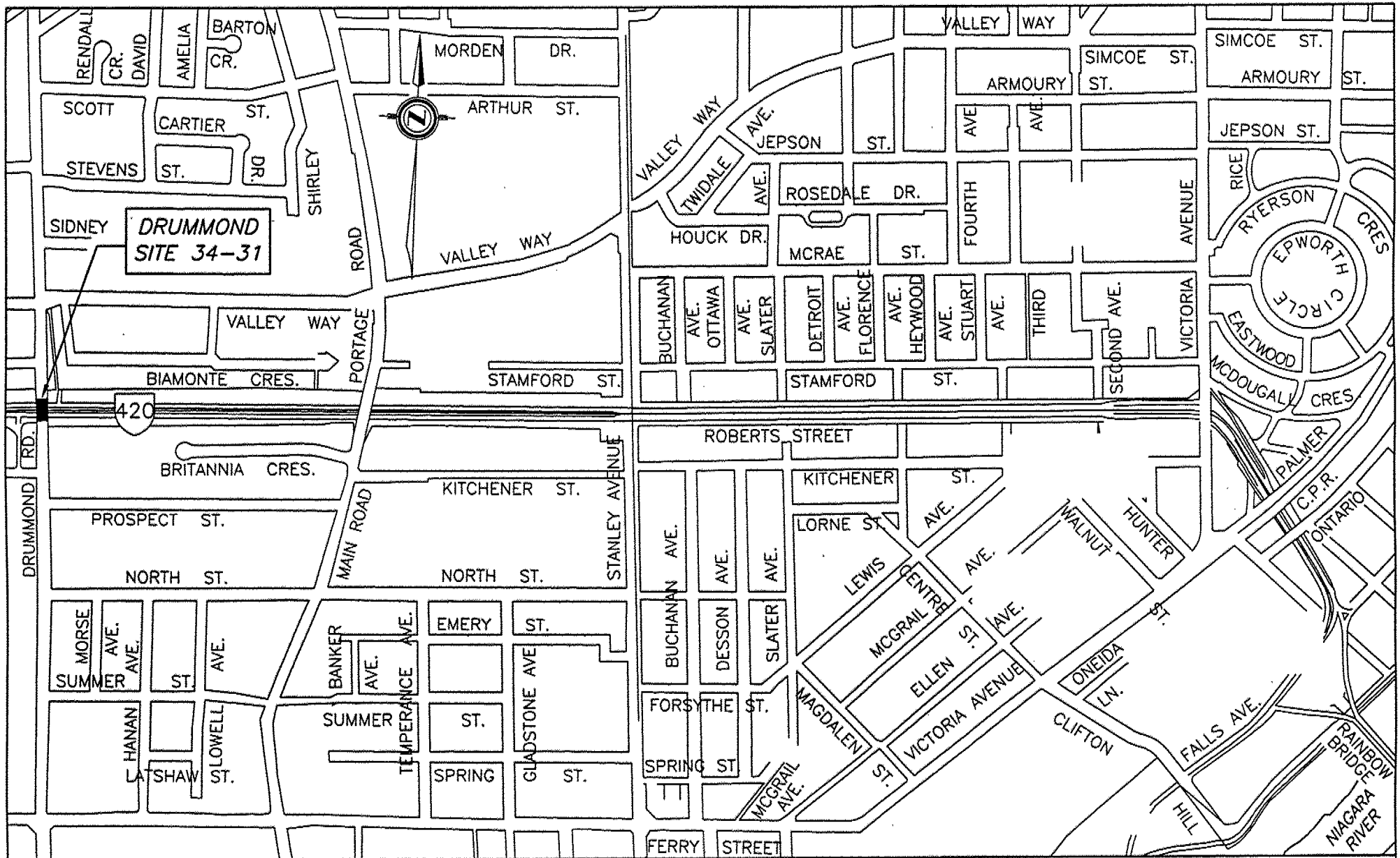
**STRUCTURAL PLANNING REPORT**

**HIGHWAY 420  
Replacement of the  
Drummond Road Underpass  
Site 34-31  
G.W.P. 309-94-00**

**September, 1999**

**PHILIPS PLANNING AND ENGINEERING LIMITED  
3215 North Service Road  
P.O. Box 220  
Burlington, ON L7R 3Y2**

**Tel: (905) 335-2353  
Fax: (905) 335-1414**



SITE PLAN

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**Appendices**

- 1) General Arrangement Drawings
- 2) Preliminary Estimates
- 3) Roadway Protection

## 1.0 INTRODUCTION

The Drummond Road Underpass on Highway 420 is located in the City of Niagara Falls and in the Regional Municipality of Niagara. The bridge structure, originally constructed in 1941 under Contract 39-123, is a two span concrete rigid frame structure with 15 metre spans.

In 1978, under Contract 77-87, the bridge was rehabilitated. The rehabilitation consisted on a concrete overlay, epoxy repair of the soffit, asphalt and waterproofing system, new sidewalks and parapet walls.

This bridge has since deteriorated and is currently a candidate for replacement under the Highway 420 Rehabilitation/Reconstruction Project. The project is scheduled for construction in the year 2000.

## 2.0 GEOMETRIC LAYOUT

### 2.1 Description

The Drummond Underpass on Highway 420 is an interchange with button-hook ramps. The east-bound ramp is located just west of the Drummond Road structure and the west-bound ramp is located just east of the structure.

Due to the relative location of the on ramps and abutments, the bridge's abutment restricts view from the ramp resulting in substandard site lines.

The cross section of Highway 420 at this location consists of 2 - 3.5 metre through lanes in each direction, 2 - 4.75 metre on ramps, 2.5 metre right shoulders and a centre median of 8.2 metres.

At the bridge, Highway 420 is on a long horizontal curve with a radius of 6,985.5 metres. The vertical alignment under the bridge is 0.5% rising to the east.

Over Highway 420, Drummond road is on a tangent horizontal alignment. The bridge is located on the summit of a vertical curve, which had been splined during the 1978 rehabilitation of the structure. The grades to the structure, prior to the 1978 construction, were 5% and 2.8% on the north and south approaches respectively.

The critical vertical clearance under this structure is sub-standard, with clearances of 4.1 metres over the south-east ramp and 4.3 metres over the north/south-west ramp.



As the existing clearance is presently substandard, the structure must be shallow to economically minimize the roadwork required on the Drummond Road or Highway 420. The constraints are the 450mm diameter sanitary sewer crossing Highway 420 with minimum cover limiting the lowering and the approach grades on Drummond Road which limits raising the Drummond Road. Considering this criteria, the following alternative structure types were considered:

- *CPCI -900 precast girder*
- *BI-1220 concrete box precast girder*
- *Reinforced concrete rigid frame*

General arrangement drawings are prepared for each of the alternatives and preliminary estimates are presented in the appendix to this report.

### ***CPCI-900 Girders***

The CPCI-900 girder bridge is a 2 span structure with a composite 225mm concrete deck providing continuity over the pier. A semi-integral abutment system is proposed for this structure, eliminating the deck joint at the abutments and providing a more durable structure.

The abutments of the girder bridge will be supported on a piled foundation and the earth will be restrained by a Retained Soil System (RSS). The centre pier will be founded on drilled caissons, with columns extending to the underside of a pier cap. The caisson reduces the size of the footings minimizing the disturbance of the soil adjacent to the utilities near the pier.

The major advantage of the precast girder structure is the constructability of the structure over active traffic, without an elaborate false work system and the construction clearances which will be required. This girder alternative combined with a semi-integral abutment system allows for the elimination of the deck joints, improving the durability of the structure.

The CPCI girder provides an ideal location between the girders to support the electrical utilities presently carried under the sidewalk encased in the structure.

The grade raise proposed for this alternative is 0.95 m, and grades are 6% on the north and 4.7% on the south approaches to the bridge.



### ***Pre-stressed, Precast Concrete Box Girder Bridge***

The Pre-cast Concrete Box girder bridge is a 2 span structure with a composite 150mm concrete deck providing continuity over the pier. A semi-integral abutment system is proposed for this structure, eliminating the deck joint at the abutments, and hence improving the durability of the structure.

The substructure for this alternative is similar to the alternative previously presented.

The major advantage of this pre-stressed, precast girder structure over the previous is the girder's depth of 686mm and the 150mm concrete deck. The improved structure depth minimizes the profile change on Drummond Road. The structure's constructability is similar to the CPCI girder structure described above.

The utilities will be carried through the structure in a similar location as the existing bridge, i.e. under the sidewalk encased in the structure.

The grade raise proposed is 0.7 m, and approach grades are 4.8% on the north, and 4.7% on the south approaches.

### ***Concrete Rigid Frame***

A concrete rigid frame structure similar to the existing structure has proven durable in the past, and deserves to be considered as an alternative. The structure provides a minimal depth of 600mm at the centre of the span making it the thinnest structure. and required no additional concrete topping. With minor cross slope correction of Highway 420, the vertical clearance can be obtained without any significant changes in the profile of Drummond Road.

The abutment can be placed on spread footings similar to the existing structure and the centre pier will be founded on drilled caissons as existing utilities will conflict with spread footings.

The grade raise proposed is 0.3 m, with approach grades of 5.1% and 4.5%.

## **3.2    *Aesthetics***

The Drummond Underpass is the first of two similar bridges on the section of the highway where motorists start slowing down and realize that they are approaching the destination. These structures provide an opportunity to improve the aesthetics of the structure with the introduction of a textured masonry stone finish.

The textured finish includes exposed aggregate, embossed sections, grooves and fins. Irregular patterns are achieved by form liners to the precast units of Retained Soil Systems (RSS) or Rigid Frame Concrete Structure.

Masonry concrete stone finishes on RSS are created by casting concrete directly against a wire cage finish available with RSS. The stone masonry will be attached to this concrete wall in a conventional manner. It is important to note that the RSS must be fully constructed, allowing the soil to consolidate prior to pouring the concrete face, reducing the possibility of overstressing in the concrete veneer.

The cost comparison of the finishes are provided in the following table:

	Additional Cost	
	RSS	Rigid Frame
Architectural Finished Panels	50	\$125/m <sup>2</sup>
Stone Masonry Panels	500	\$400/m <sup>2</sup>

As indicated in the table, the cost of the textured finishes are significantly less than the stone masonry alternatives. However, the masonry stone finish may provide more of a calming effect on the motorists and provide a stimulating influence to reduce speed as the traffic leaves the higher speed freeway and enter the slower speed arterial section of the highway.

The advantages of patterned concrete treatment on the wall are:

- *Economical (\$70,000 less than stone masonry)*
- *Wide variety of patterns including custom patterns*
- *Low maintenance*

Masonry stone walls become more prevalent as the highway approaches the Falls and introducing stone masonry will complement these structures. However, the additional costs and the additional maintenance on the stone masonry walls will offset some of the benefits gained. As this is the first structure passed the QEW/Highway 420 interchange bridges, textured concrete substructures would provide a transition between a plain exposed concrete finish in the interchange and the stone walls to the east.

We therefore recommend a textured finish on this structure. The final selection or extent of the textured finishes should be coordinated with the architect currently working on the Highway 420 Reconstruction in order to maintain a constant theme throughout the corridor.

### 3.3 *Removal of Structure*

The removal of the structure will be staged with the reconstruction of Highway 420. The first task is to close Drummond Road and to detour traffic around the site. The detour route is described in the appendix. Philips Planning and Engineering Ltd., in consultation with the City selected this route because of the existing infrastructure present at the intersections along the route.

Once traffic has been diverted, the earth pressure will be eliminated by the removal of the embankment directly behind the rigid frame structure. Excavation shall be carried out simultaneously on both ends of the bridge thus maintaining equal pressure on the abutments. Abutment legs will be restrained while the deck is being removed. The deck will be removed by, either saw cutting and lifting with a large crane or by more conventional means such as closing or protecting the lanes and demolishing the bridge by mechanical methods.

Removal of the structure will require false work to be placed under the bridge for the protection of the public and the highway. The false work will impose a restricted vertical clearance resulting in the rerouting of trucks and buses.

### **3.4 Roadway Protection**

The removal of the substructure will require roadway protection, at the pier and abutment locations. Shoring protection sketches are provided in the appendix of this report. Roadway protection should also be extended down Drummond Road to protect property owners to the north and south of the structure.

### **3.5 Utility Protection**

During the demolition care must be taken to protect the utilities in, under and over the structure. These include the following:

- *525 mm diameter storm sewer laid down the median of Highway 420 adjacent to the centre pier footing*
- *450 mm diameter sanitary sewer crossing Highway 420 beneath the structure crossing through or just under the three footings of the structure*
- *Electrical ducts in the structure*
  - *Niagara Hydro 3 ducts (100 mm diameter, plastic)*
  - *Bell Canada 4 ducts (100 mm diameter, fibre glass)*
  - *Traffic Signal 1 duct (on design drawings – not uncovered in verification of service conduit survey)*
- *Overhead Hydro just west of the structure*
- *Under ground hydro (75 mm duct) providing power to light standards position in the median of highway 420*
- *Storm sewers, complete with 2 catch basins and 2 manholes on each approach to the bridge on Drummond Road*

### 3.6 Noise Barrier

The existing noise barrier along the north-west and south-east corner of the bridge will require removal to allow construction of the bridge. Following construction, the noise wall should be installed and connected to the structure to catch noise escaping from the freeway.

### 3.7 Cost Comparisons

Preliminary estimates of the three alternatives have been prepared and are presented in the appendix of this report. The estimates are summarized as follows.:

<u>Alternatives</u>	<u>Estimate</u>
CPCI-900	\$1,129,000
Box B1-1220	\$1,114,000
Rigid Frame	\$1,102,000

## 4.0 SELECTED ALTERNATIVE

The alternatives are presented in the following matrix for easy comparison:

Alternative Comparison Drummond Road Underpass			
	CPCI	Box	Rigid Frame
Cost	\$1,129,000	\$1,114,000	\$1,102,000
Vertical Alignment	> 6%	4.8%	4.8%
Maximum Grade Raise	0.95 m	0.7 m	0.3 m
Construction Duration	3 mos.	3 mos.	4½ mos.
Traffic Stages	3	3	4

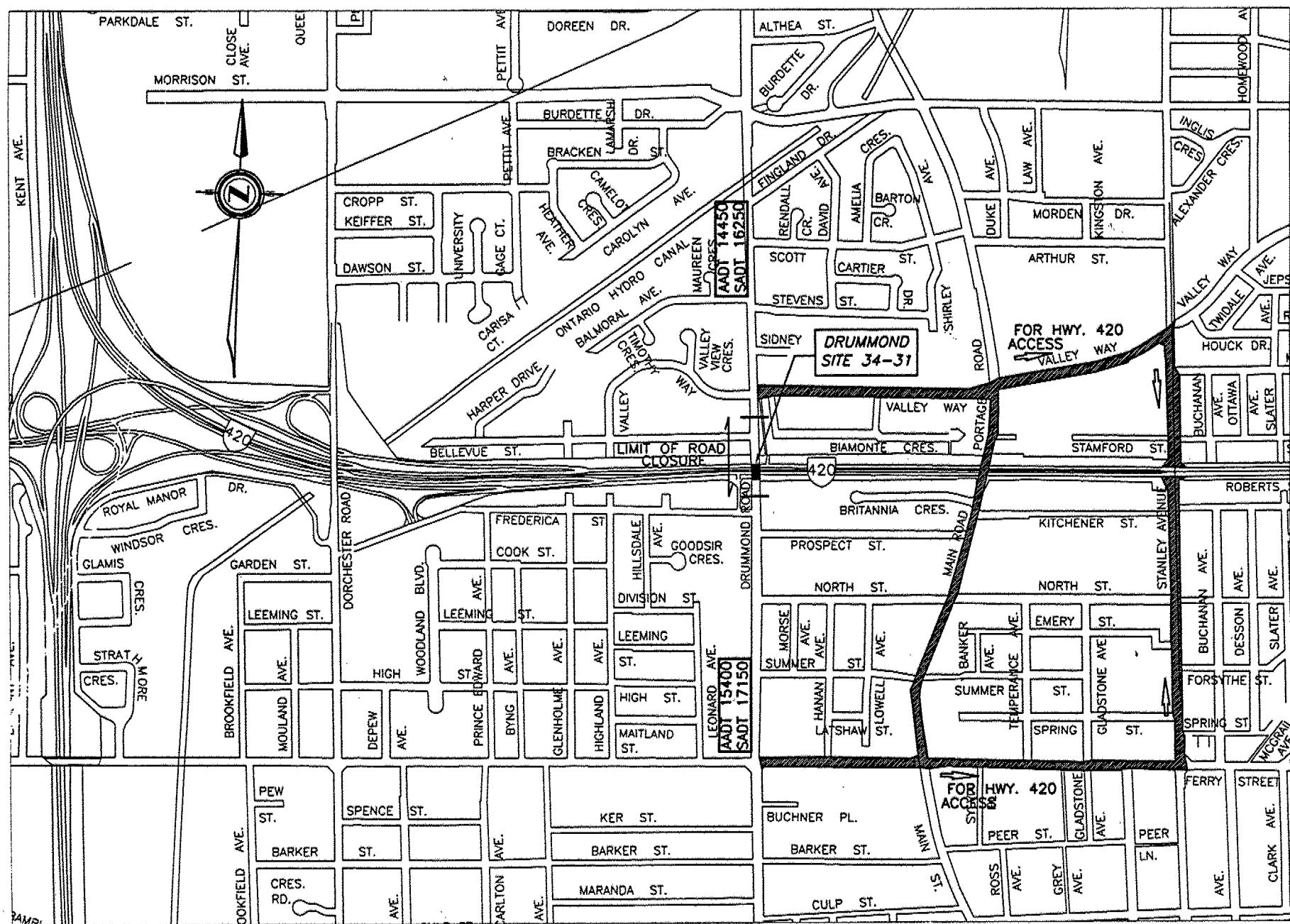
The matrix indicates the estimates are within 3%, therefore the selection will be based on design criteria and speed of construction. The precast girders alternative provides construction speed as the major advantage. As the construction time frame is minimal, the rigid frame alternative will be deleted from further considerations.

The proposed bridge type selected for the replacement of the structure is alternative two (2), the concrete box beam. This structure provides the best vertical alignment and offers an economical design that can allow construction to be carried out with minimal disruption to traffic and affords the opportunity to add the architectural treatment, adding to the character of this tourist area.

## **5.0 TRAFFIC STAGING**

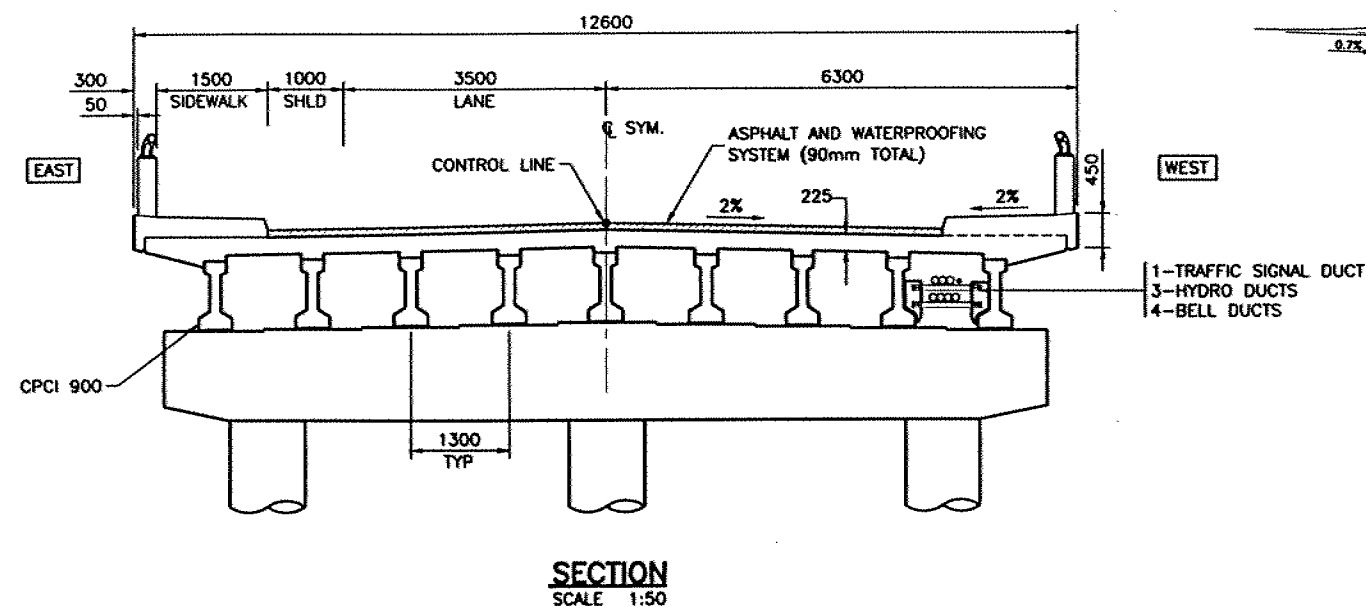
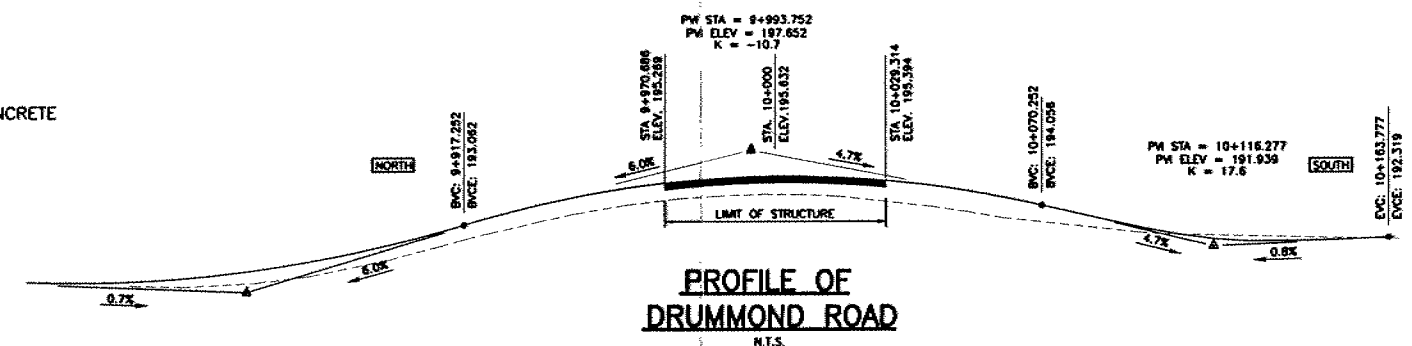
The structural construction will be staged in association with the roadwork. Traffic on highway 420 will be restricted to 1 lane in each direction therefore maximizing the vertical clearance available for the traffic on Highway 420. Restriction of traffic to one lane has been addressed in the preliminary design report prepared for the reconstruction of Highway 420.

Shifting of lanes may be required for the removal and placing of the footings for the structure. As ample room is available, this will not pose a problem. Traffic barriers will be installed to separate traveled areas. Traffic on Drummond Road will be rerouted as shown on the figure presented on the following page. The detour route plan has been developed with the City of Niagara Falls and provides an acceptable route during non-peak tourist seasons.



DETOUR PLAN

## **APPENDICES**

REVISIONS					
DATE	BY				
DESIGN: D.M.	CHK:	CODE	LOAD	DATE: SEP, 1998	
DRAWN: CNB	CHK: D.M.	SITE 34-31	STRUCT	SCHEME	DWG: A1






REVISIONS					
DATE	BY				
DESIGN: D.M.	CHK:	CODE	LOAD	DATE: SEP. 1999	
DRAWN: CNB	CHK: D.M.	SITE 34-31	STRUCT	SCHEME	DWG: A2



### GENERAL ARRANGEMENT

DRUMMOND ROAD UNDERPASS - HWY. 420

SHEET

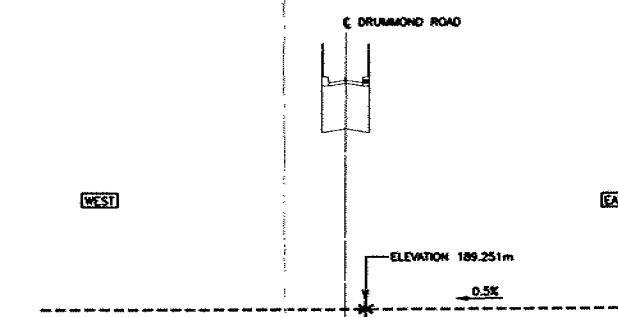
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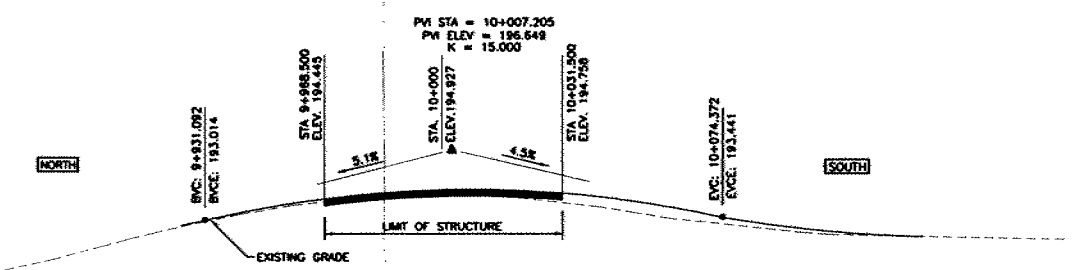
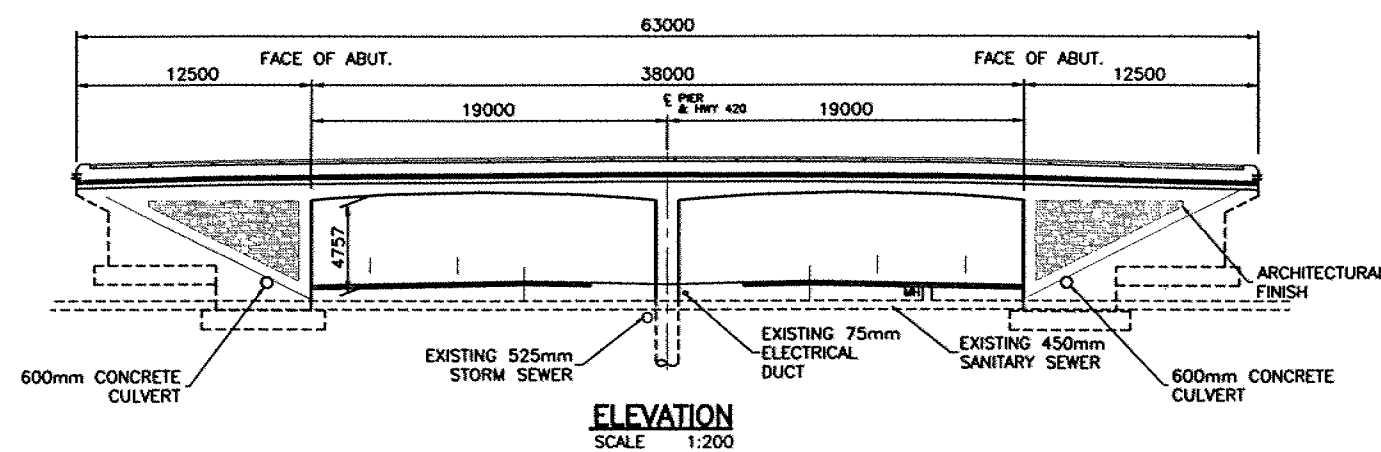
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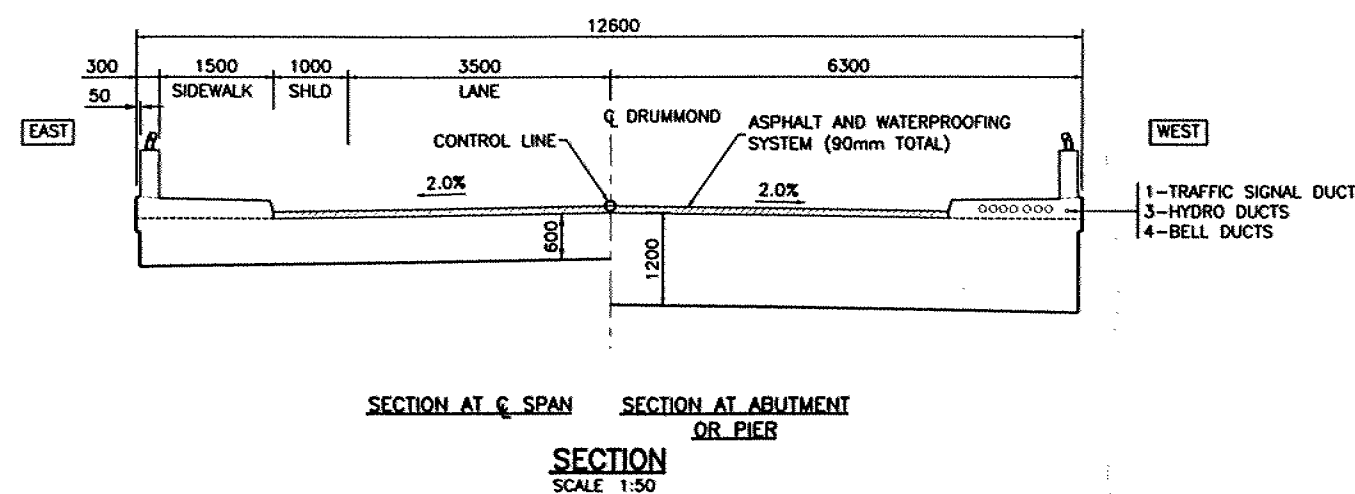
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# PROFILE OF HIGHWAY 420 N.T.S.



PROFILE OF  
DRUMMOND ROAD  
N.T.S.

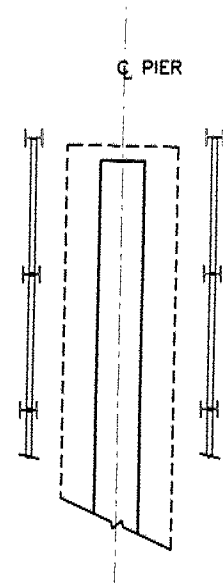
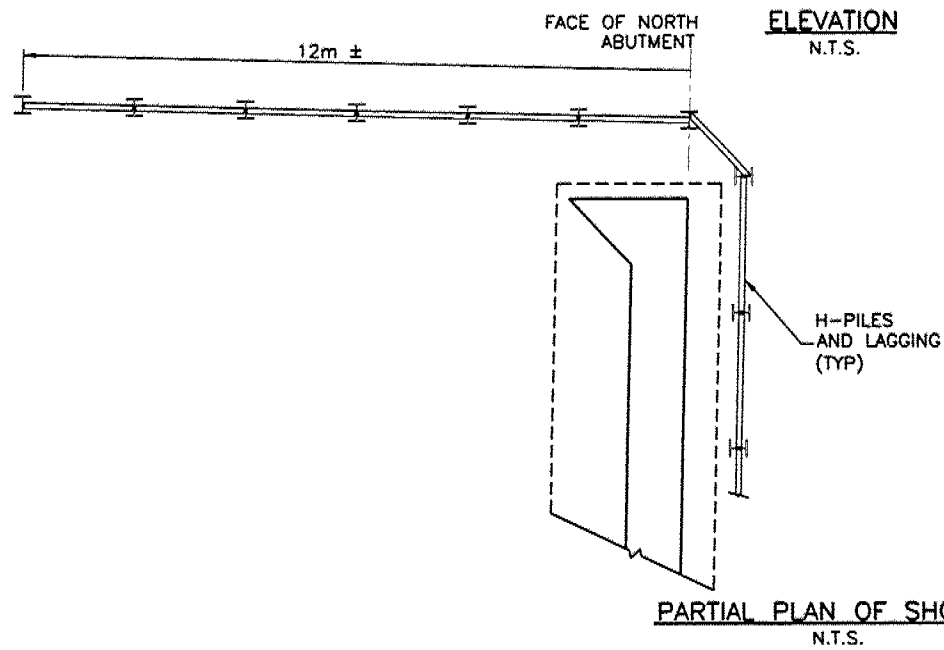
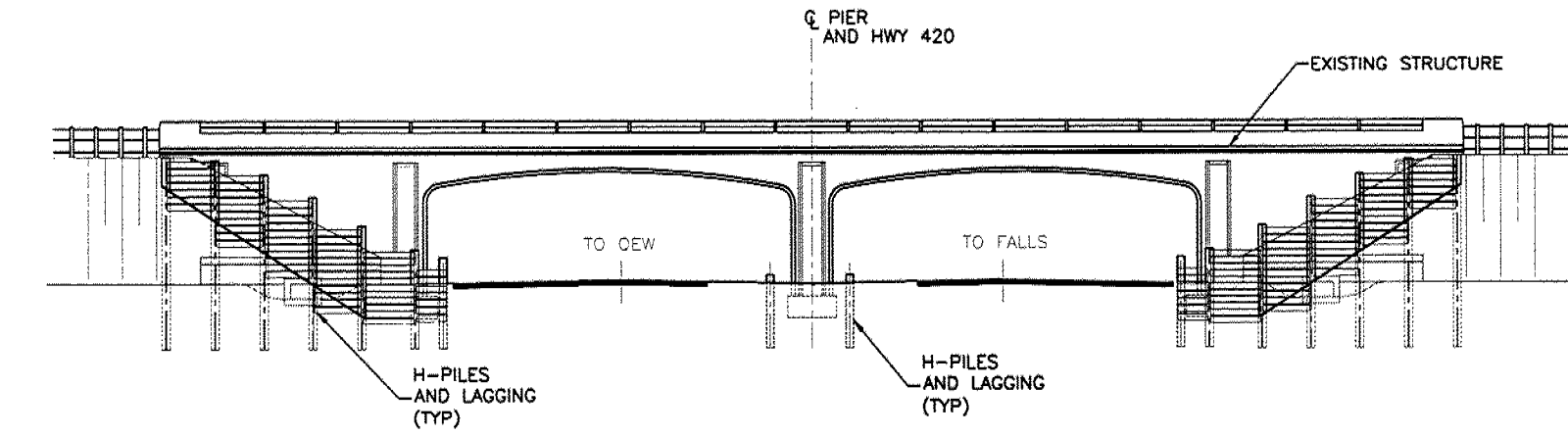


REVISIONS					
DATE	BY				
DESIGN: D.M.	CHK:	CODE	LOAD	DATE: SEP. 1998	
DRAWN: CHB	CHK: D.M.	SITE 34-31	STRUCT	SCHEME	DWG: A3

## Drummond Road Underpass Replacement

### *Preliminary Estimates*

	Alternative 1 <i>CPCI 900 Girders</i>				Alternative 2 <i>Box Girders B1-1220</i>				Alternative 3 <i>Rigid Frame</i>			
Description	Quantity	Units	Unit Price	Costs	Quantity	Units	Unit Price	Costs	Quantity	Units	Unit Price	Costs
Removal of Existing Structure	600	cu.m.	600	360,000	600	cu.m.	600	360,000	600	cu.m.	600	360,000
New Structure	513	sq.m.	1,100	564,102	513	sq.m.	1,100	564,102	479	sq.m.	1,200	574,560
Re-work of Drummond Road		l.s.		140,000		l.s.		125,000				65,000
Architectural Finishes	200	sq. m.	50	10,000	200	sq. m.	50	10,000	200	sq.m.	125	25,000
Traffic Stages		l.s.		30,000		l.s.		30,000		l.s.		40,000
Traffic staging	500,000	km	0.05	25,000	500,000	km	0.05	25,000	750,000	km	0.05	37,500
Rounding			-	102			-	102			-	60
<b>Total</b>				<b>1,129,000</b>				<b>1,114,000</b>				<b>1,102,000</b>



# ROADWAY PROTECTION