

**PRELIMINARY FOUNDATION
INVESTIGATION AND DESIGN REPORT
LUNDY'S LANE I/C UNDERPASS
QUEEN ELIZABETH WAY, NIAGARA FALLS
ONTARIO
G.W.P. 2482-04-00**

Geocres Number: 30M3-261

Report to

GENIVAR Ltd.

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation at the Lundy's Lane Underpass structure on the existing four-lane Queen Elizabeth Way (QEW) in Niagara Falls, Ontario.

The existing Lundy's Lane bridge spanning the QEW is proposed to be replaced. The replacement will include lengthening the spans over the QEW to accommodate future widening of the QEW.

A previous foundation investigation (GEOCREs number of 30M3-91) was carried out by MTO in January 1962 consisting of two boreholes drilled at the SW and NE quadrants within the QEW right of way. These boreholes were used to supplement the data collected in the current investigation. Data from the previous MTO investigation can be found in Appendix E.

The purpose of the investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A model of the subsurface conditions was developed through considering the data obtained in the course of the present investigation and that from GEOCREs 30M3-91.

Thurber carried out the investigation as a sub-consultant to GENIVAR under the Ministry of Transportation Ontario (MTO) Agreement Number 2006-E-0014.

2 SITE DESCRIPTION

The site is located where Lundy's Lane crosses the QEW in the City of Niagara Falls.

The existing Lundy's Lane underpass is a two-span structure approximately 30 m long and carries four lanes of traffic. The approach embankments for the existing structure are approximately 3 m high above original ground level and the QEW is in a shallow cut.

Geographically, the site is located in the area south of the Niagara Escarpment. The general site area is located within the eastern portion of the physiographic region known as the Haldimand Clay Plain, characterized by deep water glacio-lacustrine silts and clays. The local topography is a level plain, typical of lake sediments.

The site is located in an urban area and the surrounding land use consists of commercial properties in each of the four quadrants with residential areas beyond. The Niagara Power Canal runs parallel to the QEW and passes under Lundy's Lane approximately 280 m east of the site.

3 SITE INVESTIGATION AND FIELD TESTING

3.1 General

The site investigation and field testing for this project were carried out on March 29, 2010. The site investigation consisted of drilling and sampling a total of four boreholes to depths ranging from 7.9 m to 11.0 m. The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix D.

3.2 Drilling and Sampling

Solid stem auger drilling techniques were used to advance the boreholes and samples were obtained using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

Each of the boreholes was advanced to probe 3 m of soil with a SPT "N" value of at least 100 blows per 0.3m.

A member of Thurber's technical staff supervised the drilling and sampling operations on a full-time basis. The supervisor logged the boreholes and the recovered samples and processed the samples for transport to Thurber's Oakville office.

3.3 Piezometer Installation and Backfilling

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Two 19 mm diameter standpipe piezometers were installed in BH10-02 and

BH 10-03 to allow long term monitoring of groundwater levels. The screened portion of the standpipe piezometer was surrounded with sand prior to backfilling with bentonite holeplug to the ground surface. The remaining boreholes were backfilled using a mixture of bentonite holeplug and drill cuttings. The location and completion details of the boreholes and standpipe piezometers are shown in Table 3.1.

Table 3.1 – Borehole Details

Borehole Number	Piezometer Tip Details		Backfill
	Depth (m) / El. (m)	Stratum	
10-01	None installed	-	Borehole backfilled with holeplug to 5.8 m, drill cuttings to 0.15 m, then asphalt cold patch to surface.
10-02	10.8/188.4	Silt	Sand filter from 10.8 m to 8.4 m, holeplug to 0.15 m, then asphalt cold patch to surface.
10-03	10.7/188.9	Silt	Sand filter from 10.7 m to 7.8 m, holeplug to 0.15 m, then asphalt cold patch to surface.
10-04	None installed	-	Borehole backfilled with holeplug to 5.9 m, drill cuttings to 0.15 m, then asphalt cold patch to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

A number of samples were subjected to grain size distribution analyses (sieve and hydrometer) and Atterberg Limits testing where appropriate. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A for details of the encountered soil stratigraphy. A stratigraphic profile is presented on the Borehole

Locations and Soil Strata Drawing, Appendix D. An overall description of the stratigraphy is given in the following paragraphs; however, the factual data presented in the borehole logs governs any interpretation of the site conditions.

The soil stratigraphy encountered at the borehole locations typically consists of asphalt pavement of a variable thickness, overlying 2.4 to 3.0 m of sand fill underlain by a thin layer of clayey silt till followed by silt. More detailed descriptions of the individual strata are presented below.

5.2 Asphalt Pavement

Asphalt was encountered in all the boreholes ranging in thickness between 175 and 220 mm. The asphalt thickness may vary between the borehole locations and in other areas of the site.

5.3 Gravelly Sand (Fill)

A layer of gravelly sand fill, which is part of the pavement structure, underlies the asphalt in all boreholes. The gravelly sand fill contains trace to some silt and clay. This upper fill layer extended to depths ranging from 1.0 to 1.1 m below the pavement surface (Elevation 198.1 to 198.8).

A grain size analyses was conducted on one sample retrieved from the gravelly sand layer, the results of which are presented on the Record of Borehole sheets and Figure B1 of Appendix B. A summary of the results of the laboratory gradation test are as follows:

Gravel %	36
Sand %	47
Silt and Clay %	17

SPT N-values obtained in the gravelly sand fill ranged from 26 to 42 blows per 0.3 m of penetration, indicating a compact to dense relative density.

The moisture content of the gravelly sand fill samples ranged from 4 to 7%.

5.4 Sand (Fill)

A layer of sand fill, some silt to silty, trace gravel was encountered below the gravelly sand fill. The sand fill extended to depths ranging from 2.6 to 3.2 m below the pavement surface (Elevation 195.9 to 196.7).

Grain size analyses conducted on four samples retrieved from the sand fill are presented on the Record of Borehole sheets and Figure B2 of Appendix B. A summary of the results of

laboratory tests carried out on the sand fill samples are as follows:

Gravel%	0 to 2
Sand %	71 to 80
Silt and Clay %	19 to 29

SPT N-values obtained in the sand fill ranged from 12 to 22 blows per 0.3 m of penetration, indicating a compact relative density.

The moisture content of the sand fill samples ranged from 4 to 11%.

5.5 Clayey Silt Till

A thin layer of clayey silt till was encountered below the sand fill to depths of 3.2 to 4.1m (Elevation 195.5 to 196.0) in all of the boreholes except BH10-02. The thickness of the till layer varied from 0.6 to 1.1 m in the boreholes.

SPT N-values in the clayey silt till varied from 12 to 43 blows/0.3 m penetration. The N-values indicate that the consistency of the clayey silt till is stiff to hard.

Grain size distribution results for the clayey silt till are presented on the Record of Borehole sheets and Figure B3 of Appendix B. Atterberg Limits testing carried out on two samples is presented on the Record of Borehole sheets and Figure B6 of Appendix B. A summary of the results of laboratory tests carried out on samples of the clayey silt till were as follows:

Gravel %	0
Sand %	7 to 8
Silt %	68 to 69
Clay %	23 to 25
Liquid Limit %	21 to 23
Plastic Limit %	14 to 15

Moisture contents in the silty clay till deposit varied from 14 to 22%.

5.6 Silt

Below the fill and clayey silt till, all boreholes encountered a layer of silt and all boreholes were terminated in this layer at a depth of 7.9 to 11 m (Elevation 188.2 to 191.9). The soil is described as silt, trace to some sand, trace gravel, and trace clay.

SPT N-values in the silty clay till deposit were greater than 100 blows/0.3 m of penetration. The N-values indicate that the relative density of the silt is very dense.

Grain size distribution results for the silt are presented on the Record of Borehole sheets and Figures B4 and B5 of Appendix B. A summary of the results of laboratory tests carried out on samples of the silt were as follows:

Gravel %	0 to 6
Sand %	4 to 13
Silt %	82 to 90
Clay %	5 to 6

Moisture contents in the silty clay till deposit varied from 7 to 23%.

5.7 Water Levels

Following completion of drilling, the groundwater levels were observed in the open boreholes and a 19 mm diameter standpipe piezometer was installed in each of BH10-02 and BH10-03.

Groundwater was observed at a depth of 7.2 m in BH10-01 upon completion of drilling and the remainder of the boreholes were dry. The groundwater levels in the piezometers were measured approximately 1 month after completion of drilling.

The observed groundwater levels are shown in Table 5.1.

Table 5.1: Groundwater Levels

	BH 10-01		BH 10-02		BH 10-3	
Date	Depth (m)	Elev.	Depth (m)	Elev.	Depth (m)	Elev.
Mar 29/10	7.2*	191.9	-	-	-	-
Apr 26/10	-	-	9.4	189.8	9.9	189.7

*Water level in open borehole

It should be noted that these piezometric levels are based on short term observations and groundwater levels will be affected by seasonal fluctuations and severe weather events.

6 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the as-drilled locations, and provided northing and easting coordinates and ground surface elevations using a differential GPS.

Thurber also obtained all necessary highway occupancy permits prior to any drilling being carried out. Traffic control was provided by Niagara Traffic Services out of Niagara Falls, Ontario.

Elite Drilling Limited, a licensed well contractor in Ontario, of Fort Erie supplied and operated a truck-mounted CME 55 drill rig to conduct the drilling, sampling and in-situ testing operations at the borehole locations. Elite is a licensed well driller in Ontario.

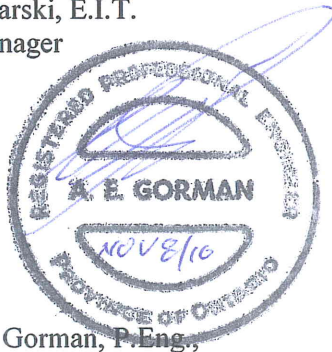
The drilling and sampling operations in the field were supervised on a full time basis by Ms. Eckie Siu of Thurber.

Laboratory testing was carried out by Thurber Engineering Ltd. in its MTO-approved Oakville laboratory.

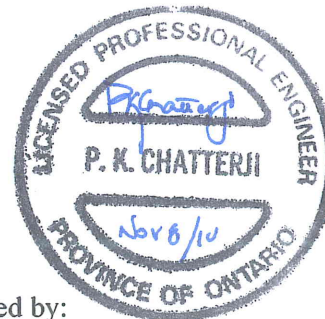
Interpretation of the field data and preparation of the investigation report was completed by Mr. Lukasz Gilarski, E.I.T. and Mr. Alastair E. Gorman, P. Eng. Overall supervision of the field program was performed by Mr. Alastair E. Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Lukasz Gilarski, E.I.T.
Project Manager



Alastair E. Gorman, P.Eng.,
Associate, Senior Foundation Engineer



Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents the interpretation of the geotechnical data in the factual report and presents preliminary geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

The project will consist of the replacement of the existing two-lane, 30 m long, two-span structure with a two-lane, two-span structure 51 m long to accommodate the widening of QEW.

At the site, the QEW runs north-south and Lundy's Lane runs east-west. The grade of QEW lies in a shallow cut, 1 to 3 m deep, relative to original ground surface. Based on the drawing provided for the project, the existing approach embankments for Lundy's Lane are approximately 2 to 3 m high and they will be raised by 2.0 to 2.5 m.

The replacement structure will overlap the footprint of the existing structure and construction will require either a closure of Lundy's Lane or staged replacement of the existing structure if traffic has to be maintained. Staged construction will require roadway protection on Lundy's Lane.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the investigation.

8 STRUCTURE FOUNDATIONS

8.1 Foundation Alternatives

Five foundation types have been considered:

- Spread footings on native soil
- Spread footings on engineered fill
- Drilled shafts (also referred to as caissons or bored piles)
- Steel H-Piles
- Steel pipe piles

These foundations alternatives are discussed below.

8.2 Spread Footings on Native Soil

From a geotechnical perspective, spread footings bearing on native soil are feasible at the site.

Spread footings may be designed on the basis of the geotechnical resistances and founding elevations given in Table 8.1. The geotechnical resistances apply at or below the stated elevations.

Table 8.1 Spread Footing Design Parameters

Foundation Element	Elevation	SLS (kN)	ULS_f (kN)
West abutment	195.9	400	600
Pier	193.5	400	600
East abutment	195.5	400	600

The resistance values above are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The SLS resistances are based on settlements not exceeding 25 mm. Differential settlements between foundation elements are not expected to exceed 15 mm.

Initial calculations of the sliding resistance may be carried out using a value of 0.55 for the ultimate friction factor of concrete poured on native soil.

8.3 Spread Footings on Engineered Fill

The use of spread footings bearing on engineered fill pads is considered to be feasible provided that the engineered fill pad is founded on the undisturbed, very dense, native soil.

The engineered fill must be founded at elevations no higher than those given in Table 8.1. Lower elevations may be necessary to accommodate the minimum thickness of engineered fill.

Provided the engineered fill is constructed as described in this section of the report, footings may be designed on the basis of the following vertical, geotechnical resistances:

- 900 kPa at factored ULS
- 350 kPa at SLS

The engineered fill must consist of OPSS Granular "A" or Granular "B" Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content and generally conforming to the geometry illustrated in the attached Figure 1.

The resistance values above are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

For footings designed on the basis of the geotechnical resistance values given above, total settlement under a footing is not expected to exceed 25 mm. Differential settlements are not expected to exceed 15 mm between foundation elements.

The sliding resistance of mass concrete poured on a compacted Granular "A" pad may be computed on the basis of an ultimate friction factor of 0.7.

8.4 Drilled Shafts (Caissons or Bored Piles)

The foundations may also be supported on drilled shafts founded in the very dense, native soil. For preliminary design purposes, it must be assumed that caissons must be socketed to a depth equal to 3 caisson diameters below the following elevations:

West abutment	195.9
Pier	195.5
East abutment	195.5

Preliminary design of drilled shafts may be based on a concentric, vertical geotechnical resistance of 2,000 kPa at SLS and 3,000 kPa at factored ULS.

The caissons must be installed in accordance with OPSS 903 (November 2009).

As it is not considered practical to install caissons on a batter, lateral loads must be resisted by the socketed portion in the very dense, native soil.

8.5 Steel H-Piles

The soil conditions encountered at the site are considered to be suitable for the support of steel H-piles. The recommended minimum pile length is 5.0 m from the underside of the abutment stem to the pile tip, or from the finished grade in front of the abutment, whichever results in the lower founding elevation. The length of a driven H-pile at this site will be governed by the elevation of the underside of the abutment stem. The anticipated elevations at which driven piles will achieve refusal are shown in Table 8.2.

Table 8.2 – Pile Tip Elevations

Foundation	Pile Tip Elevation
West abutment	194.0
Pier	194.3
East abutment	194.5

If the final bridge design results in the driven pile tip being at least 5.0 m below the abutment stem or the finished grade, then driven piles may be utilized. It is unlikely that the 5.0 m length can be achieved and accordingly driven piles are not the first recommendation as the preferred alternative from a geotechnical point of view.

However, in the interests of developing an integral abutment design, H-piles could be used in conjunction with drilled sockets in the very dense soil as follows:

1. Prepare the site to accept the 3.0 m long CSPs under the abutment. This may require excavating a trench, or pre-drilling appropriate diameter holes to accept the CSPs.
2. For each pile, drill a suitable diameter borehole, e.g. 600 mm, to a depth of 2.0 m below the underside of the CSP or the following elevations, whichever is deeper:

Foundation	Pile Tip Elevation
West abutment	194.0
East abutment	194.5

3. Place the piles into the sockets
4. Grout the bottom 1.0 m using 30 MPa concrete.

5. Install the CSPs and backfill with sand in accordance with the typical integral abutment construction procedures.

Piles driven to refusal in the very dense soil or grouted in rock sockets at, or below the elevations given above may be designed on the basis of the following geotechnical resistances:

Pile Section	SLS	ULS _f
HP 310 X 110	1,600 kN	1,800 kN
HP 360 X 132	1,800 kN	2,000 kN

The structural resistance of the pile must be checked by the structural designer.

8.6 Steel Pipe Piles

The use of steel pipe piles at this site is not recommended due to the shallow depth to very dense soil. The length of pipe pile that could be driven below the abutment stem is not considered to be sufficient to develop resistance.

8.7 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

8.8 Abutment Considerations

The soil conditions at this site are considered to be suitable for semi-integral or conventional abutments. The conditions are also suitable for integral abutment design provide the pre-drilling steps described above are implemented.

The preliminary GA drawing shows a false integral abutment design and this is considered to be acceptable from a geotechnical standpoint.

8.9 Recommended Foundation

The preliminary GA for the bridge shows an RSS false abutment. From a geotechnical standpoint, the following foundations are recommended:

East Abutment	H-piles in drilled sockets
Pier	A spread footing founded on the native soil A drilled shaft can be used as this will eliminate roadway protection requirements in the QEW median.

West abutment	H-piles in drilled sockets
---------------	----------------------------

A comparison of foundation alternatives based on advantages and disadvantages of each foundation alternative is included in Table C1, Appendix C.

9 APPROACH EMBANKMENTS

9.1 Stability

The existing Lundy's Lane approach embankments are composed of dense gravelly sand fill over compact sand fill that in turn overlies very dense native soil. These embankments appear to have performed satisfactorily and a grade raise, and associated embankment widening, constructed using similar materials can be assumed also to have satisfactory stability. The side slopes of embankment widening should be no steeper than 2H:1V.

9.2 Settlement

Considering that the nature of the soils encountered throughout the site, it is expected that the approach embankments will not experience significant long term settlements. Immediate settlements induced by the loading from the grade raise will be less than 25 mm and will be completed at the end of construction.

10 ROADWAY PROTECTION

The preliminary GA provided to Thurber indicates that the footprint of the proposed structure will overlap that of the existing structure. If traffic has to be maintained during construction and staged replacement of the structure is carried out, then it may be necessary to provide roadway protection to support the portion of the roadway remaining in service. Roadway protection must be provided in accordance with OPSS 539 (November, 2009).

This issue should be addressed during detail design after the GA and construction sequence have been finalized.

11 DEWATERING

On the basis of the preliminary investigation and in view of the low-permeability of the soils encountered on this site, dewatering is not expected to be required.

However, it must be anticipated that surface runoff could enter the excavations and steps must be taken to intercept surface runoff and near-surface seepage water.

All foundation excavations must be unwatered prior to placing concrete as required by OPSS 902 (November 2009).

12 INVESTIGATION DURING DETAIL DESIGN

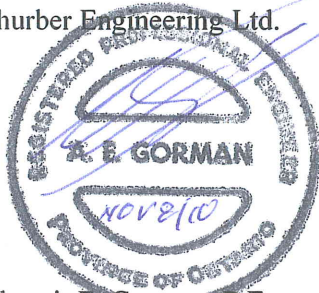
The requirements for foundation investigation during detail design must be determined after the location and GA of the bridge have been finalized. At that time, the existing pattern of boreholes should be superimposed on the GAs in order to determine the extent of additional investigation that may be required.

Typically, it is recommended that there be a minimum of two sampled boreholes at each foundation element for deep foundation design and a minimum of two for shallow foundation design.

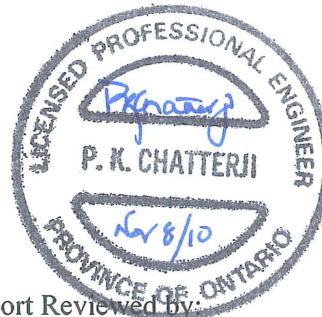
13 CLOSURE

Engineering analysis and preparation of this preliminary foundation design report was carried out by Mr. Alastair E. Gorman, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.



Alastair E. Gorman, P.Eng.,
Associate, Senior Project Engineer



Report Reviewed by:
P. K. Chatterji, P.Eng.,
Review Principal, Designated MTO Contact

Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE No 10-01

1 OF 1

METRIC

G.W.P. 2482-04-00 LOCATION N 4 772 220.9 E 335 777.5 ORIGINATED BY ES
 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY AN
 DATUM Geodetic DATE 2010.03.29 - 2010.03.29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
199.1													
0.0	ASPHALT: (175mm)												
0.2	Gravelly SAND, trace to some silt and clay Dense Brown Damp (FILL)												
198.1			1	SS	41								
1.0	SAND, trace gravel, trace to some silt and clay Dense to Compact Brown Damp (FILL)												
			2	SS	22								
196.5			3	SS	12								
2.6	Clayey SILT, trace sand Stiff Grey (TILL)												
196.0			4	SS	27								
3.2	SILT, trace gravel, trace sand, trace clay Very Dense Reddish Brown Damp												
			5	SS	103/ 0.300								
			6	SS	100/ 0.175								

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 10-02

2 OF 2

METRIC

G.W.P. 2482-04-00 LOCATION N 4 772 220.9 E 335 797.5 ORIGINATED BY ES
 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY AN
 DATUM Geodetic DATE 2010.03.29 - 2010.03.29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					W P W W L 20 40 60					
188.2	Continued From Previous Page SILT , trace sand, trace clay, trace gravel Very Dense Reddish Brown Damp		9	SS	105/		189										
11.0	END OF BOREHOLE AT 11.0m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Apr. 26,10 9.37 189.83				0.300												

RECORD OF BOREHOLE No 10-03

1 OF 2

METRIC

G.W.P. 2482-04-00 LOCATION N 4 772 204.3 E 335 851.9 ORIGINATED BY ES
 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY AN
 DATUM Geodetic DATE 2010.03.29 - 2010.03.29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	PLASTIC LIMIT w _P	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L		
199.6													
0.0	ASPHALT: (220mm)												
0.2	Gravelly SAND , trace to some silt and clay Dense Brown Damp (FILL)		1	SS	40			○					36 47 17 (SI+CL)
198.5								○					
1.1	SAND , trace gravel Dense to Compact Brown Damp (FILL)		2	SS	18			○					
			3	SS	20			○					0 71 29 (SI+CL)
196.6													
3.0	Clayey SILT , trace sand Very Stiff Reddish Brown Moist (TILL)		4	SS	26			○					0 8 69 23
195.5													
4.1	SILT , some sand, trace clay Very Dense Reddish Brown Damp		5	SS	110/ 0.225			○					
			6	SS	100/ 0.150			○					
			7	SS	100/ 0.275			○					0 13 82 5
	Trace sand		8	SS	105/ 0.300			○					

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
 20
 15
 10
 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 10-03

2 OF 2

METRIC

G.W.P. 2482-04-00 LOCATION N 4 772 204.3 E 335 851.9 ORIGINATED BY ES
HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY AN
DATUM Geodetic DATE 2010.03.29 - 2010.03.29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
188.6	SILT, some sand, trace clay Very Dense Reddish Brown Damp		9	SS	105/		189									0 4 90 6	
10.9	END OF BOREHOLE AT 10.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Apr. 26,10 9.93 189.67				0.275												

RECORD OF BOREHOLE No 10-04

1 OF 1

METRIC

G.W.P. 2482-04-00 LOCATION N 4 772 205.1 E 335 871.9 ORIGINATED BY ES
 HWY Q.E.W. BOREHOLE TYPE Solid Stem Auger COMPILED BY AN
 DATUM Geodetic DATE 2010.03.29 - 2010.03.29 CHECKED BY LPG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
199.9								<div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div>					<div><div>PLASTIC LIMIT</div><div>NATURAL MOISTURE CONTENT</div><div>LIQUID LIMIT</div><div>W_P W W_L</div></div>				
0.0	ASPHALT: (175mm)							20	40	60	80	100	20	40	60		
0.2	Gravelly SAND , trace to some silt and clay Compact Brown Damp (FILL)		1	SS	26		199										
198.8																	
1.1	SAND , trace gravel Compact Brown Damp (FILL)		2	SS	17		198										
	Occasional oxide staining		3	SS	19		197										1 80 19 (SI+CL)
196.7																	
3.2	Clayey SILT , trace sand Hard Reddish Brown (TILL)		4	SS	43		196										0 7 67 25
196.0																	
3.8	SILT , some sand, trace clay Very Dense Reddish Brown Damp		5	SS	100/ 0.175		195										
			6	SS	100/ 0.250		194										0 12 83 5
							193										
			7	SS	115/ 0.300		192										
191.9																	
7.9	END OF BOREHOLE AT 7.9m. BOREHOLE OPEN TO 7.5m AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH HOLEPLUG TO 5.9m, THEN CUTTINGS TO 0.17m, THEN CONCRETE TO SURFACE.																

ONTMT4S 7455.GPJ 10/8/17

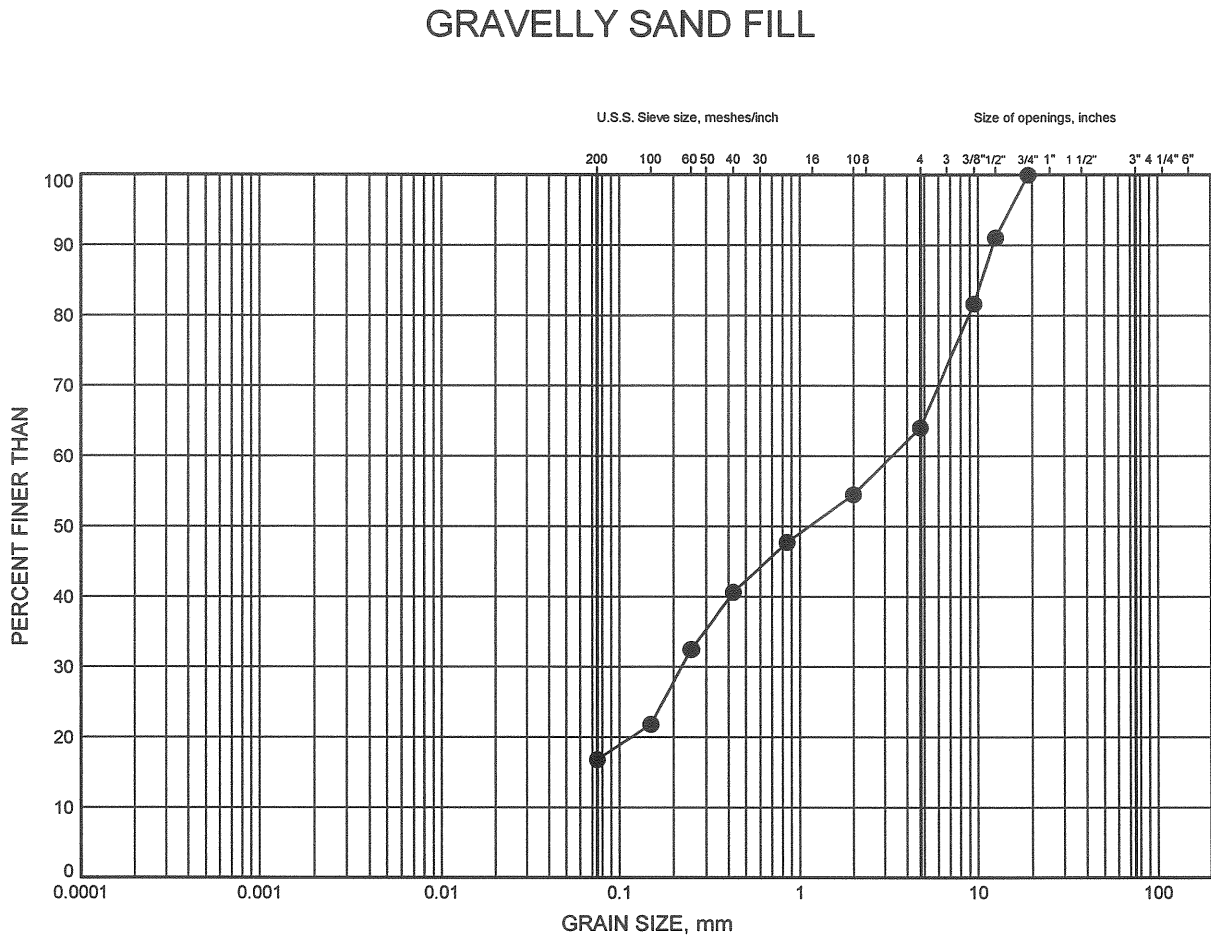
Appendix B

Laboratory Test Results

Lundy's Lane Underpass

GRAIN SIZE DISTRIBUTION

FIGURE B1



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-03	0.94	198.64

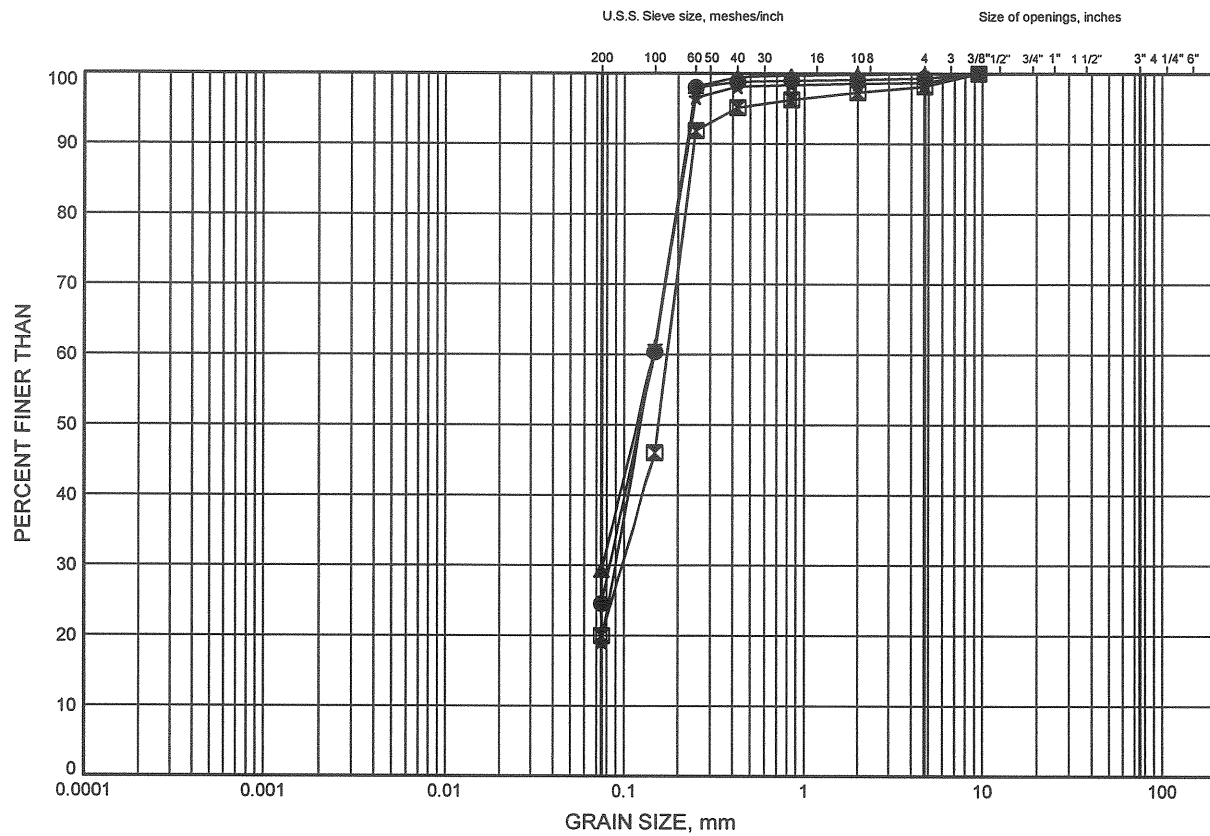


G.W.P.# 2482-04-00
Prepared By AN
Checked By LPG

Lundy's Lane Underpass GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-01	1.75	197.39
⊠	10-02	2.51	196.64
▲	10-03	2.51	197.07
★	10-04	2.51	197.34

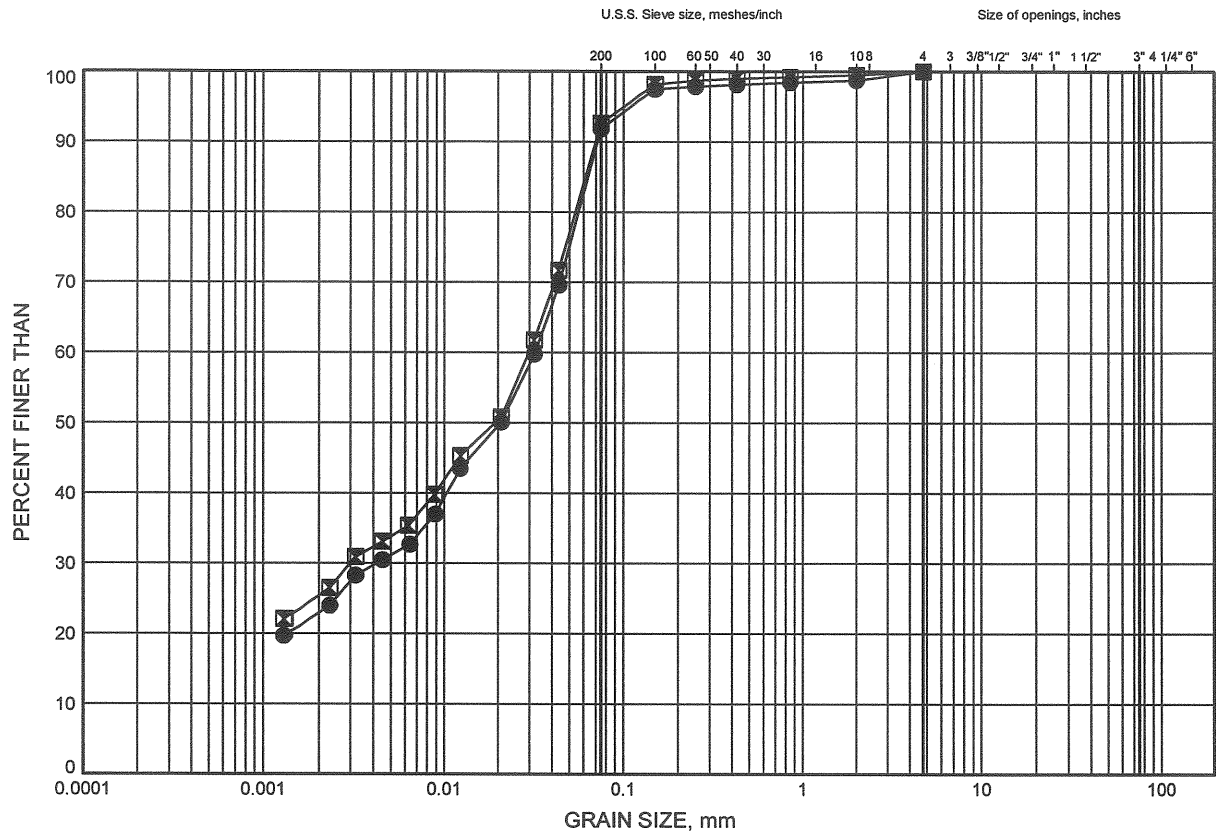


G.W.P.# 2482-04-00
Prepared By AN
Checked By LPG

Lundy's Lane Underpass GRAIN SIZE DISTRIBUTION

FIGURE B3

CLAYEY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

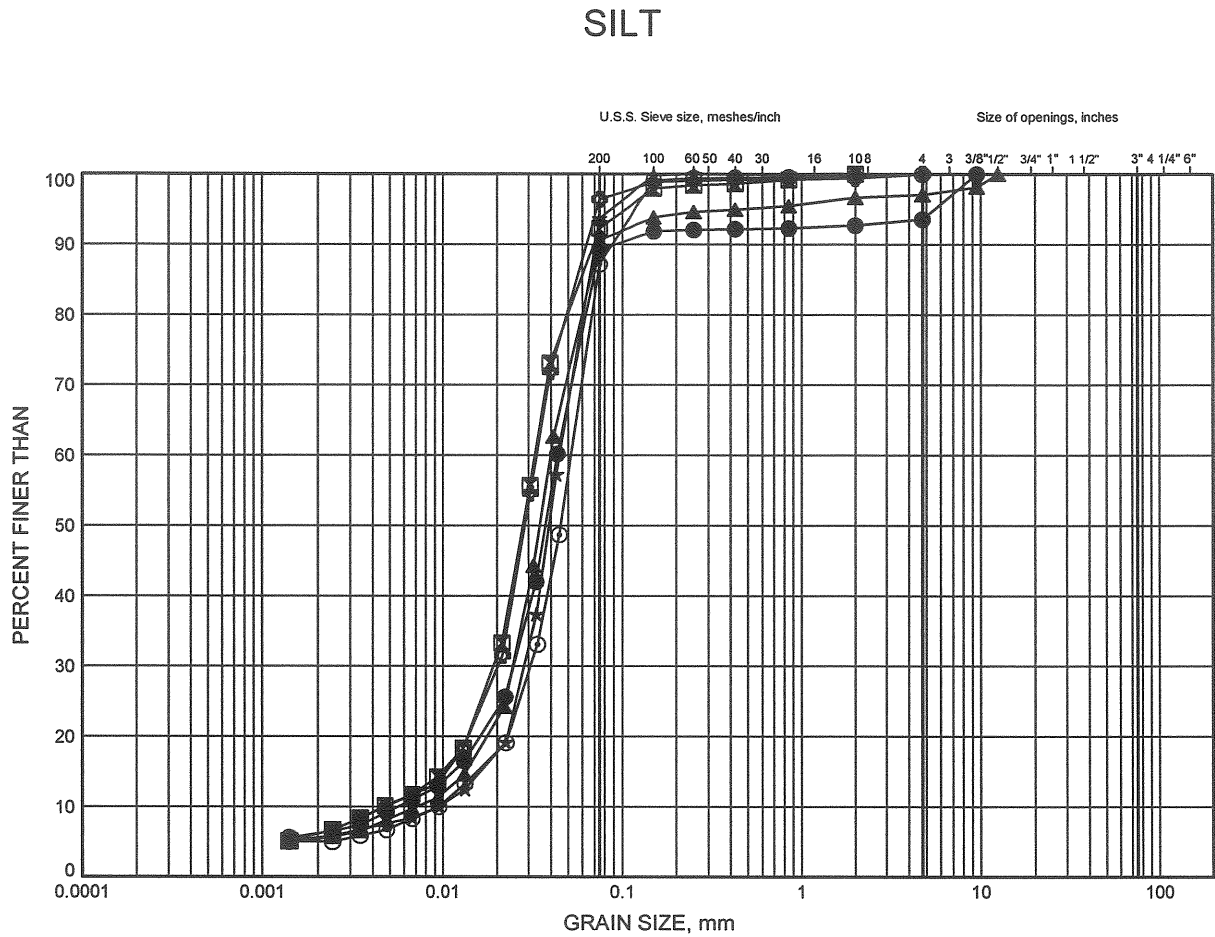
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-03	3.28	196.30
⊠	10-04	3.28	196.58



G.W.P.# 2482-04-00
Prepared By AN
Checked By LPG

Lundy's Lane Underpass GRAIN SIZE DISTRIBUTION

FIGURE B4



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

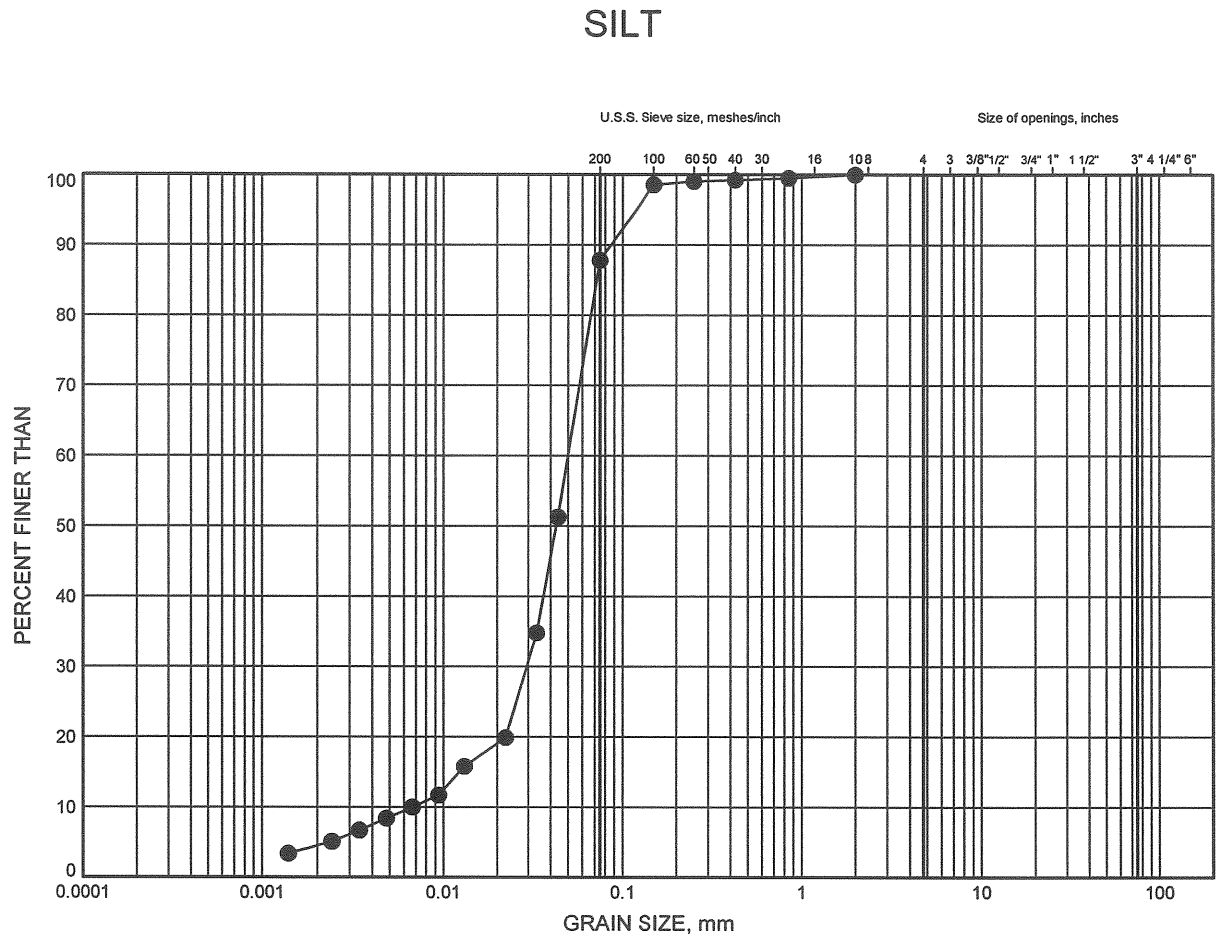
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-01	4.80	194.35
⊠	10-01	7.76	191.39
▲	10-02	4.60	194.55
★	10-02	9.27	189.88
⊙	10-03	7.76	191.82
⊕	10-03	10.81	188.77



G.W.P.# 2482-04-00.....
Prepared By AN.....
Checked By LPG.....

Lundy's Lane Underpass GRAIN SIZE DISTRIBUTION

FIGURE B5



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	10-04	6.22	193.63

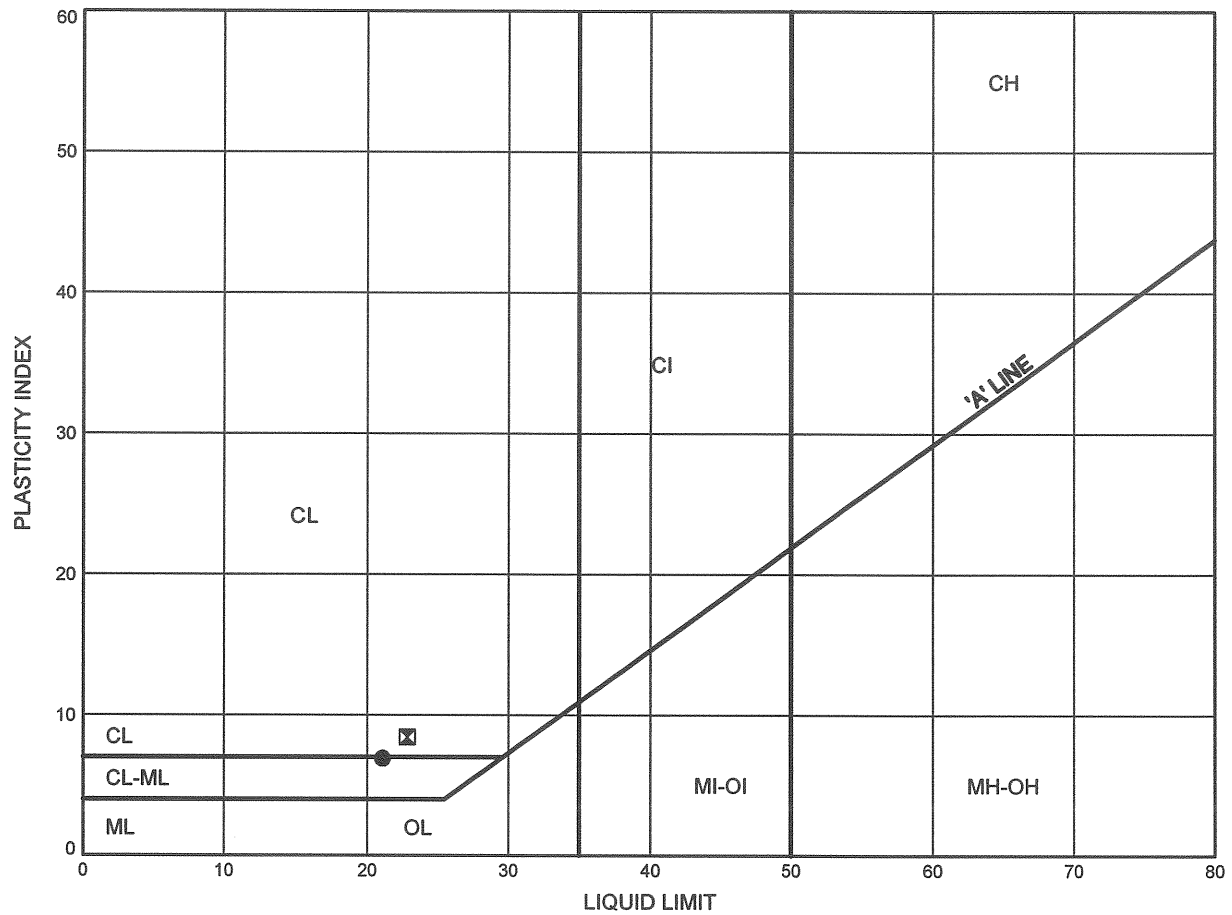


G.W.P.# 2482-04-00
Prepared By AN
Checked By LPG

Lundy's Lane Underpass
ATTERBERG LIMITS TEST RESULTS

FIGURE B6

CLAYEY SILT TILL



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	10-03	3.28	196.30
⊠	10-04	3.28	196.58

Date August 2010

Project 2482-04-00



Prep'd AN

Chkd. LPG

Appendix C

Foundation Comparison

TABLE C1 - COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation	Spread Footing	Caissons	Driven H-Piles	H-Piles in Sockets	Pipe Piles
West Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> • Feasible bearing capacity on undisturbed native soil or engineered fill • High bearing capacity on very dense native soil • Relatively straightforward installation • Least costly <p>Disadvantages:</p> <ul style="list-style-type: none"> • Lower bearing resistance 	<p>Advantages:</p> <ul style="list-style-type: none"> • High bearing capacity in very dense soil • Reduces requirements for roadway protection <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles driven into the very dense soil • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles grouted into sockets in the very dense soil • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings • Additional costs incurred through drilling 	<i>Not applicable at this site.</i>
Pier	<p>Advantages:</p> <ul style="list-style-type: none"> • Feasible bearing capacity on undisturbed native soil or engineered fill • High bearing capacity on very dense native soil • Relatively straightforward installation • Least costly <p>Disadvantages:</p> <ul style="list-style-type: none"> • Lower bearing resistance • Larger excavation in median • Requires roadway 	<p>Advantages:</p> <ul style="list-style-type: none"> • High bearing capacity in very dense soil • Reduces requirements for roadway protection <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings 	<i>Not applicable at the pier.</i>	<i>Not applicable at the pier.</i>	<i>Not applicable at this site.</i>

Lundy's Lane Underpass
Niagara Falls, Ontario






	protection				
East Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> • Feasible bearing capacity on undisturbed native soil or engineered fill • High bearing capacity on very dense native soil • Relatively straightforward installation • Least costly <p>Disadvantages:</p> <ul style="list-style-type: none"> • Lower bearing resistance 	<p>Advantages:</p> <ul style="list-style-type: none"> • High bearing capacity in very dense soil • Reduces requirements for roadway protection <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles driven into the very dense soil • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings 	<p>Advantages:</p> <ul style="list-style-type: none"> • High capacity for piles grouted into sockets in the very dense soil • Relatively straightforward installation <p>Disadvantages:</p> <ul style="list-style-type: none"> • Higher cost than spread footings <p>Additional costs incurred through drilling</p>	<i>Not applicable at this site.</i>

Appendix D

Borehole Locations and Soil Strata Drawing



LEGEND

	Borehole
	Borehole from Previous Investigation
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

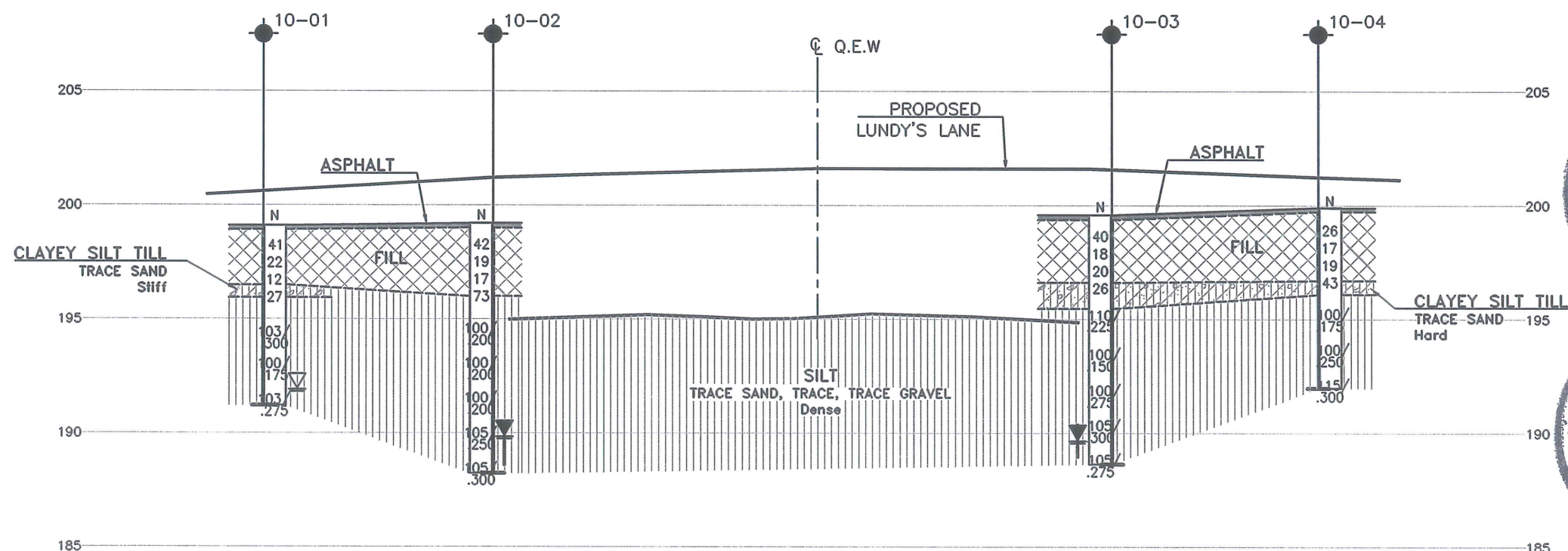
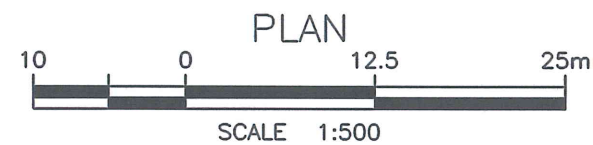
NO	ELEVATION	NORTHING	EASTING
10-01	199.1	4 772 220.9	335 777.5
10-02	199.2	4 772 220.9	335 797.5
10-03	199.6	4 772 204.3	335 851.9
10-04	199.9	4 772 205.1	335 871.9

-NOTES-

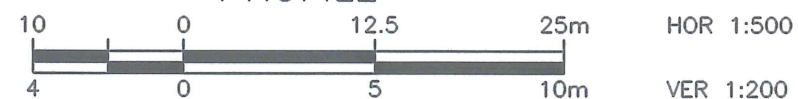
- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 30M3-261

REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	LPG	CHK	AEG	CODE	LOAD		DATE AUG. 2010		
DRAWN	AN	CHK	PKC	SITE	STRUCT				



PROFILE



Appendix E

Previous Investigation

Contract # 23-64-16.

Mr. A. M. Toye,
Bridge Engineer.
Materials & Research Division,
(Foundation Section)

March 7, 1962.

D.H.O. FOUNDATION INVESTIGATION
REPORT.
W.J. 62-F-2 -- W.P. 76-61.

Attention: Mr. S. McCombie.

Re: Widening of Structure at Lundy's Lane
(Hwy. 20 & 3A) and Q.E.W. Interchange,
Niagara Falls, Township of Stamford,
District #4.

Attached, we are forwarding to you, our detailed report on the subsoil conditions existing at the above structure site.

We believe the factual data and recommendations contained therein, should prove adequate for your future design work. If further assistance is required in connection with this project, please do not hesitate to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. A. M. Toye (2)
H. A. Tregaskes
H. D. McMillan
I. C. Campbell
J. C. Thatcher
T. J. Kovich
J. Roy
J. E. Gruspier
E. R. Saint
F. Norman
A. Watt
Foundations Office ✓
Gen. Files.

A. G. Stermac
A. G. Stermac,
PRINCIPAL FOUNDATION ENGINEER

TABLE OF CONTENTS

1. INTRODUCTION
 2. DESCRIPTION OF SITE
 3. DESCRIPTION OF FIELD AND LABORATORY WORK
 4. SUBSOIL CONDITIONS:
 - 4.1) General
 - 4.2) Silt
 5. GROUND WATER CONDITIONS
 6. DISCUSSION AND RECOMMENDATIONS
 7. SUMMARY
 8. MISCELLANEOUS
-

FOUNDATION INVESTIGATION

For

Widening of Structure at Lundy's Lane
(Hwys. 20 & 3A) and Q.E.W. Interchange,
Niagara Falls, Township of Stamford,
District #4.
W.J. 62-F-2 -- W.P. 76-61.

1. INTRODUCTION:

It is proposed to widen the existing underpass which takes Hwys. 3A & 20 (Lundy's Lane) over Queen Elizabeth Highway near Niagara Falls.

A foundation investigation was carried out by this Section to determine the subsoil conditions at the above-mentioned site where 20 ft. widenings on both sides of the existing structure are contemplated. Presented in this report, are the results of this investigation, together with the recommendations pertaining to the foundations of the new structure.

2. DESCRIPTION OF SITE:

The existing bridge is a 110-ft. long two span concrete structure built in 1938 for two lane traffic. It was observed during the time of investigation, that the present condition of the existing structure is satisfactory.

Public utility cables and pipes are located underground near Lundy's Lane and Q.E.W. in close proximity of the structure. At the time of this investigation, the ground on either side of Q.E.W. was covered with snow.

Geographically, the site is located in the area South of the Niagara Escarpment.

cont'd. /2 ...

3. DESCRIPTION OF FIELD AND LABORATORY WORK:

Field work consisted of two sampled boreholes. Dynamic cone penetrations were tried, but it was not possible to go deeper than four feet because of the high relative density of the material.

Conventional wash boring procedure was followed. Samples were recovered at depths required by means of a 2-inch O.D. split spoon sampler. The dimension of the spoon sampler and the energy used in driving it, conform to the requirements of the Standard Penetration Test. Samples were visually examined and identified in the field. Upon receipt in the laboratory, moisture content and grain size distribution curves of typical samples were determined. Laboratory and field test results have been summarized and are given in Appendix I. Elevations of the boreholes are taken from the given Plan No. D-2653-1. The location plan, the dimensions for the proposed widening and the subsoil profile, are shown on Plan 62-F-2A.

4. SUBSOIL CONDITIONS:

4.1) General:

The stratification of the subsoil is generally uniform. The site is covered with a thin layer of topsoil and sand which is underlain by a dense silt layer of undetermined thickness.

cont'd. /3 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Silt:

This deposit was encountered in both borings below the topsoil at approx. elevation 639, and established to elev. 613.5. Traces of fine sand were observed throughout this stratum. In boring 2, traces of fine gravel were observed between elevation 620 and 628. Grain size distribution curves obtained from this deposit, show that it contains 96% silt and 4% sand and gravel. Standard Penetration Tests carried out in this stratum gave values of 'N' in excess of 100 blows/foot. Based on these results, this material may be described as very dense. The average moisture content is 21%, and density 125 p.c.f.

5. GROUND WATER CONDITIONS:

No ground water was encountered in any of the borings during this investigation.

6. DISCUSSION AND RECOMMENDATIONS:

The existing structure is founded on spread footings in the dense silt stratum approx. elevation 636. Subsoil conditions encountered at the site are favourable to support the extended part of the footings for the proposed widening of the structure. A safe bearing capacity of 3 T.S.F. can be used for footing design. Footings can be founded in the dense silt stratum as high as the frost protection requirement will allow.

cont'd. /4 ...

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS 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 / FT.</u>	<u>c LB. / SQ. FT.</u>	<u>DENSENESS</u>	<u>'N' BLOWS / FT.</u>
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

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
	P.M.		SAMPLE ADVANCED MANUALLY
	P.H.		SAMPLE ADVANCED HYDRAULICALLY

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	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)
γ_s	UNIT WEIGHT OF SOLID PARTICLES
γ_w	UNIT WEIGHT OF WATER
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)
γ'	UNIT WEIGHT OF SUBMERGED SOIL
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$
e	VOID RATIO
n	POROSITY
w	WATER CONTENT
S_r	DEGREE OF SATURATION
w_L	LIQUID LIMIT
w_P	PLASTIC LIMIT
I_P	PLASTICITY INDEX
s	SHRINKAGE LIMIT
I_L	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
e_{max}	VOID RATIO IN LOOSEST STATE
e_{min}	VOID RATIO IN DENSEST STATE
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
	RELATIVE DENSITY D_r IS ALSO USED
h	HYDRAULIC HEAD OR POTENTIAL
q	RATE OF DISCHARGE
v	VELOCITY OF FLOW
i	HYDRAULIC GRADIENT
k	COEFFICIENT OF PERMEABILITY
j	SEEPAGE FORCE PER UNIT VOLUME
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$
c_v	COEFFICIENT OF CONSOLIDATION
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)
U	DEGREE OF CONSOLIDATION
τ_f	SHEAR STRENGTH
c'	EFFECTIVE COHESION INTERCEPT
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION
c_u	APPARENT COHESION
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION
μ	COEFFICIENT OF FRICTION
S_t	SENSITIVITY

GENERAL

π	$= 3.1416$
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
$\bar{\sigma}$	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ϵ	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNG'S MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

FOUNDATION SECTION

CHECKED BY G.M.

SOIL PROFILE			SAMPLES			DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W		BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F.	WATER CONTENT %			
							20 40 60 80 100	20 40 60			
640.0	Groundlevel					640.0					
639.5	Top Soil										
0.5			1	S.S.	>100						
			2	S.S.	>100						
			3	S.S.	>100						
	Silt Trace of fine sand v. dense Br. red.		4	S.S.	>100						
			5	S.S.	>100						
5			6	S.S.	>100						
55.5	End of borehole.					610.0					

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 62-E-2

LOCATION

ORIGINATED BY B.M.G.

W. P. 76-61

BORING DATE Jan. 19/62.

COMPILED BY H.S.

DATUM 640.0

BOREHOLE TYPE Washboring, BX Casing

CHECKED BY G.M.

SOIL PROFILE		SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT	LIQUID LIMIT ——— WL PLASTIC LIMIT ——— WP WATER CONTENT ——— W	BULK DENSITY P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER TYPE					
640.0	Groundlevel							
639.2	Top Soil							
0.8								
			1 S.S. >100					
			2 S.S. >100					
			3 S.S. >100	630.0				119.0
	Silt with trace of fine sand and gravel.		4 S.S. >100					
			5 S.S. >100					
			6 S.S. >100	620.0				
614.0			7 S.S. >100					
610.0	End of borehole.							



ONTARIO
DEPARTMENT OF HIGHWAYS

Bridge Division

File 947

PM 170

Memo to Mr. A. G. Stermac Date June 22, 1962
Principal Foundation Eng.
Room 107, Lab. Bldg. Subject W.P. 76-61, Hwy. 20 Underpass
1 Mile West of Niagara Falls
From F. DeVisser West Limits, Q.E.W. Dist. 4

Enclosed is one print of our preliminary plan D 5055-1 for the Hwy. 20 underpass. If you have any comments please let us know.

F. DeVisser

FDeV/et

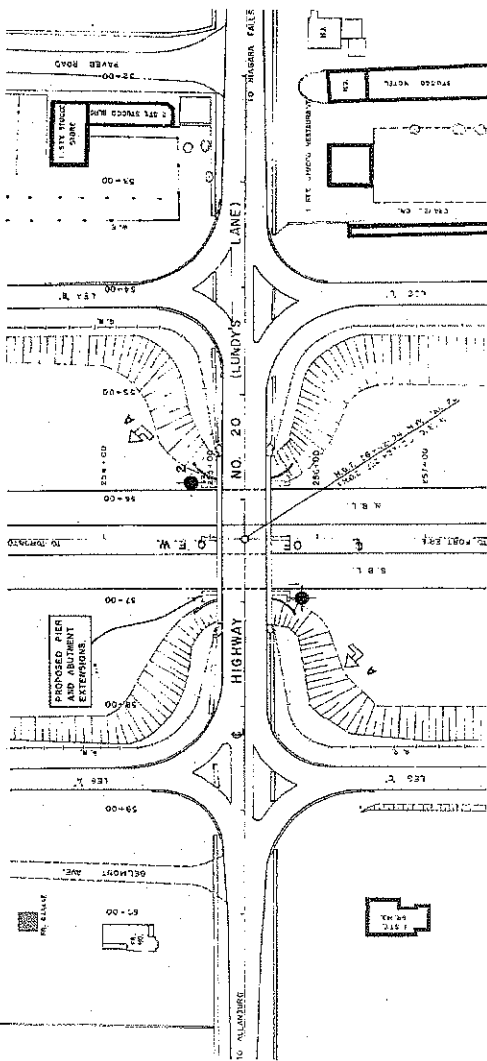
F. DeVisser,
Bridge Location Engineer.

F. DeVisser advised by phone that from the foundation point of view there are no comments.

Bridge drawing poor - no elevations

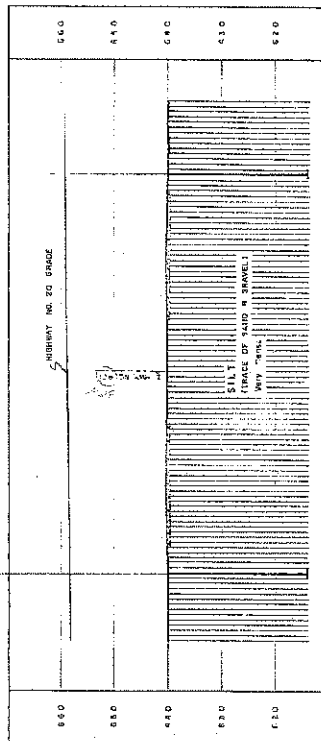
July 11, 1962.

A. G. Stermac



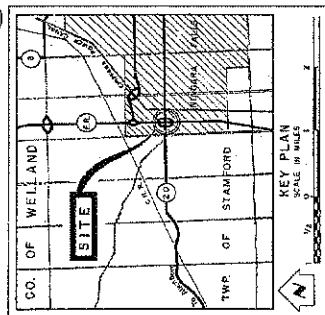
PLAN

SCALE IN FEET
0 25 50 100



A - A

SCALE IN FEET
0 25 50 100
VERTICAL
0 20 40 60
HORIZONTAL



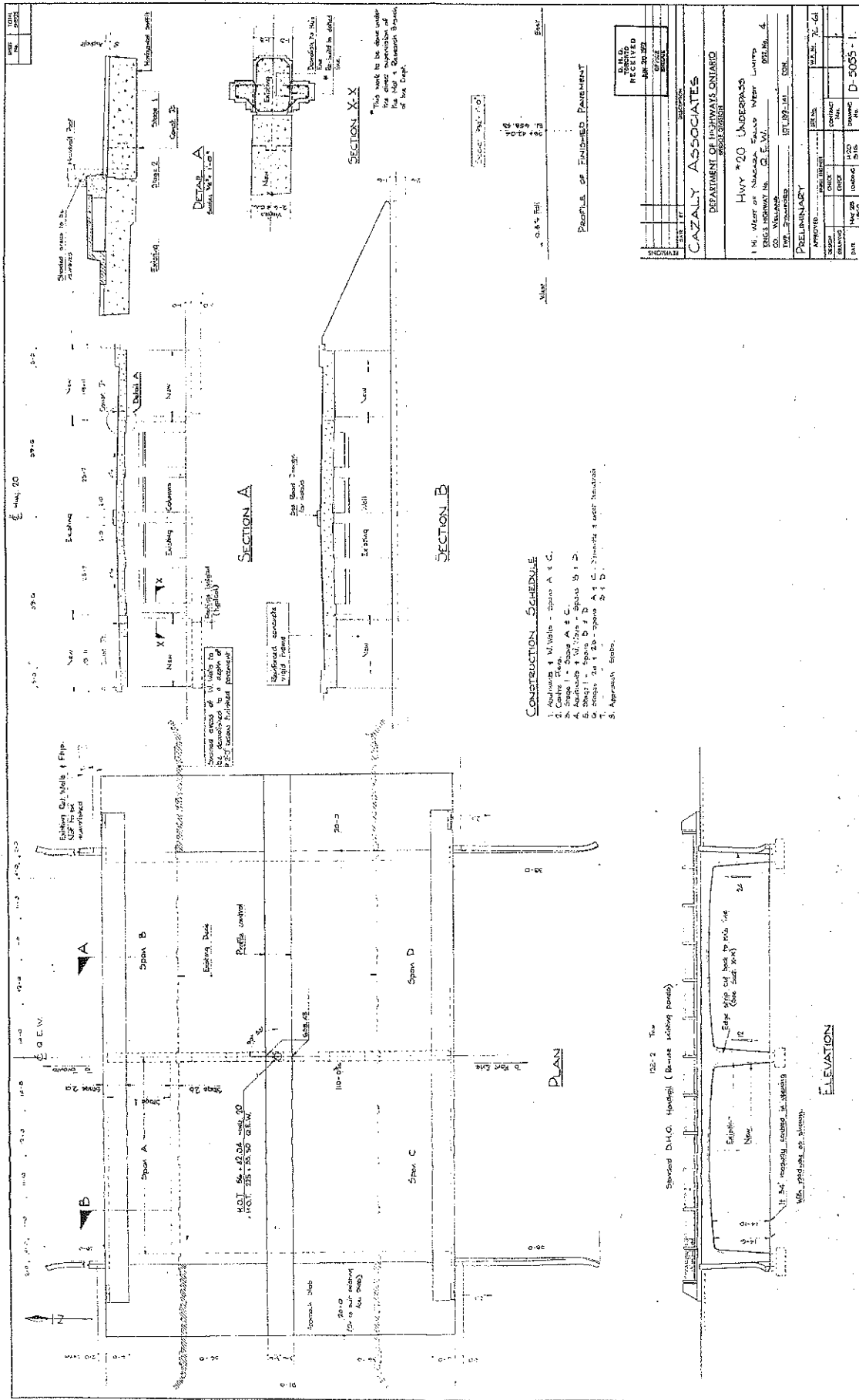
LEGEND			
	Bore Hole		
	Core Penetration Hole		
	Bore & Core Penetration Hole		
	Water Levels established at time of field investigation		

NOTE -
The boundaries between all grades have been established only at their hole location. Between bore holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH DIVISION - TORONTO REGION

HIGHWAY NO. 20 (LUNDY'S LANE)
AND
QUEEN ELIZABETH WAY

DESIGNED BY: G. H. G. H. G.	DISTRICT NO. 4	DATE: 12 FEB 1962
DRAWN BY: H. H. H. H. H.	PROJECT NO. 78-1-2	
CHECKED BY: H. H. H. H. H.	CONTRACT NO.	
APPROVED BY: H. H. H. H. H.	62-F-2A	



D.M.O.
RECEIVED
APR 20 1972
OFFICE
BRIDGE

DEPARTMENT OF HIGHWAYS ONTARIO
CAZALY ASSOCIATES
CONSULTING ENGINEERS

HWY #20 UNDERPASS
1 M. WEST OF NACON, TOWN OF WYND
PROJ. NO. 14. G.E.W.
CO. WILSON
PRELIMINARY
DATE: 12/19/71
BY: [Signature]
CHECKED: [Signature]
APPROVED: [Signature]
SCALE: 1/4" = 1'-0"

NO.	DESCRIPTION	QTY	UNIT	PRICE	TOTAL
1	CONCRETE	100	CU YD	10.00	1000.00
2	STEEL	100	LB	0.50	50.00
3	PAVEMENT	100	SQ YD	1.00	100.00
4	EMBANKMENT	100	CU YD	2.00	200.00
5	GRASS	100	SQ YD	0.10	10.00
6	LANDSCAPING	100	SQ YD	0.20	20.00
7	UTILITIES	100	FT	0.10	10.00
8	CONCRETE	100	CU YD	10.00	1000.00
9	STEEL	100	LB	0.50	50.00
10	PAVEMENT	100	SQ YD	1.00	100.00
11	EMBANKMENT	100	CU YD	2.00	200.00
12	GRASS	100	SQ YD	0.10	10.00
13	LANDSCAPING	100	SQ YD	0.20	20.00
14	UTILITIES	100	FT	0.10	10.00