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ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 62-82-01

DIST 9

HWY 417

STR SITE -

St. Laurent Blvd. Interchange
High Mast Lighting

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FOUNDATION INVESTIGATION REPORT

For

St. Laurent Blvd Interchange

High Mast Lighting

W.P. 62-82-01, Hwy, 417

District 9, Ottawa

INTRODUCTION

This report summarizes the factual information obtained from a foundation investigation carried out at the above-mentioned site between 85-06-12 and 85-06-17. The fieldwork consisted of advancing one borehole at each of the 13 high-mast lights location. The boreholes included sampling the overburden at 0.75 m intervals and coring the shale bedrock for depths of up to 3.0 m

SITE DESCRIPTION

The site is located from approximately 1 km west to 1 km east of the St. Laurent interchange along the south side of Hwy. 417. The site is located in the eastern end of Ottawa in the Regional Municipality of Ottawa-Carleton (RMOC). Land use in the vicinity of the site is predominantly developed as urban-commercial. Topography across the site is generally flat.

The site lies on a glacial till plain characterized by glacial till and silt/sand deposits. In addition, however, silty clay deposits were also identified. The underlying bedrock in the area consists of a dark-grey to black shale of the Billings Formation and is found between Elev. 55.3 and 68.7 along the 2 km section investigated.

SUBSURFACE CONDITIONS

General

The predominant material across the site consists of a heterogeneous mixture of clay, silt, sand and gravel. The deposit is generally cohesive and the fines can be described as a plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may, however, include random seams of non-cohesive waterbearing material. When excavated into, the non-cohesive materials may cave in if the sides of an augered excavation are not supported (lined).

Other non-cohesive materials were also encountered across the site. It should be noted that any non-cohesive soil which is below the prevailing groundwater level will experience cave-in and sloughing when excavated into.

Bedrock at this site consists of dark-grey to black shale of the Billings Formation. A description of the recovered core, prepared by MTC Geologist E. Magni, is included in the Appendix for reference.

The borehole logs for each of the thirteen boreholes (BH C-1 to BH C-13) along with a key plan are included in the Appendix.

The following is a brief description of the subsurface conditions encountered at each borehole.

BH C-1

Extending from the ground surface (Elev. 66.5) to a depth of 1.4 m is a non-cohesive deposit of sand. Based on visual observation traces of clay and silt are also evident. Based on a Standard Penetration Test 'N' value of 15 blows/0.3 m, this deposit is considered to be in a compact state. No laboratory tests were conducted on samples of this material.

Underlying this material and at a depth of 3.7 m below the ground surface is a non-cohesive deposit of silt, trace sand, clay. Grain size distribution tests were conducted on 2 samples of this material. The distribution curves are shown on Fig. 2 in the Appendix and the results can be summarized as follows:

	<u>Sample #2</u>	<u>Sample #3</u>
Gravel	0%	0%
Sand	5%	5%
Silt	89%	86%
Clay	6%	9%

The natural moisture content of this material is approximately 22%. Based on 'N' values ranging from 9 to 23 blows/0.3 m, this material is considered to be in a compact state.

Extending from a depth of 3.7 m to 7.9 m is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally, this deposit is cohesive and the fines can be described as a plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig 1 in the Appendix shown a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across this site.

Based on 'N' values ranging from 15 to over 100 blows/0.3 m, this material is considered to have a consistency of very stiff to hard.

Grain size distribution tests were conducted on 2 samples of this material. The distribution curves are shown on Fig. 2 in the Appendix and the results can be summarized as follows:

	<u>Sample #5</u>	<u>Sample #8</u>
Gravel	12%	32%
Sand	44%	37%
Silt	35%	24%
Clay	9%	7%

It should be noted that an organic matter content test was conducted on this material. The results indicate that at this location, this deposit contains 1.3% organics which gives this material the dark-grey to black colour.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with percent recoveries and RQD's are shown on the log sheet.

At the time of the investigation, the groundwater table was measured at Elev. 62.4.

BH C-2

Extending from the ground surface (Elev. 67.8) to a depth of 3.6 m is a non-cohesive deposit of sand. Based on a grain size distribution test the sand has some silt and trace of clay. The gradation curve is shown on Fig. 3 in the Appendix and can be summarized as follows:

Gravel	0%
Sand	72%
Silt	21%
Clay	7%

Standard Penetration test 'N' values ranging between 4 and 13 blows/0.3 m indicate this granular material is in a loose state.

Extending from a depth of 3.6 m to 5.1 m is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally, this deposit is cohesive and the fines can be described as a plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams

of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits test conducted on samples of this heterogeneous mixture from various locations across this site.

Based on 'N' values of 11 and 23 blows/0.3 m, this dark grey material is considered to have a consistency of stiff to very stiff.

A grain size distribution test was conducted on one sample of this material and the result is shown on Fig. 3 in the Appendix. In summary the one sample tested included 25% gravel, 33% sand, 27% silt and 15% clay.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheet.

At the time of the investigation the groundwater table was measured at Elev. 64.9.

BH C-3

Extending from the ground surface (Elev. 72.1) to a depth of 2.9 m is a fill composed of a silty clay with sand and gravel. Occasional boulder-sized fragments of shale were also encountered within this fill material. The results of an Atterberg Limits test carried out on one sample of this material indicates the following:

Natural Moisture content:	11 %
Plastic Limit	: 19 %
Liquid Limit	: 25.5%
Plasticity Index	: 6.5%

These results indicate that the fines of this fill material are a slightly plastic silt to a silty clay of low plasticity (ML-CL).

A gradation test was carried out on the same sample with the following distribution: 42% gravel, 31% sand, 21% silt, and 6% clay.

Based on 'N' values of 7 and 10 blows/0.3 m, this fill is considered to be in a firm to stiff state. It is to be noted that one 'N' value of 60 blow/0.3 m was obtained at Elev. 71, however, it is not indicative of the character of this material. It is believed that the high 'N' value was obtained as a result of a boulder.

Underlying the fill and extending down from Elev. 69.2 to 66.5 is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally, this deposit is cohesive and the fines can be described as a plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix show a plasticity chart with the results of Atterberg Limits tests carried out on samples of this heterogeneous mixture from various locations across this site.

Based on 'N' values ranging between 9 and 20 blows/0.3 m this dark grey material is considered to have a stiff to very stiff consistency.

Two grain size distribution tests were carried out on samples of this material and the results are shown on Fig. 4 in the Appendix.

It should be noted that an organic content test was conducted on this material. The results indicate that at this location, this deposit contains 4.5% organics which gives the material the dark grey to black colour.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation, the groundwater table was measured at Elev. 68.7.

BH C-4

Extending from the ground surface (Elev. 68.0) down to a depth of 2.1 m is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally, this deposit is cohesive and the fines can be described as a plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 35 and 44 blows/0.3 m, this material is considered to have a hard consistency.

A grain size distribution test was carried out on one sample of this material and the results are shown on Fig. 5. The distribution can be summarized as follows: 23% gravel, 48% sand, 21% silt, and 8% clay.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 66.5.

BH C-5

Extending from the ground surface (Elev. 68.1) to a depth of 1.4 m is a non-cohesive deposit of silt, some clay, sand, gravel. With depth, the clay content in this deposit increase and consequently the material becomes slightly cohesive. No laboratory tests were carried out on this material. Based on the field 'N' values, this material is in a compact state.

From a depth of 1.4 m down to a depth of 2.2 m is a deposit consisting of a heterogeneous mixture of clay, silt, sand, and gravel. Generally this deposit is cohesive and the fines can be described as a plastic silt (ML group) to silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of over 35 blows/0.3 m, this material is considered to have a hard consistency.

A grain size distribution test was carried out on one sample of this material and the results are shown on Fig. 5. The distribution can be summarized as follows: 16% gravel, 42% sand, 30% silt, and 12% clay.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 66.6.

BH C-6

Extending from the ground surface (Elev. 68.8) to a depth of 4.6 m is a deposit consisting of a heterogenous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as a plastic silt (ML group)

to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 9 to 26 blows/ 0.3 m, this material is considered to have a stiff to very stiff consistency.

Two grains size distribution tests were carried out on samples of this material and the results are shown on Fig. 6.

It is to be noted that between Elev. 66 and 67.5, this material had slight to no plasticity and could be considered to behave as a granular material.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 66.8.

BH C-7

Extending from the ground surface (Elev. 67.9) to a depth of 2.4 m is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 16 and 47 blows/ 0.3 m, this material is considered to have a stiff to very stiff consistency.

A grain size distribution test was carried out on one sample of this material and the results are shown on Fig. 6. The distribution can be summarized as follows: 13% gravel, 28% sand, 34% silt, and 25% clay.

It should be noted that an organic content test was conducted on this material. The results indicate that at this location, this deposit contains 5.8% organics which gives this material the dark grey to black colour.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 65.7.

BH C-8

Under approximately 0.5 m of sand and gravel shoulder fill, and extending down to Elev. 64.5 is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 24 to over 30 blows/ 0.3 m, this material is considered to have a very stiff to hard consistency.

No grain size distribution tests were carried out on samples of this material.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation, the groundwater table was measured at Elev. 65.7.

BH C-9

Extending from the ground surface (Elev. 71.4) down to a depth of 2.1 m is a silty clay fill, with some sand, gravel. Small pockets of silt were also encountered in this fill.

Underlying this fill, and from Elev. 69.3 to 67.4 is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as plastic silt (ML group) to a silty clay of low plasticity (CL group). However, one tested sample (#3, Elev. 69) yielded the following results:

Natural Moisture content:	22.5%
Plastic Limit	: 36.5%
Liquid Limit	: 17.0%
Plasticity Index	: 19.5%

This indicates that the fines of this sample are a silty clay of intermediate plasticity.

This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture (including the previously noted sample) from various locations across the site.

Based on 'N' values of 15 and 29 blows/ 0.3 m, this material is considered to have a stiff to very stiff consistency.

Three grain size distribution tests were carried out on samples of this material and the results are shown on Fig. 7.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater level was measured at Elev. 69.8.

BH C-10

Under 0.6 m of compact sand with silt, trace clay is a deposit extending from Elev. 70.8 to 68.7 composed of a heterogeneous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 29 and 45 blows/ 0.3 m, this material is considered to have a very stiff to hard consistency.

A grain size distribution test was carried out on one sample of this material and the results are shown on Fig. 7. The distribution can be summarized as follows: 18% gravel, 39% sand, 27% silt, and 16% clay.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 70.5.

BH C-11

Under 0.6 m of compact silt, some clay, sand, trace gravel in a deposit extending from Elev. 71.4 to 67.8 composed of a heterogenous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 21 to over 30 blows/ 0.3 m, this material is considered to have a very stiff to hard consistency.

A grain size distribution test was carried out on one sample of this material and the results are shown on Fig. 7. The distribution can be summarized as follows: 6% gravel, 39% sand, 43% silt, and 12% clay.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 70.8.

BH C-12

Extending from the ground surface (Elev. 70.6) down to a depth of 1.1 m is a silty clay deposit of low plasticity. Based on visual observation, this deposit also contains sand and gravel and occasional silt seams.

From Elev. 69.5 to Elev. 66.8 is a deposit consisting of a heterogenous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of 8 to 11 blows/ 0.3 m, this material is considered to have a stiff consistency.

Two grain size distribution tests were carried out on samples of this material and the results are shown on Fig. 8.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater level was measured at Elev. 69.4.

BH C-13

From the ground surface (Elev. 63.8) down to a depth of 3.6 m is a silt deposit with traces of clay and sand. Within this deposit are seams of sand and pockets of silty clay of low plasticity.

Two grain size distribution test were conducted on samples of this material and the results are shown on Fig. 9 in the Appendix.

Based on Standard Penetration test 'N' values of 6 to 15 blows/0.3 m this generally non-cohesive material is considered to be in a loose to compact state.

Extending from Elev. 60.2 down to Elev. 55.3 is a deposit consisting of a heterogeneous mixture of clay, silt, sand and gravel. Generally this deposit is cohesive and the fines can be described as a plastic silt (ML group) to a silty clay of low plasticity (CL group). This deposit may include random thin seams of non-cohesive silt. Fig. 1 in the Appendix shows a plasticity chart with the results of Atterberg Limits tests conducted on samples of this heterogeneous mixture from various locations across the site.

Based on 'N' values of approximately 20 to over 30 blows/ 0.3 m, this material is considered generally to have a very stiff to hard consistency.

Two grain size distribution tests were carried out on samples of this material and the results are shown on Fig. 9.

Underlying this generally cohesive deposit and extending to an undetermined depth is shale bedrock. The zones of various weathering together with the percent recoveries and RQD's are shown on the log sheets.

At the time of the investigation the groundwater table was measured at Elev. 59.5.

DISCUSSION AND RECOMMENDATIONS

In conjunction with the proposed interchange at St. Laurent Blvd. it is proposed to provide illumination utilizing 13 high mast light pole installations. The following table indicates the location and pole height of each installation in addition to the existing and proposed grade at each location.

POLE	STATION	OFFSET (m)	PROPOSED GRADE	EXISTING	POLE HEIGHT (m)
			AT POLE LOCATION	GRADE	
C1	32+565	58 South	69.91	66.54	25
C2	32+716	58 South	69.31	67.76	25
C3	32+879	58 South	69.44	72.08	25
C4	33+041	48 South	68.10	67.97	30
C5	33+195	50 South	68.25	68.09	30
C6	33+340	37 South	68.30	68.80	30
C7	33+522	24 North	70.20	69.61	30
C8	33+496	70 South	66.41	66.27	30
C9	33+698	42 South	71.30	71.37	30
C10	33+868	35 South	71.36	71.35	30
C11	34+040	35 South	71.87	72.03	30
C12	34+205	35 South	69.75	70.14	30
C13	32+418	58 South	63.90	63.80	25

Conventional spread footings for these light poles would likely be quite expensive. However, high mast light poles have been installed economically in many areas of North America and Europe using a design method proposed by B.B. Broms and others in which the poles are supported on a concrete caisson pile. The Structural Office has decided to adopt this same method described by Broms in two separate papers;

Broms, B.B.

"Lateral Resistance of Piles in Cohesive Soils",

Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM2, Paper 3825, March 1964.

and

Broms, B.B.

"Lateral Resistance of Piles in Cohesionless Soils",

Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM3, Paper 3909, May, 1964.

In the following paragraphs, the feasibility of constructing concrete caissons at the site is discussed and the various parameters to be used in the caisson design are provided.

At most locations the poles are to be installed in the original ground in areas where no significant amounts of cut or fill is required. The exceptions are as follows:

- at C-1, 3.4 m± fill
- at C-2, 1.6 m± fill
- at C-3, 2.6 m± fill

DESIGN

The contribution of fill material should be ignored from a lateral resistance point of view at those locations where fill is proposed or fill exists. Similarly, in all cases, the soil within the zone of frost penetration (1.8 m) should not be counted on to provide lateral strength.

For the cohesive soils located at this site, the coefficient of horizontal subgrade reaction should be computed in accordance with the following formula: (The design parameters are presented in Imperial Units, since the design example provided by the Structural Office used Imperial Units throughout).

$$K_h = \frac{n_1 n_2 80 q_u}{D}$$

Where:

- K_h - Coefficient of horizontal subgrade reaction (lb/in³)
- D - Diameter of concrete caisson pile (in)
- n_1 - Coefficient as defined below:

Unconfined Compressive Strength	
q_u (psi)	n_1
Less than 7	0.32
7 to 28	0.36
Greater than 28	0.40

- n_2 - Coefficient based on pile material = 1.15 for concrete
- q_u - Unconfined compressive strength (psi)

For the non-cohesive soils, K_h should be computed from the following formula:

$$K_h = n_h \frac{z}{D}$$

K_h - Coefficient of horizontal subgrade reaction (tons/ft³)

z - Depth below ground surface (ft.)

D - Diameter of caisson (ft)

n_h - Coefficient evaluated as follows:

Coefficient n_h in tons/ft³

Relative Density	Loose	Compact	Dense
Above Groundwater table	7	21	56
Below Groundwater table	4	14	34

The following table gives the soil parameters which are recommended for the design of the high-mast light caisson:

Note: ϕ = apparent angle of internal friction for non-cohesive soils
 q_u = unconfined compressive strength in psi
 $(q_u = 2 c_u)$
 γ = unit weight in pcf

Pole	Elev.(m) <u>From-to</u>	Type of Soil	ϕ	q_u <u>psi</u>	γ <u>pcf</u>
C-1	66.5-62.8	non-cohesive	30°	-	125
	62.8-60.0	cohesive	-	28	125
	60.0-58.6	cohesive	-	70	140
	58.6-	shale	see note 1		
C-2	67.8-64.2	non-cohesive	26°	-	120
	64.2-62.7	cohesive	-	15	125
	62.7-	shale	see note 1		
C-3	72.1-69.2	fill	0	0	-
	69.2-68.5	non-cohesive	33°	-	130
	68.5-66.5	cohesive	-	15	125
	66.5-	shale	see note 1		
C-4	68.0-65.9	cohesive	-	60	135
	65.9-	shale	see note 1		
C-5	68.1-66.7	non-cohesive	28°	-	122
	66.7-65.9	cohesive	-	55	135
	65.9-	shale	see note 1		
C-6	68.8-64.2	cohesive	-	28	125
	64.2-	shale	see note 1		
C-7	67.9-66.5	cohesive	-	15	125
	66.5-65.5	cohesive	-	65	135
	65.5-	shale	see note 1		
C-8	66.3-65.8	fill	0	0	-
	65.8-64.5	cohesive	0	40	130
	64.5-	shale	see note 1		
C-9	71.4-69.3	fill	0	0	-
	69.3-67.4	cohesive	-	35	130
	67.4-	shale	see note 1		

Pole	Elev. (m) <u>From-to</u>	Type of Soil	ϕ	q_u <u>psi</u>	<u>pcf</u>
C-10	71.4-70.8	non-cohesive	32°	-	125
	70.8-68.7	cohesive	-	60	135
	68.7-	shale	see note 1		
C-11	72.0-71.4	non-cohesive	32°	-	125
	71.4-67.8	cohesive	-	70	140
	67.8-	shale	see note 1		
C-12	70.6-69.5	cohesive	-	10	125
	69.5-66.8	cohesive	-	15	125
	66.8-	shale	see note 1		
C-13	63.8-60.2	non-cohesive	28°	-	122
	60.2-59.0	cohesive	-	4	120
	59.0-55.3	cohesive	-	50	130
	55.3-	shale	see note 1		

NOTES:

1. If caissons are to be augered into the shale the following values are to be used:

highly weathered	$q_u = 80 \text{ psi}, \checkmark = 140 \text{ pcf}$
slightly weathered	$q_u = 700 \text{ psi}, \checkmark = 150 \text{ pcf}$
unweathered	$q_u = 1400 \text{ psi}, \checkmark = 160 \text{ pcf}$

The attached log sheets should be referred to for the zones of the various degree of weathering.

2. If rock anchors are to be used in the design, the following bond stresses can be used.

highly weathered	15 psi
slightly weathered	30 psi
unweathered	100 psi

3. As previously noted, even though strength parameters may be given in the previous table the contribution of existing or proposed fill material, and 1.8 m frost penetration zone should be neglected in the design.
4. Groundwater levels are as follows (as measured at the time of the investigation)

<u>POLE</u>	<u>ELEV.</u>	<u>POLE</u>	<u>ELEV.</u>
C-1	62.4	C-8	65.7
C-2	64.9	C-9	69.8
C-3	68.7	C-10	70.5
C-4	66.5	C-11	70.8
C-5	66.6	C-12	69.4
C-6	66.8	C-13	59.5
C-7	65.7		

It is to be noted that the groundwater level may differ at the time of construction.

5. In zones where the soil is slightly or non-plastic, and below the prevailing groundwater table, it is possible that the sides of an unsupported augered hole will cave in. If cave-in, and as a result loosening, occurs, the lateral strength of the soil may be drastically reduced. Zones of this type of material are encountered throughout the stratigraphy.

Therefore, given the nature of the subsoils across the site, it is recommended to construct all caissons utilizing a temporary liner which is withdrawn as the concrete is poured.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of L. Politano, Project Foundations Engineer, and I. Richardson, Student Engineer, utilizing equipment owned and operated by Marathon Soil Drilling Ltd. and Johnston Drilling Inc., both of Ottawa. This report was written by L. Politano and reviewed by M. Devata, Chief Foundations Engineer (East).



A handwritten signature in cursive script, reading "L. Politano", followed by a long horizontal line ending in a small circle.

L. Politano, P. Eng.
Project Foundations Engineer

A handwritten signature in cursive script, reading "M. Devata".

M. Devata, P. Eng.
Chief Foundations Engineer
(East)

APPENDIX

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

DESCRIPTION OF ROCK CORE - W.P. 62-82-01

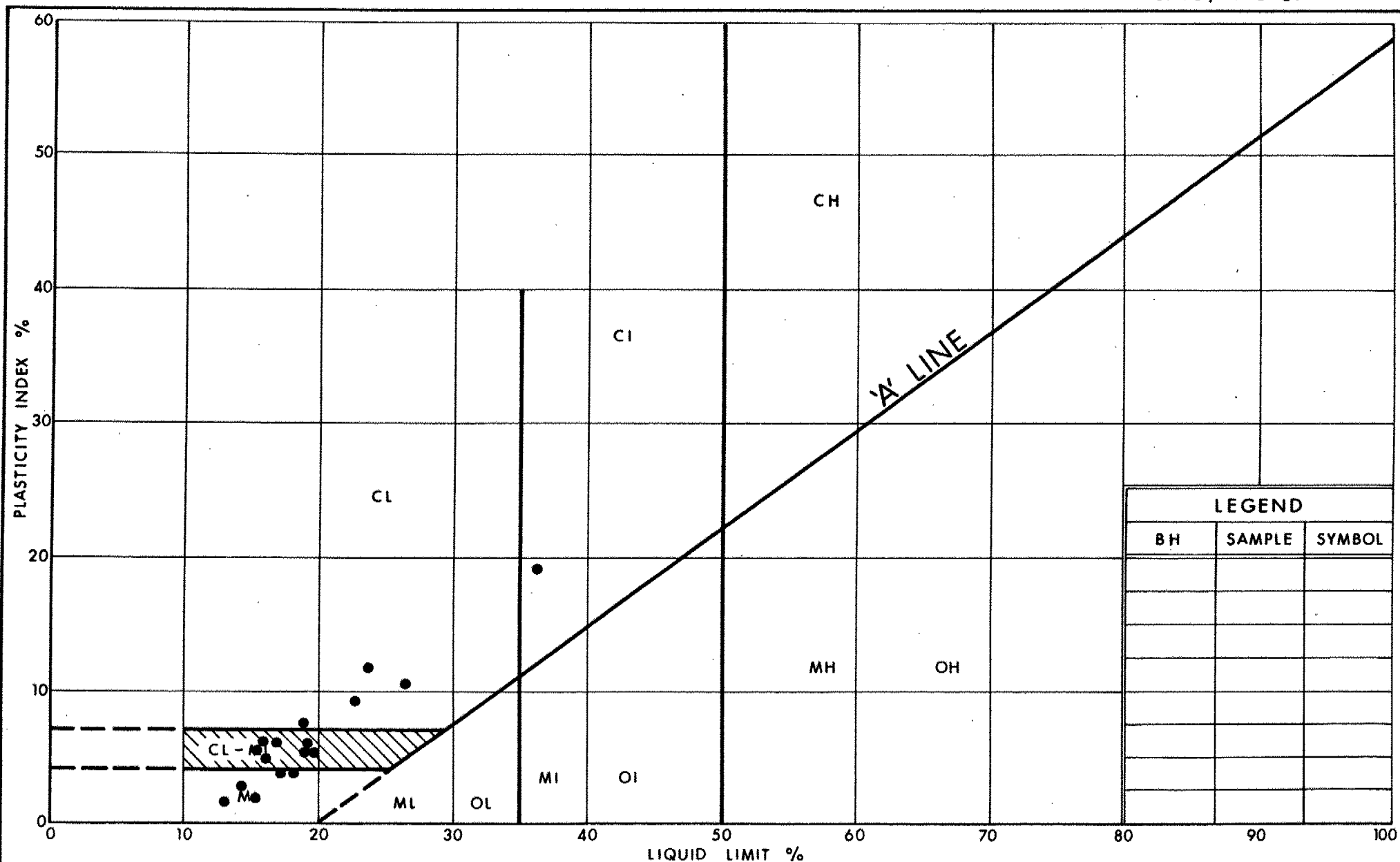
BOREHOLE NUMBER				CORE DESCRIPTION	
	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
C1	8.84m-10.34m 10.34m-11.89m	63 59	0 59	8.84 - 11.89	High core loss apparently due to poor drilling; Shale, black, slightly weathered, becoming unweathered with occ. thin limestone layers. Top of sound bedrock difficult to define.
C2	5.13m- 6.58m 6.58m- 8.18m	100 98	23 46	5.13m - 6.22m 6.22m - 8.18	Shale, black, slightly weathered, very closely spaced joints, with occasional thin limestone layers Shale, black, unweathered, closely spaced joints, with occasional (5%) thin limestone layers (6 cm)
C3	6.20 - 7.75 7.75 - 8.92	100 100	34 72	6.20 - 6.86 6.86 - 8.92	Shale, black, slightly weathered, very closely spaced joints Shale, black, unweathered, closely spaced joints, with occasional (2%) thin limestone layers (9 cm)
C4	2.21 - 3.68 3.68 - 5.28	100 100	66 67	2.21 - 2.44 2.44 - 5.28	Shale, black, slightly weathered, very closely spaced joints Shale, black, unweathered, closely to medium spaced joints, with occasional (5%) thin limestone layers (4 cm)
C5	2.31 - 3.89 3.89 - 5.36	100 100	8 67	2.31 - 3.43 3.43 - 5.36	Shale, black, moderately weathered, very closely spaced joints, with occasional (2%) thin limestone layers (2 cm) Shale, black, unweathered, closely spaced joints, with occasional (10%) thin limestone layers (7 cm)
C6	4.72 - 6.15 6.15 - 7.75	100 100	54 27	4.72 - 7.75	Shale, black, unweathered, with occasional zones of slightly weathered shale
C7	2.54 - 3.56 3.56 - 5.05	95 100	20 61	2.54 - 3.00 3.00 - 5.05	Shale, black, slightly weathered, close to very closely spaced joints Shale, black, unweathered, close to medium spaced joints

* CR= CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

DESCRIPTION OF ROCK CORE - W.P. 62-82-01

BOREHOLE NUMBER				CORE DESCRIPTION	
	DEPTH (m)	% CR *	% RQD *	DEPTH (m)	DESCRIPTION
C8	1.98 - 3.51	93	60	1.98 - 2.29	Shale, black, moderately weathered, very closely spaced joints
	3.51 - 5.03	100	47	2.29 - 5.03	Shale, black, unweathered, very closely spaced joints
C9	3.99 - 5.49	100	78	3.99 - 5.49	Shale, black, unweathered, closely spaced joints
C10	2.67 - 4.25	100	31	2.67 - 2.90	Shale, black, slightly weathered, very closely spaced joints
	4.25 - 5.38	100	62	2.90 - 5.38	Shale, black, slightly weathered, becoming unweathered, closely spaced joints with zones of very close spacing
C11	4.19 - 5.76	98	84	4.19 - 7.21	Shale, black, unweathered, closely to medium spaced joints
	5.76 - 7.21	100	72		
C12	4.27 - 5.84	100	6	4.27 - 5.84	Shale, black, slightly weathered, very closely spaced joints, with moderately weathered, very closely spaced zones at 4.57-4.77, 5.21-5.26 and 5.36-5.77
C13	8.46 - 9.98	100	53	8.46 - 9.14	Shale (80%), black, slightly weathered, very closely spaced joints, with limestone (20%) layers up to 10 cm thick
	9.98 - 11.51	100	50	9.14 - 11.51	Shale (90%), black, unweathered, closely spaced joints, with limestone (10%) layers up to 12 cm thick

* CR = CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION



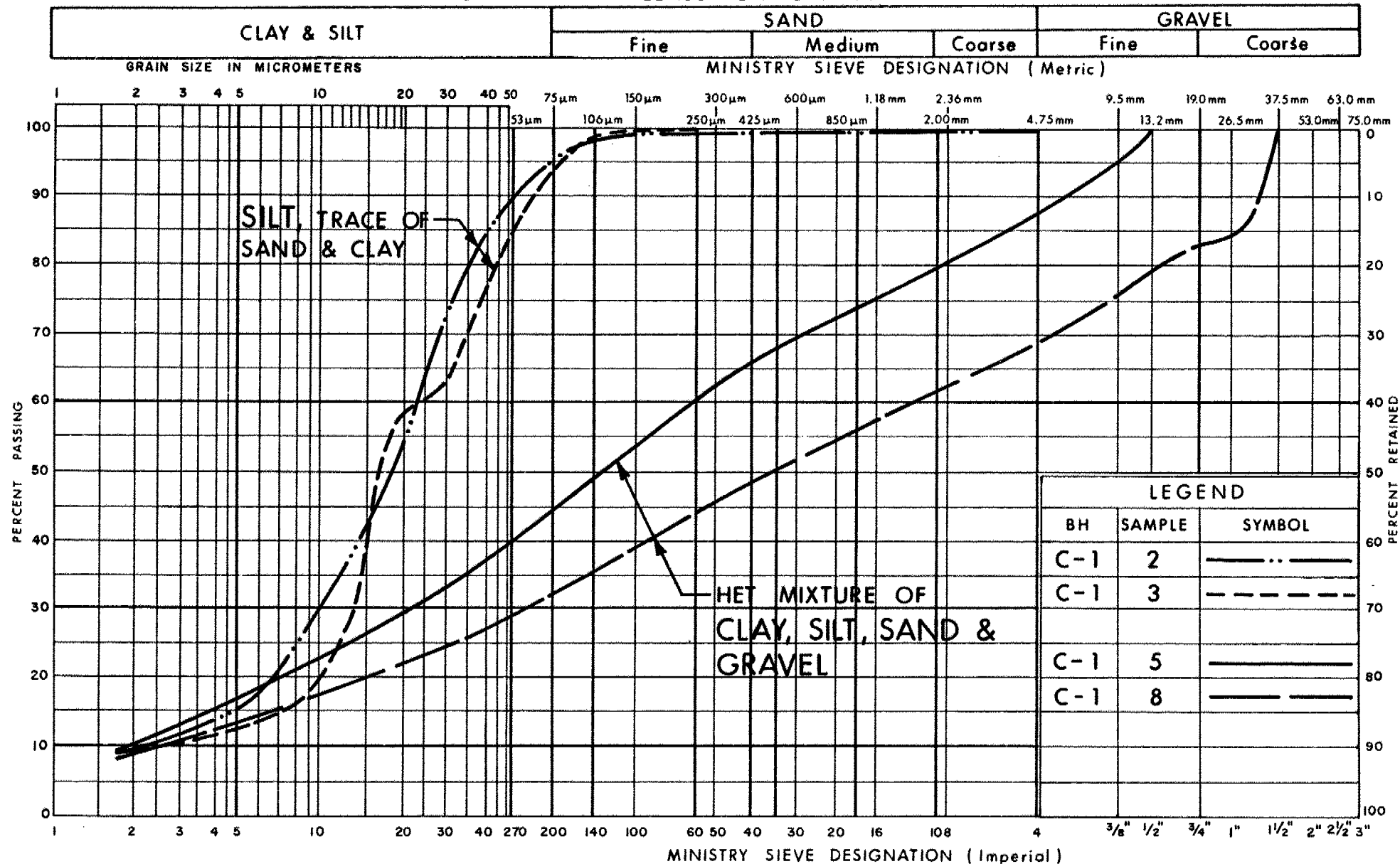
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PLASTICITY CHART HET MIXTURE OF CLAY, SILT, SAND & GRAVEL

FIG No 1

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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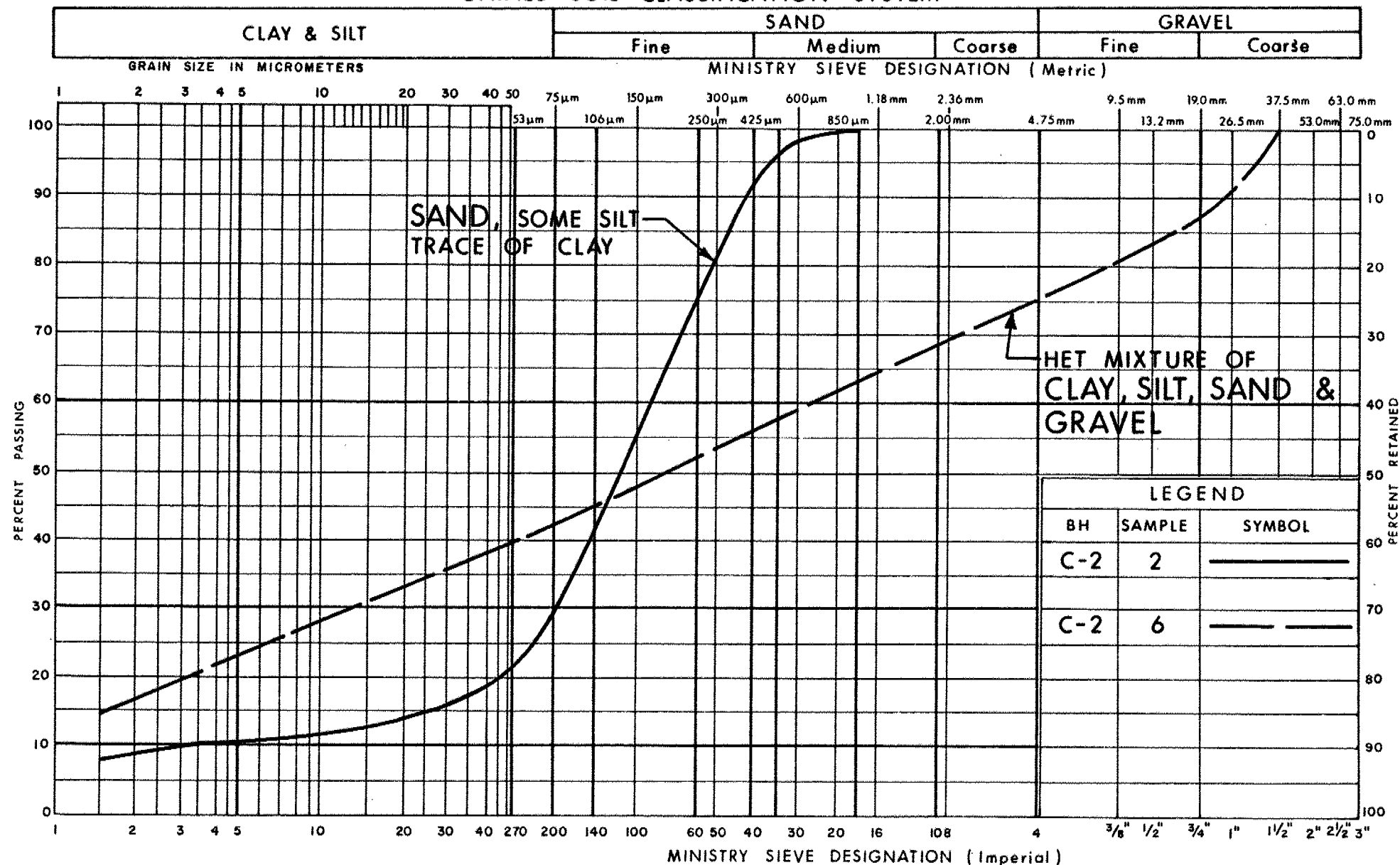
GRAIN SIZE DISTRIBUTION

BOREHOLE C-1

FIG No 2

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

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Transportation and
Communications

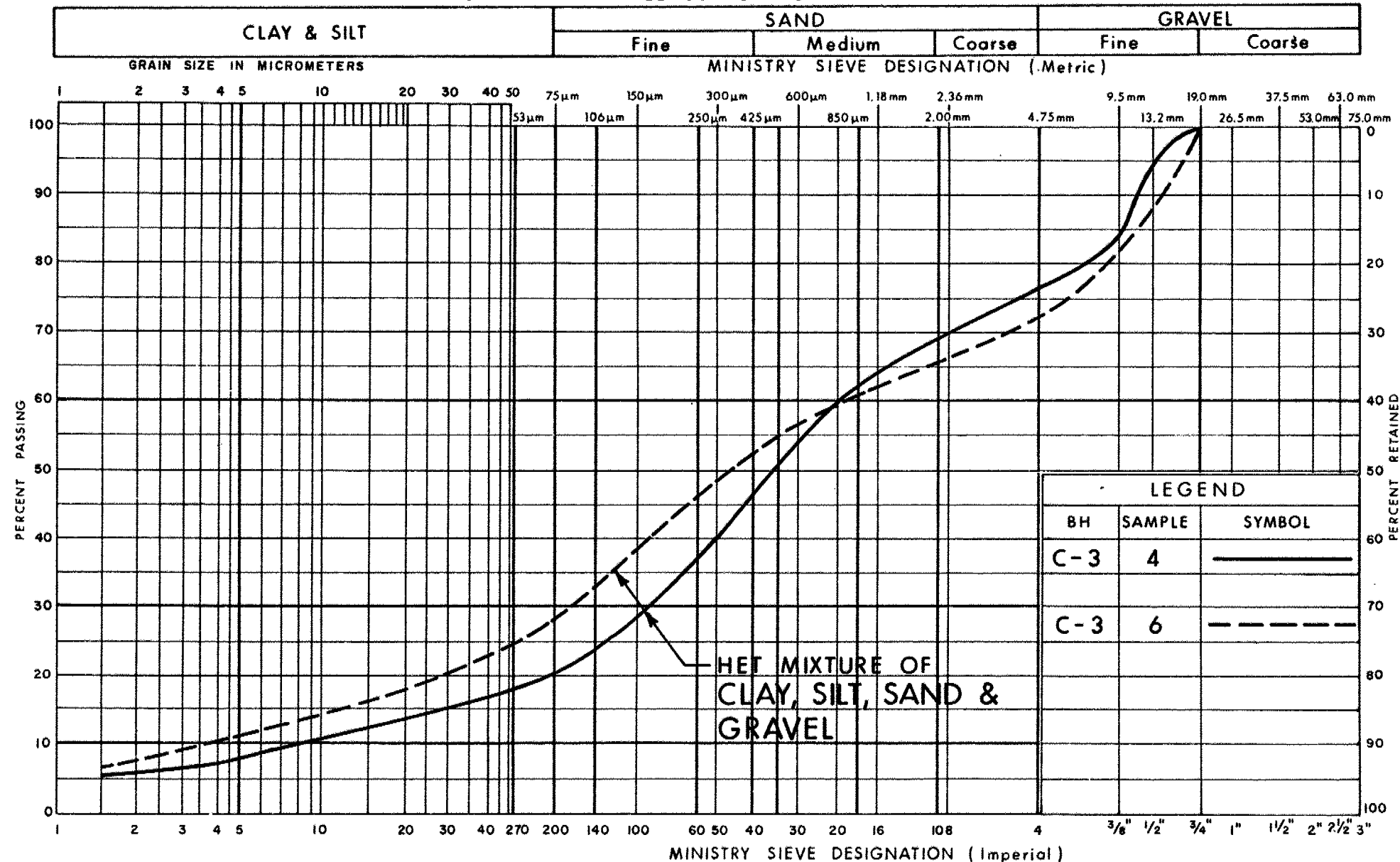
GRAIN SIZE DISTRIBUTION

BOREHOLE C-2

FIG No 3

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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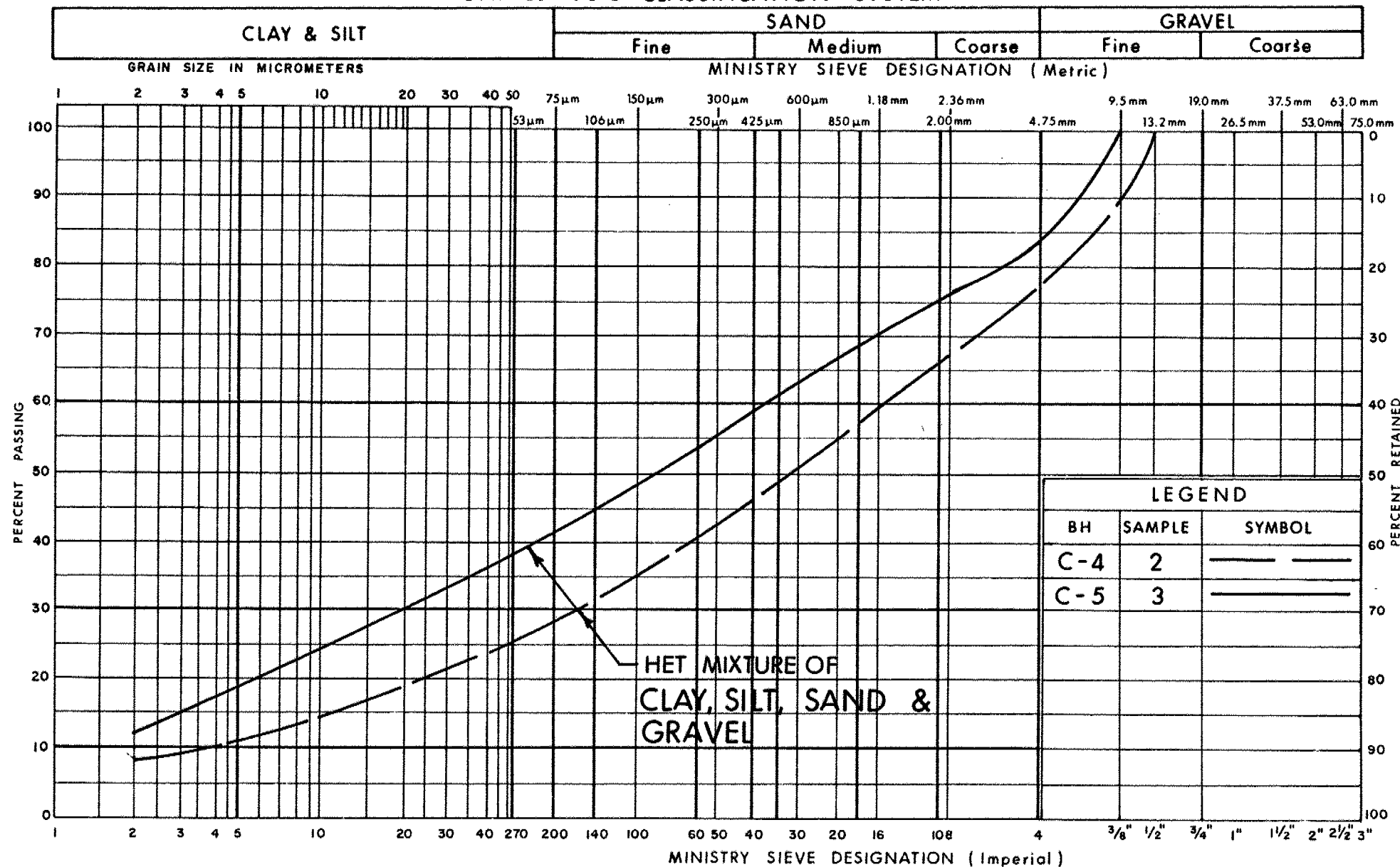
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 GRAIN SIZE DISTRIBUTION
BOREHOLE C-3

FIG No 4

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



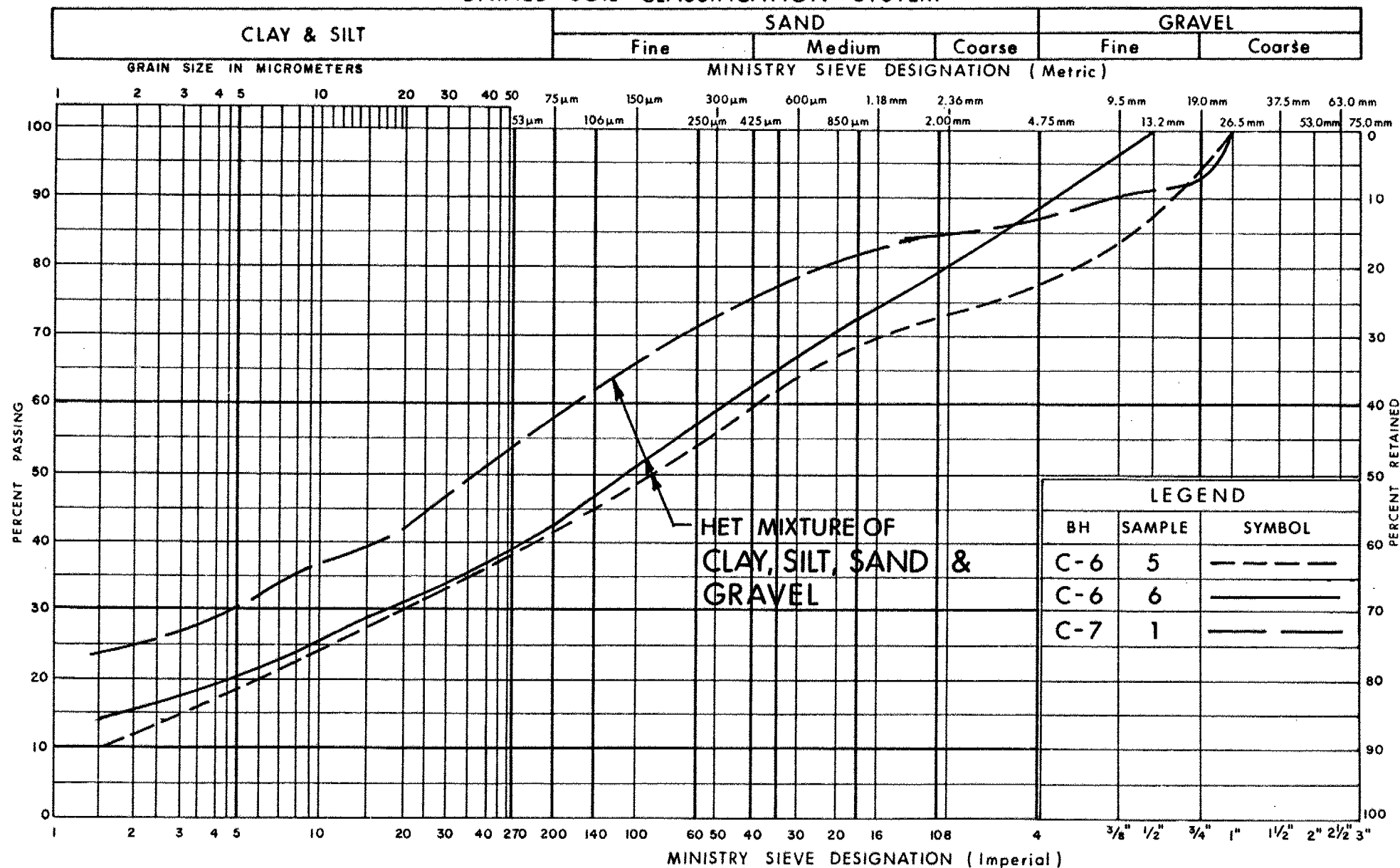
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Communications

GRAIN SIZE DISTRIBUTION BOREHOLES C-4 & C-5

FIG No 5

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

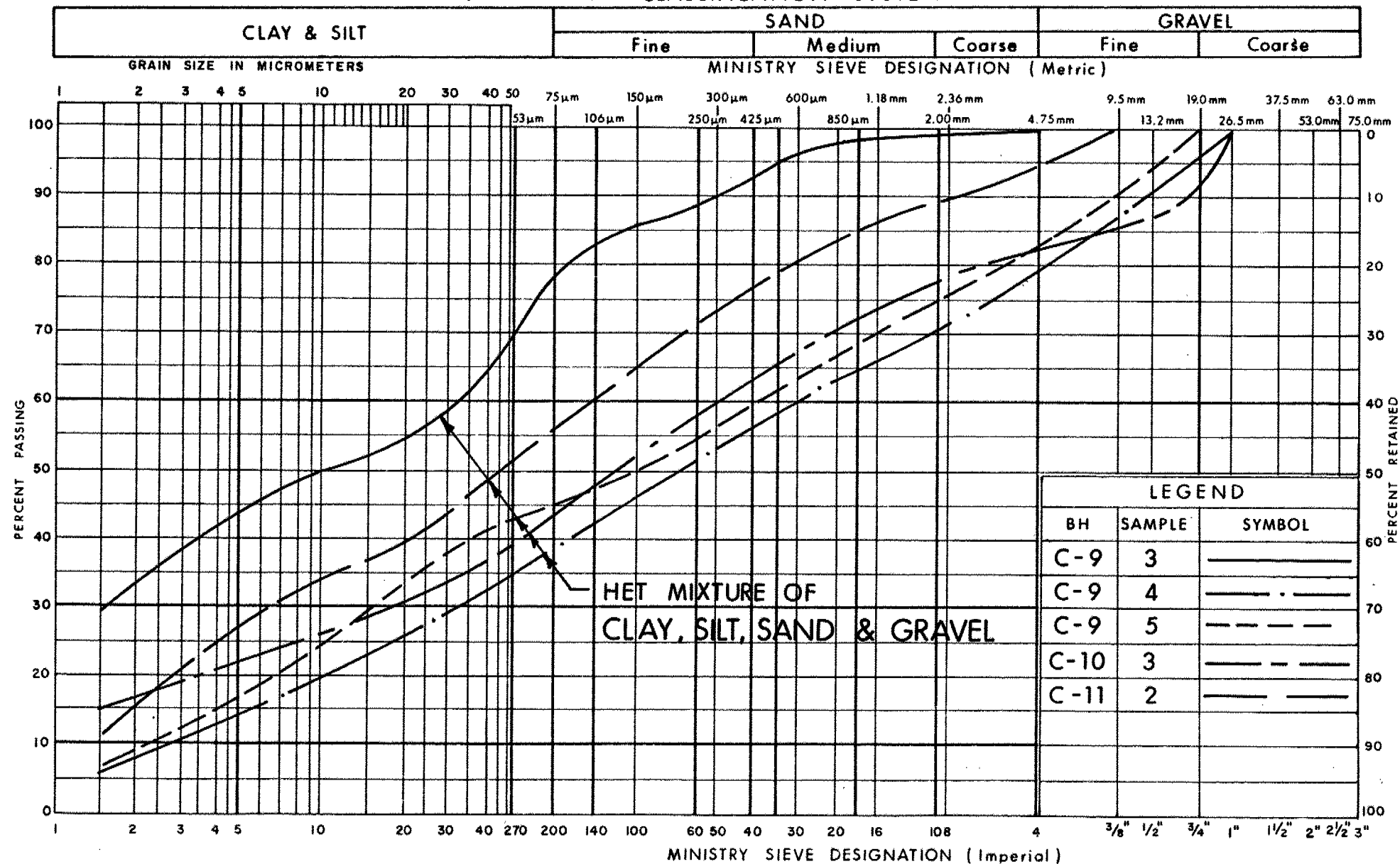
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Communications

 GRAIN SIZE DISTRIBUTION
BOREHOLES C-6 & C-7

FIG No 6

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ontario

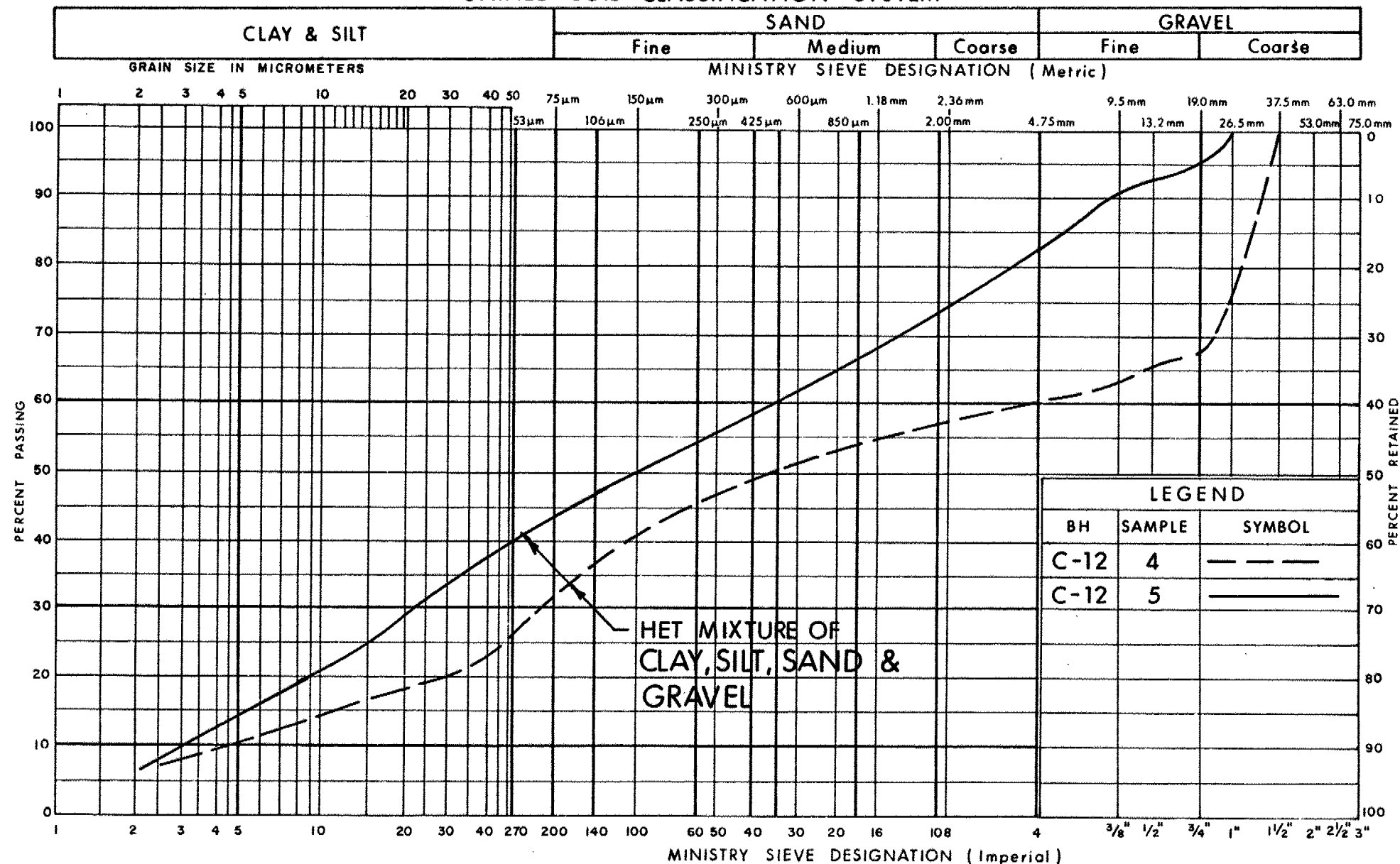
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Transportation and
Communications

GRAIN SIZE DISTRIBUTION
BOREHOLES C-9, C-10 & C-11

FIG No 7

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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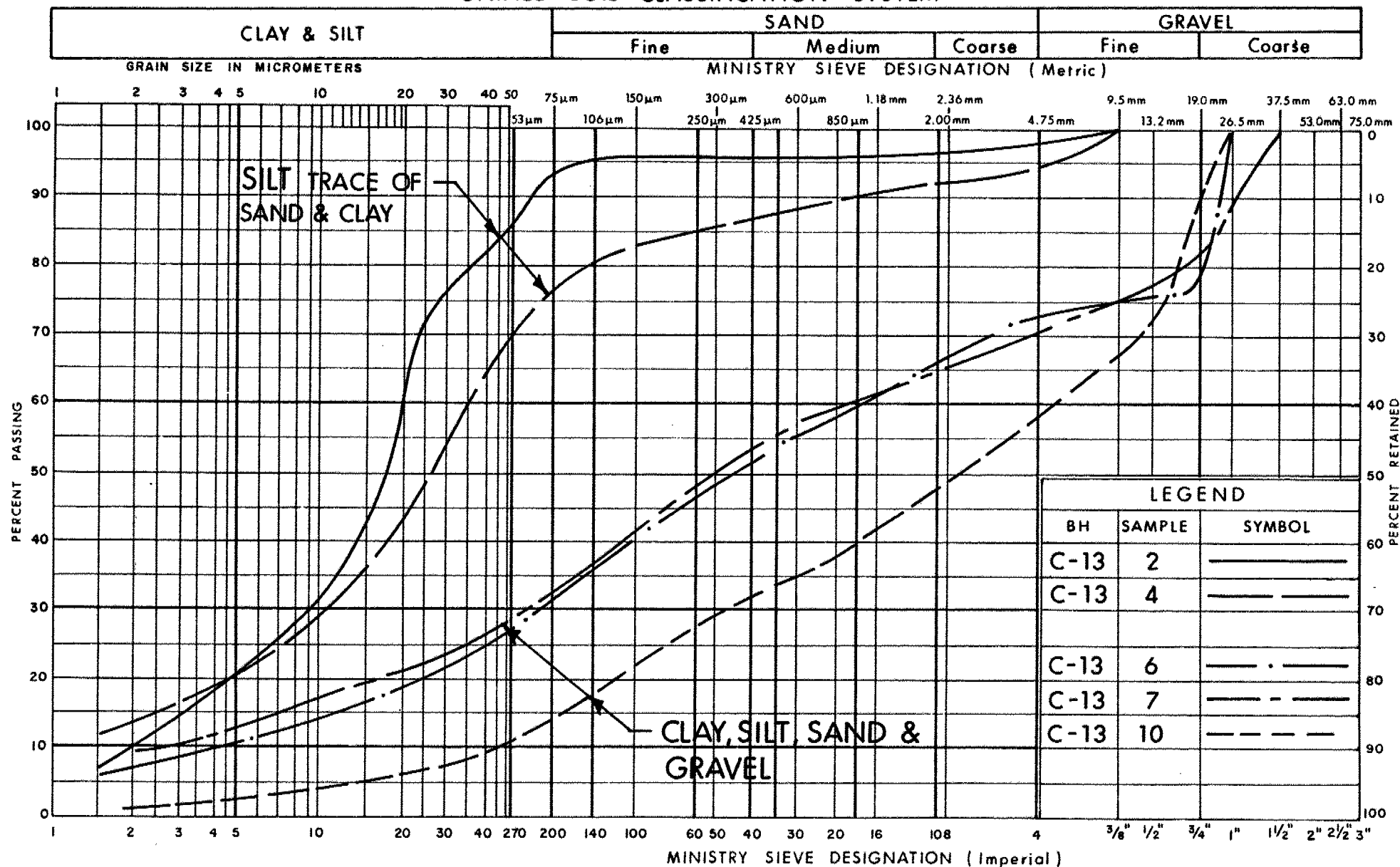
GRAIN SIZE DISTRIBUTION

BOREHOLE C-12

FIG No 8

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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Transportation and
Communications

 GRAIN SIZE DISTRIBUTION
BOREHOLE C-13

FIG No 9

W P 62-82-01

RECORD OF BOREHOLE No C-1

METRIC

W P 62-82-01 LOCATION Sta. 32 + 565; O/S 58.0 m RT 4 Hwy. 417 ORIGINATED BY IR
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BX Core & Cone Test COMPILED BY IR
DATUM Geodetic DATE 85 06 13 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ Org. Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100		W _p	W	W _L		
66.5	Ground Level													
0.0	Sand, some silt trace clay Compact Brown		1	SS	15									
65.1														
1.4	Silt, trace sand clay Compact Grey		2	SS	23									0 5 89 6
			3	SS	9									0 5 86 9
			4	SS	10									
62.8														
3.7	Heterogeneous Mixture Clay, Silt, Sand, Gravel with random silt seams		5	SS	16								1.3%	12 44 35 9
			6	SS	15									
			7	SS	20									
	Very Stiff		8	SS	28									32 37 24 7
	Hard Black		9	SS	100/25 cm									
58.6			10	SS	160/28 cm									
7.9	Shale Bedrock Highly Weathered		11	SS	100/12 cm									
			12	RC	BX									RQD = 0%
	Slightly Weathered Unweathered		13	RC BXL	REC 63%									
			14	RC BXL	REC 59%									RQD = 59%
54.6														
11.9	End of Borehole													



RECORD OF BOREHOLE No C-2

METRIC

W P 62-82-01 LOCATION Sta. 32 + 716; O/S 58.0 m RT & Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core & Cone Test COMPILED BY IR
DATUM Geodetic DATE 85 06 13 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
67.8	Ground Level															
0.0	Sand some silt trace clay		1	SS	13									0 72 21 7		
	Compact		2	SS	9											
	Loose		3	SS	4											
			4	SS	4											
64.2																
3.6	Heterogeneous Mixture Clay, Silt, Sand, Gravel Stiff to Very Stiff Dark Grey		5	SS	11										25 33 27 15	
62.7			6	SS	23											
5.1	Shale Bedrock Slightly Weathered Unweathered		7	RC BXL	REC 100%										RQD = 23%	
			8	RC BXL	REC 98%									RQD = 46%		
59.6																
8.2	End of Borehole															

RECORD OF BOREHOLE No C-3

METRIC

W P 62-82-01 LOCATION Sta. 32 + 879; O/S 58.0 m RT & Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core & Cone Test COMPILED BY TR
DATUM Geodetic DATE 85 06 13 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y Org. Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
72.1	Ground Level															
0.0	<u>Fill</u> Silty Clay with Sand and Gravel occasional boulders Firm to Stiff		1	SS	60	*										
			2	SS	10											
			3	SS	7											
69.2																
2.9	Heterogeneous Mixture Clay, Silt, Sand, Gravel Stiff to Very Stiff		4	SS	20											
			5	SS	9											
			6	SS	12											
66.5			7	SS	32											
5.6	Shale Bedrock <u>Highly Weathered</u> Slightly Weathered Unweathered		8	SS	79/15cm											
			9	RC BXL	REC 100%											
			10	RC BXL	REC 100%											
63.2																
8.9	End of Borehole * Note: High 'N' Value due to boulder															



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RECORD OF BOREHOLE No C-4

METRIC

W P 62-82-01 LOCATION Sta. 33 + 041; O/S 48.0 m RT & Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core & Cone Test COMPILED BY IR
DATUM Geodetic DATE 85 06 14 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
68.0	Ground Level										
0.0	Heterogeneous Mixture Clay, Silt Sand, Gravel with random silt seams Dark Brown Black Hard		1	SS	44						23 48 21 8
65.9			2	SS	35						
2.1	Shale Bedrock Slightly Weathered Unweathered		3	RC BXL	REC 100%						RQD = 66%
			4	RC BXL	REC 100%						RQD = 67%
62.7											
5.3	End of Borehole										

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to
Sensitivity

20
15
10
5
0
5
10
15
20
(%) STRAIN AT FAILURE



RECORD OF BOREHOLE No C-5

METRIC

W P 62-82-01 LOCATION Sta. 33 + 195; O/S 50.0 m RT 417
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core
DATUM Geodetic DATE 85 06 14

ORIGINATED BY LP
COMPILED BY IR
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
68.1	Ground Level																
0.0	Silt, some clay, sand gravel Becoming more cohesive with depth (clay content increasing) Compact Brown		1	SS	19		68										
66.7			2	SS	8		67										
1.4	Heterogeneous Mixture Clay, Silt Sand, Gravel		3	SS	35		66										16 42 30 12
65.9	Hard Dark Gray		4	SS	40/	2 cm	66										
2.2	Highly Weathered Shale Bedrock		5	RC BXL	REC 100%		65										RQD = 8%
	Moderately Weathered Unweathered		6	RC BXL	REC 100%		64										RQD = 67%
62.8							63										
5.3	End of Borehole																



RECORD OF BOREHOLE No C-6

METRIC

W P 62-82-01 LOCATION Sta. 33 + 340; O/S 37.0 m RT 4 Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core COMPILED BY IR
DATUM Geodetic DATE 85 06 14 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH										WATER CONTENT (%)		
								20 40 60 80 100										10 20 30		
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE														
68.8	Ground Level																			
0.0	Heterogeneous Mixture Clay, Silt Sand, Gravel trace organics		1	SS	26		68													
			2	SS	16		67													
	Slightly Cohesive		3	SS	21		66													
	Grey		4	SS	19		65													
	Dark Grey		5	SS	14		64													
	Stiff to Very Stiff		6	SS	9		63													
64.2			7	SS	20/		62													
4.6	Highly Weathered Shale Bedrock																			
	Unweathered with occasional zones of slightly weathered shale	8	RC BXL	REC 100%													RQD = 54%			
		9	RC BXL	REC 100%												RQD = 27%				
61.0																				
7.8	End of Borehole																			

+3, x5: Numbers refer to
Sensitivity


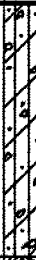


20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No C-7

METRIC

W P 62-82-01 LOCATION Sta. 33 + 522; O/S 27.9 m LT & Hwy. 417 ORIGINATED BY IR
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core COMPILED BY IR
DATUM Geodetic DATE 85 06 18 CHECKED BY [Signature]

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 Org. Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100	SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE					WATER CONTENT (%) 10 20 30	
67.9	Ground Level																		
0.0	Heterogeneous Mixture Clay, Silt, Sand, Gravel Sand and Gravel content increasing with depth Very Stiff to Hard		1	SS	16	 20 cm													
			2	SS	47														
65.5		3	SS	33/															
2.4	Highly Weathered Shale Bedrock Slightly Weathered Unweathered		4	RC BXL	REC 95%														
				5	RC BXL	REC 100%													
62.8																			
5.1	End of Borehole																		

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No C-8

METRIC

W P 62-82-01 LOCATION Sta. 33 + 496; O/S 70.0 m RT 4 Hwy. 417 ORIGINATED BY IR
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core & Cone Test COMPILED BY IR
 DATUM Geodetic DATE 85 06 14 CHECKED BY SP.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
66.3	Ground Level																
0.0	Sand and Gravel (Fill)																
65.8																	
0.5	Heterogeneous Mixture Clay, Silt Sand, Gravel Very Stiff to Hard		1	SS	24												
64.5			2	SS	103/	30 cm											
1.8	Shale Bedrock Moderately Weathered Unweathered		3	RC BXL	REC 93%												RQD = 60%
			4	RC BXL	REC 100%												RQD = 47%
61.3																	
5.0	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15
10

5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C-9

METRIC

W P 62-82-01 LOCATION Sta. 33 + 698; O/S 41.0 m RT Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core COMPILED BY IR
DATUM Geodetic DATE 85 06 14 CHECKED BY *CP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100				
								SHEAR STRENGTH				
71.4	Ground Level											
0.0	(Fill) Silty Clay some sand, gravel with small pockets of silt		1	SS	24		71					
			2	SS	4		70					
69.3	Heterogeneous Mixture Clay, Silt, Sand, Gravel Grey		3	SS	15		69					0 23 47 30
2.1	Very Stiff Dark Grey		4	SS	29		68					22 40 32 6
67.4			5	SS	28	18 cm	67					17 38 39 6
4.0	Shale Bedrock Unweathered		6	RC BXL	REC 100%		66					RQD = 78%
65.9												
5.5	End of Borehole											

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No C-10

METRIC

W P 62-82-01 LOCATION Sta. 33 + 868; O/S 35.0 m RT & Hwy. 417 ORIGINATED BY IR
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core COMPILED BY IR
DATUM Geodetic DATE 85 06 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
71.4	Ground Level																
0.0	Sand with Silt trace clay Compact		1	SS	20		71										
70.8																	
0.6	Heterogeneous Mixture Clay, Silt, Sand, Gravel		2	SS	29		70										
	Grey																
	Very Stiff to Hard		3	SS	45		69										18 39 27 16
	Dark Grey		4	SS	24/	23 cm	68										
68.7																	
2.7	Shale Bedrock Slightly Weathered becoming unweathered with depth		5	RC BXL	REC 100%		67										RQD = 31%
			6	RC BXL	REC 100%												RQD = 62%
66.0																	
5.4	End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No C-11

METRIC

W P 62-82-01

LOCATION Sta. 34 + 040; O/S 35.0 m RT Hwy. 417

ORIGINATED BY IR

DIST 9 HWY 417

BOREHOLE TYPE Hollow Stem Augers, BXL Core

COMPILED BY IR

DATUM Geodetic

DATE 85 06 18

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100									
								SHEAR STRENGTH									
72.0	Ground Level																
0.0	Silt, some clay, sand trace gravel Compact		1	SS	24												
71.4																	
0.6	Heterogeneous Mixture Clay, Silt, Sand, Gravel		2	SS	21												
	Silt some sand		3	SS	87/	15 cm											
	Sand and Gravel content increasing with depth		4	SS	91												
	Very Stiff to Hard		5	SS	35												
67.8			6	SS	47/	23 cm											
4.2	Shale Bedrock Unweathered		7	RC BXL	REC 98%											RQD = 84%	
			8	RC BXL	REC 100%											RQD = 72%	
64.8																	
7.2	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No C-12

METRIC

W P 62-82-01

LOCATION Sta. 34 + 205; O/S 36.4 m RT & Hwy. 417

ORIGINATED BY IR

DIST 9 HWY 417

BOREHOLE TYPE Hollow Stem Augers, BXL Core

COMPILED BY IR

DATUM Geodetic

DATE 85 06 18

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100		
70.6	Ground Level													
0.0	Silty Clay with Sand trace gravel Occasional silt seams Brown		1	SS	49		70							
69.5														
1.1	Heterogeneous Mixture Clay, Silt Sand, Gravel Stiff Dark Grey		2	SS	9		69							
			3	SS	8		68							
			4	SS	11		67							39 28 26 7
66.8			5	SS	82		66							19 37 38 6
3.8	Highly Weathered Shale Bedrock Slightly to Moderately Weathered		6	RC BXL	REC 100%		65							RQD = 6%
64.8														
5.8	End of Borehole													

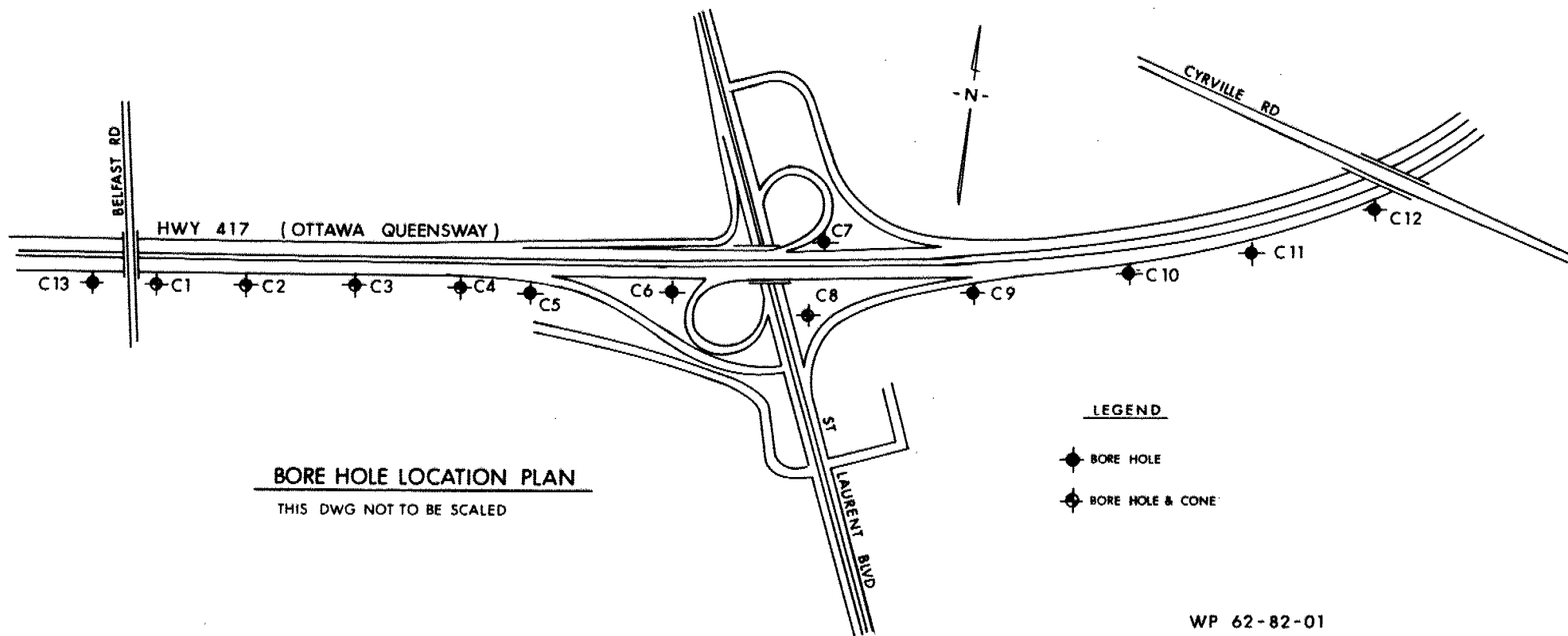
OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No C-13

METRIC

W P 62-82-01 LOCATION Sta. 32 + 418; O/S 58.0 m RT & Hwy. 417 ORIGINATED BY LP
 DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Augers, BXL Core COMPILED BY IR
 DATUM Geodetic DATE 85 06 12 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100									
								SHEAR STRENGTH									
								○ UNCONFINED	+ FIELD VANE								
								● QUICK TRIAXIAL	x LAB VANE								
								WATER CONTENT (%)									
63.8	Ground Level																
0.0	Silt, trace clay sand with sand seams and pockets of silty clay Loose to Compact																
			1	SS	12		63										
			2	SS	15		62							3 4 83 10			
			3	SS	7		61										
			4	SS	6		60							6 16 64 14			
60.2			5	SS	2		59										
3.6	Soft Heterogeneous Mixture Clay, Silt Sand, Gravel with random silt seams Very Stiff to Hard		6	SS	22		58							28 40 24 8			
			7	SS	43		57							30 37 23 10			
			8	SS	21		56										
			9	SS	37		55										
			10	SS	48		54							42 43 13 2			
			11	SS	75/8 cm		53										
55.3			12	RC BXL	REC 100%		52							RQD = 53%			
8.5	Shale Bedrock Slightly Weathered Unweathered		13	RC BXL	REC 100%		51							RQD = 53%			
52.3							50										
11.5	End of Borehole						49										



WP 62-82-01



Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO

McCORMICK RANKIN & ASSOCIATES LTD.

PRELIMINARY

GEOTECHNICAL CONSIDERATIONS

EAST TRANSITWAY CORRIDOR

RIVERSIDE DRIVE TO BLAIR ROAD

STAGE 1 - STATION 8+300 TO 11+310

REGIONAL MUNICIPALITY

OF OTTAWA-CARLETON

Distribution:

6 copies - McCormick Rankin & Associates Ltd.

Ottawa, Ontario

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November 1983

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ABSTRACT

Part I of this report presents the factual results of subsurface borings and data compilation carried out along the proposed East Transitway Stage 1 alignment between Riverside Drive and Michael Street in the Regional Municipality of Ottawa-Carleton, Ontario. Part II provides general geotechnical recommendations for planning and/or preliminary design purposes.

The subsurface conditions along the proposed East Transitway Stage 1 route consist of some 1 to 6 metres of recent fill material overlying variable and discontinuous deposits of clayey silt, sandy silt, silty sand, and/or fine to medium sand. These deposits are in turn underlain by a 1 to 5 metre thick stratum of glacial till which mantles shale bedrock. The surface of the shale bedrock varies from about 1.5 to 9.5 metres below existing ground surface along the route between Riverside Drive and Michael Street. Stabilized groundwater levels in this area are typically in the order of 1 to 6 metres below ground level.

The East Transitway within the study area will consist of a mixture of elevated, at grade, and depressed sections and as such, overpass, underpass, and retaining wall structures will be required at various locations along the route. No problems are anticipated with the elevated or at grade sections of the roadway. However, the depressed sections will involve both overburden and bedrock excavation to well below the groundwater level in many areas.

Where space permits, both short term and long term overburden slopes may be left in open cut at the recommended side slopes. Full height temporary shoring will be required for overburden support where space restrictions dictate near vertical cuts. Excavation below the groundwater level will, in all cases, require prior groundwater lowering and control.

It is considered that the shale bedrock along the route may be cut near vertically but, because of its marked degradation potential upon exposure, will need immediate protection both in the short and long term. Wire mesh, rock bolts, and shotcrete may be used in the short term while anchored wire mesh and shotcrete, concrete retaining walls, or facing panels will be required for long term support. Preliminary recommendations are also given for overburden slope/bedrock cut setback dimensions and rock catchbench requirements.

It is considered that the majority of the retaining wall, bridge, and underpass structures along the route may be founded on spread footings resting on either compact to dense overburden materials or on the shale bedrock. Subexcavation and/or the use of piled foundations may be required in localized areas where these structures are underlain by loose or compressible materials. As outlined in the report, concrete retaining wall structures should not be planned at the crest of rock cuts unless significant precautions are taken.

Preliminary recommendations are given for flexible pavement design for the transitway structure, and for the associated realigned Queensway ramps, new bus loops, and station access roads.

Additional detailed subsurface information will be required as the East Transitway Stage 1 facilities go to final design. An outline of the recommended investigative work is given in this report.

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1. INTRODUCTION

Golder Associates has been retained by McCormick Rankin & Associates Ltd., Consulting Engineers to the Regional Municipality of Ottawa-Carleton, to carry out a preliminary geotechnical investigation along the proposed corridor for the East Transitway from Riverside Drive to Michael Street. The purpose of this investigation was to confirm and summarize the general soil, bedrock, and groundwater conditions along the proposed corridor by means of both a limited number of detailed, but widely spaced, boreholes and by compiling existing available subsurface information within and adjacent to the proposed corridor area. Based on an interpretation of this factual data, an evaluation was to be carried out to outline general geotechnical considerations, concerns, and constraints to a degree sufficient to allow alignment, grade, and design plans to be formulated and recommended. Any special geotechnically related construction considerations which could have a significant influence on planning or design decisions were also to be outlined.

Verbal authorization to proceed with this investigation was given on January 18, 1983. The section of the East Transitway under study in this report is Stage 1, extending from Riverside Drive at Station 8+300 to Michael Street at Station 11+310.

The report is presented in two parts. Part I details the subsurface conditions encountered along the proposed East Transitway, Stage 1 corridor while Part II provides our general geotechnical recommendations for planning and/or preliminary design purposes.

PART I

SUBSURFACE CONDITIONS

2. DESCRIPTION OF PROJECT

The section of the proposed East Transitway under study in this report extends from near Riverside Drive in the west (Station 8+300) to Michael Street in the east (Station 11+300), as shown on the attached Key Plan, Figure 1.

As presently recommended, the East Transitway corridor will run north from Riverside Drive along the west side of the Canadian National/Canadian Pacific railway tracks to cross over Alta Vista Drive immediately north of the railway bridge and south of the Ottawa Queensway ramps. The route will then run eastwards immediately in front of the Ottawa train station in cut section, crossing under the station Ring Road system and swinging north to cross under Tremblay Road at the intersection with Pickering Place. North of Tremblay Road, the corridor will again swing east to parallel the south side of the Ottawa Queensway, being basically in cut section and crossing under Belfast Road at the south end of the existing Queensway-Belfast Road bridge. Approximately 500 metres east of Belfast Road, the route will swing north to cross under the Ottawa Queensway via a long underpass structure. The corridor will then stay depressed, running parallel to the north side of the Queensway, crossing under St. Laurent Boulevard and returning to near grade at Michael Street.

As indicated above, the East Transitway within the study area will consist of a mixture of elevated, at grade, and depressed sections. As such, overpass, underpass, and retaining wall structures will be required at various locations along the route. In particular, bridge structures will be required at Alta Vista Drive and ultimately at the future Industrial Avenue (at present Transitway/Riverside Drive intersection). Underpass structures will be required at the Ottawa Station Ring Road system, at Tremblay Road,

at Belfast Road, and at St. Laurent Boulevard. As well, a long covered underpass structure will be required under the Queensway and within a proposed Transitway Station area immediately west of St. Laurent Boulevard. In addition, several hundred metres of retaining wall structures will be required along this section of the route.

In keeping with the present transitway design, it is expected that the transitway will be of two-lane paved cross-section with associated curbs, shoulders, ditching and/or storm water handling facilities. As well, several kilometres of realigned Queensway ramps, new bus loops and station access roads will be constructed in association with this section of the East Transitway.

3. DESCRIPTION OF SITE AND GEOLOGY

The topography along the proposed East Transitway corridor between Riverside Drive and Michael Street is relatively flat; the minor undulations which do occur along the route being generally man-made due to the predominantly built-up nature of the area. Overall, the ground surface shows a gentle rise to the east, from about elevation 60 to 61 metres at Riverside Drive to elevation 69 to 70 metres at Michael Street.

Published surficial soils information and previous experience in this general area indicates that the transitway corridor runs across a glacial till plain, characterized by variable thicknesses of glacial deposits (silts, sands, and glacial till). These native deposits are often overlain by several metres of recent fill materials. The bedrock along the corridor is indicated to consist of black fissile shale of the Billings formation. Geological evidence indicates the possibility of minor faults or fault zones within the shale, oriented in a generally northwest-southeast direction and being near vertical to steeply inclined.

4. INVESTIGATION PROCEDURES

4.1 Previous Investigations

Several previous investigations have been carried out within and/or adjacent to the corridor area by both Golder Associates and others. The majority of these investigations were carried out several years ago in association with the design of the Ottawa Queensway structures. A detailed review of these investigations was carried out as part of the present work, and the locations of pertinent boreholes have been plotted on the Boring Plan, Figure 2. As well, the subsurface stratigraphy shown as being encountered in these previous borings has been plotted onto the subsurface profile shown on Figure 3. It should be noted that in several cases the stratigraphic terminology used on the previous logs has been adjusted for clarity and conformity with the present investigation. If specific details from the previous investigations are required, these may be obtained by reading the respective previous reports outlined in the attached Table I.

4.2 Present Investigation

Based on the results of the previous investigations, a program of seven (7) widely spaced borings was laid out along the Stage 1 route between Riverside Drive and Michael Street. These borings were put down between February 22 and March 9, 1983 using a track mounted hollow stem auger machine supplied and operated by the Marathon Drilling Co. Ltd. of Ottawa. Following evaluation of the results of these borings with respect to the proposed transitway alignment, a further program of six (6) borings was laid out to infill the existing data. These additional borings were put down between November 14 and 18, 1983. Standard penetration tests were carried out in the overburden soils in all of the borings at near continuous intervals of depth and samples of the subsoils encountered recovered using standard drive open sampling equipment. All of the borings were taken to auger refusal and the bedrock proven

in all but borehole 83-17 either by augering or by diamond core drilling for depths of 3 to 6 metres. During the core drilling a careful record was kept of drill water return, drilling pressure, rock core recovery, and R.Q.D. (Rock Quality Designation). Standpipes were sealed into each boring in order to measure the stabilized groundwater level(s) along the route. The field work was carried out under the full time supervision of a member of our engineering staff.

The subsoil and bedrock samples recovered were carefully packaged and returned to our laboratory for detailed examination and classification testing. The results of the laboratory testing are shown on the respective Record of Borehole sheets and on Figures 4 to 9.

A detailed log of each borehole put down at this time along this section of the transitway corridor is given on the Record of Borehole sheets following the text of this report. It should be noted that boreholes 83-8 to 83-12 inclusive were put down within the Stage 2 portion of the East Transitway as part of the present field program. The results of these borings will be presented in a subsequent report for the Stage 2 section of the East Transitway. The borehole locations, with respect to existing site features, are shown on the Boring Plan, Figure 2. The Subsurface Profile, Figure 3, presents a vertical plot of the subsoils along the route, as encountered during the present investigation and as indicated by previous borings within the corridor area.

The ground surface elevations at the present borehole locations have been extrapolated from the large scale topographic maps used to locate the borings in the field. The elevations shown for the previously drafted borehole logs have, in most cases, been converted from Imperial units. It should be noted that the present ground surface elevations, and possibly the present surficial soil stratigraphy, may have changed since the time of these past investigations.

5. SUBSURFACE CONDITIONS

5.1 General

The detailed stratigraphy encountered in each borehole is shown on the Record of Borehole sheets and is illustrated on the Subsurface Profile, Figure 3.

In summary, the subsurface conditions along the proposed Stage 1 route of the East Transitway consist of some 1 to 6 metres of recent fill material overlying variable and discontinuous deposits of clayey silt, sandy silt, silty sand, and/or fine to medium sand. These deposits are in turn underlain by a 1 to 5 metre thick stratum of glacial till which mantles shale bedrock. The surface of the shale bedrock varies from about 1.5 to 9.5 metres below existing ground surface along the Stage 1 corridor between Riverside Drive and Michael Street.

Following is a brief detailed description of the subsurface conditions encountered in the thirteen boreholes along the route.

5.2 Fill, Topsoil, Peat, Alluvium

All of the boreholes put down along the route encountered surficial deposits of recent fill. The fill materials were found to range widely in texture, density, and thickness; from some 0.6 metres of crushed stone in borehole 83-5, through about 2.9 metres of silty sand embankment material in borehole 83-6, to some 6.1 metres of silty clay and silty sand fill in borehole 83-1. The results of a grading analysis carried out on a sample of the fill material recovered from borehole 83-1 are shown on Figure 4. Standard penetration tests carried out within the fills along the route gave N values ranging from 1 to 18 blows per 0.3 metre, indicating the generally very loose to compact density of these materials. In several of the borings, the fill deposits were found to be underlain by the original organic topsoil layer, while in others the fill had apparently been covered with a layer of topsoil.

In all cases the topsoil was limited in thickness to about 90 to 250 millimetres. In borehole 83-2, the embankment fill was found to be underlain by about 1.2 metres of stiff dark brown fibrous peat. This deposit is indicated to be limited in areal extent as it was not encountered in other borings put down in the area. Likewise, the fill in borehole 83-13 was underlain by an alluvial deposit consisting of some 0.6 metres of organic clayey silt with wood.

5.3 Clayey Silt, Sandy Silt, Silty Sand, Sand

Variable and discontinuous deposits of clayey silt, sandy silt, silty sand, fine sand, and fine to medium sand were encountered below the fills in several of the borings along the route. In general, these deposits were in the order of 0.5 to 5 metres thick where encountered and were often interlayered. Grading analyses carried out on representative samples of sandy silt, silty sand, fine sand, and fine to medium sand recovered from the boreholes are shown on Figures 5 to 8 and indicate the generally fine grained and uniform nature of these deposits. Standard penetration resistance N values of about 7 to 25 blows per 0.3 metres indicate that the silts and sands are in a generally loose to compact state of packing.

5.4 Glacial Till

Glacial till was encountered in all the borings put down along the route, either directly below the fill materials or underlying the silt and sand strata. The glacial till mantles the underlying bedrock and was found to range significantly in thickness at the borehole locations, from only about 0.4 metres thick in boreholes 83-7 and 83-15 to about 4.8 metres thick in borehole 83-3. The till consists basically of sandy silt in a matrix of clay, gravel, cobbles and some boulders. Grading analyses carried out on representative 40 millimetre I.D. split barrel samples of the till are shown on Figure 9. Standard penetration tests carried out within the glacial till gave N values ranging from 2 to 97 blows per 0.3 metres, indicating

the quite variable, very loose to very dense, nature of this deposit.

5.5 Shale Bedrock

Bedrock was encountered in all but one of the boreholes put down during the present investigation at depths ranging from 1.2 to 1.9 metres (boreholes 83-15 and 83-7) to 9.5 metres (borehole 83-13) below present ground surface. As well, a very small exposure of bedrock exists near borehole 83-7, immediately adjacent to the north end of the Michael Street pedestrian underpass structure. The previous borings put down within the corridor area recorded similar depths to the bedrock. These depths correspond to an elevation range of from 55 to 56 metres at the west end to about 66 to 69 metres at the east end of the corridor area. In many of the borings it was possible to auger into the upper highly weathered surface of the shale. The bedrock core retrieved from the present boreholes is typical of the Billings formation comprising dark almost black fissile shales with a few intercalated bands of light grey microcrystalline limestone and calcareous siltstone. Throughout most of the area the Billings formation shale is near horizontally bedded.

A measure of the quality of the bedrock core retrieved from the boreholes is shown on the respective Record of Borehole sheets in the form of percent core recovery and R.Q.D. The percent core recovery attained in the boreholes was generally high, being generally in the order of 97 to 100 percent, with the majority of the core recovered being solid core. R.Q.D. is defined as the total length of intact core "sticks" which are greater than 100 millimetres in length divided by the total core run length drilled, expressed as a percentage. Because the shale bedrock in this area was known to have a tendency to break and crack along the bedding planes when subjected

to cyclical changes in moisture content, two R.Q.D. values have been determined; one was made in the field immediately upon removing the core from the core barrel and a second was made later in the laboratory after the core had dried out. These values are shown on the Record of Borehole sheets for each core run. As can be seen, R.Q.D. values "after coring" were found to vary considerably (from 0 to 98 percent) but to be generally in the order of 25 to 75 percent. However, the R.Q.D. "after wetting and drying" was consistently zero percent, indicating the marked susceptibility of the shale to degradation and slaking, which preferentially develops along microfabric weaknesses and bedding planes within the rock material.

In some of the boreholes, the core recovered exhibits vertical to near vertical joints. Where tight, such joints were often infilled with calcite. In other cases broken zones occur along such joints. However, often in these structures limonate coatings are evident. Notable in this respect is the core from borehole 83-7 in which several limonate stained joints were encountered. As well, in several of the borings, weathered and fractured seams up to 100 millimetres thick were noted throughout the cored depth. Although the shale bedrock in this area is essentially horizontally bedded (bedding generally at less than 5 degrees from horizontal), the core from borehole 83-4 indicates the possible presence of minor faulting or zones of glacial disruption (apparent dip of bedding in borehole 83-4 was 25 to 30 degrees from horizontal). This bedding dip, along with the general trends of faulting shown on the regional geological mapping, would suggest the possible presence of northwest trending faulting in the area.

5.6 Groundwater

The groundwater conditions along the proposed Stage 1 East Transitway corridor were observed within the open boreholes during drilling and within standpipes installed in the completed borings. The stabilized groundwater level as measured in the standpipes at the time of this investigation in late February and mid-November 1983 ranged from about 0.5 to 5.8 metres below existing ground surface. These depths correspond to elevations of between 58 to 60 metres near Alta Vista Drive (west end of Stage 1), rising to between 67 to 69 metres at Michael Street (east end of Stage 1). In general, the groundwater level was found to be within the lower portion of the deeper fill deposits, or within the underlying silts, sands, or glacial till. In areas where the bedrock surface is at a shallow depth, the stabilized groundwater level was at or within the upper part of the shale bedrock.

PART II

PRELIMINARY DESIGN CONSIDERATIONS

6. INTRODUCTION

This section of the report provides our geotechnical engineering recommendations for preliminary design and for planning of the proposed Stage 1 section of the East Transitway between Riverside Drive and Michael Street, together with our comments on special construction considerations which may influence design decisions.

The information on subsurface conditions obtained for purposes of this study is of a general nature, based on widely spaced borings and available previous data. At present, no site specific details have been given on proposed structures. As such, the following discussions and recommendations should be considered as preliminary and are subject to revision pending the results of detailed investigations during the final design stage.

7. GENERAL

Overall preliminary evaluation of the East Transitway Stage 1 route suggests that there should be no major geotechnical constraints to the alignment, grade, or design as presently recommended and as shown on the attached Figures 2 and 3. Although a number of significant geotechnical considerations and concerns are outlined in subsequent sections of this report, the subsurface conditions, being somewhat consistent along the length and width of the proposed corridor, suggest that changes in alignment and grade, though solving some concerns, could simply create the same or new concerns in different areas. A possible exception would be the grade of the East Transitway in the vicinity of the Ottawa train station. As the grade drops from Alta Vista Drive, unacceptable fill and/or alluvium materials will be encountered at subgrade level requiring some subexcavation. A slight lowering of the grade in this area could possibly reduce the amount of required subexcavation. However, based on the results of this preliminary investigation, we do not feel there are any geotechnical reasons for shifting the horizontal alignment from that proposed.

8. FILL SECTIONS

Embankment approach fills up to 6 metres in height, will be required at the Transitway/Alta Vista Drive and (ultimately) at the Transitway/Industrial Avenue bridge structures. The section of the Transitway between these two structures will also be in fill section. As well, embankment fills will be required adjacent to the pedestrian underpass structure at Michael Street and along at least some of the realigned Queensway ramps and new bus loops and access roadways. No major problems are envisaged at this time in the construction of these embankments provided they are constructed of acceptable earth borrow, granular material, or rock fill, adequately compacted in place with stable side slopes or retaining wall support where appropriate. It is considered that embankment side slopes of 2 horizontal to 1 vertical should be stable in the long term, provided they are properly seeded and/or sodded for erosion protection.

Some minor settlement of the embankments may be experienced due to densification of either the loose fills or the loose to compact silts and sands which are considered to exist below much of the proposed embankment area. Although the majority of this settlement should take place during and immediately following the construction period, consideration could be given to delaying final paving for several months following embankment construction in order to limit possible long term pavement distress. At the Alta Vista Drive bridge approaches, the proposed embankments will encroach on the existing railway bridge approach fills. Additional investigation may be required in this area to determine what, if any, effect the new fills may have on the existing approach fills and/or on the performance of the existing railway bridge structure.

9. CUT SECTIONS

The majority of the East Transitway between Alta Vista Drive and Michael Street will be in cut section, with cuts of up to 8 to 9 metres below existing grade.

9.1 Overburden

Overburden excavation for the cut sections will be mainly through deposits of loose miscellaneous fill, loose to compact silts and sands, and loose to very dense glacial till. In many areas, the excavations will extend to well below the groundwater level within these essentially granular overburden deposits.

No unusual problems are anticipated in excavation of the overburden deposits above the groundwater level. Where space permits, overburden deposits above the groundwater level may be excavated in open cut using short term construction side slopes of about 1.0 horizontal to 1 vertical. Where overburden slopes above the groundwater level are to be left in open cut in the long term, slopes of 2.0 horizontal to 1 vertical (or flatter) should be used in design.

Excavation of the overburden deposits below the groundwater level will present some constraints. Groundwater inflow to excavations, if left uncontrolled, could also result in significant soil inflow, with subsequent undermining of the excavation slopes and/or adjacent structures. Again, where space permits, short term construction slopes in the order of 1.0 to 1.5 horizontal to 1 vertical may be attainable provided groundwater lowering procedures have been completed prior to excavation. This will be especially important in the Ottawa station area where the excavations will extend to well below the present groundwater level within fine, and fine to medium sands which are presently under an excess hydrostatic pressure head. Wellpoints and/or pre-augered gravel packed deep wells will probably be required for effective drawdown.

Where overburden slopes below the present groundwater level are to be left exposed in the long term, slopes of 2.5 horizontal to 1 vertical (or flatter) should be used in design. However, as for the short term case, groundwater lowering procedures will have to be used prior to excavation below the groundwater level. Following completion of the excavation and the installation of permanent groundwater control facilities (roadway subdrains, slope toe drains, and/or french drains within the slopes), the short term groundwater lowering procedures may be systematically shut down and the groundwater level allowed to restabilize within the cut slopes.

The main area in which groundwater lowering within the overburden will be required is adjacent to the Ottawa train station between about Station 9+200 to 9+800. Groundwater control will also be required within the overburden near the Belfast Road underpass at about Station 10+000 to 10+100.

In many areas along the route, space restrictions will be such that short term vertical overburden cuts will be required and long term support will be attained using full height retaining walls. In these areas, full height temporary shoring will be required for overburden support. Either the use of soldier piles and lagging in conjunction with groundwater lowering, or the use of interlocking steel sheet piling should provide the necessary support. Depending on the height of these temporary walls, deadman anchors, tie-backs, or raker supports may also be required in the short term.

A possible alternative to the above-noted shoring systems would be the use of "slurry wall" techniques. This could be a viable alternative especially where excavation of the shale bedrock is required below the overburden cut. As outlined in a later section, the ability of the shale bedrock to provide adequate toe support for the temporary

shoring system may be questionable, and it may therefore be necessary to extend the shoring system full depth to the base of the rock cut. In this case, consideration might be given to the use of full height slurry wall construction techniques.

Where long term open cuts are bottomed in overburden, a system of toe drains or subdrains will be required for cut slope and roadway drainage purposes. Sodding or seeding of all overburden slopes should be carried out to protect against erosion due to surface and groundwater runoff and ditching should be provided along the top of the cut to lead surface water runoff away from the cut. Where long term open cuts are bottomed in bedrock, the toe of the overlying overburden slope will have to be set back at least 2 to 3 metres from the face of any subsequent rock cut to provide sufficient space for the rock excavation and/or to ensure that undercutting of the overburden slope does not occur due to deterioration of the exposed rock face. As well, a drain should be provided along the interface between the overburden and the underlying bedrock to intersect water inflow at this point.

It should be noted that the relatively large scale groundwater control procedures that may be required along portions of the route could have a significant effect on adjacent structures. At the final design stage it will be necessary to determine in some detail the founding conditions of adjacent structures, and to evaluate the effect the proposed transitway construction may or may not have on these structures. In particular, the groundwater lowering could increase the effective stress on the subsoils in the area, resulting in subsoil compression with consequent settlement of adjacent structures. As well, the areal extent of the influence of the proposed groundwater lowering could be significant, depending on the continuity, hydraulic conductivity, and permeability of the subsurface strata. Monitoring of a full scale pumping test carried out in the overburden during the final design stage of the

investigation will be necessary to define the area of influence of the groundwater lowering.

9.2 Bedrock

Bedrock excavation will be required in the Queensway Underpass area, as well as within and east of the St. Laurent Station area. In addition, a minor amount of rock excavation will be required for the pedestrian underpass structure in front of the Ottawa train station. Cuts of up to 5 metres below the bedrock surface will be required in the St. Laurent Boulevard area.

The bedrock in this area, below a surface zone of highly weathered material, consists of a thinly laminated dark grey to black fissile shale and evidence of some fracturing was noted along fissile bedding planes throughout the core recovered. As well, the shale is known to slake perceptibly and to degrade significantly when exposed to the atmosphere for any length of time. This thinly bedded and jointed rock type is known, from past experience, to be highly susceptible to disturbance (overbreak and underbreak conditions) from excavation procedures. It must therefore be noted that the overall appearance and stability as well as the short term and long term durability of the transitway rock cuts will be controlled almost exclusively by the quality of the excavation practices undertaken by the contractors during construction. Given the known questionable quality of the bedrock excavation attained in the Ottawa area in the recent past, it will be necessary to stipulate and enforce very strict bedrock excavation performance specifications in order to ensure that the transitway design details are not jeopardized by the excavation procedures used.

It is considered that the upper surface of weathered shale, and at least some of the moderately weathered and fractured shale may be excavated using large hydraulic earth excavation techniques (backhoe), possibly in conjunction with line drilling techniques to define the excavation limits. Alternatively, drilling and blasting procedures will be required. Because of the sensitive nature of the shale bedrock, consideration will have to be given to line drilling, pre-shearing, and split second firing techniques. As well, stringent blasting control will be required in terms of blast hole spacing, charge weights, weight per delay, and peak particle acceleration. Without such control, it is conceivable that undermining of adjacent structures (Queensway foundations) could occur.

It is considered that the shale bedrock may be cut near vertically in the short term, but because of its degradation potential upon exposure, will need immediate protection in the form of wire mesh, rock bolts, and/or shotcrete. The use of tie-back anchors could also be necessary in some areas.

Where rock cuts are to be left open and exposed in the long term, stable slope angles could well exceed 45 degrees since the only structural weakness in the rock mass is the near horizontal bedding. (Exception would be in the area of borehole 83-4 where relatively steep bedding dips were noted, possibly associated with past minor faulting or with zones of glacial disruption). However, as for the short term case, protection of these rock cuts will be necessary to avoid rapid breakdown and weathering upon exposure. If long term rock slopes steeper than about 1 horizontal to 1 vertical are required due to space restrictions, this protection may have to be structural in nature, i.e. anchored wire mesh and shotcrete, concrete retaining walls, or facing panels.

Alternatively, rock slopes of about 1.0 to 1.5 horizontal to 1 vertical could be employed along with slope protection using gravel, paving stones, or the like. Rock slopes of 2 horizontal to 1 vertical or flatter may either be left exposed or covered with a layer of topsoil and seeded.

To avoid undue maintenance problems, it is recommended that a catchbench be provided along the toe of all rock cuts, and as outlined previously, that the toe of the overlying overburden cut be set back at least 2 to 3 metres from the crest of the rock cut. Details on the actual size and shape of these benches could be provided once final grades and slope angles are known.

Groundwater inflow into bedrock excavations at this site may be expected along major joints, bedding planes and possible faults. The results of limited packer tests carried out in borehole 83-18 indicate that the bedrock is relatively permeable in this area. To reduce the potential for groundwater movement along bedding planes and down the face of the rock cut, horizontal drains should be drilled into the rock at the toe of the cut and led to a conventional subdrain system. As for the overburden areas, the effect of this groundwater lowering on adjacent structures will have to be assessed.

Considerable care will have to be exercised where rock cuts (either short term or long term) are required adjacent to temporarily shored overburden excavations. If space restrictions necessitate near vertical rock cuts close to the toe of shored excavations, back-break of the shale to the shoring supports will undoubtedly occur. In this case, the overburden shoring system will have to be extended full depth to the base of the rock cut. Alternatively, where space permits, it may be possible to provide an adequate overburden wall/bedrock cut setback to provide anchorage support for the toe of the overburden wall, and

to protect the rock face with shotcrete and/or tie-back anchors.

Detailed exploration of subsurface conditions in the vicinity of St. Laurent Boulevard will be required in order to optimize excavation procedures for constructing the complex system of ramps and intersections shown in this area.

10. RETAINING WALLS

10.1 Ottawa Train Station Area

An extensive retaining wall system will be required in the above area for the support of the overburden cuts adjacent to the station structure, at the station Ring Road and Tremblay Road underpass structures and along the west side of Pickering Place. Reinforced concrete retaining walls may be used in this area.

The results of boreholes 83-1, 83-3, 83-16 and 83-17 put down in this area indicate that loose to compact fine sand or glacial till will exist at retaining wall founding level at a depth of about 1.5 metres below transitway grade throughout this area. Since these materials are relatively compressible at low bearing pressures, it will be necessary to either deepen the footing excavations to reach dense glacial till (indicated to exist at a relatively shallow depth below founding level) or to found the retaining walls on end bearing piles. For preliminary purposes, retaining wall footings bearing on dense glacial till (N values greater than 30 blows per 0.3 metres) may be designed using allowable bearing pressures for the serviceability limit state Type II and ultimate limit state cases of about 200 and 750 kilopascals, respectively. Alternatively, nominal 300 millimetre sized H-piles driven to practical refusal may, for preliminary purposes, be designed for an allowable load of 900 kilonewtons per pile. Ministry of Transportation and Communications (M.T.C.) standards for backfilling and drainage procedures should be used in the design of these retaining walls.

10.2 Queensway-Belfast Road Area

Retaining wall structures will be required at the Belfast Road Underpass and adjacent to the Queensway west of Belfast Road.

Boreholes 83-3, 83-4 and the previous borings at the Belfast/Queensway bridge indicate that loose to compact glacial till should be encountered at founding level. It is considered that the retaining walls in this area could be founded on the compact glacial till (N values greater than 10 blows per 0.3 metres) using allowable bearing pressures of some 100 and 350 kilopascals for the serviceability limit state Type II and ultimate limit state, respectively, or alternatively, on end bearing H-piles at the allowable load outlined previously.

10.3 Queensway-St. Laurent Boulevard Area

An extensive retaining wall system will also be required in the Queensway-St. Laurent Boulevard area, at the entrance to the Queensway underpass structure, within the St. Laurent Station area, and on both sides of the Transitway between St. Laurent Boulevard and Michael Street.

Based on the results of this investigation (boreholes 83-5, 83-6, 83-7, and 83-18), virtually all the retaining walls in this area should bear directly on the shale bedrock. For preliminary design purposes, retaining walls founded on spread footings on or within the moderately weathered shale bedrock should be designed using a maximum factored bearing capacity at the Ultimate Limit State of 2000 kilopascals.

In conjunction with the comments made in section 9.2 on the rapid weathering potential of the shale bedrock in this area, retaining wall structures should not be planned at the crest of the rock cuts unless,

- a) the excavated rock slope is flatter than 1 horizontal to 1 vertical, and
- b) the rock slope is immediately protected with concrete, shotcrete, paving stones, etc., and

- c) a minimum setback of 3 metres is maintained between the outside edge of the retaining wall footing and the crest of the rock cut.

Where retaining walls are used to support rock cuts, and where the concrete is cast directly against the rock face, provision for drainage must be provided, in the form of a system of proprietary filter drains evenly spaced down the rock face and connected to the storm drain system. Otherwise, standard granular backfill and drainage procedures should be used for all retaining wall design.

It should also be noted that the shale in this area is known to have some swelling potential. If concrete is cast directly against the rock, lateral movement of the shale layers could result in excess lateral pressures behind the retaining walls. Depending on the stress state and swelling properties of the shale, stress relief in the form of weak styrofoam or clean sand may be required between the rock face and the back of the concrete wall.

11. STRUCTURES

11.1 Bridges

Bridge structures will be required to carry the Transitway over Industrial Avenue (ultimately) and over Alta Vista Drive. This preliminary investigation has indicated that shale bedrock should exist within 1 or 2 metres of founding level for these structures. As such, a preliminary bearing pressure of 2000 kilopascals may be used for the ultimate limit state design for footings bearing directly on relatively sound shale bedrock. Consideration may have to be given to cleaning off degraded shale from founding areas prior to concrete pouring. Air-water lance techniques should be applied in conjunction with dozer excavation in order not to excessively disturb the bedrock. As was the case for the retaining walls, control of heave and swelling may need to be addressed in the design of these bridge structures.

11.2 Underpasses

As presently envisaged some six underpass structures will be required along the route. As well, the pedestrian tunnel at the Ottawa train station and the St. Laurent Station structure itself will in effect be underpass structures.

The Ring Road, Tremblay Road, and Belfast Road underpass structures may be founded on spread footings resting on compact glacial till at allowable bearing pressures of 100 kilopascals and 350 kilopascals for the serviceability limit state Type II and ultimate limit states, respectively. As indicated by the widely spaced borings put down during this investigation, minor amounts of subexcavation of loose materials (fill, sand, till) may be necessary in these underpass areas in order to reach competent bearing strata. Alternatively, these structures could be founded

on end bearing H-piles at the same allowable load of 900 kilonewtons per pile as outlined for the retaining walls in this area.

Special care will have to be exercised in the Belfast Road area where the proposed transitway underpass structure will abut the south end of the Queensway-Belfast Road bridge structure. The underpass structures at the Ottawa Station, the Queensway, and in the St. Laurent Station area may be founded directly on the shale bedrock at bearing values equal to those given previously for competent shale bedrock.

On the basis of past laboratory testing carried out on similar shales in the Ottawa area, it is anticipated that heaving and/or lateral swelling of the shale could take place upon exposure in rock cuts. As well, water seepage through the rock faces may be expected. Consideration may therefore have to be given to not using the cut rock faces as back or bottom forms for the concrete underpass structures; rather the shale may have to be overexcavated to allow the placement of weak styrofoam or clean sand as a stress relief system.

Standard backfill and drainage provisions should be provided for the underpass structures. As well, frost tapers will be required where structures are to be built at shallow depth below existing roadways.

12. PRELIMINARY PAVEMENT DESIGN

Since the Transitway, as well as the realigned ramps, bus loops, and access roadways will be in a combination of cut, fill, and transition sections, subgrade conditions will vary along the route. In keeping with the present transitway design, it is recommended that flexible pavement design be used throughout, employing the following preliminary design thicknesses.

a) Transitway

140 millimetres hot mix asphalt over

150 to 225 millimetres of M.T.C. Granular A over

450 to 600 millimetres of M.T.C. Granular B

b) Ramps, Loops and Access Roadways

90 to 140 millimetres of hot mix asphalt over

150 to 225 millimetres of M.T.C. Granular A over

300 to 450 millimetres of M.T.C. Granular B

The lower subbase thicknesses may be used in fill section where the embankment fills are comprised of acceptable granular material (select subgrade); the greater thicknesses being used along at grade and cut sections of the route. Where the subgrade consists of bedrock, consideration may be given to replacing the Granular B subbase with an equivalent thickness of rock shatter.

Full length subdrains will be required within the cut and at least some of the at grade sections of the route. Standard ditching criteria may be used along many of the realigned Queensway ramps.

13. FURTHER INVESTIGATION

As outlined in the report, a number of geotechnical concerns have been identified and preliminary recommendations given. Additional subsurface information will be required as the East Transitway Stage 1 facilities go to final design. In general, consideration will have to be given to the following investigative work.

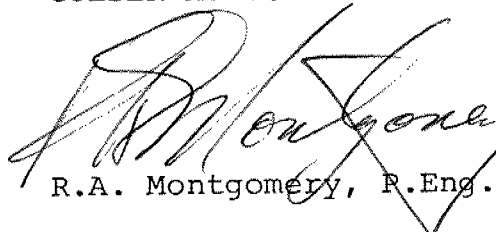
- a) additional borings along all sections of the transitway route as well as along all realigned Queensway ramps, bus loops, and access roadways to confirm the continuity of the soil, bedrock, and groundwater conditions as they relate to both design and construction,
- b) additional detailed borings at each structure location (retaining wall, bridge, underpass) to confirm subsurface conditions and to provide detailed geotechnical parameters for final design purposes,
- c) careful exploration of bedrock conditions and properties in the vicinity of the St. Laurent Station and St. Laurent Boulevard areas in order to optimize excavation and design procedures for constructing the complex system of ramps, intersections, and structures planned for this area,
- d) the installation of additional permanent groundwater level monitoring devices in cut areas, to provide up to date groundwater level data for evaluation of construction and long term drainage and groundwater control requirements,
- e) at least one large diameter borehole should be put down in the St. Laurent Station area in order to recover large diameter core for assessing the swelling and stress state properties of the shale bedrock,

- f) full scale pumping tests should be carried out in 150 millimetre diameter wells put down within the overburden in the Ottawa train station area and within bedrock in the proposed St. Laurent Station area. Monitoring of the pumping tests should be carried out from observation wells to determine the area of influence of any proposed groundwater lowering procedures,
- g) the testing of groundwater samples from the area should be carried out to assess the possibility of the existence of a corrosive environment for buried concrete and/or steel elements,
- h) accurate and detailed laboratory analyses should be carried out on representative subsurface samples to provide the necessary design parameters,
- i) a full scale test excavation should be carried out, either at final design or pre-contract time, to determine the excavation concerns and problems associated with both the overburden and bedrock along the route,
- j) the founding conditions of all adjacent structures should be determined and evaluated with respect to the proposed transitway construction.

As well, any other geotechnical concerns which come to light during the final design stage should be investigated as required. All of the data obtained, factual as well as interpretative, should be presented in detailed geotechnical reports for use by the design engineers.

We trust that this preliminary report and attachments provide the information you require at this stage. Should you have any questions concerning the geotechnical concerns and preliminary recommendations outlined in this report, please call us.

GOLDER ASSOCIATES


R.A. Montgomery, P.Eng.



RAM:TGC:cn

LIST OF REFERENCE REPORTS

1. M.T.C. Report 31G5-13, July 1957, Subsurface Investigation, Tremblay Road and Ottawa Queensway
2. M.T.C. Report 31G5-14, July 1957, Subsurface Investigation, Alta Vista Drive and Ottawa Queensway
3. M.T.C. Report 31G5-15, June 1957, Subsurface Investigation, St. Laurent Boulevard and Ottawa Queensway
4. M.T.C. Report 31G5-16, June 1957, Subsurface Investigation, Avenue M (now Belfast Road) and Ottawa Queensway
5. M.T.C. Report 31G5-124, January 1959, Subsurface Investigation, Michael Street and Ottawa Queensway
6. Golder Associates Report 64047, June 1964, Subsurface Investigation, Bridge No. 44 at Tremblay Road, Ottawa Queensway
7. Golder Associates Report 64152, December 1964, Subsurface Investigation, Tremblay Road, Alta Vista South Reconstruction, Ottawa Queensway
8. Golder Associates Report 71826, January 1972, Subsurface Investigation, Proposed Pumping Station, Hurdman Bridge
9. Golder Associates Report 791-2069, March 1980, Subsurface Investigation, Proposed Riverside Drive, Smyth Road to Industrial Avenue
10. Golder Associates Report 831-2095, April 1983, Subsurface Investigation, Proposed Hurdman Station, Southeast Transitway

LIST OF ABBREVIATIONS

The abbreviation commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance:

The number of blows by a 63.6 kg (140 lb) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 0.3 m (12 in.).

Standard Penetration Resistance, *N*:

The number of blows by a 63.6 kg (140 lb) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 0.3 m (12 in.).

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

<i>Relative Density</i>	<i>'N'</i>
	<u>Blows/0.30m</u> <u>or Blows/ft.</u>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

<i>Consistency</i>	<u>kPa</u>	<i>'Cu'</i> <u>psf.</u>
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1000
Stiff	50 to 100	1000 to 2000
Very stiff	100 to 200	2000 to 4000
Hard	over 200	over 4000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

τ	= 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

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	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

III. SOIL PROPERTIES

(a) Unit weight

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_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation

(b) Consistency

w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_s	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$

(c) Permeability

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

(d) Consolidation (one-dimensional)

m_v	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_v	coefficient of consolidation
T_v	time factor = c_v / d^2 (d , drainage path)
U	degree of consolidation

(e) Shear strength

τ_f	shear strength
c'	effective cohesion
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

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

RECORD OF BOREHOLE 83-1

LOCATION See Figure 2

BORING DATE FEB. 22, 1983

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV./N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○	WATER CONTENT, PERCENT					
										Wp	W	WL			
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	63.92	GROUND SURFACE												<p>GROUND SURFACE</p> <p>SURFACE SEAL</p> <p>PLASTIC TUBING</p> <p>NATIVE BACKFILL</p> <p>STANDPIPE</p>	
	0.00	TOPSOIL													
	0.15	VERY LOOSE TO COMPACT GREY BROWN TO GREY SILTY SAND, SOME GRAVEL, SOME SILTY CLAY, TRACE ORGANIC MATERIAL (FILL)													
			1	50	9										
			2	6											
			3	5											
			4	1											
			5	1											
			6	4											
		7	11												
		57.65	COMPACT GREY FINE SAND												
	57.01	DENSE DARK GREY SANDY SILT, SOME GRAVEL (GLACIAL TILL)													
ROTARY DRILL BXL CORE	56.59	HIGHLY WEATHERED DARK GREY SHALE BEDROCK													
	56.04	FAINTLY WEATHERED THINLY LAMINATED DARK GREY TO BLACK FISSILE SHALE BEDROCK, BEDDING < 5°. OCCASIONAL THIN CALCITE FILLED SEAMS AND STRINGERS. (NOTE: SHALE SLAKES PERCEPTIBLY WITH MOISTURE CHANGES)													
			10	BXL RC											
			11	BXL RC											
	53.12	END OF HOLE													

STA. 9+430 - 10m R+E

WETTING AND DRYING

CORE RECOVERY %

R.Q.D. (%) AFTER CORING

R.Q.D. (%) AFTER 1 CYCLE WETTING AND DRYING

7.47 - 7.80 MULTIPLE FRACTURED ZONE, SOME CLAY COATINGS IN WEATHERED SURFACE ZONE

8.71 - 8.79 BROKEN ZONE

9.45 - 9.50 BROKEN ZONE ALONG FISSILE BEDDING PLANES

0
15 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50

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DRAWN J.C.
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RECORD OF BOREHOLE 83-2

LOCATION See Figure 2

BORING DATE FEB. 22, 23, 1983

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH		WATER CONTENT, PERCENT					
								Cu, kPa	NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○	Wp	W	WL			
POWER AUGER 200mm DIAM. (HOLLOW STEM)	61.81	GROUND SURFACE					STA. 9+725 - 125m LT. 4							<p>GROUND SURFACE</p> <p>SURFACE SEAL</p> <p>NATIVE BACKFILL</p> <p>PLASTIC TUBING</p> <p>STANDPIPE</p> <p>W.L. IN STANDPIPE AT ELEV. 57.65 MAR. 16, 1983</p>	
	0.00	TOPSOIL													
	0.15	LOOSE BROWN SILTY SAND, GRAVEL, TRACE ORGANIC MATERIAL (FILL)													
	60.73			1	50	3m									
	1.07	VERY LOOSE TO LOOSE SILTY SAND, SILTY CLAY, TRACE GRAVEL AND ORGANIC MATERIAL (FILL)		2	"	4									
				3	"	4									
				4	"	1									
	57.84			5	"	8									
	3.96	STIFF DARK BROWN PEAT		6	"	12									
	56.65			7	"	16									
	5.15	COMPACT TO VERY DENSE GREY SANDY SILT, SOME GRAVEL AND CLAY (GLACIAL TILL)		8	"	63									
54.82			9	"	50										
6.98	WEATHERED SHALE BEDROCK		10	BXL RC	1										
7.16	FAINTLY WEATHERED THINLY LAMINATED DARK GREY TO BLACK FISSILE SHALE BEDROCK, BEDDING <5° SOME NEAR VERTICAL CALCITE STRINGERS (NOTE: SHALE SLAKES PERCEPTIBLY WITH MOISTURE CHANGES)		11	BXL RC	1										
51.65	END OF HOLE														
10.15															

CORE RECOVERY %
 97
 91
 R.Q.D. (%) AFTER CORING
 78
 74
 R.Q.D. (%) AFTER 1 CYCLE WETTING AND DRYING
 0
 0

9.72-9.99 CORE SIGNIFICANTLY DISCED ALONG FISSILE BEDDING PLANES

0
 15 5 10
 Percent axial strain at failure

VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED [Signature]

DATUM GEODETIC

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

DRAWN _____
CHECKED _____

RECORD OF BOREHOLE 83-5

LOCATION See Figure 2

BORING DATE MAR. 1, 1983

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○	WATER CONTENT, PERCENT Wp — W — WL				
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	69.0 ± 0.00	GROUND SURFACE												<p>GROUND SURFACE</p> <p>SURFACE SEAL</p> <p>NATIVE BACKFILL</p> <p>PLASTIC TUBING</p> <p>STANDPIPE</p> <p>W.L. IN STANDPIPE AT ELEV. 67.02 MAR. 16, 1983</p>
	68.54 ± 0.46	ORGANIC SILTY SAND		1	50	13								
	67.48 ± 1.52	COMPACT BROWN SILTY SAND, SOME FINE SAND LAYERS		2	"	32								
	66.13 ± 2.87	DENSE BROWN TO GREY SANDY SILT, SOME CLAY AND GRAVEL (GLACIAL TILL)		3	"	34								
ROTARY DRILL BXL CORE	64.43 ± 4.57	SLIGHTLY WEATHERED THINLY LAMINATED DARK GREY FISSILE SHALE BEDROCK, BEDDING <5°, NUMEROUS NEAR VERTICAL FRACTURES		4	BXL RC	—								
				5	"	—								
				6	"	—								
				7	"	—								
	60.80 ± 8.20	FAINTLY WEATHERED THINLY LAMINATED DARK GREY FISSILE SHALE BEDROCK BEDDING <5°, AND NEAR VERTICAL CALCITE FILLED STRINGERS, SOME OPEN FRACTURING (NOTE: SHALE SLAKES PERCEPTIBLY WITH MOISTURE CHANGES)												
	END OF HOLE													

STA. 10+465.5 - 75m L+L

IN WEATHERED ZONE, NUMEROUS BROKEN ZONES OCCUR COMPRISING SLIVERS OF SHALE WITH CLAYEY SILT COATING

3.66 - 5.96 SEVERAL LIGHT GREY CALCAREOUS SILTY STRINGERS

5.05 - 5.08 LIGHT GREY MICROCRYSTALLINE LIMESTONE HORIZON

5.22 - 5.33

CORE RECOVERY %

R.Q.D. (%) AFTER CORING

R.Q.D. (%) AFTER 1 CYCLE WETTING & DRYING

0 15 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50

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RECORD OF BOREHOLE 83-6

LOCATION See Figure 2

BORING DATE MAR. 3, 1983

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES			ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m		SHEAR STRENGTH Cu, kPa	NAT. V. - + Q - ● REM. V. - ⊕ U - ○	WATER CONTENT, PERCENT					
										1x10	1x10	1x10	1x10		
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	68.40	GROUND SURFACE													GROUND SURFACE SURFACE SEAL PLASTIC TUBING MH NATIVE BACKFILL STANDPIPE W.L. IN STANDPIPE AT ELEV. 66.21 MARCH 16, 1983
	0.00	BROWN SAND AND GRAVEL (FILL)													
	67.79														
	0.61	LOOSE BROWN FINE SAND SOME SILT (FILL)		1		7									
	67.18														
	1.22														
	65.50	LOOSE BROWN TO GREY SAND AND SILTY SAND SOME CLAY, TRACE ORGANIC MATERIAL (FILL)		2		10									
	2.90														
	63.71	LOOSE TO DENSE DARK GREY SANDY SILT, SOME CLAY AND GRAVEL (GLACIAL TILL)		4		8									
	4.69														
ROTARY DRILL BxL CORE	63.37	MODERATELY WEATHERED DARK GREY SHALE		7	BxL RC	1									
	5.03	FAINTLY WEATHERED THINLY LAMINATED DARK GREY FISSILE SHALE BEDROCK BEDDING < 5°, SOME NEAR VERTICAL FRACTURES		8		1									
	60.72	(NOTE: SHALE SLAKES PERCEPTIBLY WITH MOISTURE CHANGES)													
	7.68	END OF HOLE													

15 0 5 10 Percent axial strain at failure

VERTICAL SCALE 1:50

Golder Associates

DRAWN P.V.
CHECKED

NOTE: Boreholes 83-8 to 83-12 inclusive were put down within Stage 2 section of East Transitway.

RECORD OF BOREHOLE 83-14, 83-15

LOCATION See Figure 2

BORING DATE NOV. 14, 1983

DATUM GEODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION		
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	NAT. V. - + REM. V. - ⊕	Q. - ● U. - ○	WATER CONTENT, PERCENT					
											Wp	W			WL	
POWER AUGER 200mm DIAM. (HOLLOW STEM)	60.31	GROUND SURFACE														
	0.00	TOPSOIL														
	0.09	BROWN SILTY SAND, TRACE GRAVEL (FILL)														
	59.57															
	0.73															
	56.99															
	3.31	HIGHLY WEATHERED DARK GREY SHALE BEDROCK														
	56.49															
	3.81	END OF HOLE REFUSAL TO AUGER														
POWER AUGER 200mm DIAM. (HOLLOW STEM)	59.81	GROUND SURFACE														
	0.00	BROWN SILTY SAND, SOME GRAVEL AND ORGANIC MATERIAL (FILL)														
	59.19															
	0.61	DENSE BROWN SANDY SILT SOME CLAY AND GRAVEL (GLACIAL TILL)														
	58.58															
	1.22	HIGHLY WEATHERED DARK GREY SHALE BEDROCK														
	58.28															
1.52	END OF HOLE REFUSAL TO AUGER															

0
15 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50

Golder Associates

DRAWN J.C.
CHECKED [Signature]

RECORD OF BOREHOLE 83-18

LOCATION See Figure 2

BORING DATE NOV 18 1983

DATUM GLODETIC

SAMPLER HAMMER, 63.5 kg.; DROP, 760 mm

PENETRATION TEST HAMMER, 63.5 kg.; DROP, 760 mm

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m		HYDRAULIC CONDUCTIVITY, k, cm/sec.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		BLOWS/0.3m	SHEAR STRENGTH Cu, kPa	NAT. V. - + Q. - ● REM. V. - ⊕ U. - ○	WATER CONTENT, PERCENT Wp — W — WL					
POWER AUGER 200 mm DIAM. (HOLLOW STEM)	67.72	GROUND SURFACE												<p>GROUND SURFACE SURFACE SEAL NATIVE BACKFILL PLASTIC TUBING STANDPIPE</p>	
	0.00	DARK BROWN SANDY SILT SOME GRAVEL (FILL)													
	67.09	TOPSOIL													
	0.61 66.79														
	0.91														
	65.11	COMPACT TO VERY DENSE DARK BROWN SANDY SILT. SOME CLAY, GRAVEL AND BOULDERS (GLACIAL TILL)		1	50	13									
	2.59	SLIGHTLY WEATHERED THINLY LAMINATED DARK GREY TO BLACK FISSILE SHALE BEDROCK BEDDING <5°. NUMEROUS FRACTURED SEAMS		2	55	66									
	63.80			3	20	1									
	3.90			4	50	100/75									
	63.80			5	50	100/75									
ROTARY DRILL BXL CORE		FAINTLY WEATHERED THINLY LAMINATED DARK GREY TO BLACK FISSILE SHALE BEDROCK BEDDING <5°. SOME FRACTURED SEAMS		6											
				7											
				8											
	60.23	END OF HOLE													
	7.47	END OF HOLE													

STA. 11+076 - E

CORE RECOVERY (%)

R.Q.D. (%) AFTER CORING

R.Q.D. (%) AFTER 1 CYCLE WETTING AND DRYING

3.59 - 4.27
 4.75 - 4.79
 5.88 - 6.19
 6.31 - 6.49
 6.52 - 6.55
 6.57 - 6.71
 7.10 - 7.13

LIGHT GREY MICROCRYSTALLINE LIMESTONE HORIZONS

 CORE BROKEN ADJACENT TO SUBVERTICAL JOINT

 CORE BROKEN ALONG FISSILE BEDDING PLANES, SOME FRAGMENTS EXHIBIT SILTY CLAY COATINGS

WL IN STANDPIPE AT ELEV. 67.03 NOV. 22, 1983

0
15 5 10 Percent axial strain at failure

VERTICAL SCALE
1:50

Golder Associates

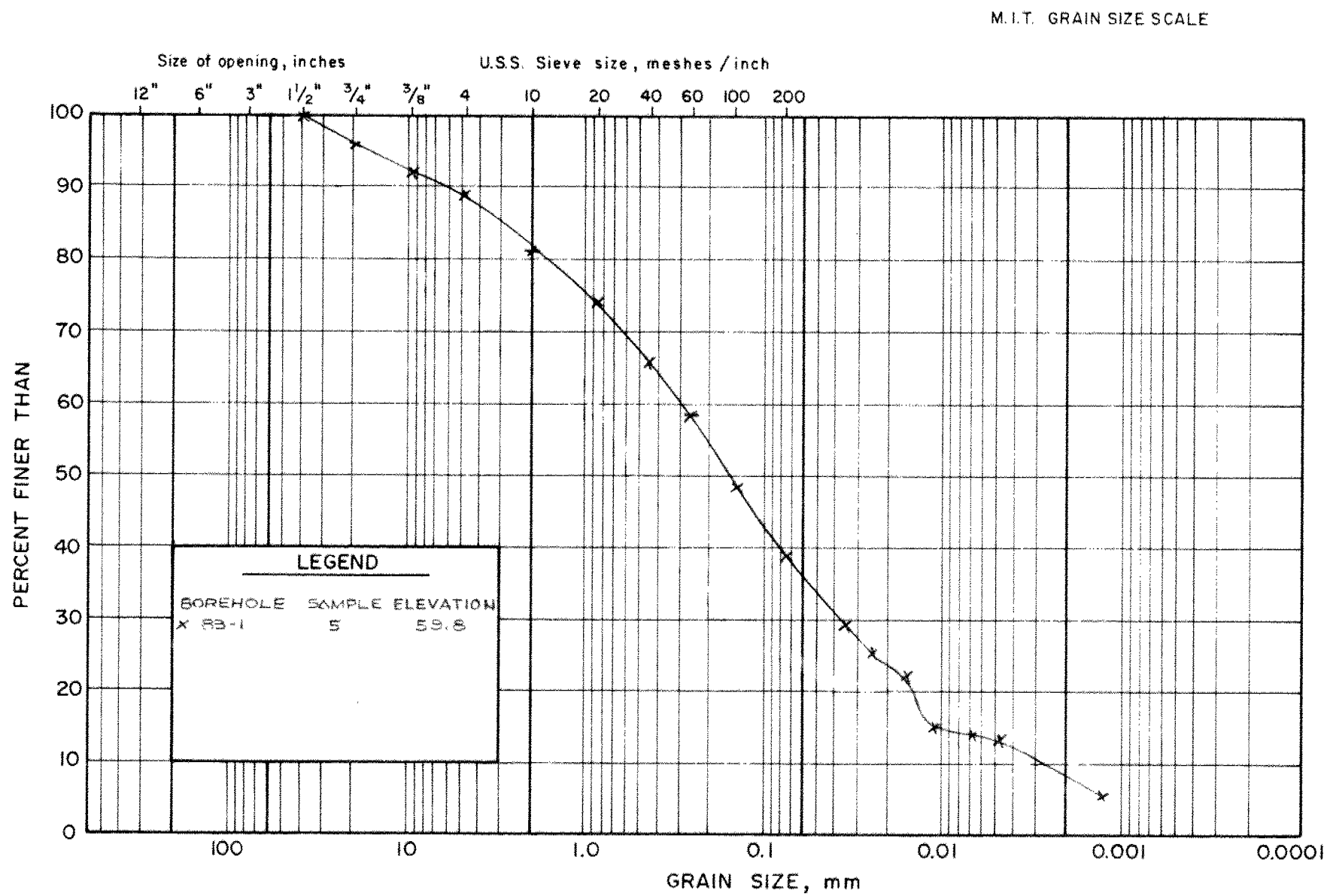
DRAWN: J.C.
CHECKED: [Signature]

OVERSIZE DRAWING

December 1983

831-2016

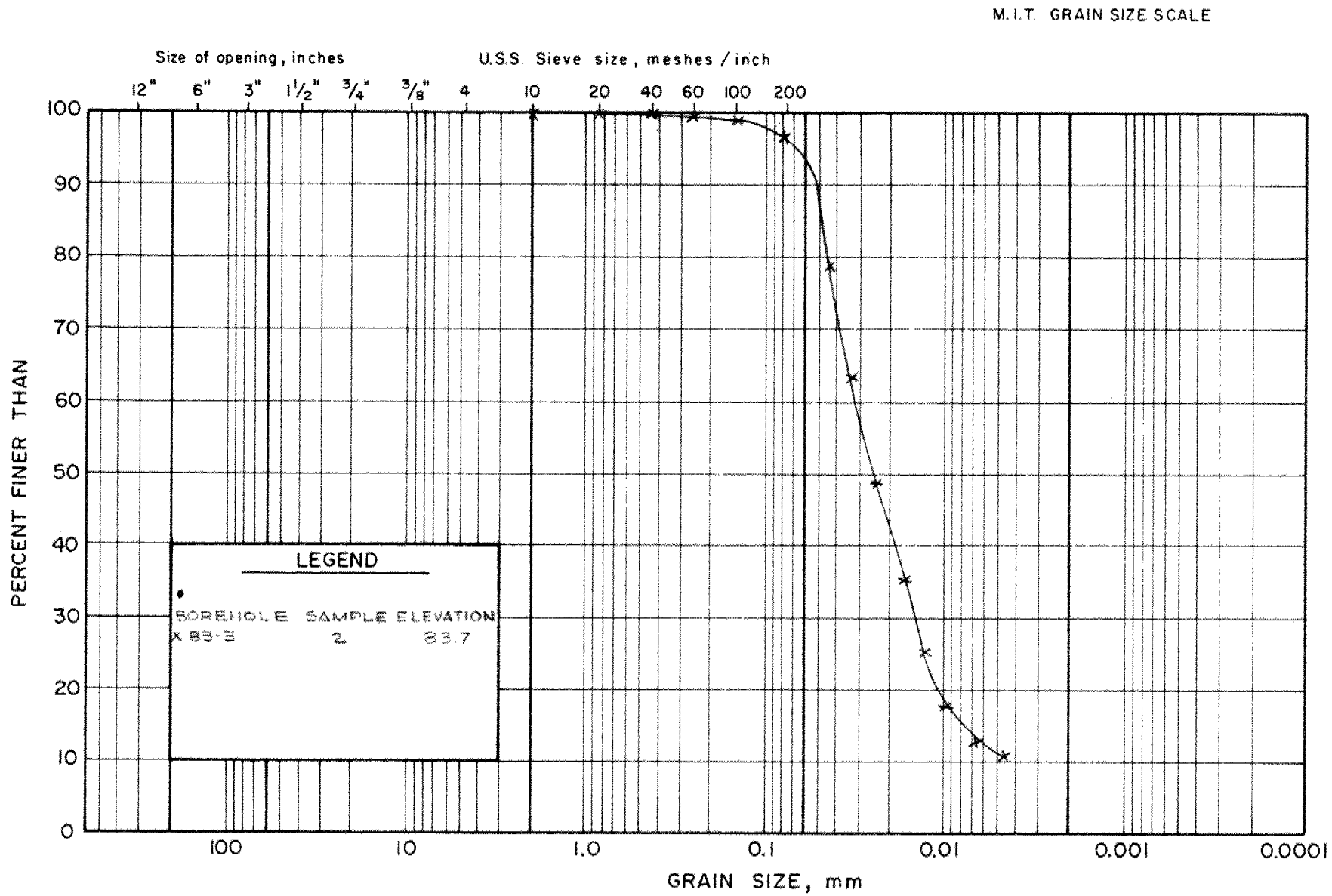
NOTE: Figure 2, BORING PLAN, and Figure 3, SUBSURFACE PROFILE, form part of this report but have been submitted under separate cover.



GRAIN SIZE DISTRIBUTION
 SILTY SAND, SOME GRAVEL, TRACE CLAY (FILL)

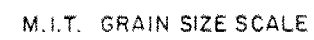
FIGURE 4

Goldier Associates



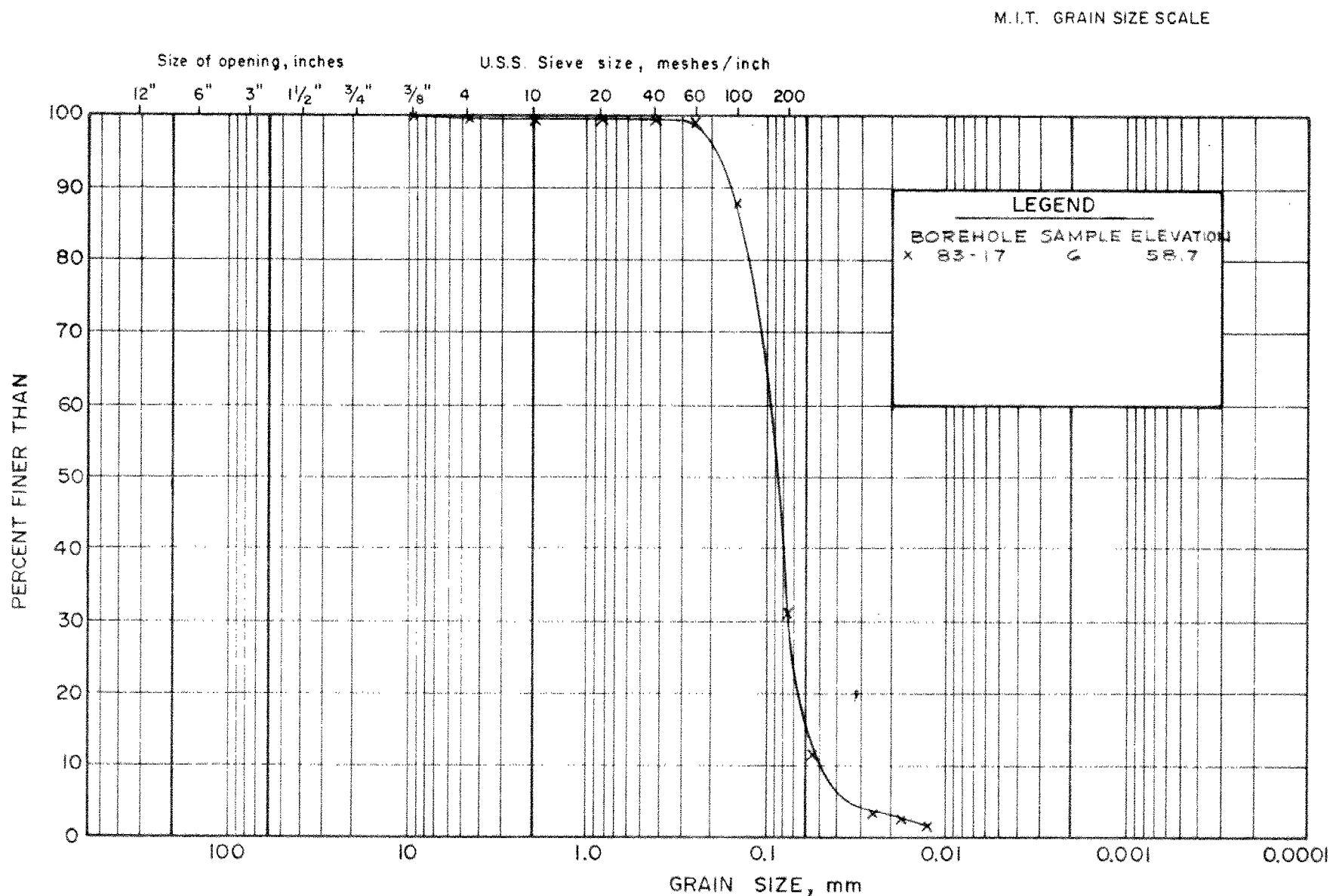
GRAIN SIZE DISTRIBUTION
SANDY SILT

FIGURE 5



BOULDER SIZE	COBBLE SIZE	COARSE GRAVEL SIZE	MEDIUM GRAVEL SIZE	FINE GRAVEL SIZE	COARSE SAND SIZE	MEDIUM SAND SIZE	FINE SAND SIZE	SILT SIZE	CLAY SIZE
-----------------	----------------	-----------------------	-----------------------	---------------------	---------------------	---------------------	-------------------	-----------	-----------

FIGURE 6

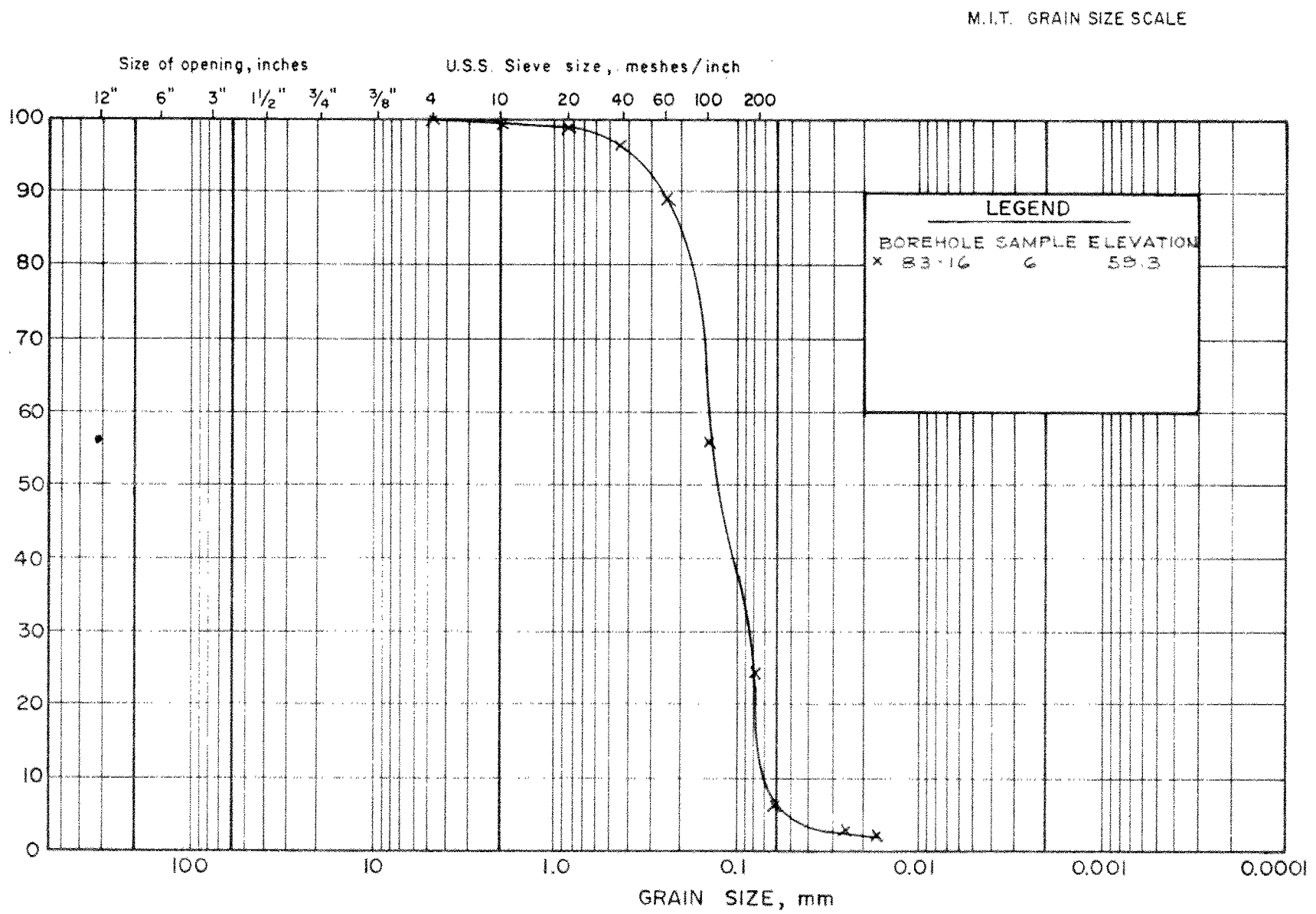


BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
		GRAVEL SIZE			SAND SIZE			FINE GRAINED			

GRAIN SIZE DISTRIBUTION

FINE SAND

FIGURE 7

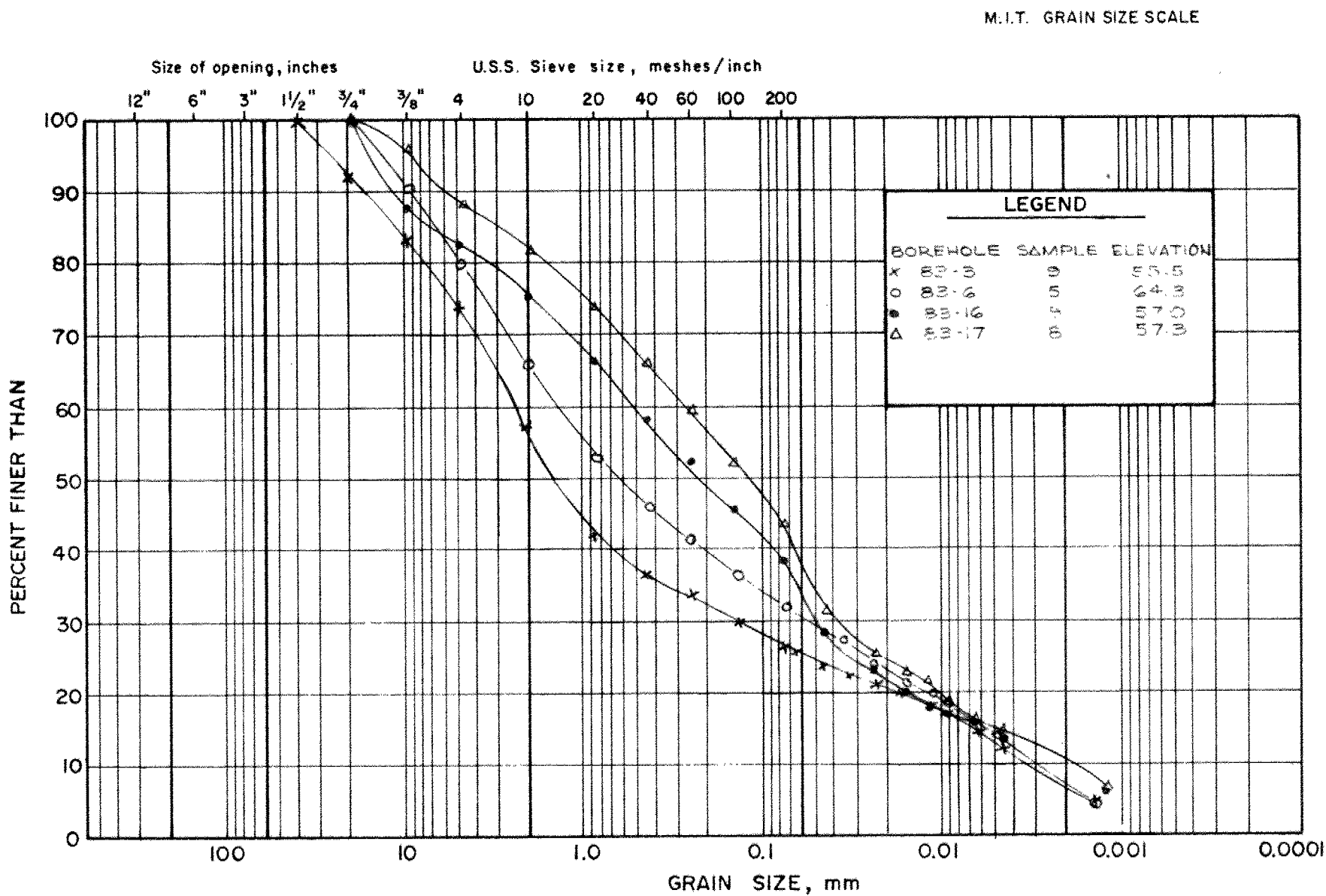


BOULDER SIZE	COBBLE SIZE	COARSE	MEDIUM	FINE	COARSE	MEDIUM	FINE	SILT SIZE		CLAY SIZE	
		GRAVEL SIZE			SAND SIZE			FINE GRAINED			

GRAIN SIZE DISTRIBUTION
FINE TO MEDIUM SAND

FIGURE 8

Goldier Associates



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

GRAIN SIZE DISTRIBUTION
SANDY SILT TILL

FIGURE 9



Golder Associates

CONSULTING GEOTECHNICAL AND MINING ENGINEERS

REPORT TO

M.M. DILLON LIMITED

GEOTECHNICAL INVESTIGATION
DESIGN DETAILS
EAST TRANSITWAY

TREMBLAY RD. TO ST. LAURENT STATION
Station 9+800 to 10+670

REGIONAL MUNICIPALITY OF
OTTAWA-CARLETON

Distribution:

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ABSTRACT

This report presents the engineering recommendations for the geotechnical and geological design aspects for the proposed structures and facilities of the East Transitway from Tremblay Road to the St. Laurent Station, between about Station 9+800 and 10+670.

The East Transitway roadway structure will be in cut section in this area requiring excavation to 9 metres below existing ground surface. It is recommended that short term open cut excavations having a slope of 3.0 horizontal to 1 vertical be used in areas where the glacial till is overlain by deposits of sand, silty sand, sandy silt. Easterly, from about Station 10+400, the temporary excavation side slopes may be increased to 1.5 horizontal to 1 vertical. Preliminary design recommendations are given for temporary shoring design should space limitations dictate. The shale bedrock at the site should, because it is susceptible to slaking and degradation, be excavated using nominal 1/4 horizontal to 1 vertical side slopes and be protected where required. Careful excavation techniques should be employed in the structure footing areas and where rock cuts are required adjacent to the toe of shored excavation.

The Ottawa Queensway underpass structure may be founded on spread footings resting on dark grey shale bedrock. The recommended allowable bearing pressure in the Ultimate Limit State is 1500 kilopascals provided that the footing areas have been properly prepared. Backfill and drainage requirements are put forward along with the soil parameters and wall loads to be used in design. The lateral loads will have to be resisted by friction along the base assuming a factored angle of friction between the concrete and shale of 20.5 degrees. Additional horizontal restraint may be provided with rock anchors or shear keys.

The Belfast Road structure may be founded on the dense glacial till located at an elevation below 60.5 metres. The footing for the structure should be designed using a maximum factored bearing capacity in the Ultimate Limit State of 700 kilopascals and a bearing pressure in the Serviceability Limit State of 240 kilopascals. Horizontal sliding resistance between mass concrete and the dense glacial till should be calculated using a factored angle of friction of 23.9 degrees. Alternatively, the Belfast Road structure may be founded on the compact glacial till at an elevation of 61.5 metres using a maximum factored bearing capacity in the Ultimate Limit State of 500 kilopascals and a bearing pressure in the Serviceability Limit State of 150 kilopascals. In this case, horizontal resistance should be calculated assuming a factored angle of friction of 19.6 degrees between the concrete and glacial till.

Recommendations are also provided for the design of the retaining walls required in this project. Consideration is given to appropriate bearing strata or elevations for the footings, bearing capacity, backfill, wall loadings and methods of horizontal restraint.

Recommendations are provided for the transitway pavement design, associated transitway shoulders' and a possible bus access ramp as well as for Belfast Road and Tremblay Road from Avenue K to Belfast Road. Special design and construction considerations are discussed regarding the need to provide pavement drainage while maintaining the shale bedrock in a wet environment.

Site service installation and the relocation of a 1050 millimetre diameter watermain are discussed with particular reference to excavation techniques, granular bedding and backfilling. For the proposed 1650 millimetre diameter storm sewer, full rock face and soft ground tunnelling procedures are discussed.

The need to provide close control over all excavation procedures is outlined and recommendations are given for the need to retain experienced and specialist supervisory personnel throughout all phases of construction.

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FIGURES 1 TO 7

1. INTRODUCTION

Golder Associates has been retained by M.M. Dillon Ltd., Consulting Engineers to the Regional Municipality of Ottawa-Carleton, to carry out a geotechnical investigation along the East Transitway between about Station 9+800, just north of Tremblay Road at Pickering Place, to Station 10+670, north of the Ottawa Queensway, as shown on the Key Plan, Figure 1. The purpose of this investigation was to determine the soil, bedrock and groundwater conditions in the area of this section of the East Transitway by means of a number of detailed borings and, based on the factual information obtained, to provide geotechnical engineering recommendations for the design of the structures and related facilities proposed for this area including any special construction considerations which could influence design decisions.

The factual information on the subsurface conditions encountered in this area is presented in Golder Associates' report 841-2295-1, "Geotechnical Investigation, Subsurface Conditions, East Transitway, Tremblay Road to St. Laurent Station, Station 9+800 to 10+670" dated October 1984. The relative borehole locations and the inferred soil stratigraphy along the route are shown on Figures 2 and 3, respectively.

This report presents our engineering recommendations for the geotechnical design aspects of the structures and facilities to be provided along this section of the East Transitway. As well, comments on special construction considerations which could influence design decisions are provided.

It should be stressed that the information contained in this report is provided for the guidance of the design engineers. Contractors bidding on or undertaking the works

should review the factual results of this investigation as outlined in report 841-2295-1, satisfy themselves as to the adequacy of the information for construction and make their own interpretation of the factual data as it effects their proposed construction techniques, schedule and equipment capabilities.

It is recommended that the final design of the facility be reviewed with the geotechnical consultant to ensure that the recommendations given in this report are applicable to the actual design requirements and have been properly interpreted by the design engineers.

2. DESCRIPTION OF PROJECT

The section of the East Transitway under consideration in this report extends from immediately north of Tremblay Road at Pickering Place to north of the Queensway, at the St. Laurent Shopping Centre (Station 9+800 to 10+670). Along this section of the transitway, the vertical alignment will be depressed, having a maximum depth of about 9 metres below existing ground surface. Additional facilities to be provided in this contract include:

- i) The Belfast Road underpass structure which may incorporate a stairway and bus loading ramps;
- ii) a 140 lineal metre section of the Ottawa-Queensway underpass structure, including an elevated sidewalk to provide access to the St. Laurent Station;
- iii) a pedestrian access structure from Tremblay Road at Avenue T to the south end of the Queensway underpass structure;
- iv) a number of permanent retaining wall structures as well as some temporary retaining structures which will be required during construction;
- v) the construction of a 1650 millimetre diameter roadway storm sewer, having an invert elevation which would be up to 8.5 metres below the transitway grade or up to 12.0 metres below existing ground surface;
- vi) the relocation of two sections of an existing 1050 millimetre diameter watermain.

3. EXCAVATION

3.1 Overburden

Excavation of overburden for the construction of the paved transitway facility and associated structures will be carried out to variable depths through surficial deposits of loose to compact fill, compact sandy silt, silty sand, loose to compact sand and up to 2.0 metres of sandy silt till. The excavation for the transitway facility will be carried out in most part to below the water table with a maximum depth below the water table of about 6 metres in the vicinity of Station 10+670. Overburden excavation will also be required for the pedestrian access structure located to the north of Tremblay Road at Avenue T. This will involve excavation of mixed fill consisting of sandy silt containing gravel, cobbles, boulders and organic matter. The variable nature of this deposit also indicates the possible presence of asphalt and chunks of reinforced concrete. Excavation of overburden will also be required for the retaining walls at the Belfast Road structure and at the Ottawa-Queensway underpass. The proposed storm sewer structure will require the excavation of sandy silts, silty sands and sands below the proposed transitway grade. As well, up to 4.7 metres of dense to very dense dark grey sandy silt till will have to be excavated to enable removal of the underlying dark grey shale bedrock to the proposed storm sewer invert level.

Excavations of the overburden deposits both above and below the groundwater level should present no major problems, but will present some constraints. For the general excavation of the transitway and possible associated access ramp, it is considered that the overburden deposits of sandy silt, silty sand, and sand above the groundwater level may be excavated in open cut using short term construction side slopes of about 1.5 horizontal to 1 vertical. However, a

recent test dig in this area has indicated that excavation of these fine grained deposits below the groundwater level will present some constraints. Groundwater inflow to the test dig, when left uncontrolled, resulted in significant soil inflow and undermining of the excavation slopes. The results of the test dig has indicated that short term excavation slopes of about 3.0 horizontal to 1.0 vertical will be required in the silts and sands below the groundwater level in order to limit soil inflow. At these relatively shallow slopes, groundwater inflow, though possibly substantial at first, should be readily handled by pumping from sumps within the excavation. If steeper short term excavation slopes are required, groundwater lowering will have to be completed prior to excavation. In this case, a full system of vacuum wellpoints and/or pre-augered gravel packed wells would probably be required for effective drawdown.

The design, installation and operation of the wellpoint system must be carried out by an experienced groundwater specialist and/or contractor. It is expected that the wellpoints would be installed on a relatively close spacing (typically 2.5 to 3.0 metres) by means of jetting within a casing. The lead time required to lower the groundwater table will depend on the design and efficiency of the system but can be estimated to be in the order of a few weeks to a month. The temporarily lowered groundwater condition must be maintained until the permanent slope drainage systems are in place and functioning.

Temporary groundwater lowering north of the proposed transitway will cause an increase in stress in the overburden soils which will result in some permanent settlement of the existing watermain. Although definitive amounts are not possible, the total differential settlement of the watermain should be in the order of about 25 millimetres or less. It is recommended, however that the watermain be monitored during the groundwater lowering procedure to ensure that settlement along the length of the watermain remains within tolerable limits.

Should the above estimated settlements not be considered acceptable, excavation in this area would have to be carried out using interlocking steel sheet piling to protect the watermain.

In contrast, excavation of the glacial till should present no major problems either above or below the groundwater level. Short term construction slopes of about 1.5 horizontal to 1 vertical should be appropriate in the glacial till. Although some water inflow may be expected, at least locally along the sand/till or till/bedrock interfaces, this should be handled by pumping from sumps within the excavation.

Where space restrictions dictate, it may be necessary to provide a temporary braced shoring system to restrain the overburden soils. The temporary shoring system should be designed using the parameters provided in Table 1.

TABLE 1
SOIL PARAMETERS FOR TEMPORARY BRACING DESIGN

Soil Description	Unit Weight (kN/m ³)	Angle of Friction (degrees)	Cohesion (kN/m ²)	Coefficient of Active Earth Pressure, K_a
<u>Serviceability Limit State</u>				
Silty Sands, Sandy Silts and Sands	20	30	-	0.33
Glacial Till	21	35	-	0.27
<u>Ultimate Limit State</u>				
Silty Sands, Sandy Silts and Sands	20	24.8	-	0.41
Glacial Till	21	29.2	-	0.34

Relatively low height cantilever shoring walls may be designed using a triangular pressure distribution plus any surcharge loading due to either construction equipment or inclined soil slopes behind the bracing system. The coefficient of earth pressure, K_a , assumes that some movement is allowed to occur at the top of the shoring system. Higher braced shoring walls should be designed to resist lateral earth pressures assuming an appropriate triangular, parabolic or rectangular distribution of stress.

The temporary shoring system could consist of either soldier piles and lagging (in conjunction with groundwater lowering to at least the top of the glacial till) or interlocking sheet piling. In the vicinity of the Belfast Road bridge structure, it is recommended that a sheet pile system be used to minimize ground loss due to soil and water inflow so that the integrity of the existing piled foundation is not jeopardized.

Where the temporary shoring wall is bottomed in overburden, the soldier piles, or sheet piles, should be driven to well below excavation level or to the surface of the bedrock in order to provide for toe support. Raker support, if used, should be provided through a system of interconnected or individual footings keyed into the glacial till and/or bedrock at the base of the excavation. For rakers inclined up to 45 degrees from vertical, an allowable bearing value, in the Serviceability Limit State, of 700 kilopascals should be used for footings keyed into the slightly weathered shale bedrock. A maximum factored bearing capacity in the Ultimate Limit State is 1500 kilopascals should be used in design.

Should tie-backs be used to support the temporary shoring walls, they should be grouted into the shale bedrock using maximum grout to shale bond values of 500 to 2000 kilopascals for the Serviceability Limit State and Ultimate Limit State, respectively. It is considered that a minimum anchored length of about 3 metres would be required and

it is recommended that all anchors be pull tested to 1.33 times their design capacity.

The ability of the shale bedrock to provide toe support for the temporary overburden shoring may be questionable in those areas where subsequent rock cuts are required adjacent to the shoring system. In these areas, the soldier piles should either be "toed-in" to the bedrock by means of churn drilling or be adequately pinned at the toe. As well, a lower set of walers, rakers, and/or tie-backs will probably have to be provided near the base of the wall to provide additional lateral resistance at the toe of the shoring system. It is also recommended that a minimum horizontal distance of 1.0 metres be provided between the toe of the piles and the line drilling for the adjacent rock excavation. Larger setback allowances should be provided if the rock in the vicinity of the toe of the piles is in a fractured state or has been disturbed or damaged due to blasting.

Should construction be carried out during freezing temperatures, freezing of the frost susceptible soil behind the temporary support walls will place additional stress on the rakers or anchors and possibly cause overloading of the support system. Accordingly, the soils behind the support walls should be protected from freezing temperatures using a combination of heaters, tarpaulins and/or insulation.

3.2 Bedrock

Bedrock cuts of up to 5 metres in depth will be required for construction of the transitway and associated structures. As well, installation of the proposed 1650 millimetre diameter storm sewer may require cuts of up to 10.5 metres in shale bedrock.

The shale in this area is known to slake perceptibly and to degrade significantly when exposed to the atmosphere for any length of time. This thinly bedded and jointed rock type is known from past experience to be highly susceptible to disturbance (overbreak and underbreak conditions) from excavation procedures. Therefore, it must be noted that the overall appearance and stability as well as the short term durability of the transitway rock cuts will be controlled almost exclusively by the quality of the excavation practices undertaken by the contractors during construction. In order to minimize degradation of the rock cuts, it will be necessary to stipulate and enforce very strict bedrock excavation performance specifications in order to ensure that the transitway design details are not jeopardized by the excavation procedures used.

It is considered that the upper surface of the shale and at least some of the weathered shale may be excavated using large hydraulic earth excavation techniques (backhoe), possibly in conjunction with line drilling techniques to define the excavation limits and provided the backhoe can undercut an exposed rock face. However, based on the measured strength of the intact shale (uniaxial compressive strength of 24 to 30 megapascals), it is unlikely that the use of even large hydraulic rippers would prove economical in breaking up the shale prior to excavation. Consequently, it is considered that drilling and blasting procedures will be required for the major portion of the bedrock excavation. Because of the sensitive nature of the shale bedrock to overbreak and underbreak, consideration will have to be given to line drilling, pre-shearing, and split second firing techniques. As well, stringent blasting control will be required in terms of blast hole spacing, charge weights, weight per delay, and peak particle acceleration.

It is recommended that minimum acceptable blasting standards be stipulated in terms of a performance specification. In this regard, we would recommend that a performance specification to ensure that "greater than 80 percent half

barrels are visible after initial scaling of the sidewalls of the excavation" will provide the necessary blasting control. To this end, the contractor should be advised to take advantage of any technical assistance provided by his blasting specialist and/or explosives supplier to program his work, to interpret requirements, to determine blasting technique and method, drilling and delay patterns, weight charges and other requirements.

Provided the above control procedures are followed, it is considered that the shale bedrock may be cut near vertical, in the short term. However, because of the degradation potential of the shale upon exposure, vertical rock cuts may need extensive protection to avoid undercutting of the adjacent overburden excavation and to protect the on-going construction work at the base of the rock cuts. Consideration should be given, therefore, to limiting short term rock cuts to slopes of about 1/4 horizontal to 1 vertical to avoid these rock face protection measures. As well, full face rock scaling procedures should be carried out on an on-going basis to maintain the exposed rock face in a stable condition.

Considerable care will also have to be exercised where rock cuts are required adjacent to the toe of temporary shored excavations in order to ensure that the rock face does not deteriorate to a point where it undermines the adjacent earth cut. As well, blasting procedures which cause back-break of the shale could result in immediate undermining of the shoring supports. In the extreme, this could result in the need to extend the overburden shoring system full depth to the base of the rock cut or to provide extensive shotcrete or rock anchor protection to the exposed rock face.

4. LONG TERM SLOPES

4.1 Overburden

Where overburden slopes above the groundwater level are to be left in open cut in the long term, slopes of at least 2.0 horizontal to 1.0 vertical should be used in design. Below the groundwater level, flatter slopes should be used i.e. 3.0 horizontal to 1.0 vertical for sandy silt, silty sand, or sand, and 2.5 horizontal to 1.0 vertical for glacial till.

As well, special protection and permanent drainage measures will have to be employed for all exposed overburden slopes below the groundwater level. A typical treatment of slope drainage facilities to be provided for cut sections through sandy silts, silty sands and sands below the groundwater level is shown on Figure 4.

Permanent drainage of the slope should be provided by a 300 millimetre thick drainage layer placed across the slope to the height of the existing groundwater level. The drainage layer should consist of granular material conforming to the Ministry of Transportation and Communications (M.T.C.) specifications for granular A or select granular B, and should be underlain by a filter fabric. Given the fine graded nature of the silts and sands in this area, it is considered that a non-woven filter fabric having an equivalent opening size of 0.08 millimetres or less, such as Texel 7612, would be suitable for use at the site. Alternatively, the drainage layer could consist of a 300 millimetre thick layer of clear crushed stone fully wrapped, both top and bottom, with a similar type of filter fabric. The slope drain should then be covered with a minimum of 300 millimetres of topsoil, which is in turn sodded and/or seeded to protect against surface erosion. The slope drain should connect to a toe drain which forms part of the roadway drainage system. Protection against soil loss from beneath the

roadway structure will be necessary in areas where the transitway subgrade consists of silty fine sand, sandy silt or sand below the existing stabilized groundwater level. In this case, a filter fabric should be placed full width across the proposed transitway structure.

Space restrictions dictate that the long term overburden slopes between the north side of the proposed transitway and the Ottawa-Queensway will have to be steeper than 3 horizontal to 1 vertical. In most cases, the design long term slope will be about 2 horizontal to 1 vertical with a maximum slope of about 1.6 horizontal to 1 vertical. During construction, it will therefore be necessary to lower the groundwater level by means of a system of vacuum wellpoints in areas indicated to have a groundwater level within the silty sands, sandy silts and sands, and above the proposed transitway subgrade. Permanent drainage of these slopes will be by means of a treatment similar to that shown on Figure 5. With the steeper slopes, it will be necessary to provide a thicker granular layer to further aid in draining the slope and to prevent piping of the native soils. Should space restrictions prevent construction of the granular drainage layer, it may be necessary to support these slopes by means of permanent retaining walls.

Where long term overburden cuts are bottomed on bedrock, the toe of the overburden slope should be set back at least 1 to 2 metres from the face of any subsequent rock cut to provide sufficient space for the rock excavation and/or to ensure that undercutting of the overburden slope does not occur. Where the rock cut has been adequately protected by wire mesh and concrete, concrete facing panels or the like, a minimum setback distance of 0.5 metres may be used between the toe of the overburden slope and the face of the concrete panel. In either case, a toe drain should be provided along the interface between the overburden and the underlying bedrock to intersect drainage at this point.

4.2 Bedrock

Where rock cuts are to be left open in the long term, it is recommended that the slopes not be steeper than 1.0 horizontal to 1.0 vertical. Even so, protection of these steep rock cuts will be necessary to avoid rapid breakdown and weathering upon exposure. In this case, the slope protection would have to be structural in nature, consisting of wire mesh and concrete, concrete retaining walls, or structural concrete facing panels. Alternatively, flatter slopes of about 1.5 horizontal to 1 vertical could be used, with the slope protection consisting of non-structural elements such as gravel, or paving stones. Should rock slopes of about 2.0 horizontal to 1.0 vertical, or flatter, be employed, they may be left exposed or simply covered with a layer of topsoil and seeded or sodded.

Regardless of the rock cut slope angle used, groundwater inflow into bedrock excavations may be expected along major joints and bedding planes and down the face of the rock cut. Permanent slope drainage facilities similar to those for the overburden slopes should be used where relatively flat (less than 1.5:1) rock slopes are employed. For steeper exposed rock faces, attempts should be made to seal and/or drain any water bearing discontinuities through subsurface drainage holes behind the face, or by a system of proprietary filter drains along the rock face and connected to the roadway storm drain system.

Permanent drainage of protected rock faces overlain by soil should be provided by means of a treatment similar to that shown on Figure 6. A 200 millimetre thick layer of select granular B or granular A can be used behind the facing panels to intercept drainage from the rock face. Alternatively, a filter blanket such as Cordrain

could be provided along the full height of the rock face. The filter blanket should be embedded in the toe drain at the rock/overburden interface. Vertical resistance to sliding of the protective concrete cover should be calculated by assuming an angle of friction of 25 degrees between concrete and granular material. If a filter blanket is used, then no friction should be assumed between the material and concrete. Additional resistance should be provided by means of rock bolts, rock dowels or through the use of a restraining footing keyed into the rock at the toe of the slope.

5. PROJECT STRUCTURES

5.1 Ottawa-Queensway Underpass

The Ottawa-Queensway underpass is to consist of a rigid frame structure founded at an elevation varying from 58.5 to 60.0 metres. For foundation design purposes, the shale bedrock that will be exposed at founding level can be considered of fair quality, and the rigid frame footings and associated elevated sidewalk footings may be designed using a maximum factored bearing capacity at the Ultimate Limit State of 1500 kilopascals.

This design value assumes that the shale at and below founding level is not disturbed by the blasting operations. In this respect, it is recommended that final rock trimming for footing areas, using either a small backhoe mounted rock hammer or by hand-held equipment, be carried out after the general excavation to the transitway subgrade level. Although bulk blasting of footing areas should not be allowed, experience has indicated that it may be necessary to employ line drilling and pre-splitting techniques to define the footing limits and avoid disturbance of the adjacent bedrock. The general conditions of the rock below the founding level indicate the presence of occasional zones of highly fractured, moderately weathered rock. In this respect, consideration should be given to carrying out a program of percussion drilled inspection holes, to at least 1.5 metres below founding level to check for possible weathered seams. If such zones are encountered and if considered detrimental to the above noted foundation design, footing excavations should be carried down to and below such zones and the area back-filled with concrete.

In order to avoid degradation, swelling and heaving of the shale upon exposure to the atmosphere, all bearing surfaces should be immediately (within hours) covered with a mud mat of concrete. This includes the vertical rock face in the area where the rigid frame thrust loadings will be applied. Since the oxidation process which causes degradation of the shale also produces detrimental chemical reactions, all concrete in contact with the shale should contain sulphate resistant cement to resist potential deterioration of the concrete.

For the elevated sidewalk footing, it is recommended that a distance of at least 750 millimetres be maintained between the edge of the footing and the crest of the cut for the lower lying rigid frame footing. A sketch of a typical footing treatment is shown on Figure 7.

Drainage of the backfill behind the rigid frame walls should be provided to prevent hydrostatic build-up. Drainage may be provided via a perforated pipe drain with a granular surround running along the top of the rigid frame footing and outletting through sleeves in the rigid frame wall into the transitway subdrain system. With full effective drainage of the backfill, the rigid frame should be designed to resist lateral earth pressure using a soil unit weight of 21.2 kilonewtons per cubic metre and the following parameters:

	Serviceability Limit State (SLS)	Ultimate Limit State (ULS)
Angle of Friction, ϕ (degrees)	32	26.6
Coefficient of Earth Pressure at Rest, K_0	0.47	0.55

The magnitude of the stress block should be calculated using the formula $K_0 \gamma H$ and assuming a triangular distribution of stress. The rigid frame walls should be backfilled with nominally compacted, free draining, non frost susceptible material. Since this material will ultimately support the Queensway traffic, it is recommended that granular material conforming to the Ministry of Transportation and Communications' specification for select granular B (100 millimetre minus quarried crushed stone) be used and that compaction be carried to 90 percent of the maximum standard Proctor dry density value. Within the top 1.5 metres of backfill, the compactive effort should be increased to provide an in situ density equivalent to 95 percent of the maximum standard Proctor dry density value. Although heavy vibratory compaction equipment should not be allowed within 0.3 metres from the back of the rigid frame wall, in order to account for some compaction induced stresses the walls of the structure should be designed to resist, in addition to the triangular earth pressure distribution, a uniformly distributed stress of 10 kilopascals applied over the total height of the wall. Live loads due to highway traffic need not be applied in the design provided that an adequately reinforced concrete approach slab supported by the underpass structure is provided. Live loads occurring during the construction phase, however, before placement of the approach slab, should be included in the design of the rigid frame structure.

The rigid frame structure will have to resist, in addition to the at rest earth pressure, a passive earth pressure load due to the outwards movement of the walls of the structure during live loading and temperature variations. The passive earth pressure should be calculated by assuming a soil unit weight of 21.2 kilonewtons per cubic metre and using the following parameters:

	Serviceability Limit State (SLS)	Ultimate Limit State (ULS)
Angle of Friction, ϕ (degrees)	32	26.6
Coefficient of Passive Earth Pressure, K_p	3.25	2.62

The magnitude of the stress block should be calculated using the formula $K_p \gamma H$ and assuming a triangular distribution of stress. Alternatively, the abutment face against the backfill could be fitted with a 25 millimetre thick sheet of high density expanded polystyrene foam insulation having a compressive strength in the order of 50 kilopascals. This material should accomodate the earth pressures generated during outward lateral movement of the rigid frame walls without dramatically increasing the lateral earth pressures on the walls.

Horizontal resistance to sliding should be calculated using a factored angle of friction of 20.5 degrees, between the shale bedrock and the rough concrete base of the wall footing. No additional resistance should be assumed for the rock in front of the wall footing. Where the frictional resistance along the base of the footing is not adequate to restrain the horizontal earth pressure forces, additional restraint will have to be provided by transferring the lateral load to the rock through the use of shear keys or rock anchors.

Key trenches, if used, should be cut with care, should extend at least 1.0 metre below the base of the rigid frame footing, and should be designed using a horizontal bearing capacity in the Ultimate Limit State of 500 kilopascals. Rock anchors, if used, should be designed based on a rock to cement grout bond of 500 kilopascals in the Serviceability Limit State and 2000 kilopascals in the Ultimate Limit State. These rock anchors should be a minimum of 3 metres in length and should be considered to develop bond stress over the lower 2 metres

only. All rock anchors should be proof loaded to 1.33 times their design load.

5.2 Belfast Road Underpass

The Belfast Road underpass will consist essentially of a rigid frame structure and may incorporate access stairways and a bus loading ramp.

Excavation for the underpass structure will be carried out through a thin layer of fill, discontinuous grey brown sand, up to 2.5 metres of grey sandy silt and compact glacial till. Footings for the underpass structure should be founded at or below elevation 60.5 metres and should bear directly on the dense glacial till. As such, further excavation of variable amounts of glacial till will be required in the footing area. Standpipe readings taken at the time of the investigation, indicate that the water level will be at about elevation 63 to 64 metres.

An open cut excavation can be used on the south side of the station provided that the water and soil inflow is controlled (see Section 3). Side slopes of 3.0 horizontal to 1.0 vertical, or flatter should be used to minimize sloughing of soil into the excavation. The bridge abutment on the north side of the underpass structure is understood to be founded on piles driven to bedrock. To prevent any loss of soil which could potentially cause pile instability problems, the excavation on the north side of the underpass will have to be carried out within driven sheeting. The compact nature of the till and the presence of boulders may necessitate an initial driving and excavation followed by further driving of the sheet piling to achieve satisfactory penetration. Reference should be made to Section 3.1 regarding further details in the design of temporary retaining systems.

The footings for the underpass structure bearing on dense glacial till at an elevation of 60.5 metres, may be designed using a maximum factored bearing pressure at the Serviceability Limit State of 240 kilopascals and a bearing pressure at the Ultimate Limit State of 700 kilopascals. Reduced bearing pressures of 150 kilopascals at the Serviceability Limit State and 500 kilopascals at the Ultimate Limit State should be used for structure footings bearing on glacial till above elevation 60.5 metres. Footings for possible stairways and bus loading ramps may bear on the glacial till (below an elevation of 61.5 metres) and should be designed using a bearing pressure at the Serviceability Limit State and Ultimate Limit State of 150 and 500 kilopascals, respectively. These design values assume that the glacial till at and below founding level has not been disturbed during construction. A minimum of 1.5 metres of frost cover should be provided to the underside of all aforementioned footings.

Comments and design parameters relating to the backfill and associated horizontal wall loading for the Queensway underpass structure are also applicable to the Belfast Road structure. If a reinforced approach slab is not provided then live loads due to traffic should be incorporated into the design by assuming a rectangular distribution of stress equal to 13 kilonewtons per square metre and applied over the full height of the wall.

Horizontal resistance to sliding should be calculated using a factored angle of friction of 23.9 degrees, between the dense glacial till and the rough concrete footing. If shear keys are incorporated in the design to provide additional restraint against the imposed horizontal forces, then a factored angle of friction of 33.9 degrees may be assumed for the shear resistance within the dense glacial till. For structure footings at an elevation higher than 60.5 metres, the factored angles of friction should be reduced to 19.6 and 29.2 degrees between compact glacial till and concrete and within the compact glacial till respectively.

5.3 Retaining Walls

Retaining wall structures are planned for several locations along this section of the transitway. These include a section on the north side of the proposed transitway east of the Belfast Road structure, a possible retaining wall along the north side of the transitway extending from about station 9+820 to station 9+911 and a wall along the north side of a possible future bus access ramp from Tremblay Road at Avenue K to the transitway structure.

The possible retaining wall north of the transitway, from Station 9+820 to 9+911 and the retaining wall north of the possible access ramp would retain up to about 4.5 metres of sandy silt and loose glacial till. These retaining walls would be founded on compact to very dense glacial till. As such, the following guide may be used in the design of the founding elevation for these retaining walls along the transitway.

<u>Station</u>	<u>Recommended Founding Elevation (metres)</u>
9+820	57.2
9+850	58.0
9+911	59.0

Allowance should also be made for a depth of cover of at least 1.5 metres for frost protection purposes. The founding level for the possible retaining wall along the bus access ramp may be taken as 59.5 metres. In the above cases, the allowable bearing pressures for Serviceability Limit State Type II and Ultimate Limit State are 150 to 500 kilopascals, respectively, for footings bearing on compact to dense glacial till. The settlement of retaining walls designed using the lower value of allowable bearing pressure should not exceed 25 millimetres provided that the soil at and below founding level is not disturbed.

These retaining walls should be designed using a soil unit weight of 21.2 kilonewtons per cubic metre. With respect to the angle of internal friction, ϕ , and coefficient of active earth pressure, K_a , the following values may be considered for design purposes.

	<u>Serviceability Limit State Type II</u>	<u>Ultimate Limit State</u>
Angle of Friction, ϕ (degrees)	30	24.8
Coefficient of Active Earth Pressure, K_a	0.33	0.41

Backfill to these retaining walls should consist of free draining granular material and the walls should be provided with standard retaining wall drainage facilities. Assuming nominal compaction of the granular materials behind the retaining walls, a triangular pressure distribution should be used in computing the lateral earth pressure. It is important to ensure that the backfill does not receive excessive compaction since this could significantly alter the distribution and magnitude of the lateral pressure to be resisted by the wall. Any surcharge loading due to inclined slopes at the top of the retaining wall or live loads both during and after construction should be added to the above mentioned pressure distribution.

For these retaining walls founded on glacial till, resistance to sliding should be calculated using a factored angle of friction, $\phi_f = 19.6$ degrees, between mass concrete and sandy silt till. Additional restraint, if required should be provided by shear keys on the base of retaining wall footings. The mobilized factored angle of friction against sliding within the compact glacial till may be taken as 29.2 degrees.

Results from Boreholes 4 and 5 indicate that the retaining wall at the Belfast Road structure may be adequately founded on the compact glacial till at an elevation of at most 61.5 metres provided 1.5 metres of earth cover is maintained for frost protection purposes. The allowable bearing pressure for the Serviceability Limit State and Ultimate Limit State may be taken as 150 to 500 kilopascals, respectively. The design parameters and comments outlined previously also apply in this case. To account for an inclined earth slope behind the wall, the coefficient of active earth pressure to be used in

design should be increased. Typically, for a 3 horizontal to 1 vertical slope behind the wall, the coefficient of active earth pressure should be increased to 0.40 for the Serviceability Limit State and 0.52 for the Ultimate Limit State.

Retaining walls may also be required in rock cut sections of the transitway near the Queensway underpass structure. The retaining walls in this case, may be founded on shale bedrock which is indicated to be slightly to fairly weathered with some zones of highly fractured and weathered rock. In this area, the retaining wall footings should be designed for an allowable bearing pressure of 1500 kilopascals for the Ultimate Limit State. This design value assumes that the shale at and below founding level is not disturbed by the excavation and blasting operations, that no significant weathered zones existing within 1.5 metres depth below founding level, that a minimum of 1.2 metres of frost cover is provided, and that the footing areas are protected from the atmosphere with a mud mat of concrete. For footings where significant zones of highly weathered and fractured rock exist within 1.5 metres below founding level, the bearing capacity should be reduced to 500 kilopascals. Backfill to this retaining wall should consist of free draining granular material compacted in place to at least 90 percent of the maximum standard Proctor density value. Compaction induced stresses should be avoided by restricting the operation of heavy vibratory rollers close to the back face of the wall. Provided that effective drainage of the backfill is provided, previously outlined active lateral earth pressure parameters may be used in design. Resistance to sliding should be calculated using a factored angle of friction of $\phi_f = 20.5$ between concrete and shale bedrock provided that the rock has not been damaged during excavation. Final trimming of the founding surface should be done with either a small backhoe mounted rock hammer or hand held equipment.

In addition, horizontal restraint may be provided by transferring the horizontal load to solid rock by the use of rock anchors or shear keys. Key trenches, if used, should be a minimum of 1.0 metres deep and should be designed using a factored angle of friction for the shale of 20.5 degrees. Rock anchors should be designed based on a rock to cement grout bond of 500 kilopascals in the Serviceability Limit State and 2000 kilopascals in the Ultimate Limit State. The rock anchors should be a minimum of 3 metres in length and should be considered to develop bond stress within the lower 2 metres only.

5.4 Pedestrian Access Ramp

A pedestrian access structure has been proposed between the Ottawa-Queensway underpass and Tremblay Road. The top of the access ramp will be at an elevation of about 66 metres at the underpass structure and will rise at about a 6 percent grade to Tremblay Road. Excavation for the proposed structure will be carried out in open cut through fill up to about 5 metres in depth, silty clay and compact glacial till. The fill is indicated to consist in most part of sandy silt containing some gravel, cobbles and boulders. The excavation should be carried out with 1 horizontal to 1 vertical, or flatter, side slopes to minimize sloughing of the sides of the excavation. Since the measured water table elevation will be at or above the bottom of the excavation, some water infiltration may be expected but should be easily handled by pumping from sumps within the excavation. The walls for the structure may be founded on the shale bedrock within about 1 metre of the bedrock surface. The shale in this area may tend to be weathered and fractured, and as such, the allowable ultimate bearing capacity used for design should be 500 kilopascals. A minimum of 1.2 metres of frost cover should be provided.

The pedestrian structure should be designed to resist a triangular pressure distribution using a soil unit weight of 21.2 kilonewtons per cubic metre. The coefficient of active earth pressure for the Serviceability and Ultimate Limit State should be taken as 0.33 and 0.41, respectively.

If it is decided to provide a covered section, then the roof should be designed to resist a vertical applied stress equal to the product of the unit weight of the fill and the height of fill above the roof of the ramp. A unit weight of 20.4 kilonewtons per cubic metre may be assumed for the fill.

Backfill to the walls should consist of free draining granular material, nominally compacted to 90 percent of the standard Proctor dry density and provided with standard drainage conduits. If compaction above the recommended 90 percent of Proctor density is attained, then an additional load of 10 kilonewtons per square metre should be applied over the entire height of the wall in the design calculations.

The stairs and ramp for the access structure should be designed as a slab on grade. Grade raises to the underside of the slab should consist of crushed stone conforming to M.T.C. specification for select granular B. A minimum of 150 millimetre thickness of crushed stone conforming to M.T.C. granular A should be provided as a base course immediately beneath the concrete slab. All granular material should be placed in maximum 250 millimetre thick lifts and should be compacted to 95 percent of the standard Proctor density using suitable vibratory equipment.

Full length drainage of the subsoil beneath the ramp should be provided by means of a slotted pipe embedded in the granular material and outletting into the transitway subdrain system.

6. SERVICE INSTALLATION AND RELOCATION

6.1 General

It is understood that a 1650 millimetre diameter storm sewer is to be installed along this section of the proposed transitway. The elevation of the invert is to vary from about 56.4 metres at Station 9+800 to about 57.2 metres at Station 10+670. These invert levels correspond to depths of about 3 to 11.5 metres below existing ground surface.

A 1050 millimetre diameter watermain is also to be relocated in conjunction with the construction of the East Transitway. This watermain presently runs parallel and about 28 metres to the south of the centreline of the six lane Ottawa-Queensway. Relocation of the watermain will be necessary in order to circumvent the proposed Belfast Road and Queensway underpass structures. At the time of this report, the proposed relocation involves running the watermain across the proposed transitway at about Station 10+000. From Station 10+020 to 10+134, the watermain would run parallel and about 26 metres to the south of the transitway. At about Station 10+155, the watermain would again traverse beneath the transitway to join with the existing watermain just south of the Queensway. Further relocation at the Queensway underpass structure involves traversing the transitway at about 10+439 and running the watermain about 22 metres to the south of the proposed transitway centreline.

6.2 Storm Sewer

Two proposals are being put forward for the construction of the proposed 1650 millimetre diameter storm sewer. These alternatives involve either tunnelling through soft, mixed and rock face conditions, or using standard open cut excavation techniques.

Full rock face tunnelling, using either drilling and blasting techniques or a tunnelling machine is considered feasible in areas where the depth of rock cover is greater than about twice the storm sewer diameter. Hence, this method would be applicable from about Station 10+210 to about Station 10+525. The bedrock at tunnel level along this section of the route consists of faintly weathered dark grey thinly bedded to laminated shale bedrock. Detailed descriptions of the rock core obtained at the proposed sewer elevation are provided in the factual report. Analysis of the rock core recovered revealed that the rock is fairly sound and should stand unsupported in the short term. However, the bedrock is laminated and has some moderately weathered highly fractured seams and, as such, some loose rock will probably develop on the tunnel roof immediately upon exposure. As well, a considerable amount of overbreak may occur in areas where the bedrock is indicated to be inclined to the horizontal plane and in areas shown to contain some near vertical fracturing. The use of a tunnel boring machine may not be practical in such areas. Tunnelling would therefore have to be carried out using drill and blast techniques.

Since the rock is quite sensitive to moisture and confining stress changes and is susceptible to slaking, it will be necessary to provide temporary support such as ribs and lagging, ribs and liner plates, liner plates, or the like in areas where the rock is highly fractured and for those areas of the tunnel with minimal rock cover over the crown. Temporary support in the form of rock bolts in combination with wire mesh and shotcrete could probably be used in areas of sound rock exhibiting little or no fractures. It should also be noted that extended anchor bolts would have to be used due to the laminated nature of this bedrock.

Based on the results of the in situ secondary permeability tests performed in the shale at the design invert level, water inflow should be easily handled with standard pumping techniques. The seepage will be local in nature, and confined mostly to inclined fractures and zones of fractured rock. It is recommended that the water inflow from any open joints or fissures encountered during tunnelling be treated on an individual basis and that rock joints or fissures be packed and grouted as required.

Mixed face tunnelling will have to be carried out from about Station 9+800 to about Station 10+125 with the exception of a short section, from Station 9+875 to 10+210 which would be entirely through overburden. The soil through which the tunnel will pass in this area consists of compact to very dense glacial till comprising dark grey sandy silt and containing a trace to some clay and gravel. However, in some localized areas some sand or sand and gravel may be expected at the glacial till/bedrock interface. Advance of the tunnel along this section would have to be carried out by hand mining. Slow progress through areas of glacial till may be anticipated due to the dense nature of this material. To prevent ground loss and running associated with the presence of sand and/or sand and gravel zones within the till, it may prove necessary to tunnel using poling plates in conjunction with ribs and lagging or liner plates.

Firm contact between the temporary lining and the surrounding ground or rock must be ensured; therefore provision must be made to fill any over-mined areas with grout. Further details on the expected loading and design of the temporary lining and the final concrete lining could be provided if tunnelling is deemed to be the most viable construction technique.

Open cut excavations for the proposed storm sewer would probably be carried out from the bottom of the general excavation to transitway subgrade level. Although, it may be possible to excavate the overburden deposits (silts, sands and glacial till) below transitway grade using flat side slopes, space restrictions will probably dictate that overburden excavation for the storm sewer take place within a protective steel trench box. Ground-water inflow into the excavated trench should be anticipated, particularly where the excavations extend within the sandy silts, silty sands, and sands. However, in general, water inflow should be handled by pumping from sumps located within the excavated trench.

Special precautions will have to be taken in areas where the transitway side slopes are temporarily supported or where structure footings have already been poured in order to ensure that the wall toe support is not removed and the subsoils disturbed during excavation for the storm sewer. In this respect, it may be prudent to carry out the storm sewer construction and backfilling prior to full depth excavation or pouring of the transitway structure footings.

Excavations through shale bedrock should be carried out in accordance with the recommendations outlined in Section 3.2 of this report. Although drilling and blasting procedures will probably be required, these will have to be carried out with care in order to avoid disturbing the bedrock within any adjacent transitway structure footing areas. The use of the line drilling and pre-split techniques to define the sewer trench cut and to act as a buffer discontinuity will probably be necessary to avoid excess vibration levels from penetrating beyond the trench excavation limits.

In order to avoid drawdown of the water level and subsequent drying out and heaving of the shale, the storm sewer trench bedding and backfill should be interrupted with seepage barriers placed at about 100 metre intervals along the transitway easterly from Station 10+250. These barriers could consist of a 1.0 to 1.5 metre width of concrete, extending from trench wall to trench wall, and from trench invert to roadway subgrade level.

It is recommended that a minimum of 300 millimetres of crushed stone bedding (50 millimetres minus) be used where the storm sewer pipe rests on undisturbed subsoil or rock. Provision should be made, however, for providing an additional 300 millimetre thickness of bedding should the pipe invert be founded on subsoil unavoidably disturbed during digging operations below the water table. The bedding material should be placed to at least the spring line of the pipe and compacted to at least 95 percent of the standard Proctor dry density value. Well compacted granular backfill conforming to M.T.C. specifications for granular C, should be placed to at least 300 millimetres above the top of the storm sewer. Backfill below the zone of seasonal frost action could consist of either rock shatter or imported sand and gravel conforming to M.T.C. granular C specifications. All backfill should be placed in shallow 300 millimetre loose lifts compacted to 95 percent of the respective standard Proctor dry density value using suitable compaction equipment.

In order to minimize differential frost heave between areas over the trenches and the adjacent sections of the roadway, it is recommended that acceptable native backfill be used between the frost line (2 metres below the transitway profile grade) and transitway subgrade level. As far as is practical, the native backfill should match the native materials exposed on the adjacent trench walls. It should be noted that considerable effort, including compaction in thin lifts with suitable compaction equipment will be required to achieve 95 percent of the standard Proctor dry density for these native materials.

6.3 Watermain Relocation

Excavation for the relocation of the 1050 millimetre diameter watermain will be taken to a depth ranging from about 6 to 7.5 metres below ground. Within the east portion of this section of the transitway, excavation will be within sandy silts, silty sands up to 3 metres in depth and glacial till having a thickness of about 4 metres. Towards the west, excavation will be carried out through shale bedrock to a depth of about 6.5 metres. The stabilized groundwater level will be about 5 to 6 metres above invert level over most of the length of the watermain.

In order to minimize disturbance of the subgrade and loss of ground, it is recommended that the excavations for this watermain relocation be carried out within a braced steel trench box. This should serve to control the sloughing of the native soils for the period of the watermain installation. Groundwater inflow from the silts, silty sands and glacial till should be handled by pumping from within the excavation.

It is understood that pipe bedding material specified by the Regional Municipality of Ottawa-Carleton (R.M.O.C.) consists of limestone screenings, which have a maximum size of 9.5 millimetres. The specified bedding thickness is 150 millimetres below the pipe and 200 millimetres on each side of the pipe up to the springline. Clean sand backfill is then to be used to 300 millimetres above the crown of the watermain.

Where invert levels will be in overburden or bedrock above the groundwater table, placement and compaction of the R.M.O.C. standard bedding design should generally be possible. In the event that wet conditions exist at subgrade level, provision should be made in the contract for the use of a coarser bedding material (such as 50 millimetre minus crushed stone) where necessary to firm up the bottom of the trench and serve as a base on which to place the screenings.

Where excavations will be below the water table in sands/silts and glacial till, it is expected that wet and somewhat disturbed conditions at subgrade level will not permit successful placement and compaction of limestone screenings. In this section, the bedding should consist of 300 millimetres of well compacted 50 millimetre minus crushed stone topped with the R.M.O.C. bedding standard.

The trench should then be backfilled with either well compacted sand and gravel or acceptable native materials. All trench backfill in the area of the proposed transitway should be placed in maximum 300 millimetre lifts and compacted to at least 95 percent of the standard Proctor dry density. To minimize differential frost heave in areas where the trench crosses the proposed transitway, it will also be necessary to place native backfill between the frost line and roadway subgrade material. As noted previously, this backfill should match the exposed native materials on the adjacent trench walls and should be compacted in thin lifts to 95 percent of the standard Proctor dry density value.

7. PAVEMENT DESIGN

7.1 Asphalt and Granular Depths

In keeping with present transitway design, it is recommended that a flexible pavement be used throughout this section of the proposed East Transitway. Based on the projected traffic volumes, axle loadings, the anticipated service life and the nature of the subgrade soils, the following asphalt and granular depths are recommended for the various sections of roadway involved in this project.

i) Transitway from Station 9+800 to 10+330

From Station 9+800, located near Tremblay Road at Pickering Place to Station 10+330, the profile grade is to be from about 2 to 5 metres below existing ground surface. Subgrade soils will consist of silty sand, sand and glacial till. Stabilized groundwater levels are indicated to be above the proposed grade level along this section of the Transitway. In preparation for pavement construction, it is recommended that the exposed subgrade be proof rolled with suitable equipment. Any soft areas evident from proof rolling should be subexcavated as necessary and backfilled with well compacted granular material. Drainage of the subexcavated area within 1.5 metres of the pavement surface should be ensured by providing a granular taper between the subexcavation and the proposed subgrade level or between the subexcavation and the adjacent subdrain trench. The bottom of the tapers should have a slope of at least 1 percent. The pavement structure along this section of the Transitway should consist of the following:

140 millimetres of hot mix asphalt over
225 millimetres of M.T.C. Granular A over
610 millimetres of M.T.C. Select Granular B
(100 millimetres minus crushed stone)

ii) Transitway From Station 10+350 to 10+670

Along this section of the transitway, the subgrade material will consist of shale bedrock. To ensure that the shale bedrock remains in a wet environment, it is recommended that a three-layer pavement structure be used. The following pavement design is recommended:

140 millimetres of hot mix asphalt over
225 millimetres of M.T.C. Granular A over
450 millimetres of M.T.C. Select Granular B
(100 millimetres minus crushed stone)

A transition zone between about Station 10+330 and 10+530 will be required as the subgrade changes from overburden to bedrock. The excavation should follow the bedrock, with the granular B thickness decreasing up chainage.

iii) Future Entry/Exit Ramp

A possible future entry/exit ramp is planned to extend from the Transitway at about Station 9+875 to Tremblay Road just north of Avenue L and will be in cut over most of its length. The subgrade soils are indicated to consist of silty sand and glacial till. The pavement structure can be designed to a somewhat reduced granular base equivalent and as such should be comprised of the following:

140 millimetres of hot mix asphalt over
150 millimetres of M.T.C. Granular A over
450 millimetres of M.T.C. Select Granular B
(100 millimetres minus crushed stone)

iv) Transitway Shoulders

A portion of the transitway will be flanked by mountable curbs and a paved shoulder which is to be used for emergency bus traffic, vehicle breakdown and snow storage. Under these circumstances, it is recommended that the pavement design for these shoulder areas consist of the following:

90 millimetres of hot mix asphalt over
150 millimetres of M.T.C. Granular A over
300 millimetres of Select Granular B
(100 millimetres minus crushed stone)

v) Belfast Road

A resurfacing and raising of Belfast Road will be carried out in conjunction with the jacking of the north and south abutments of the Ottawa-Queensway overpass. The proposed profile grade at both abutments is understood to be 0.5 metres above the existing grade and will taper north and south from these points. At Tremblay Road, a grade raise of about 300 millimetres will be required. In preparation for resurfacing and raising, it is recommended that the existing asphalt surface be removed or scarified to facilitate drainage of the new pavement structure. It is recommended that the pavement structure consist of the following

140 millimetres of hot mix asphalt over
150 millimetres of M.T.G. Granular A

Grade raise material should consist of well compacted M.T.C. select granular B (100 millimetres minus crushed stone).

vi) Tremblay Road

It is understood that Tremblay Road is to be rebuilt and widened to a 12 metre wide structure from about Avenue K to Belfast Road. Expansion of the roadway is to be on the north side of the existing roadway.

The present pavement structure, west of Avenue K is indicated to consist of 120 to 150 millimetres of asphalt over 180 to 200 millimetres of sand and gravel. The subgrade material consists of sandy silt fill containing some gravel.

The existing pavement structure is not adequate for the existing traffic loads and the asphalt surface is showing signs of distress and deterioration. It is therefore recommended that reconstruction of Tremblay Road be considered.

In preparation for reconstruction, the existing pavement structure should be removed and the exposed subgrade proof rolled to expose any soft areas. In view of the significant truck traffic on Tremblay Road, the pavement structure should consist of:

140 millimetres of hot mix asphalt over
150 millimetres of M.T.C. Granular A over
375 millimetres of Select Granular B
(100 millimetre minus crushed stone)

Full length subdrains are not considered necessary. However, 3 metre lengths of perforated stub drains should be provided from the catch basins out into the roadway subbase material to provide drainage of the pavement structure.

The existing pavement structure may be rebuilt to provide a short term alternative to full reconstruction. In preparation for widening, it is recommended that all topsoil and fill material be removed adjacent to the existing pavement structure and the subgrade be proof rolled. Although the exact pavement structure along this section of Tremblay Road has not been defined, the pavement structure along the widened section of the roadway should match, as close as practical, the existing pavement structure. A resurfacing layer, having a minimum thickness of 40 millimetres, should be provided on the existing structure. Should any grade raise be required, it is recommended that the existing pavement structure be removed or scarified and the profile raised with granular A material and topped with 140 millimetres of hot mix asphalt.

All roadway granular materials for the transitway, the associated ramp and shoulders, Belfast Road and Tremblay Road should be compacted in even lifts in accordance with M.T.C. Form 501. Maximum compacted densities of the various materials should be in accordance with M.T.C. Form 501, Section 501.06.02.

7.2 Pavement Type

The recommended composition of the asphalt pavement for the transitway and ramps is as follows:

Surface Course	-	40 millimetres HL3 (modified)
Upper Binder Course	-	50 millimetres HL8
Lower Binder Course	-	50 millimetres HL8
TOTAL		<u>140 millimetres</u>

For the shoulders, the following pavement composition is recommended:

Surface Course -	40 millimetres HL3
Binder Course -	<u>50 millimetres HL8</u>
TOTAL	<u>90 millimetres</u>

For Belfast Road and Tremblay Road, the following pavement composition may be considered:

Surface Course	-	40 millimetres HL3 or HL4
Upper Binder Course	-	50 millimetres HL8
Lower Binder Course	-	<u>50 millimetres HL8</u>
TOTAL		<u>140 millimetres</u>

7.1 Pavement Drainage

In order to achieve satisfactory pavement performance with the designs recommended, effective drainage of the pavement structure will be necessary. In this regard, the roadway granular materials should be extended full width and the subgrades shaped to promote drainage. Full length perforated subdrains should be installed along the entire length of this portion of the transitway and for the proposed entry/exit ramp. To facilitate drainage of the groundwater in a lateral direction, the surface of the subgrade should be shaped with a 3 percent fall towards the subdrains.

For bedrock cut areas, it is imperative that the surface of the shale bedrock subgrade remain in a wet environment to avoid possible degradation, swelling and heaving problems. Subdrains in shale cut areas (from Station 10+300 to 10+670) must be set within the roadway base material (select granular B). At no time should the subdrain invert level be lower than the surface of the bedrock at subgrade level.

To provide a uniform subgrade with respect to frost heaving and performance characteristics, backfilling of catch basin laterals located below subgrade level (in overburden) should be completed using acceptable native material. All trench backfill should be compacted to at least 95 percent of the standard Proctor dry density value to provide an acceptable subgrade for the paved areas.

CONSTRUCTION CONSIDERATIONS

It is recommended that allowance be made for participation by experienced geotechnical personnel throughout this project. The bearing surfaces for all foundation elements of the proposed structures should be inspected to ensure that they have been properly prepared prior to placement of concrete or granular base material. As well, inspection of all check holes drilled in foundation areas should be carried out prior to the finalization of the as-built founding elevation. Inspection should also be carried out on an ongoing basis to confirm that the shale bedrock within the footing areas is being adequately protected and that no detrimental swelling or heaving of the shale is taking place.

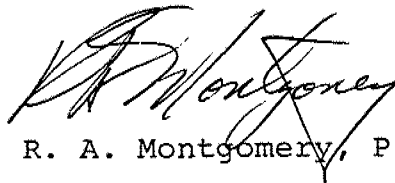
Control on the placement and compaction of all granular fills on the project should be carried out to ensure that grading and compaction requirements are achieved. Careful control on the placement of native materials above the storm sewer and relocated watermain is also recommended.

As indicated in the report, care will have to be exercised by the contractors in the excavation of all overburden and bedrock materials at this site in order to ensure that the procedures used do not jeopardize any of the design details or parameters outlined in this report. Therefore, it is recommended that allowance be made for participation by experienced geotechnical and geological engineering personnel during all phases of construction and that the contractors be required to take advantage of any technical assistance provided by explosives' suppliers and/or blasting specialists. In particular, every effort should be made to reduce damage caused by blasting, both to the overlying overburden slopes and shoring, and to adjacent areas of bedrock which will ultimately support structures.

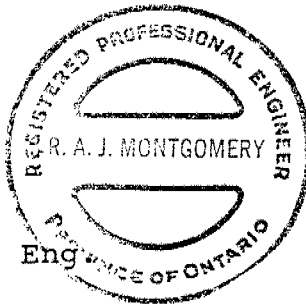
GOLDER ASSOCIATES



A. F. Chevrier



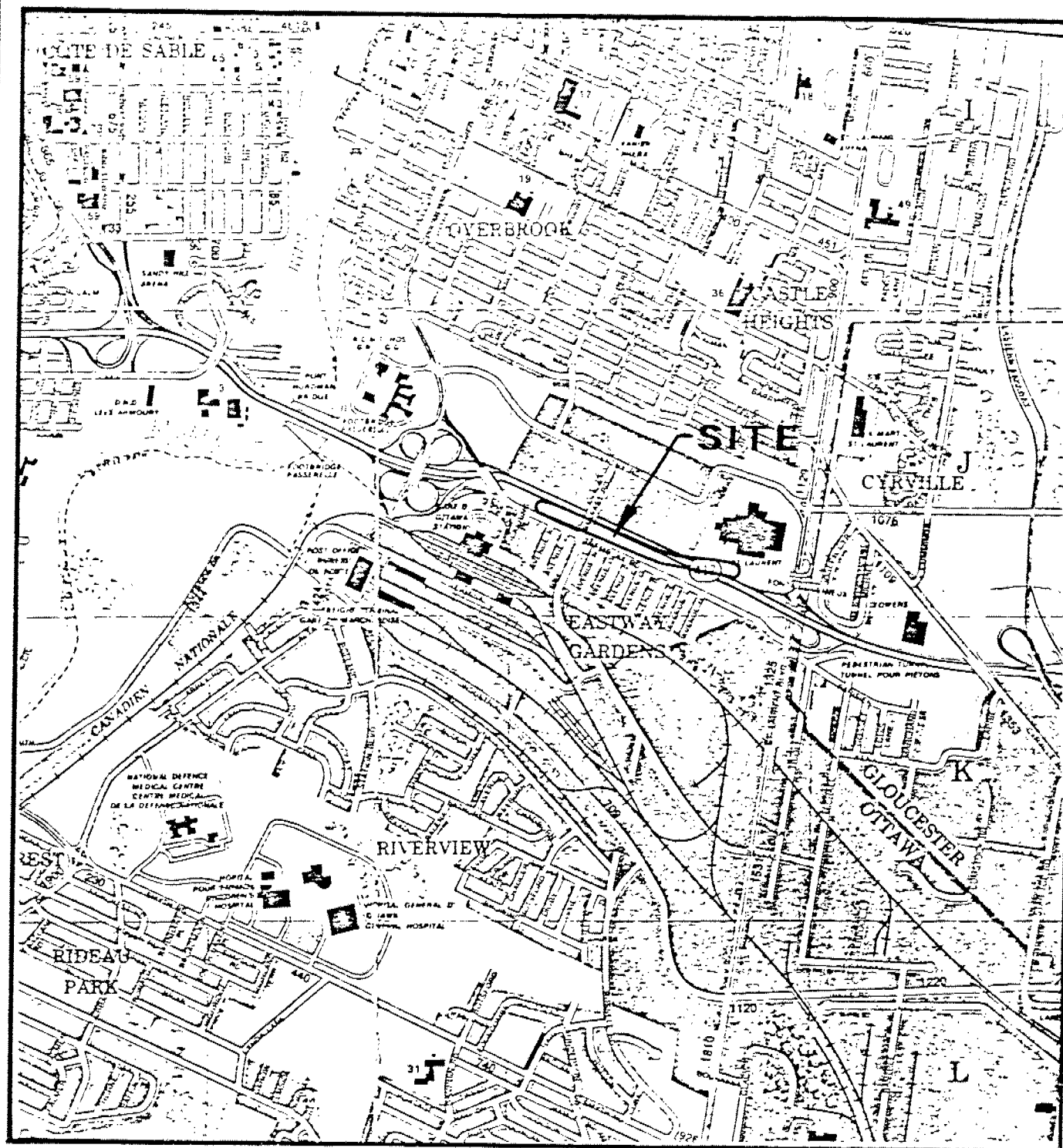
R. A. Montgomery, P. Eng



AFC/RAM/sue

KEY PLAN

FIGURE 1



SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

SCALE
1:25000

Date OCT 3 1984
Project 841-2295

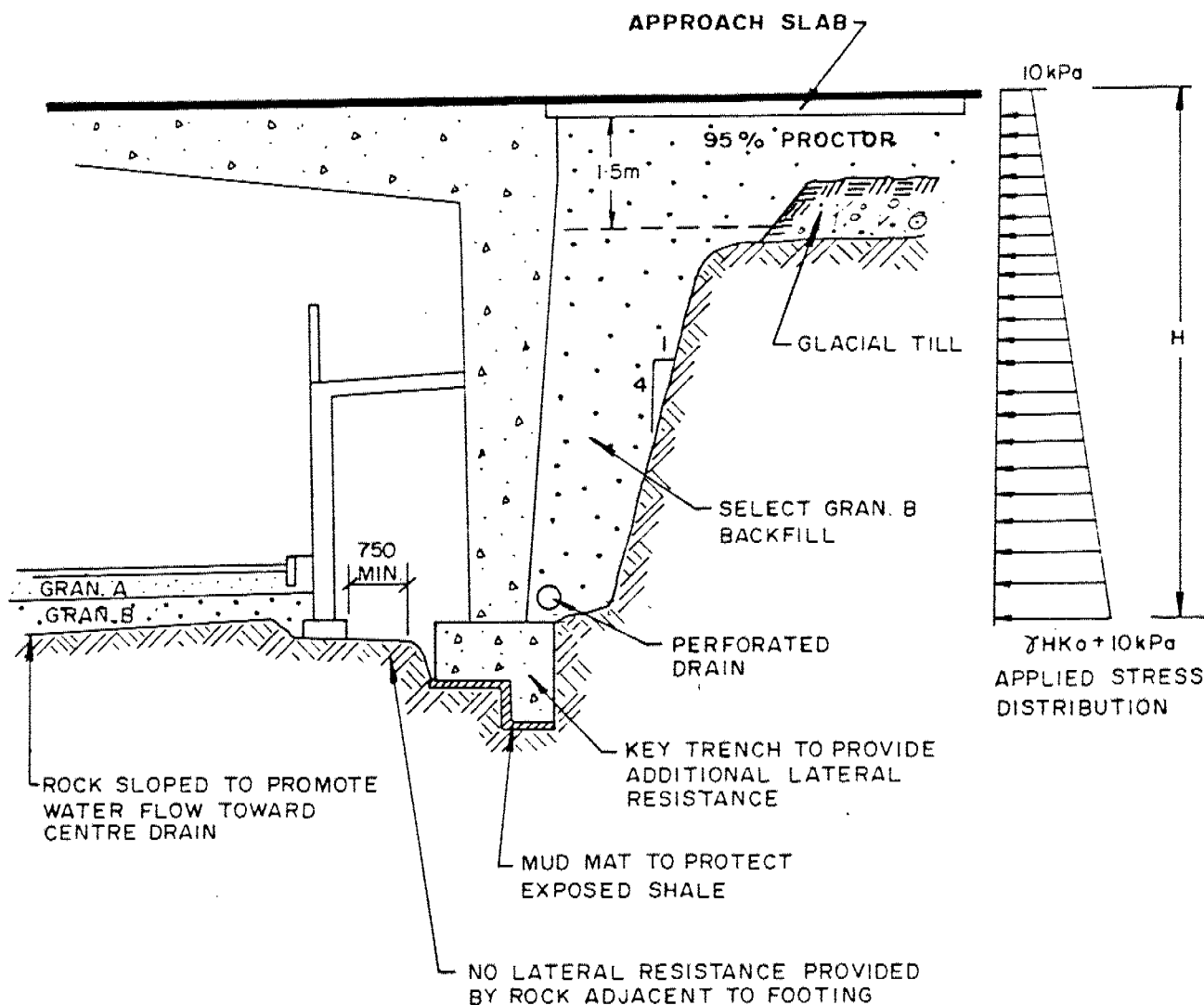
Goldier Associates

Drawn J.C.
Chkd J.C.

OVERSIZE DRAWING

SKETCH OF TYPICAL FOOTING TREATMENT (QUEENSWAY UNDERPASS)

FIGURE 7



SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

NOT TO SCALE

Date NOV 29, 1984
Project 841-2295

Golder Associates

Drawn JC
Chkd J

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST. 9

CONT No

WP No 62-82-02

ST. LAURENT BOULEVARD
INTERCHANGE OVERPASS
GENERAL ARRANGEMENT

SHEET

NOTES

CLASS OF CONCRETE

PRESTRESSED BOX BEAMS	35 MPa
ABUTMENT FOOTINGS	30 MPa
PIERS	30 MPa
ABUTMENTS AND WINGWALLS	30 MPa
DECK	30 MPa
BARRIER WALLS	30 MPa
APPROACH SLABS	30 MPa
REMAINDER	20 MPa

CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100 ± 25
ABUTMENTS & WINGWALLS:	
FRONT FACE	50 ± 20
BACK FACE	70 ± 20
PIERS	80 ± 20
DECK:	
TOP	70 ± 20
BOTTOM	40 ± 10
REMAINDER UNLESS OTHERWISE NOTED	70 ± 20

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS.

CONSTRUCTION NOTES

THE CONTRACTOR SHALL FINISH THE BEARING SEATS TO THE SPECIFIED ELEVATIONS TO A TOLERANCE OF ± 3 mm.
CONSTRUCTION STAGING IS TO BE AS INDICATED ON DWG. 2.

LIST OF DRAWINGS

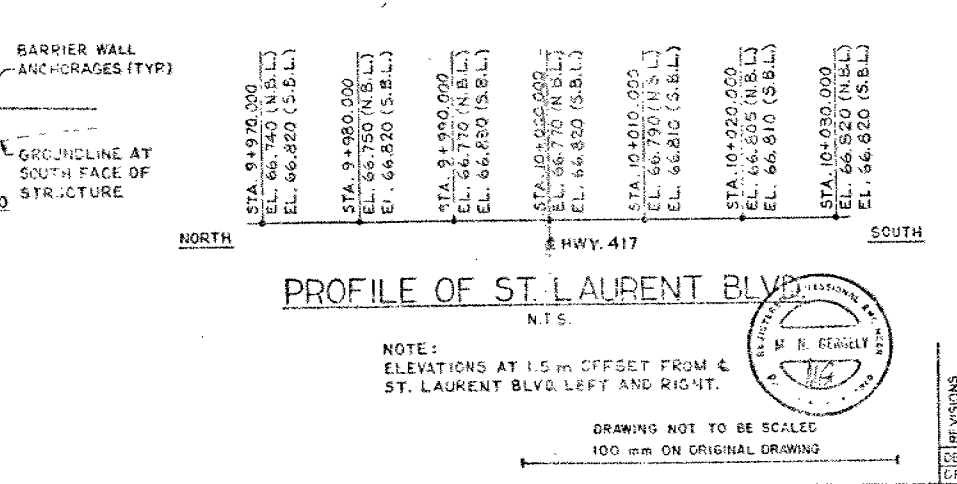
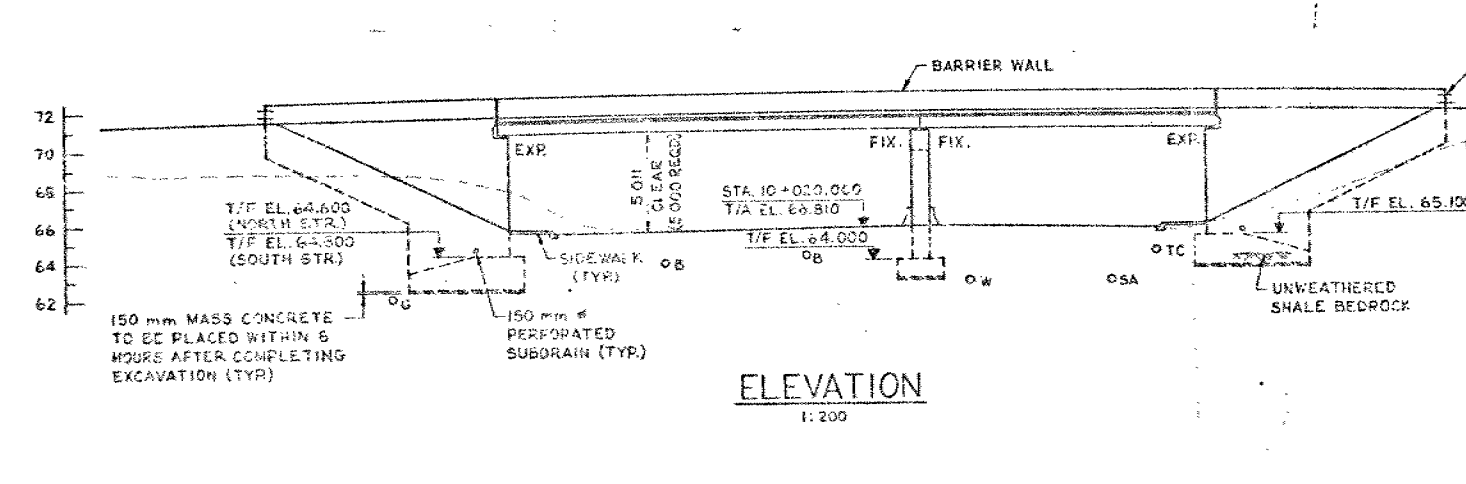
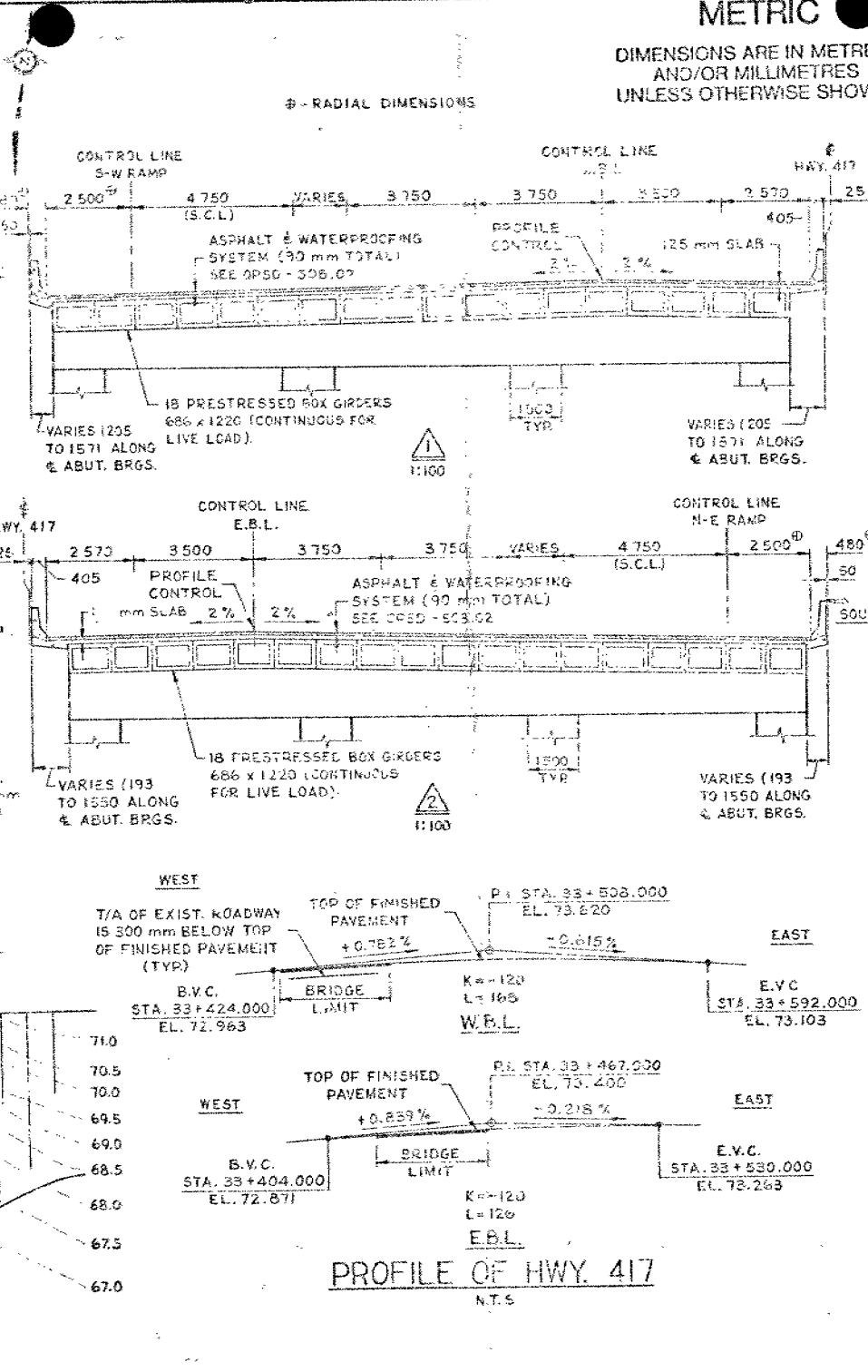
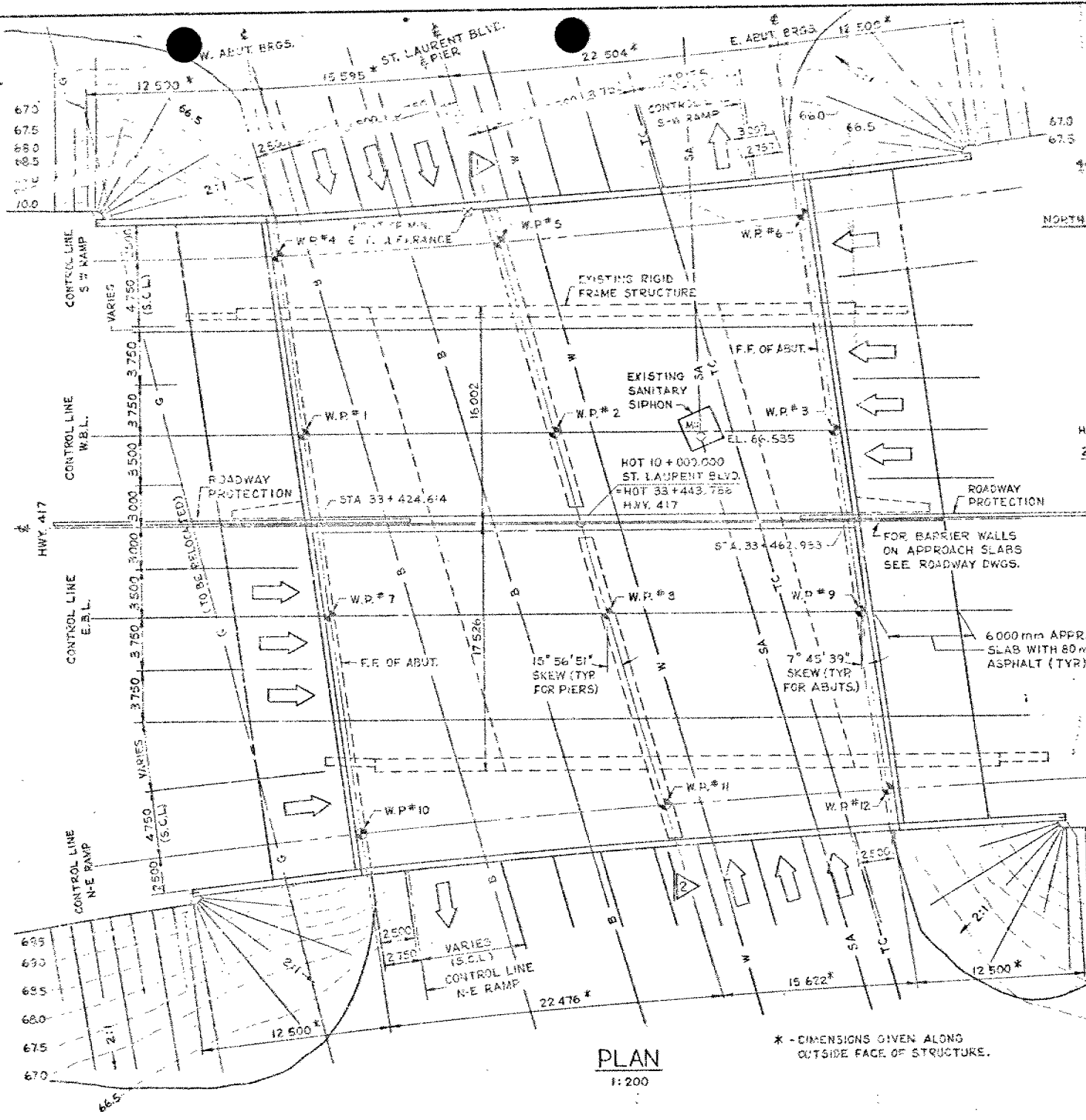
1. GENERAL ARRANGEMENT
2. CONSTRUCTION STAGES
3. BOREHOLE LOCATION & SOIL STRATA
4. FOOTING LAYOUT & REINF.
5. W. ABUTMENT DIMENS. & REINF.
6. E. ABUTMENT DIMENS. & REINF.
7. WINGWALL DIMENS. & REINF.
8. NORTH STRUCTURE PIER
9. SOUTH STRUCTURE PIER
10. PRESTRESSED GIRDERS 'NW'
11. PRESTRESSED GIRDERS 'NE'
12. PRESTRESSED GIRDERS 'SE'
13. PRESTRESSED GIRDERS 'SW'
14. NORTH STRUCTURE DECK DETAILS
15. NORTH STRUCTURE DECK REINF.
16. SOUTH STRUCTURE DECK DETAILS
17. SOUTH STRUCTURE DECK REINF.
18. NORTH STR. NORTH BARRIER WALL
19. NORTH STR. SOUTH BARRIER WALL
20. SOUTH STR. NORTH BARRIER WALL
21. SOUTH STR. SOUTH BARRIER WALL
22. NORTH STRUCTURE 6000 mm APPROACH SLAB
23. SOUTH STRUCTURE 6000 mm APPROACH SLAB
24. EXPANSION JOINTS
25. TEMPORARY ROADWAY PROTECTION
26. AS CONSTRUCTED ELEV. & DIM.
27. BRIDGE DATE & SITE NUMBER DATA
28. ELECTRICAL EMBEDDED WORK
29. QUANTITIES
30. QUANTITIES

LIST OF ABBREVIATIONS

W.P. = WORKING POINT	G = GAS LINE
T/A = TOP OF ASPHALT	B = BELL CANADA
T/F = TOP OF FOOTING	W = WATERMAIN
F.F. = FRONT FACE	SA = SANITARY SEWER
B.F. = BACK FACE	TC = TELECOMMUNICATION
E.F. = EACH FACE	

BM NCC 140

EL. 72.904
TABLET IN TOP OF CONC. S.W.
16.887 RT. 33 + 448.376 HWY 417 & MED.



NOTE:
ELEVATIONS AT 1.5 m OFFSET FROM &
ST. LAURENT BLVD. LEFT AND RIGHT.

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN	W. J.	CHECK
CHECK	W. J.	CHECK
DATE	DATE	DATE

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

DIST. 9

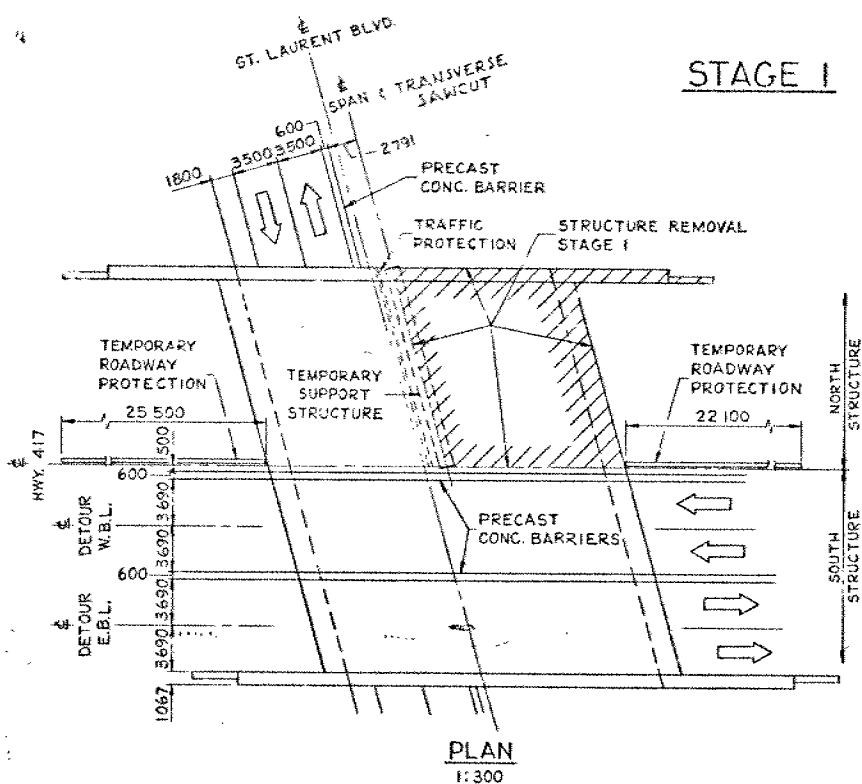
CONT No

WP No 62-82-02

ST. LAURENT BOULEVARD
INTERCHANGE OVERPASS
CONSTRUCTION STAGES

SHEET

STAGE 1

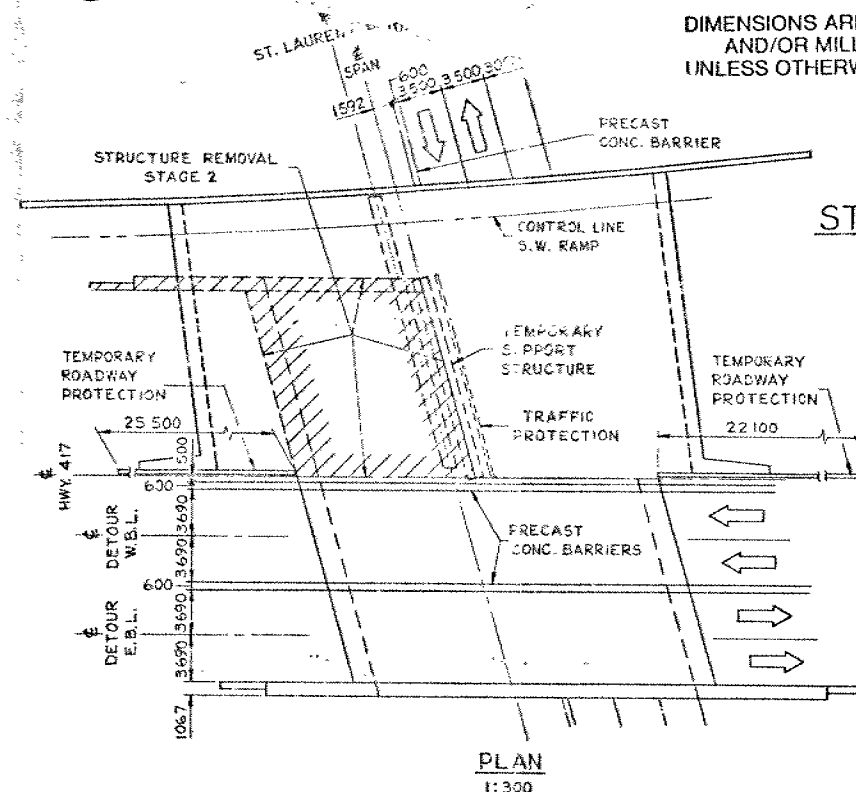


NOTES

- DETOUR HWY. 417 TRAFFIC TO EXISTING SOUTH STRUCTURE, HAVING INSTALLED TEMPORARY PRECAST CONCRETE BARRIERS.
- INSTALL TEMPORARY ROADWAY PROTECTION ALONG & HWY. 417, TO THE HORIZONTAL LIMITS SHOWN.
- EXCAVATE BACKFILL BEHIND BOTH ABUTMENTS OF THE NORTH STRUCTURE SIMULTANEOUSLY, TO THE ELEVATION OF TOP OF FOOTINGS.
- DETOUR ST. LAURENT BLVD. TRAFFIC TO UNDER WEST SIDE OF EXISTING STRUCTURE.
- ERECT TEMPORARY SUPPORT STRUCTURE UNDER MIDSPAN OF NORTH STRUCTURE, AND INSTALL TRAFFIC PROTECTION FOR DETOURED TRAFFIC.
- SAWCUT NORTH STRUCTURE TRANSVERSELY AT MIDSPAN.
- REMOVE EAST PART OF NORTH STRUCTURE.

* THE TEMPORARY SUPPORT STRUCTURE MUST BE CAPABLE OF SUPPORTING AN UNFACTORED VERTICAL UNIFORMLY DISTRIBUTED LOAD OF 180 kN/m ON EACH SIDE OF THE TRANSVERSE SAWCUT.

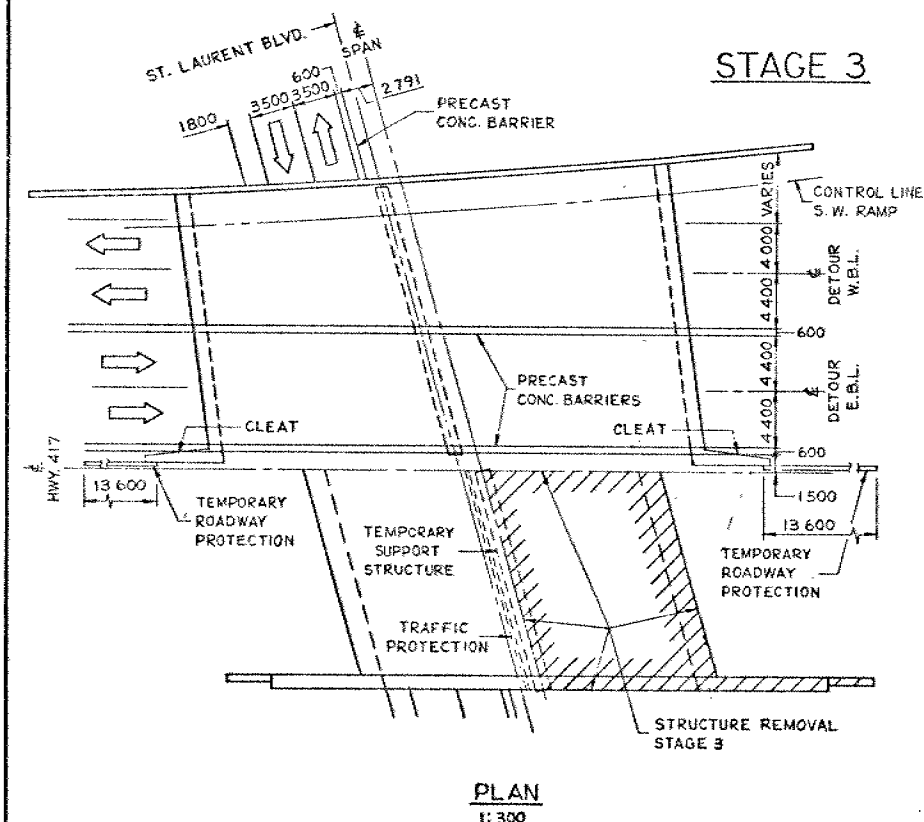
STAGE 2



NOTES

- DETOUR ST. LAURENT BLVD. TRAFFIC TO UNDER EAST SIDE OF STRUCTURE AND INSTALL TRAFFIC PROTECTION.
- REMOVE WEST PART OF NORTH STRUCTURE, AND TEMPORARY SUPPORT STRUCTURE.
- CONSTRUCT NEW NORTH SUBSTRUCTURE.
- ERECT BEAMS OVER WEST SPAN.
- DETOUR ST. LAURENT BLVD. TRAFFIC TO UNDER WEST SPAN OF STRUCTURE.
- ERECT BEAMS OVER EAST SPAN.
- FINISH CONSTRUCTION OF NEW NORTH STRUCTURE.

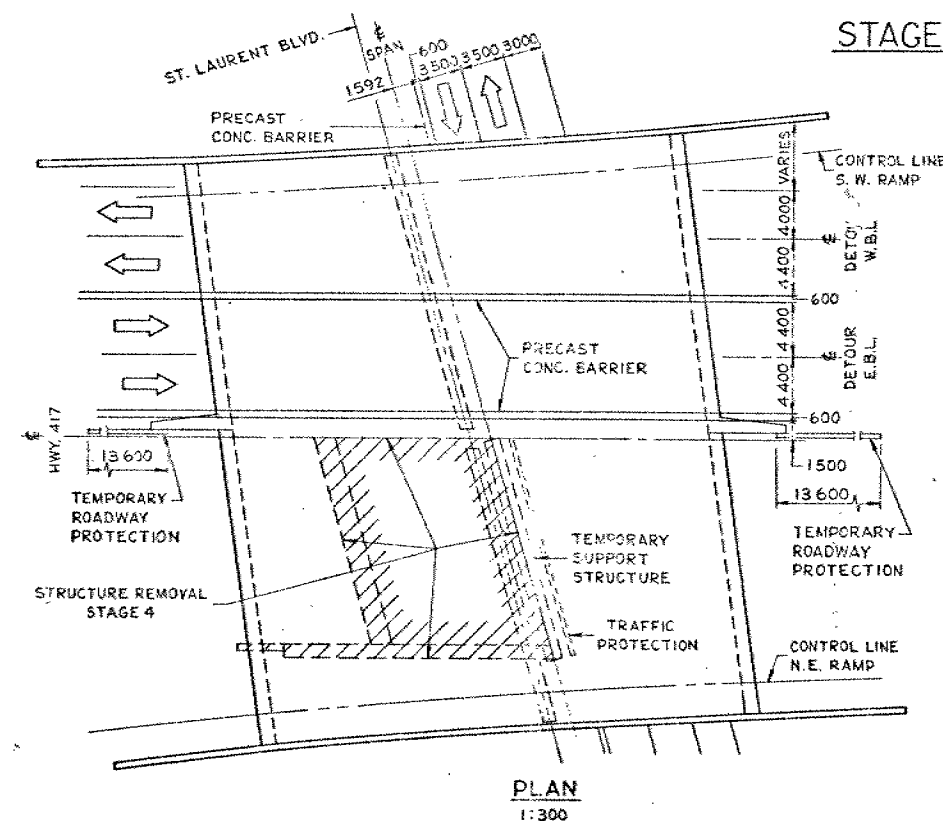
STAGE 3



NOTES

- DETOUR HWY. 417 TRAFFIC TO NEW NORTH STRUCTURE, HAVING INSTALLED TEMPORARY PRECAST CONCRETE BARRIERS.
- REPEAT STEPS 3, 5 AND 6 OF STAGE 1 FOR EXISTING SOUTH STRUCTURE.
- REMOVE TEMPORARY ROADWAY PROTECTION BETWEEN EXISTING SOUTH STRUCTURE ABUTMENT AND NEW NORTH STRUCTURE CLEATS.
- REMOVE EAST PART OF SOUTH STRUCTURE.

STAGE 4



NOTES

- DETOUR ST. LAURENT BLVD. TRAFFIC TO UNDER EAST SIDE OF STRUCTURE AND INSTALL TRAFFIC PROTECTION.
- REMOVE WEST PART OF SOUTH STRUCTURE, AND TEMPORARY SUPPORT STRUCTURE.
- CONSTRUCT NEW SOUTH SUBSTRUCTURE.
- REPEAT STEPS 4 TO 6 OF STAGE 2.
- REMOVE TEMPORARY ROADWAY PROTECTION.
- FINISH CONSTRUCTION OF NEW SOUTH STRUCTURE.
- REMOVE TEMPORARY BARRIERS.
- OPEN ST. LAURENT BLVD. AND HWY 417 TO TRAFFIC.

WORKING POINT DATA

WORKING POINTS	W.P.#1	W.P.#2	W.P.#3	W.P.#4	W.P.#5	W.P.#6	W.P.#7	W.P.#8	W.P.#9	W.P.#10	W.P.#11	W.P.#12
CONTROL LINE	HWY. 417	HWY. 417	HWY. 417	S.W. RAMP	S.W. RAMP	S.W. RAMP	HWY. 417	HWY. 417	HWY. 417	N.E. RAMP	N.E. RAMP	N.E. RAMP
STATION	33+423.728	33+441.929	33+462.097	33+422.012	33+438.041	33+460.110	33+425.500	33+445.643	33+463.869	33+427.483	33+449.524	33+465.581
CO-ORDINATES												
NORTH	5 031 294.479	5 031 296.486	5 031 298.709	5 031 307.046	5 031 309.737	5 031 313.930	5 031 291.753	5 031 283.974	5 031 285.984	5 031 266.572	5 031 270.691	5 031 273.442
EAST	372 443.618	372 461.907	372 481.952	372 440.663	372 456.454	372 478.132	372 447.012	372 467.033	372 485.147	372 450.823	372 472.475	372 488.295
ELEVATION	72.961	73.090	73.201	72.691	72.788	72.836	73.032	73.148	73.223	72.737	72.894	72.973

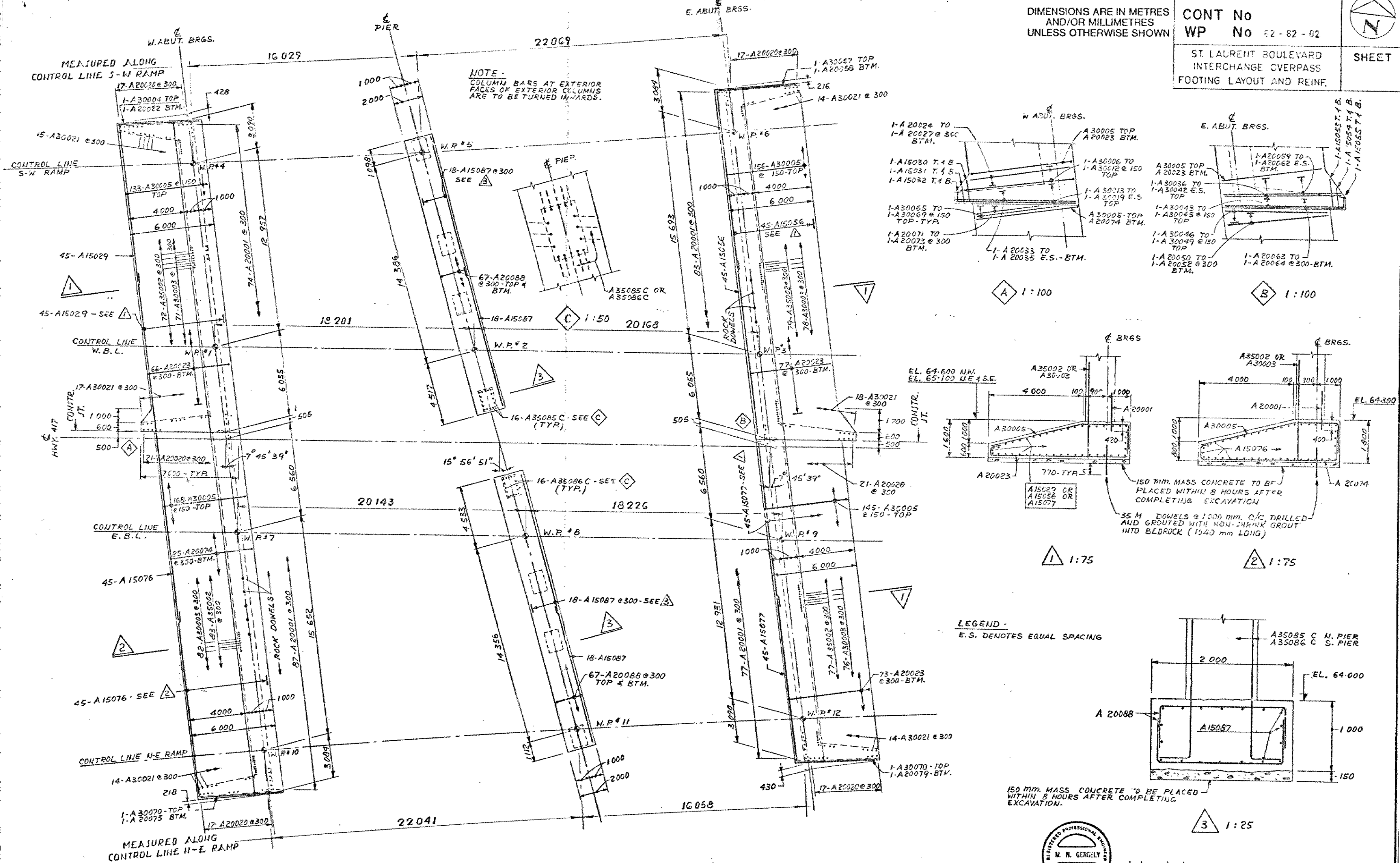
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	W.S.	CHECK	LOADING
DRAWING	W.S.	CHECK	SITE No
			DATE MAY '85
			DWG

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWNCONT No
WP No 62-82-02ST. LAURENT BOULEVARD
INTERCHANGE OVERPASS
FOOTING LAYOUT AND REIN.

SHEET

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN M.G.	CHECK	LOADING 84800 C-A-83	DATE May '85
DRAWING M.G.	CHECK M.G.	SITE No 03-72	DWG 4

SEND
TO

Foundation Design Section
Downsview

FROM

P.A. Waterman Planning + Design

85-07-25

SUBJECT

W.P. 62-82-01 St. Laurent Blvd / Hwy 417

As requested I have attached the following drawings:

1) original plan showing design (photomosaic). This plan does not cover the entire project limits however.

2) original covering entire project limits. (This is the closest thing we have to an 8 1/2" x 11" size plan!) Perhaps the Reprographic Office in Downsview can reduce the plan to an acceptable size for you.

Please return these originals to me once you have finished with them.

Thank-you

REPLY FROM

REPLY DATE

(C O P Y)

Foundation Investigation - St. Laurent Blvd. at Queensway

1. FIELD WORK

Three boreholes were made to supplement the information obtained in the one previous borehole. Since the shale will be the supporting strata for the structure, holes were carried down into the shale until about ten feet of core was recovered having percentage recoveries above 75%.

2. SAMPLE TESTING

Standard penetration tests were made in the boreholes and samples visually classified.

Cores recovered from diamond drilling were examined in detail for the slope and thickness of the bedding planes since a non-uniform slope of the planes indicates a broken condition.

3. OBSERVATIONS

A few feet of loose or organic soils are found beneath the surface, under these is a thin layer of glacial till which is in turn underlain by shale at 5 to 6 feet.

The upper few feet of the shale is broken and indicates weathering or ancient ice action during the glacial period. The soundness of the shale increases in general with depth and at about ten feet is not weathered or broken. The shale is, however, basically a soft laminated deposit with bedding planes only a few inches thick and hence cannot be loaded to high rock bearing values.

Groundwater levels were within a foot or two of the surface and can be considered as nearly at the seasonal low. During wet weather the site is subject to flooding.

4. DESIGN RECOMMENDATIONS

4.1 Strengths

Above El. 210 - Soils not likely for support of structures.

MAR 21 1984
Pavement and Foundation Design Section

El. 210 - 206 - broken shale considered as granular material, bearing capacity 5,000 pounds per sq. ft. except at Hole 2 where sound shale at El. 208.

Below El. 206 - bearing capacity of shale 15,000 pounds per sq. foot.

4.2 Soil Compressibility

Not a factor at this site.

4.3 Soil Shrinkage and Swelling

Not a factor at this site.

4.4 Foundation type

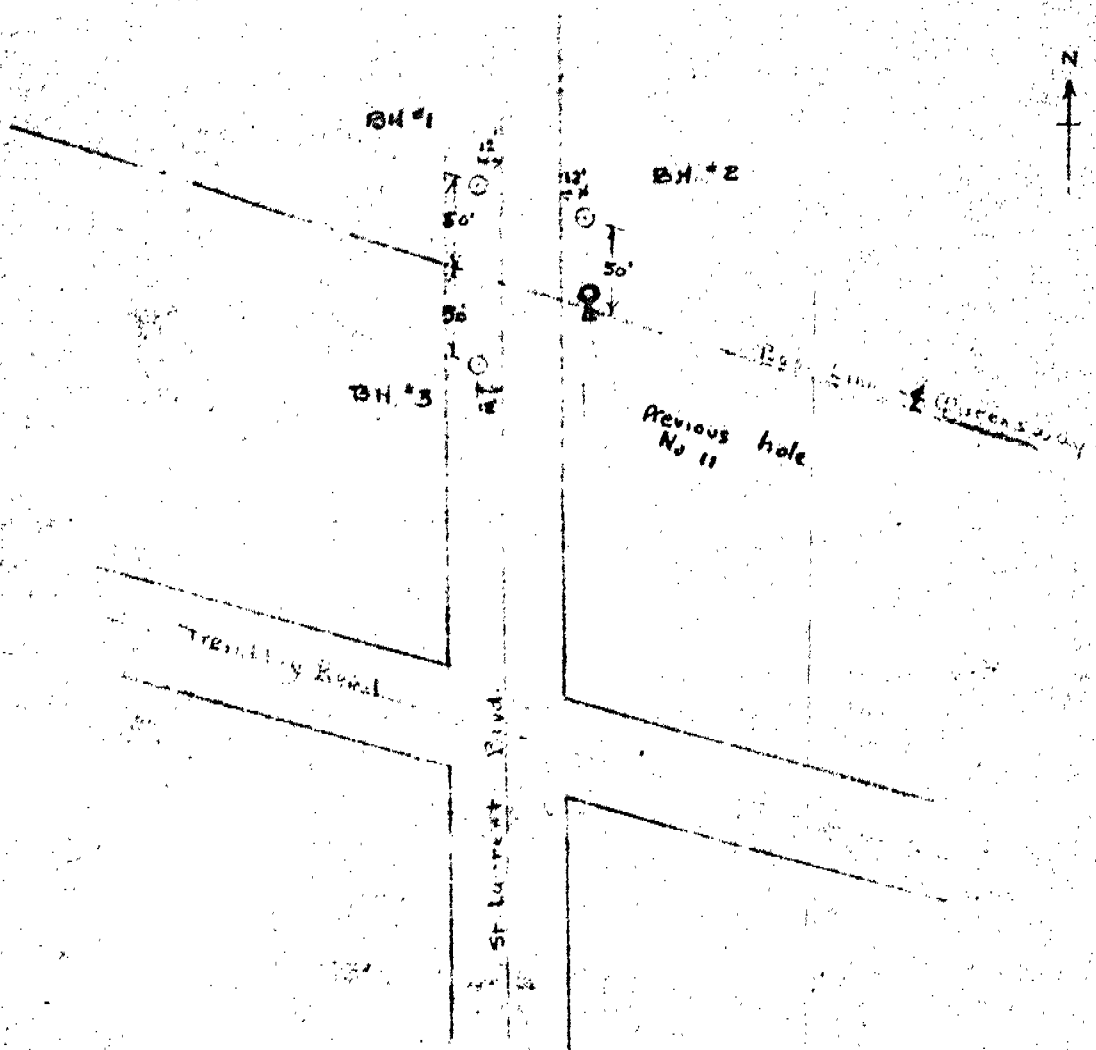
The poor quality of the upper shale layers may make the use of piles, driven from the present ground surface, an interesting alternative to pier foundations, excavated into the shale. We have found that most pile types will penetrate several feet into the soft shale and when pile lengths are estimated, penetration to El. 200 can be assumed. This reasoning should not, however, be used in the contract documents since different pile types have different effects on broken shale and the unbroken shale.

5. CONSTRUCTION PRECAUTIONS

The loose upper soils will be below groundwater during construction and an enlarged excavation will probably be found necessary. A flow of water in the upper shale is also to be expected but can usually be controlled by pumping from pits inside the excavation.

6. CO-ORDINATION

We would be glad to discuss further any points arising from the report.



McROSTIE & ASSOCIATES
CONSULTING ENGINEERS

BOREHOLE LOCATIONS
ST. LAURENT AT QUEENSWAY

SCALE 1" = 100'

PLATE 1

MASTINE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

ST. LAURENT BLVD.
 & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 216.1
 REMARKS GEODETIC DATUM

HOLE NO. 1

DATE JUNE 3-4 '57

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST			
							LINE HAMMER	NO CASING	INCH DIA. ROD	
							BLOWS PER FOOT			
GROUND SURFACE					0	216.1				
				TOP SOIL	1.0					
				LOOSE	2					
				ORGANIC SOIL	4.0	212.1				
				MEDIUM-DENSE TILL	5.5					
				VERY DENSE TILL	6.1	210.0				
				BRITISH HALE 120 H	13					
				(DRILLED 60% RECOVERY)	10.1	206.0				
				SHALE ROCK	12					
				(DRILLED - 90% RECOVERY)	13.7					
				SHALE ROCK	18					
				(DRILLED - 100% RECOVERY)	20					
					21.2	184.9				
				BOTTOM OF HOLE						
							% WATER CONTENT			
							PLATE			
							2			

ST. LAURENT BLVD.
QUEENSWAY

HOLE No.

DATE JUNE 5, 1957

2

100-101814-10

memorandum



To: Mr. M. Devata
Chief Foundation Engineer
Foundation Design Section
Central Building, Room 315

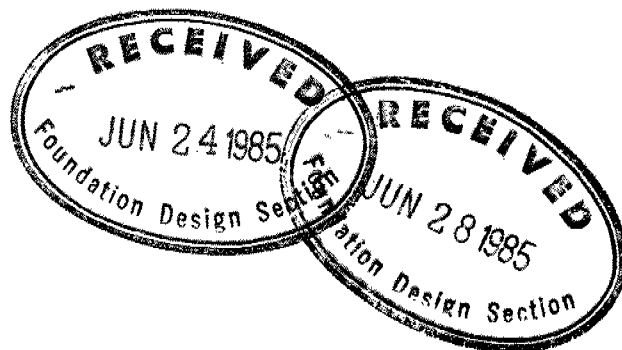
Attn: Mr. L. Politano

From: Soils & Aggregates Section
Engineering Materials Office
Central Building, Room 311

Date: 85 06 21

File No.: 3162-2-4-113

Re: Borehole Core Descriptions
High Mast Lighting,
St. Laurent Blvd. and Hwy. 417,
Ottawa,
W.P. 62-82-01



As requested by your section, core from thirteen(13) boreholes was logged, and descriptions are appended. Depth to top of bedrock and depth to top of sound rock in each borehole are tabulated below:

BOREHOLE NUMBER	DEPTH TO BEDROCK (in metres below ground surface)	DEPTH TO SOUND ROCK (in metres below ground surface)
C1	8.84	*
C2	5.13	6.22
C3	6.20	6.86
C4	2.21	2.44
C5	2.31	3.43
C6	4.72	4.72
C7	2.54	3.00
C8	1.98	2.29
C9	3.99	3.99
C10	2.67	2.90
C11	4.19	4.19
C12	4.27	4.27*
C13	8.46	9.14

* See notes

If you have any questions, please contact me.

E.R. Magni,
Geologist.

ERM/jlo
Attachment

NOTES:

- 1) Bedrock is black shale of the Billings Formation. You are reminded of the potential heave characteristics of this shale. I have previously supplied you with information on this subject.
- 2) Rock in the zone between top of bedrock and top of sound rock is typically weathered and/or highly jointed. Sound rock is typically unweathered, and less severely jointed.
- 3) Borehole C1: High core losses were recorded in this borehole, which are apparently a result of poor drilling. In my opinion, this rock mass is probably no different than any other. If rock anchors are used at this site, I recommend, however, that anchor lengths in rock be increased by 50 percent to be safe.
- 4) Borehole C12: The rock mass in this borehole consists of alternating sound and weathered zones. If rock anchors are used at this site, I recommend anchor lengths in rock be increased by 50 percent to be safe.

DESCRIPTION OF ROCK CORE - W.P. 62-82-01

BOREHOLE NUMBER				CORE DESCRIPTION	
	DEPTH (m)	% CR *	% RQD *	DEPTH (m)	DESCRIPTION
C1	8.84m-10.34m 10.34m-11.89m	63 59	0 59	8.84 - 11.89	High core loss apparently due to poor drilling; Shale, black, slightly weathered, becoming unweathered with occ. thin limestone layers. Top of sound bedrock difficult to define.
C2	5.13m- 6.58m 6.58m- 8.18m	100 98	23 46	5.13m - 6.22m 6.22m - 8.18	Shale, black, slightly weathered, very closely spaced joints, with occasional thin limestone layers Shale, black, unweathered, closely spaced joints, with occasional (5%) thin limestone layers (6 cm)
C3	6.20 - 7.75 7.75 - 8.92	100 100	34 72	6.20 - 6.86 6.86 - 8.92	Shale, black, slightly weathered, very closely spaced joints Shale, black, unweathered, closely spaced joints, with occasional (2%) thin limestone layers (9 cm)
C4	2.21 - 3.68 3.68 - 5.28	100 100	66 67	2.21 - 2.44 2.44 - 5.28	Shale, black, slightly weathered, very closely spaced joints Shale, black, unweathered, closely to medium spaced joints, with occasional (5%) thin limestone layers (4 cm)
C5	2.31 - 3.89 3.89 - 5.36	100 100	8 67	2.31 - 3.43 3.43 - 5.36	Shale, black, moderately weathered, very closely spaced joints, with occasional (2%) thin limestone layers (2 cm) Shale, black, unweathered, closely spaced joints, with occasional (10%) thin limestone layers (7 cm)
C6	4.72 - 6.15 6.15 - 7.75	100 100	54 27	4.72 - 7.75	Shale, black, unweathered, with occasional zones of slightly weathered shale
C7	2.54 - 3.56 3.56 - 5.05	95 100	20 61	2.54 - 3.00 3.00 - 5.05	Shale, black, slightly weathered, close to very closely spaced joints Shale, black, unweathered, close to medium spaced joints

* CR= CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

DESCRIPTION OF ROCK CORE - W.P. 62-82-01

BOREHOLE NUMBER				CORE DESCRIPTION	
	DEPTH (m)	% CR*	% RQD*	DEPTH (m)	DESCRIPTION
C8	1.98 - 3.51	93	60	1.98 - 2.29	Shale, black, moderately weathered, very closely spaced joints
	3.51 - 5.03	100	47	2.29 - 5.03	Shale, black, unweathered, very closely spaced joints
C9	3.99 - 5.49	100	78	3.99 - 5.49	Shale, black, unweathered, closely spaced joints
C10	2.67 - 4.25	100	31	2.67 - 2.90	Shale, black, slightly weathered, very closely spaced joints
	4.25 - 5.38	100	62	2.90 - 5.38	Shale, black, slightly weathered, becoming unweathered, closely spaced joints with zones of very close spacing
C11	4.19 - 5.76 5.76 - 7.21	98 100	84 72	4.19 - 7.21	Shale, black, unweathered, closely to medium spaced joints
C12	4.27 - 5.84	100	6	4.27 - 5.84	Shale, black, slightly weathered, very closely spaced joints, with moderately weathered, very closely spaced zones at 4.57-4.77, 5.21-5.26 and 5.36-5.77
C13	8.46 - 9.98	100	53	8.46 - 9.14	Shale (80%), black, slightly weathered, very closely spaced joints, with limestone (20%) layers up to 10 cm thick
	9.98 - 11.51	100	50	9.14 - 11.51	Shale (90%), black, unweathered, closely spaced joints, with limestone (10%) layers up to 12 cm thick

* CR = CORE RECOVERY ; RQD = ROCK QUALITY DESIGNATION

JUL 5 1 37 PM '85

DNA697

KIN

JULY 5/85 125 P

T KINGSLAND HD STRUCT SECT

- ATN E C LANE

RE WP 62-82-01 - HWY 417, ST LAURENT INTERCHANGE - HIGH MAST
LIGHTING - FOUNDATION DESIGN SECT. MEMO
DATED 85 07 02 (PRELIM. FOUND. RECS.)

PLEASE NOTE THE FOLLOWING CORRECTIONS TO THE ABOVE MENTIONED MEMO

1) PG. (D) (7TH LAST LINE)

FOURTH WORD IN SENTENCE SHOULD READ PARAMETERS NOT PAVEMENTS

2) PG (4) (3RD LAST LINE)

PU IN PSI (NOT IN PIS)

3) PG (5) UNDER C1, 3RD LINE

60.0 TO 60.6 SHOULD BE 60.0 TO 58.6

4) PG (5) UNDER C2, 2ND LINE 64.2 TO 62.5 SHOULD READ 64.2 TO 62.7

5) PG (5) UNDER C8 1ST LINE 68.3 SHOULD READ 66.3

6) PG (5) UNDER C9, 1ST LINE 63.9 SHOULD READ 69.3

NOTE

AT C7 AND C12 THE BH WERE NOT DRILLED AT THE EXACT POLE LOCATIONS

L. POLITANO FOUNDATION DESIGN SECT

K

ACC TPA 697 07051334

642581

JUL 5 2 53 PM '85

DNA702

KIN

JULY 5/85 250 P (CORRECTION TO DNA 697 125 P)

T KINGSLAND

ATN E C LANE

RE WP 62 81 01 HWY 417

THE 2ND LINE SHOULD READ
1) PG. 4 (7TH LAST LINE)

5TH LINE SHUD READ

QU IN PSI (NOT IN PIS)

L POLITANO FOUNDATION DESIGN SECT
K



memorandum



To: T.C. Kingsland
Head, Structural Section
Eastern Region
(Kingston)

Date: 1985 07 02

From: Foundation Design Section
Room 315, Central Building
Downsview

ATT: E.C. Lane

Re: W.P. 62-82-01, Hwy 417 (Dist. 9 Ottawa)
St. Laurent Blvd. Interchange
High Mast Lighting
Foundation Investigation Preliminary Recommendations

The fieldwork for the above-noted project has been completed. The investigation was conducted between 85-06-12 and 85-06-17 and consisted of advancing one borehole at each high-mast lighting location. The boreholes included sampling the overburden at 0.75 m intervals and coring the shale bedrock for depths of up to 3.0 m.

Our complete and detailed foundation investigation report will be issued in the future. This memo outlines the preliminary recommendations pertaining to the design and construction of the high-mast lighting foundations.

DISCUSSION AND RECOMMENDATIONS

In conjunction with the proposed interchange at St. Laurent Blvd. it is proposed to provide illumination utilizing 13 high mast light pole installations. The following table indicates the location and pole height of each installation in addition to the existing and proposed grade at each location.

.../2

<u>POLE</u>	<u>STATION</u>	<u>OFFSET (m)</u>	<u>PROPOSED GRADE AT POLE LOCATION</u>	<u>EXISTING GRADE</u>	<u>POLE HEIGHT (m)</u>
C1	32+565	58 South	69.91	66.54	25
C2	32+716	58 South	69.31	67.76	25
C3	32+879	58 South	69.44	72.08	25
C4	33+041	48 South	68.10	67.97	30
C5	33+195	50 South	68.25	68.09	30
C6	33+340	37 South	68.30	68.80	30
C7	33+522	24 North	70.20	69.61	30
C8	33+496	70 South	66.41	66.27	30
C9	33+698	42 South	71.30	71.37	30
C10	33+868	35 South	71.36	71.35	30
C11	34+040	35 South	71.87	72.03	30
C12	34+205	35 South	69.75	70.14	30
C13	32+418	58 South	63.90	63.80	25

Conventional spread footings for these light poles would likely be quite expensive. However, high mast light poles have been installed economically in many areas of North America and Europe using a design method proposed by B.B. Broms and others in which the poles are supported on concrete caisson pile. The Structural Office has decided to adopt this same method described by Broms in two separate papers; Broms B.B. "Lateral Resistance of Piles in Cohesive Soils", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM2, Paper 3825, March 1964.; and "Lateral Resistance of Piles in Cohesionless Soils", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM3, Paper 3909, May, 1964.

In the following paragraphs, the feasibility of constructing concrete caissons at the site is discussed and the various parameters to be used in the caisson design are provided.

At most locations the poles are to be installed in the original ground in areas where no significant amounts of cut or fill is required. The exceptions are of follows:

at C-1, 3.4 m± fill

at C-2, 1.6 m± fill

at C-3, 2.6 m± cut

DESIGN

The contribution of fill material should be ignored from a lateral resistance point of view at those locations where fill is required. Similarly, in all cases, the soil within the zone of frost penetration (1.8 m) should not be counted on to provide lateral strength.

For the cohesive soils located at this site, the coefficient of horizontal subgrade reaction should be computed in accordance with the following formula: (The design parameters are presented in Imperial Units, since the design example provided by the Structural Office used Imperial Units throughout.

$$K_h = \frac{n_1 n_2 80 q_u}{D}$$

D

Where:

K_h - coefficient of horizontal subgrade reaction (lb/in³)

D - Diameter of concrete caisson pile (in)

n_1 - coefficient as defined below:

Unconfined Compressive Strength

q_u (psi)	n_1
Less than 7	0.32
7 to 28	0.36
Greater than 28	0.40

n_2 - coefficient based on pile material = 1.15 for concrete

q_u - unconfined compressive strength (psi)

For the non-cohesive soils, K_h should be computed from the following formula:

$$K_h = n_h \frac{z}{D}$$

K_h - coefficient of horizontal subgrade reaction (tons/ft³)

z - depth below ground surface (ft.)

D - diameter of caisson (ft)

n_h - Coefficient evaluated as follows:

Coefficient n_h in tons/ft³

Relative Density	Loose	Compact	Dense
Above Groundwater table	7	21	56
Below Groundwater table	4	14	34

The following soil ^{parameters} ~~pavements~~ are recommended for the design of the high-mast light caisson:

(note: ϕ = apparent angle of internal friction for non-cohesive soils

q_u = unconfined compressive strength in ~~psi~~ ^{psf}

($q_u = 2c_u$)

γ = unit weight in pcf

<u>Pole</u>	<u>Elev.(m)</u> <u>From-to</u>	<u>Type of Soil</u>	<u>ϕ</u>	<u>q_u</u> <u>psi</u>	<u>γ</u> <u>pcf</u>
C-10	71.4-70.8	non-cohesive	32°	-	125
	70.8-68.7	cohesive	-	60	135
	68.7-	shale	see note 1		
C-11	72.0-71.4	non-cohesive	32°	-	125
	71.4-67.8	cohesive	-	70	140
	67.8-	shale	see note 1		
C-12	70.6-69.5	non-cohesive	35°	-	135
	69.5-66.8	cohesive	-	15	125
	66.8-	shale	see note 1		
C-13	63.8-60.2	non-cohesive	28°	-	122
	60.2-59.0	cohesive	-	4	120
	59.0-55.3	cohesive	-	50	130
	55.3-	shale	see note 1		

NOTES:

1. If caissons are to be augered into the shale the following values are to be used:

highly weathered	$q_u = 80$ psi, $\gamma = 140$ pcf
slightly weathered	$q_u = 700$ psi, $\gamma = 150$ pcf
unweathered	$q_u = 1400$ psi, $\gamma = 160$ pcf

Attached log sheets should be referred to for the zones of the various weathering

2. If rock anchors are to be installed, the following bond stresses can be used.

highly weathered	15 psi
slightly weathered	30 psi
unweathered	100 psi

3. As previously noted, the contribution of existing or proposed fill material, and 1.8 m frost penetration zone should be neglected in the design.

4. Groundwater levels are as follows:

<u>POLE</u>	<u>ELEV.</u>	<u>POLE</u>	<u>ELVE.</u>
C-1	62.4	C-8	65.7
C-2	64.9	C-9	69.8
C-3	68.7	C-10	70.5
C-4	66.5	C-11	70.8
C-5	66.6	C-12	69.4
C-6	66.8	C-13	59.5
C-7	65.7		

If you have any questions please contact the undersigned. The final foundation report will be issued in the future.

L. Politano, P. Eng.
Project Foundations Engineer
for
M. Devata, P. Eng.
Chief Foundations Engineer
(East)

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 62-82-⁰²~~01~~ DIST 9
HWY 417 STR SITE 3-72

St. Laurent Blvd. Overpass

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GEOCRES 31G5-137

DATE JUN 20 1984

JUN 20 1984

FOUNDATION INVESTIGATION REPORT

For

St. Laurent Blvd. Overpass

W.P. 62-82-01, Site 3-72

Hwy. 417, District 9, Ottawa

INTRODUCTION:

This report summarizes the factual information obtained from a foundation investigation carried out at the above-mentioned site between 84 04 17 and 84 04 19. The fieldwork consisted of 11 sampled boreholes of which 6 were accompanied by cone penetration tests. In addition, 2 boreholes also accompanied by cone tests, were advanced by only augering to locate bedrock. In total, the 13 boreholes ranged in depth from 2.4 to 10.1 m.

Bedrock was proven in 5 of the 13 boreholes by obtaining up to 3.4 m of BXL rock core.

SITE DESCRIPTION AND GEOLOGY

The site is located at the existing Hwy.417 - St. Laurent Blvd. Overpass in the eastern end of Ottawa in the Regional Municipality of Ottawa-Carleton (RMOC). Land use in the vicinity of the site is predominantly developed as urban commercial. Topography across the site is generally flat.

The site lies on a glacial till plain characterized by glacial till and silty sand deposits. In addition, however, silty clay and organic deposits were also identified. The underlying bedrock in the area consists of black shale of the Billings formation and is found some 3 m below the existing St. Laurent Blvd. grade.

SUBSURFACE CONDITIONS

General

The subsurface conditions are quite variable at this site. The boundaries between the soil types, insitu and laboratory test results, and groundwater levels, are shown on the attached Record of Borehole Sheets. The locations and elevations of the borings, along with profiles showing estimated stratigraphical sections based on borehole data, are shown on Drawings 628201-A and 628201-B.

The various soil types encountered are briefly described in the following paragraphs.

Peat

Dark brown to black peat was found in BH #4, 5 and 6 as the surficial deposit. In BH #4 and 5, the peat deposit is 1.4 and 1.1 m thick respectively. In BH #6 the peat deposit is 3 m thick and extends down to the layer of silty clay at elevation 65.9.

The peat found at this site appears to be at an intermediate stage of decomposition as it does not have a totally fibrous texture. Root fibres and pieces of decomposed wood are however, still evident. This organic deposit has a spongy consistency and is quite compressible.

Results of moisture content testing on two samples of the material indicate 98% and 119.5% natural moisture contents.

Silty Clay

Silty clay was found in BH #4, 5, 6 and 8. This stratum varies in thickness from 0.3 to 0.7 m and is found at an elevation of 65.5 to 66.1.

The results of Atterberg Limits testing carried out on 4 samples of this cohesive material are plotted on Fig.1 in the Appendix and indicate that stratum is generally composed of a silty clay of low plasticity (CL group).

The result of a grain size distribution test carried out on one sample of this material can be summarized as follows:

Clay	39%
Silt	55%
Sand	5%
Gravel	1%

Based on this information, this stratum can be described as a silty clay, trace sand, gravel. Testing for organic content indicates 1.8 to 6.1% organics in this silty clay. However, the organic material was usually encountered as intrusions in the matrix.

Based on the interpretation of Standard Penetration test 'N' values, this material is considered to have a consistency of stiff to very stiff.

Organic Clay, Organic Silty Clay

Organic clay or organic silty clay was encountered in BH #7, 8, 9, 10 and 11. The stratum varied from 0.3 to 2.3 m in thickness, and was found between elevations of 65.2 and 67.3. In BH #8, 9, and 10, the material was found immediately beneath the existing structure approach fills.

Results of Atterberg Limits testing conducted in two samples of this material are shown on Fig.2 in the Appendix.

Based on the interpretation of Standard Penetration Test 'N' values, the consistency of this material is described as soft to stiff.

Glacial Till

Glacial till was found only at the east side of the project site in 4 boreholes. At all locations, the till was immediately overlying the shale bedrock.

In BH #7 and 8, the glacial till is of a cohesive type and has a thickness of 0.7 m. This till was found at elevations 65.0 and 65.5 in BH #7 and 8 respectively.

This till is described as a heterogeneous mixture of silty clay, sand and gravel. Testing for organic content indicates 1.4 to 2.1% organics in the till. However, the organic material was usually encountered in the upper zones. The results of Atterberg Limits testing carried out on samples from this deposit are plotted on Fig.3 and indicate that the till matrix is a silty clay of low plasticity (CL group).

Based on Standard Penetration test 'N' values of 19-29 blows/0.3 m, the consistency of the till is interpreted as being very stiff.

In BH #4 and 5, the glacial till is non-cohesive and has a thickness of 1.0 m in BH #4 and 0.5 m in BH #5. This till was found at an elevation of approximately 65.

This till is described as a silty sand, trace clay, some gravel. The results of grain size distribution testing conducted on 2 samples from this stratum indicate a reasonably uniform distribution as described below:

Gravel	18-22%
Sand	40-49%
Silt	28-33%
Clay	5%

Based on the interpretation of Standard Penetration test 'N' values, this stratum is considered to be very dense.

Fill

Fill used for the existing structure approaches was encountered in BH #8, 9 and 10. The height of the fill ranged from 5.3 m in BH #3 to 6.1 m in BH #10. In all cases the fill overlies the organic silty clay stratum.

The results of Atterberg Limits testing carried out on 5 samples of this cohesive material are plotted on Fig.4 and indicate that the fill is composed of a silty clay of intermediate plasticity (CI group) to a clay of high plasticity (CH group).

Field vane tests indicate that the shear strength of the fill ranges from 44 kPa to over 100 kPa. Sensitivity of the fill varies from 5 to 12. Based on Standard Penetration test 'N' values of 3-13 blows/0.3 m, the fill is considered to have a low to high degree of compaction. Natural moisture content of the fill ranges from 39-46.5%.

The results of grain size distribution tests carried out on 5 samples of the fill material indicate the following results:

Clay	55-69%
Silt	18-32%
Sand	3-24%
Gravel	0%

An isolated pocket of organic silty clay was encountered in BH #8 in the upper 1.0 m of the fill.

Silty Sand

Silty sand was found in BH #1, 2 and 11. The stratum varied in thickness from 1.0 m in BH #11 to 1.8 m in BH #1. In BH #1 and 2, the silty sand deposit immediately overlies the shale bedrock. In BH #11, the silty sand deposit overlies an organic silty clay deposit.

Results of grain size distribution tests conducted on 4 samples are shown in envelope form in Fig.5 and indicate variance in the sand and silt contents and reasonably uniform gravel and clay contents. The distribution can be summarized as follows:

Gravel	6-10%
Sand	56-75%
Silt	12-30%
Clay	4-6%

Based on the above observations, this material can be described as silty sand, trace sand, gravel.

Interpretation of the Standard Penetration test 'N' values indicate that this non-cohesive material is in a compact to very dense state.

Shale Bedrock

Bedrock at the site was proven in 5 of the 13 boreholes by obtaining up to 3.4 m of BXL rock core. In the remaining boreholes, split-spoon samples of the weathered bedrock were recovered or augering was advanced to refusal.

Bedrock at the site was found some 3 m below the native overburden or up to 8.4 m below the existing structure approach fills. These depths correspond to a bedrock elevation ranging from 63.9 to 65.7.

Bedrock at this location consists of black fissile shale of the Billings Formation of the Ordovician Period. The upper 0.3 to 1.7 m zone is in a highly weathered state. In most boreholes, it was possible to drive a split-spoon through the weathered zone or auger through it.

The thinly and horizontally bedded shale of the Billings Formation may, in some instances, be susceptible to slaking and degradation when exposed to the atmosphere. Consequently, the bottom of an excavation in this type of shale may experience heaving if the excavation is kept open for a considerable length of time.

The core recovery attained in the cored boreholes (BH #1, 4, 6, 7, 12) ranged from 68 to 100%. Borehole 4, sample RC-4 yielded a recovery of 38%. This unrealistic value can be attributed to mechanical problems experienced during the coring process.

Rock Quality Designation (RQD) values for the weathered bedrock is as low as 0%, and from 50-89% for the unweathered shale. Based on the RQD, the unweathered shale is considered to be of fair to good quality.

Groundwater Conditions

Overnight stabilized water level readings taken in open boreholes indicated the general groundwater table to vary between elevation 66.1 and 66.7 during the period of the investigation.

DISCUSSION AND RECOMMENDATIONS

General

Presently, the Hwy. 417 - St. Laurent Blvd. Overpass structure is a single-span reinforced concrete rigid frame type with a span of $24 \pm \text{m}$ and a width of $33 \pm \text{m}$.

The Ministry intends to replace this existing structure with a $40 \pm \text{m}$, 2-span precast concrete box girder structure with retaining walls at each quadrant. The replacement of the existing structure is part of the interchange revisions which are being undertaken to accommodate the RMOC transit-way system.

Soil types at the site are quite variable and overlie a black shale bedrock which is found at about elevation $64 \pm$.

Recommendations for the design and construction of the structure and associated retaining walls are as follows:

Structure Component	Possible Founding Elev. (m)	Bedrock Condition	Loadings	
			U.L.S. (kPa)	S.L.S. II (kPa)
N-W Ret. Wall	64.0	weathered	700	400
	63.5	unweathered	1500	-
N-E Ret. Wall	64.0	weathered	700	400
	63.5	unweathered	1500	-
S-W Ret. Wall	64.0	weathered	700	400
	62.5	unweathered	1500	-
S-E Ret. Wall	64.5	weathered	700	400
	63.8	unweathered	1500	-
West Abutment south end to C	63.5	weathered	700	400
	62.5	unweathered	1500	-
C to north end	63.5	weathered	700	400
	63.0	unweathered	1500	-
Pier	64.0	weathered	700	400
	63.0	unweathered	1500	-
East Abutment	64.0	weathered	700	400
	63.5	unweathered	1500	-

In the situations where the unweathered shale is used as the founding stratum, S.L.S. Type II loadings do not apply as the unweathered bedrock is considered to be unyielding.


Other Considerations

1. The adhesion between concrete and the weathered shale can be assumed to be 75 kPa. The adhesion between concrete and the unweathered shale can be taken as 200 kPa.
2. In view of the nature of the soils at the site, groundwater can be expected to seep into open excavations. The seepage can, however, be controlled by pumping from sumps.
3. As previously stated, the surface of the shale bedrock is susceptible to slaking and possible heaving and degradation when exposed to the atmosphere. In view of this, it is recommended that all excavation bases be protected with a 150 mm concrete working pad as soon as possible after the excavation is opened.
4. Free-draining backfill should be used within a wedge behind all abutments and retaining walls bounded by a plane rising at 60° to the horizontal as per Section 6-9.6.1 of the 1983 O.H.B.D.C. The backfill material should be drained by perforated subdrains and/or weepholes.
5. All underside of footings should be protected for frost by a minimum 1.8 m earth cover.
6. This structure will be constructed in stages and as a result, temporary slopes will be in place for a reasonable length of time. In view of this, all slopes should be constructed at 2:1.
7. Considering the staging involved, it is important that the first stage excavations into bedrock do not extend further out than necessary. The integrity of adjacent bedrock should be maintained for the second stage of construction.


8. Abutment and retaining walls are normally designed for the active earth pressure conditions. However, the at-rest conditions may be used in structures where the deflection of the abutment is prevented by the propping action of the deck or where abutments or retaining walls are founded on unyielding materials by means of spread footings.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of L. Politano, Project Foundations Engineer, and D. Thanasse, Student Engineer, utilizing equipment owned and operated by Johnson Drilling Inc., Ottawa. This report was written by L. Politano and reviewed by M. Devata, Chief Foundations Engineer (East).

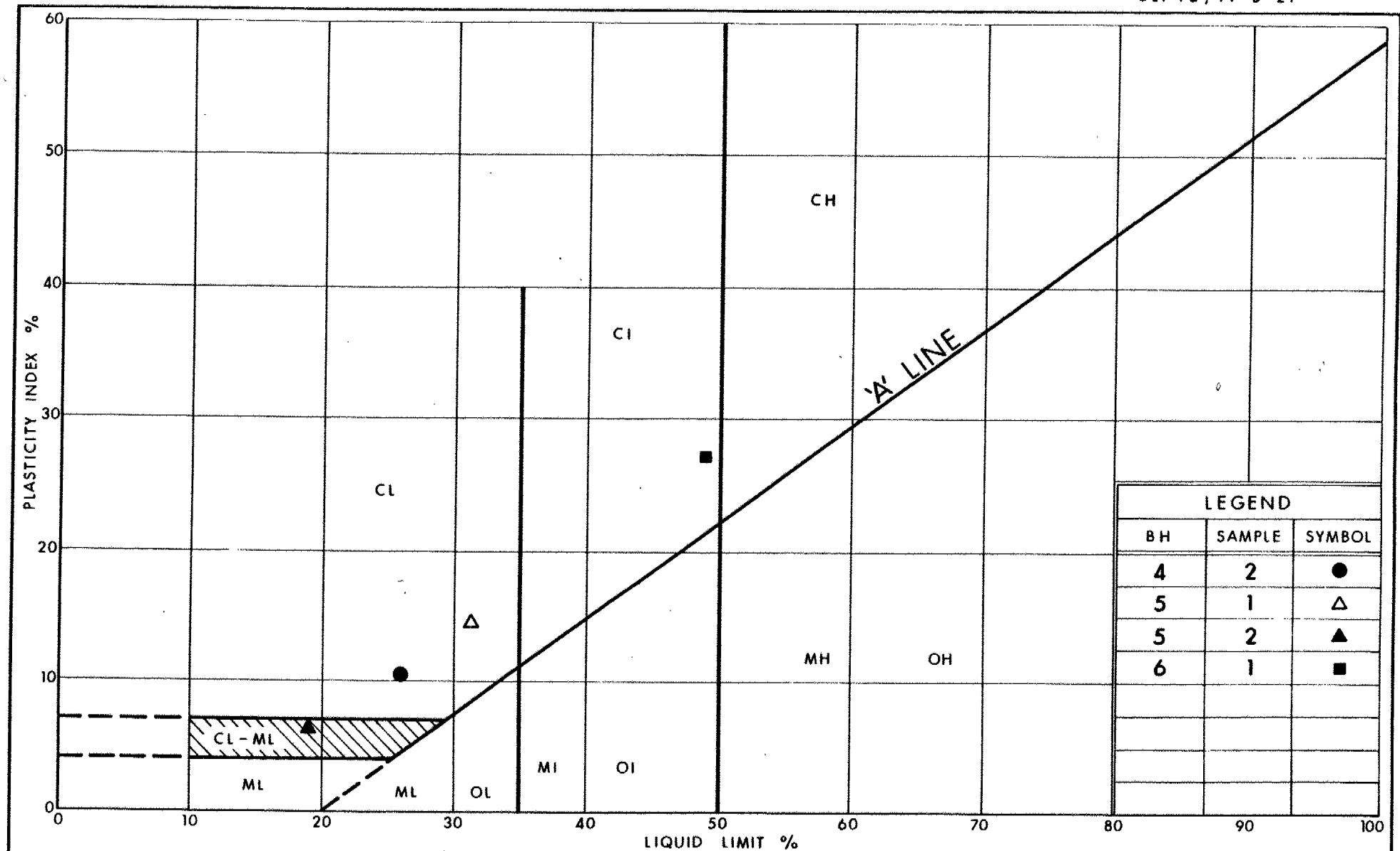

L. Politano
Project Foundations Engineer




M. Devata, P.Eng.
Chief Foundations Engineer (East)

June 1984

APPENDIX



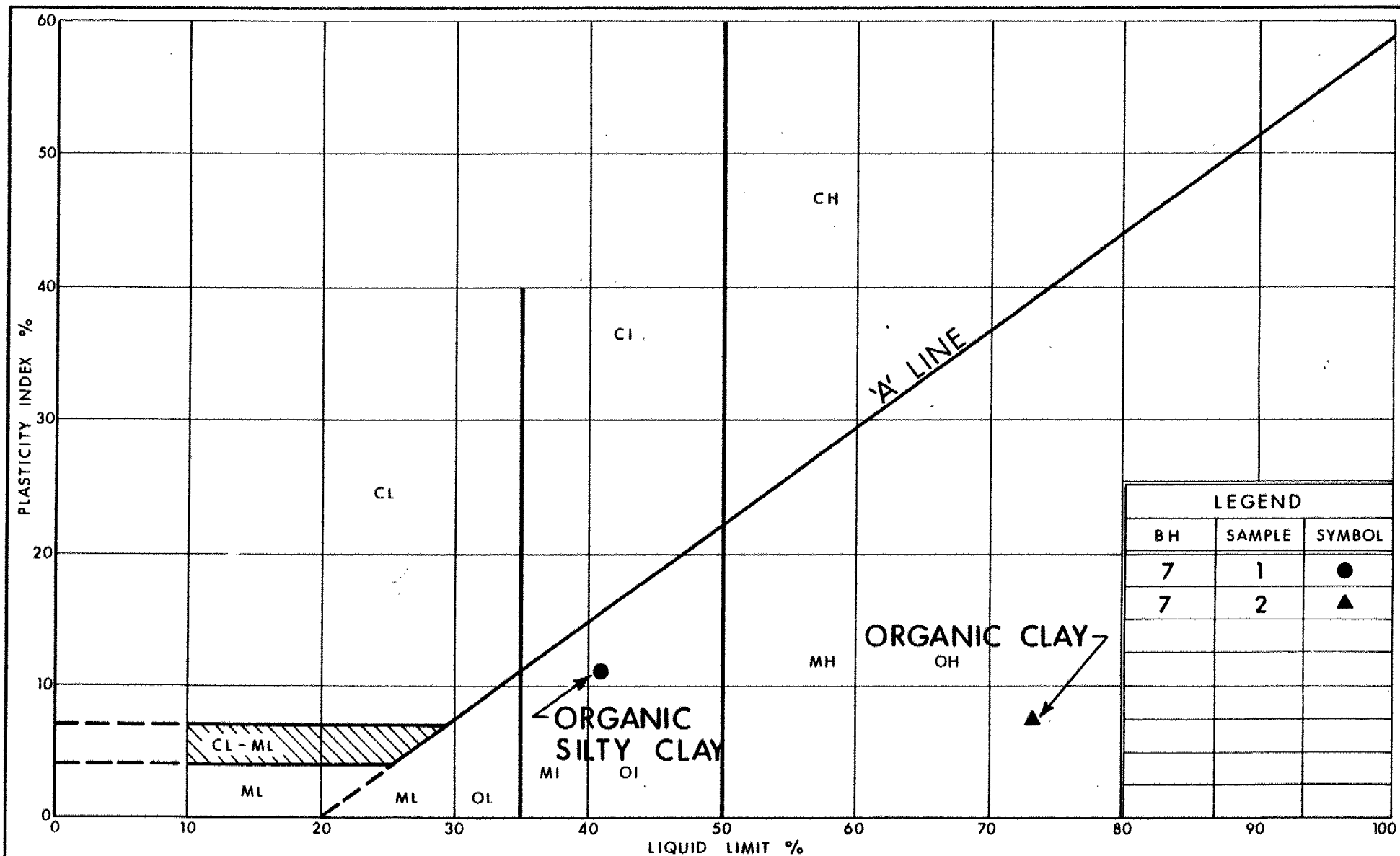
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PLASTICITY CHART

SILTY CLAY, TRACE OF SAND AND GRAVEL

FIG No 1

W P 62-82-01

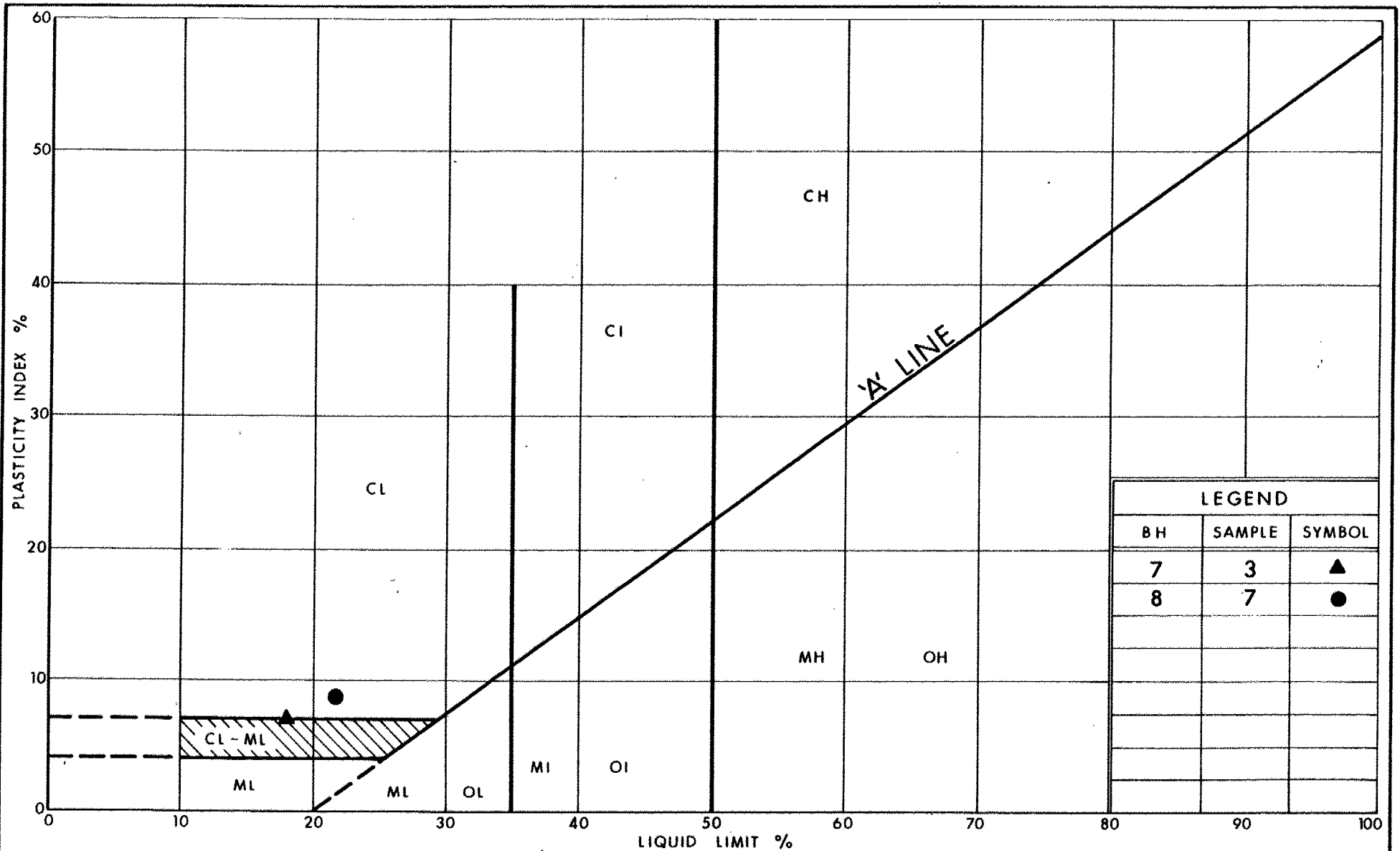


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PLASTICITY CHART

FIG No 2

W P 62-82-01

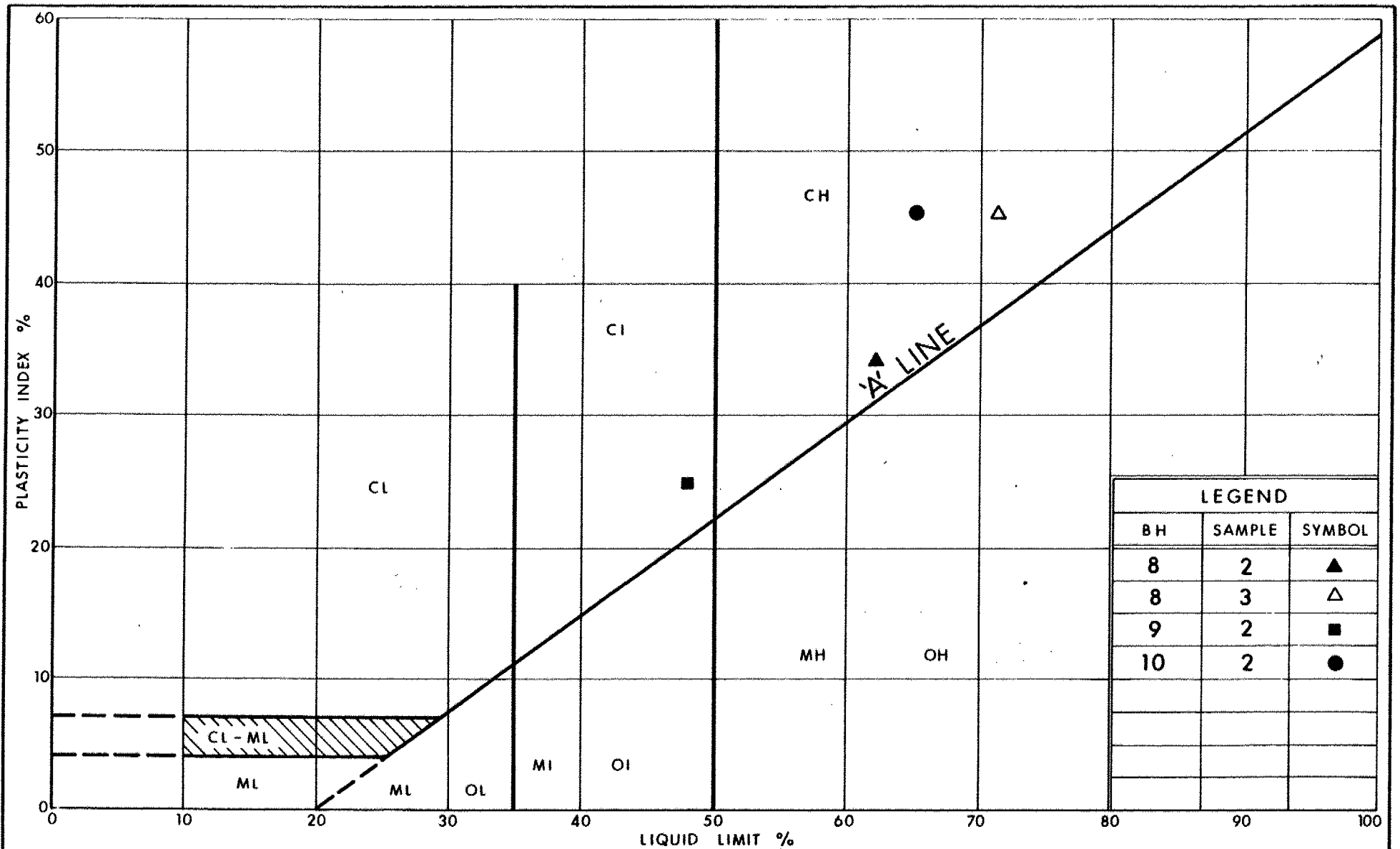


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PLASTICITY CHART
HET MIXTURE OF
SILTY CLAY, SAND & GRAVEL (Glacial Till)

FIG No 3

W P 62-82-01



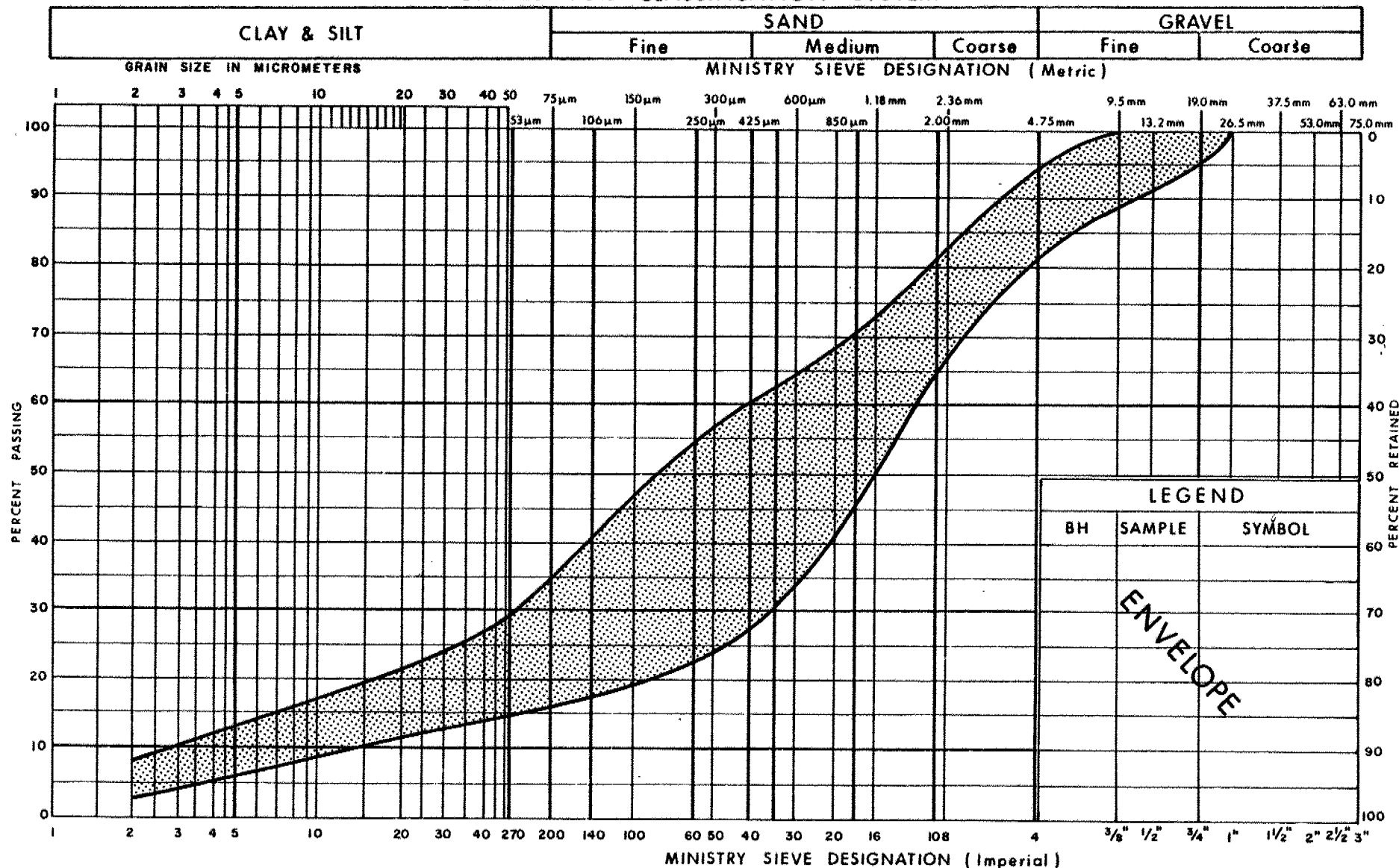
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PLASTICITY CHART
FILL
(SILTY CLAY TO CLAY, TRACE TO WITH SAND)

FIG No 4

W P 62-82-01

UNIFIED SOIL CLASSIFICATION SYSTEM



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GRAIN SIZE DISTRIBUTION
SILTY SAND, TRACE TO SOME GRAVEL

FIG No 5

W P 62-82-01

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 1

METRIC

W P 62-82-01 LOCATION Sta. 33 + 437.0; O/S 16.7 m LT & Hwy. 417 ORIGINATED BY DT
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger & BXL Rock Core COMPILED BY DT
DATUM Geodetic DATE 84 04 17 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ Org. Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
66.8	Asphalt Surface													
0.0	Asphalt Pavement													
66.0	Sand and Gravel Subbase Very Dense		1	SS	90/	15 cm								
0.8			2	SS	95/	23 cm								
	Silty Sand trace gravel Very Dense		3	SS	74									7 69 18 6
64.2			4	SS	15/	10 cm								6 66 22 6
2.6	Black Shale Bedrock Weathered Unweathered		5	BXL RC	REC 90%									RQD = 50%
62.1														
4.7	End of Borehole Note: Water Table Not Stabilized													

+3, x5: Numbers refer to
Sensitivity

20
15
10

5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 2

METRIC

W P 62-82-01 LOCATION Sta. 33 + 447.3; O/S 18.8 m RT & Hwy. 417 ORIGINATED BY DT
 DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger COMPILED BY DT
 DATUM Geodetic DATE 84 04 17 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
66.7	Asphalt Surface																
0.0	Asphalt Pavement																
65.9	Sand and Gravel Subbase																
0.8	Silty Sand trace gravel Very Dense to Compact		1	SS	60		66										
64.4			2	SS	18		65										9 74 13 4
2.3	Black Shale Bedrock		3	SS	85/20 cm												
64.1	Weathered																
2.6	End of Borehole Refusal to Auger																
	Note: Water Table Not Stabilized																

+3, x5: Numbers refer to
Sensitivity

20
15
10

5 (%) STRAIN AT FAILURE



METRIC

ORIGINATED BY DT

COMPILED BY DT

CHECKED BY CP.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100		W _p W W _L			
72.8	Asphalt Surface												
0.0	Asphalt Pavement												
64.4													
8.4	Black Shale Bedrock												
62.7	Weathered												
10.1	End of Borehole Refusal to Auger												
Notes:													
1. Water Table Not Stabilized													
2. No Samples Taken from this Borehole													

+3, x⁵: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 4

METRIC

W P 62-82-01 LOCATION Sta. 33 + 461.6; O/S 25.8 m LT & Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger, BXL Rock Core & Cone Test COMPILED BY DT
DATUM Geodetic DATE 84 04 18 CHECKED BY *EP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
66.9 0.0	Ground Surface												
65.5 1.4	Peat, with Layer of Organic Clay Stiff		1	SS	9							W=98%	
65.2 1.7	Silty Clay		2	SS	10							2.9%	
64.2 2.7	Silty Sand, Trace Clay some gravel (Glacial Till) Very Dense		3	SS	73								22 40 33 5
	Black Shale Bedrock		4	BXL RC	REC 38%								RQD = 0%
62.5 4.4	Weathered Unweathered		5	BXL RC	REC 88%								RQD = 89%
	End of Borehole												

OFFICE REPORT ON SOIL EXPLORATION



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Communications
Ontario

RECORD OF BOREHOLE No 5

METRIC

W P 62-82-01 LOCATION Sta. 33 + 473.6; O/S 28.7 m LT & Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger & Cone Test COMPILED BY DT
DATUM Geodetic DATE 84 04 18 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
66.8	Ground Surface										
0.0	Peat										
65.7			1	SS	15						0 5 55 40
1.1	Silty Clay, Trace Sand										
65.0	Stiff to Hard		2	SS	77/20 cm					1.8%	
1.8	Silty Sand, Trace Clay										
64.5	some gravel (Glacial Very Dense fill)		3	SS	70/15 cm						18 48 29 5
2.3	Black Shale Bedrock Weathered										
63.8			4	SS	60/8 cm						
3.0	End of Borehole Refusal to Auger										

+3, x5: Numbers refer to
Sensitivity

20
15
10
5
5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 6

METRIC

W P 62-82-01 LOCATION Sta. 33 + 435.0; O/S 19.0 m RT of Hwy. 417 ORIGINATED BY LP
 DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger & BXL Rock Core COMPILED BY DT
 DATUM Geodetic DATE 84 04 18 CHECKED BY JP

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
66.9	Ground Surface													
0.0	Peat													
65.9	Silty Clay, trace sand,		1	SS	20									
65.5	gravel													
1.4	Peat, traces of fibres, roots		2	SS	16									
	Very Stiff to Soft		3	SS	4									
63.9	Black Shale Bedrock		4	SS	73									
3.0			5	SS	21									
	Weathered		6	SS	50/3									
	Unweathered		7	BXL RC	REC 68%									
62.1	End of Borehole													
4.8	Note: Water Table Not Stabilized													

+3, x⁵: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 7

METRIC

W P 62-82-01 LOCATION Sta. 33 + 468.6; O/S 22.5 m RT 4 Hwy. 417 ORIGINATED BY LP
 DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger, BXL Rock Core & Cone Test COMPILED BY DT
 DATUM Geodetic DATE 84 04 17 CHECKED BY [Signature]

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
67.3 0.0	Ground Surface											
	Organic Clay to Silty Clay trace sand, fibres Stiff		1	SS	11							
			2	SS	9							
65.0												
2.3	Heterogeneous Mixture of Silty Clay, Sand, Gravel, trace organics (Glacial Till) V. Stiff		3	SS	21							
64.3												
3.0	Weathered Unweathered		4	BXL RC	REC 90%							
	Black Shale Bedrock		5	BXL RC	REC 100%							
62.3												
5.0	End of Borehole											

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



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RECORD OF BOREHOLE No 8

METRIC

W P 62-82-01 LOCATION Sta. 33 + 482.3; O/S 20.8 m RT & Hwy. 417 ORIGINATED BY LP
DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger & Cone Test COMPILED BY DT
DATUM Geodetic DATE 84 04 18 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa						
71.9	Ground Surface							20 40 60 80 100						GR SA SI CL
0.0	Fill							○ UNCONFINED + FIELD VANE						
	Organic Silty Clay							● QUICK TRIAXIAL x LAB VANE						
								20 40 60 80 100	20 40 60					
			1	SS	5									
			2	SS	5									0 12 32 56
	Clay of High Plasticity													
	Trace to some sand with silt													
	Firm to Stiff		3	SS	6									0 2 28 70
			4	SS	13									
66.6														
5.3	Organic Silty Clay, trace of Fibrous Material, Wood		5	SS	20									
66.1														
5.8	Silty Clay, trace sand, gravel		6	SS	18									
65.5	Very Stiff													
6.4	Heterogeneous Mixture of Silty Clay, Sand, Gravel trace organics		7	SS	29									
64.7	(Glacial Till) v. Stiff													
7.2	End of Borehole Refusal to Auger Probable Bedrock													

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



Ministry of
Transportation and
Communications

RECORD OF BOREHOLE No 9

METRIC

W P 62-82-01 LOCATION Sta. 33 + 418.4; O/S 23.5 m RT & Hwy. 417 ORIGINATED BY DT
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger & Cone Test COMPILED BY DT
DATUM Geodetic DATE 84 04 19 CHECKED BY *DT*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W_L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
71.1	Ground Surface												
0.0	Topsoil					*							
	Fill		1	SS	3								
	Silty Clay												
	some sand												
	Stiff to		2	SS	4								0 24 18 58
	Very Stiff												
			3	SS	12								0 14 30 56
65.2													
64.9	Organic Silty Clay		4	SS	30/	8 cm							
6.2	End of Borehole Refusal to Auger Probable Bedrock												
	* Note: Water Level Not Established												

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



Ministry of
Transportation and
Communications

RECORD OF BOREHOLE No 10

METRIC

W P 62-82-01 LOCATION Sta. 33 + 402.2; O/S 18.9 m LT 4 Hwy. 417 ORIGINATED BY DT
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger & Cone Test COMPILED BY DT
DATUM Geodetic DATE 84 04 17 CHECKED BY CP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
72.2	Ground Surface										
0.0	Topsoil										
	Fill										
	Clay of High Plasticity										
	some sand silt		1	SS	5						
	Very Stiff		2	SS	3						
			3	SS	6						
66.1	Organic Silty Clay										
65.7	trace sand Stiff		4	SS	35						
65	Black Shale Bedrock Weathered										
64.9	End of Borehole Refusal to Auger										
7.3											

100/23 cm

+3, x5: Numbers refer to Sensitivity
20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 11 & 12

METRIC

W P 62-82-01 LOCATION Sta. 33 + 422.7; O/S 28.7 m LT Hwy. 417 ORIGINATED BY DT
DIST 9 HWY 417 BOREHOLE TYPE Hollow Stem Auger, BXL Rock Core & Cone Test COMPILED BY DT
DATUM Geodetic DATE 84 04 18 CHECKED BY *BP*

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
66.6	Ground Surface												
0.0	Topsoil												
65.7	Silty Sand some gravel Compact		1	SS	14								10 56 30 4
0.9	Organic Silt some sand trace gravel Loose		2	SS	3								
			3	SS	6								
64.0			4	SS	100/	15 cm							
2.6	Black Shale Bedrock Weathered Unweathered		5	BXL RC	REC 83%								RQD = 70%
62.0													
4.6	End of Borehole												

+3, x⁵: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



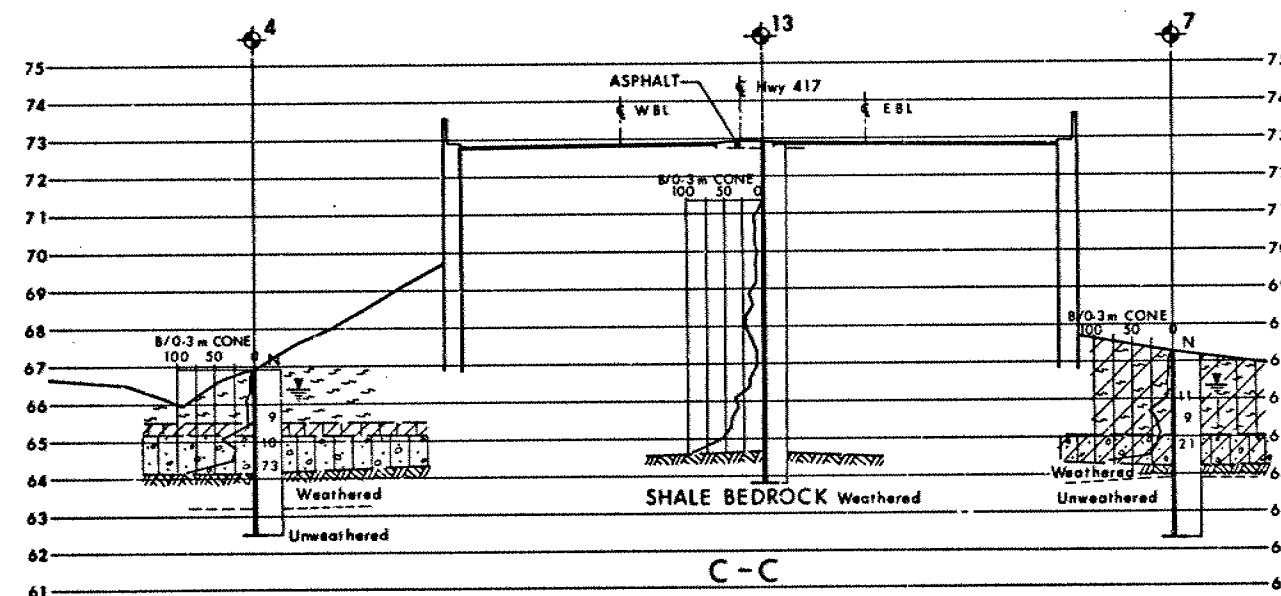
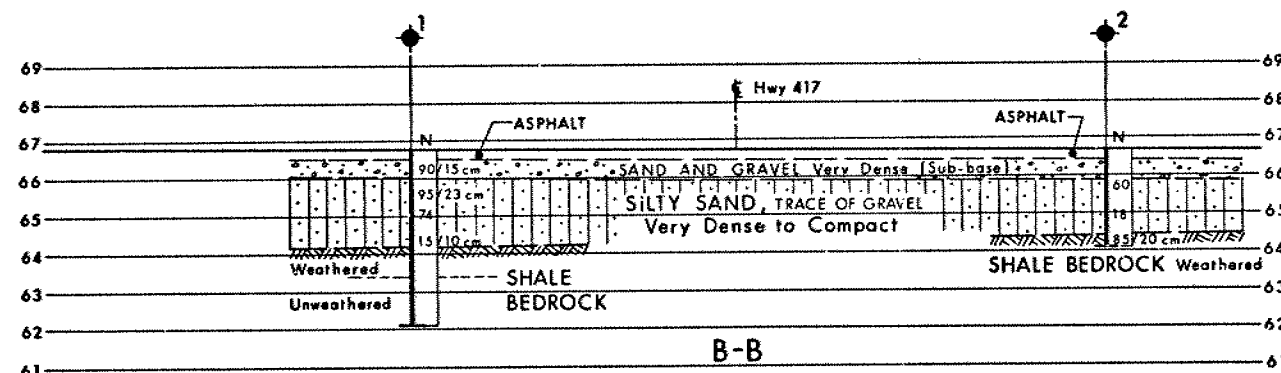
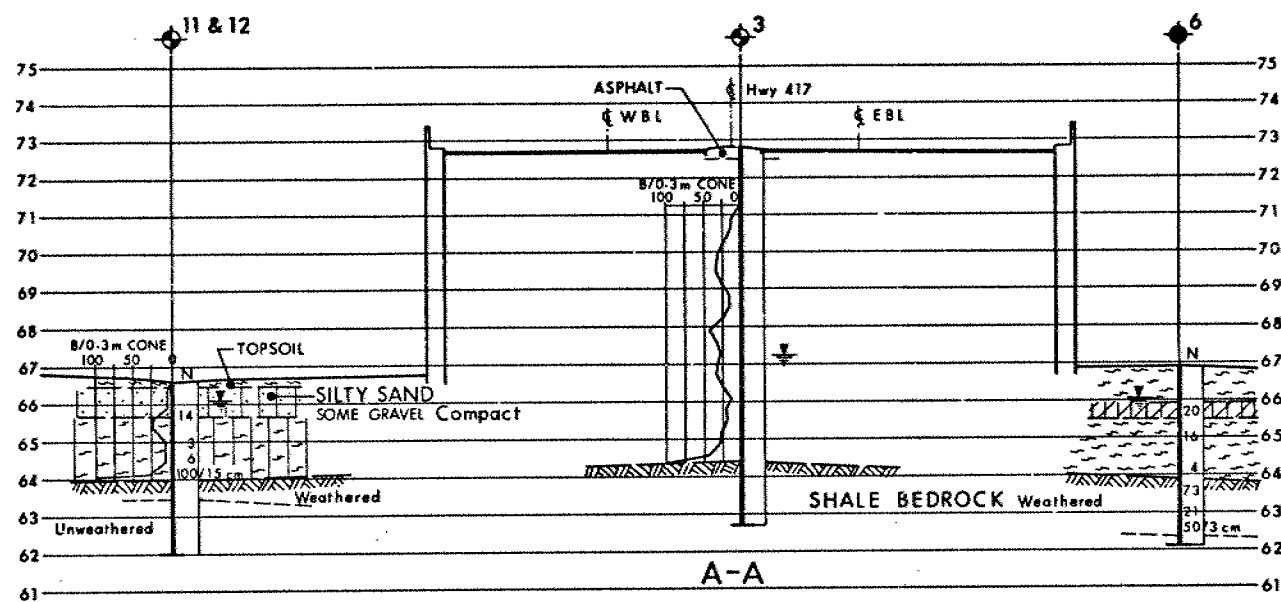
METRIC

ORIGINATED BY DT

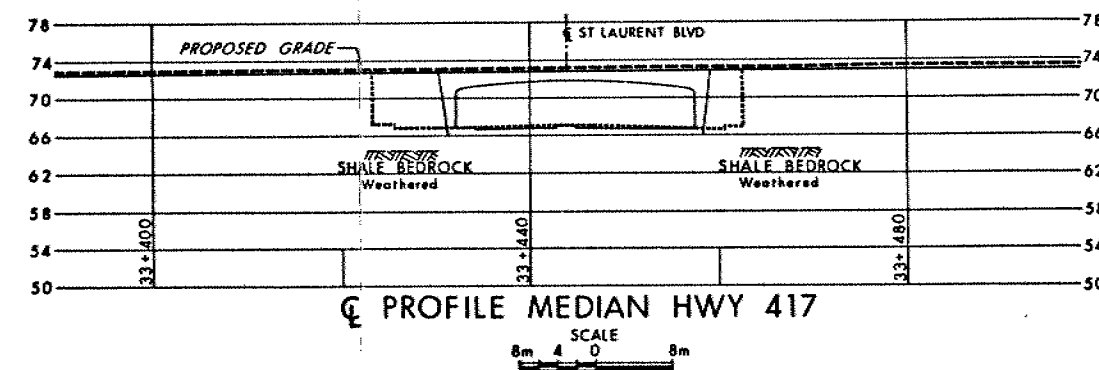
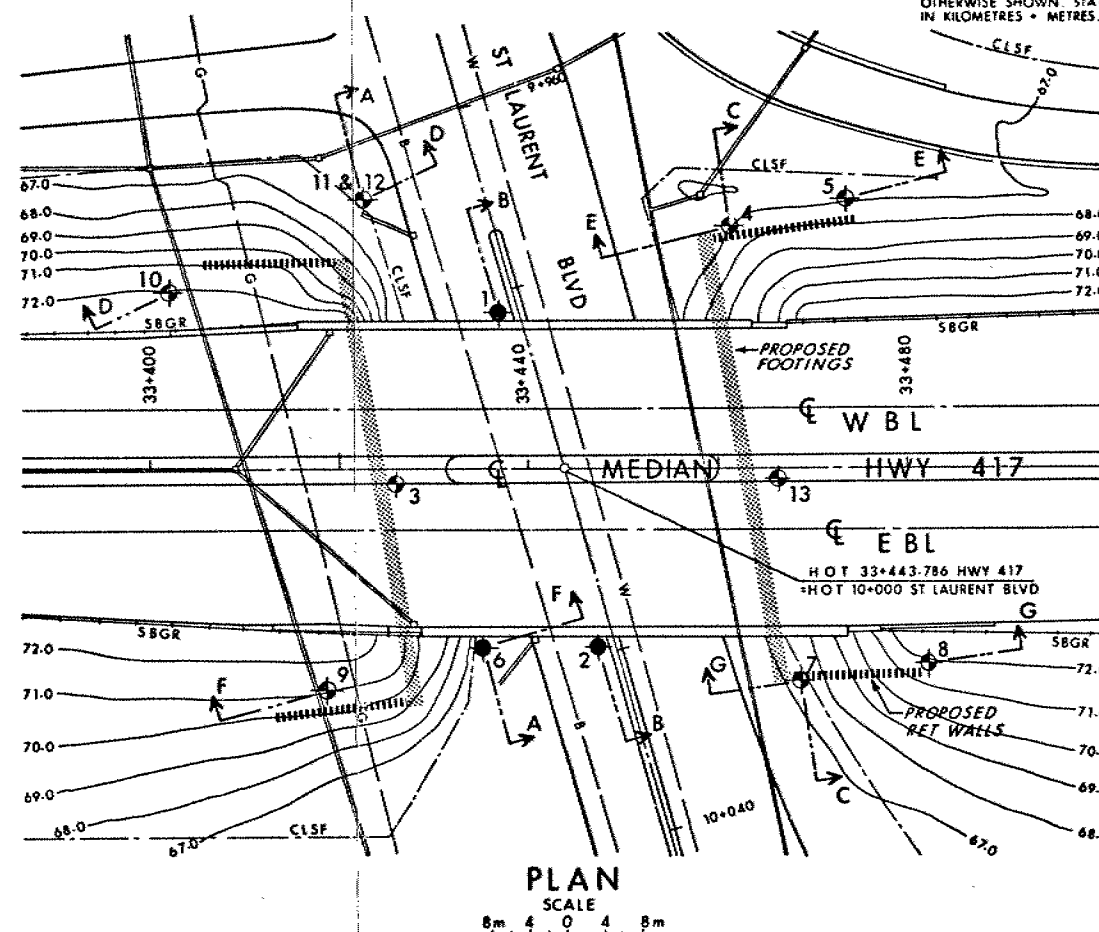
COMPILED BY DT

CHECKED BY

+3, x5: Numbers refer to Sensitivity



SECTIONS
SCALE
HOR 4m 2 0 4m
VERT 2m 1 0 2m



SOIL STRATIGRAPHY LEGEND

	SILTY CLAY TRACE OF SAND & GRAVEL		ORGANIC SILTY CLAY TRACE OF FIBROUS MATERIAL, WOOD
	ORGANIC SILT SOME SAND, TRACE GRAVEL Loose		PEAT TRACES OF FIBRES, ROOTS Very Stiff to Soft
	SILTY SAND, SOME GRAVEL TRACE OF CLAY (Glacial Till) V. Dense		WET MIXTURE OF SILTY CLAY, SAND & GRAVEL TRACE OF ORGANICS (Glacial Till) Very Stiff

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

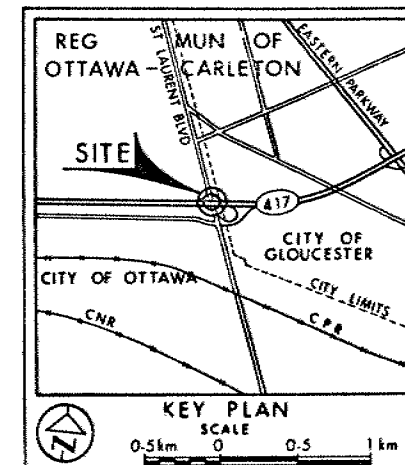
CONT No
WP No 62-82-01

ST LAURENT BLVD OVERPASS

BORE HOLE LOCATIONS & SOIL STRATA



SHEET



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation 1984 04

No	ELEVATION	STATION	OFFSET
1	66.8	33+437.0	16.7m Lt
2	66.7	33+447.3	18.8m Rt
3	72.8	33+426.1	1.4m Rt
4	66.9	33+461.6	25.8m Lt
5	66.8	33+473.6	28.7m Lt
6	66.9	33+435.0	19.0m Rt
7	67.3	33+468.6	22.5m Rt
8	71.9	33+482.3	20.8m Rt
9	71.1	33+418.4	23.5m Rt
10	72.2	33+402.2	18.9m Lt
11 & 12	66.6	33+422.7	28.7m Lt
13	72.9	33+466.4	1.2m Rt

NOTE

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

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

REV	DATE	BY	DESCRIPTION
1			

Geocres No 31G5-137

HWY No 417	DIST 9
SUBMD LP	CHECKED DATE 1984 06 08 SITE 3-72
DRAWN	CHECKED DATE 1984 06 08 DWG 628201-A

REF No E-7009-1 ; 1984 01

METRIC

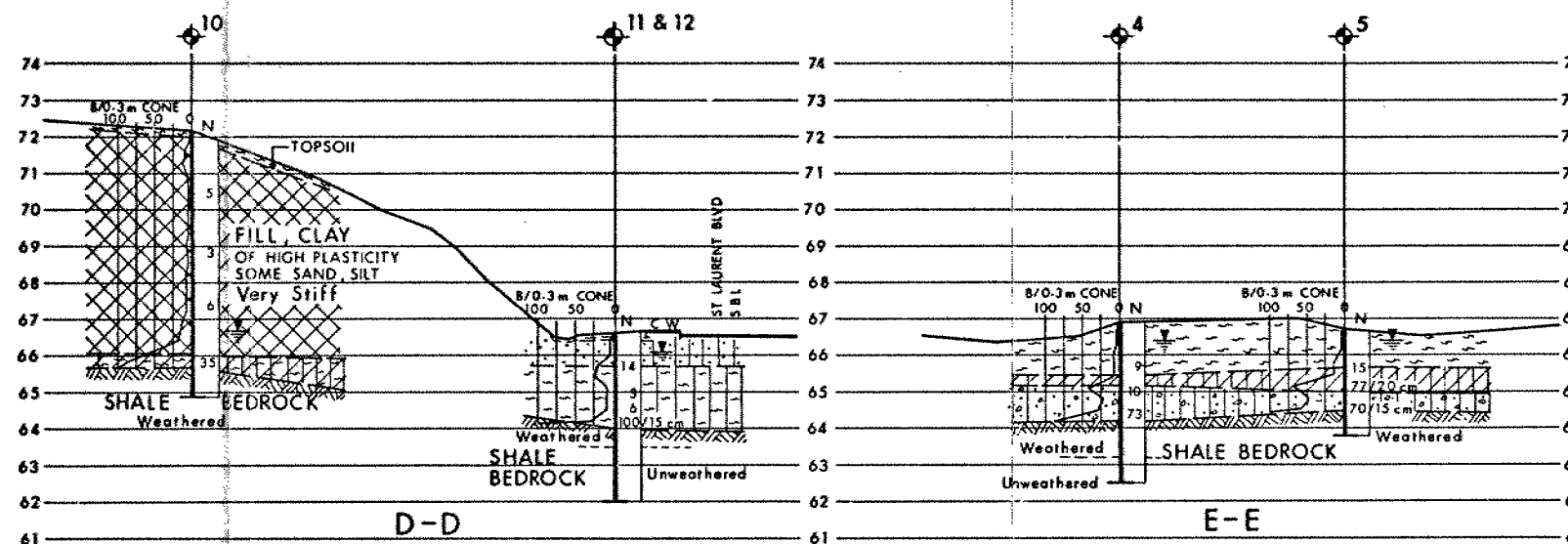
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES.

CONT No
WP No 62-82-01

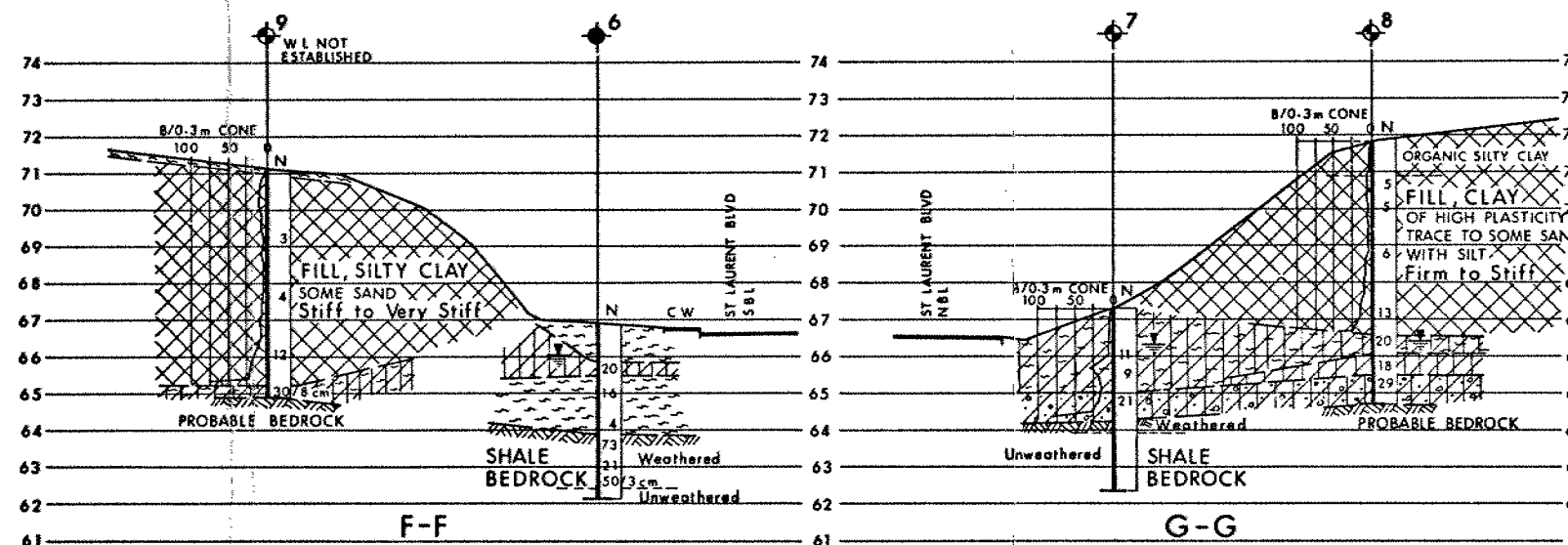
ST LAURENT BLVD OVERPASS

BORE HOLE LOCATIONS & SOIL STRATA

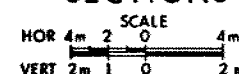
SHEET



NOTE:
FOR PLAN & PROFILE REFER
TO DWG No 628201-A



SECTIONS



SOIL STRATIGRAPHY LEGEND

	SILTY SAND SOME GRAVEL Compact		ORGANIC SILTY CLAY TRACE OF FIBROUS MATERIAL, WOOD		SILTY CLAY, TRACE OF SAND & GRAVEL Stiff to Hard
	ORGANIC SILT SOME SAND, TRACE GRAVEL Loose		PEAT, TRACES OF FIBRES, ROOTS Very Stiff to Soft		
	SILTY SAND, SOME GRAVEL TRACE OF CLAY (Glacial Till) V. Dense		HET MIXTURE OF SILTY CLAY, SAND & GRAVEL TRACE OF ORGANICS (Glacial Till) Very Stiff		

SEE DWG 628201-A

KEY PLAN
SCALE

LEGEND

- ◆ Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ◆ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- W.L. at time of investigation 1984 04
- W.L. Not Established in BH 9

No	ELEVATION	STATION	OFFSET
4	66.9	33+461.6	25.8m Lt
5	66.8	33+473.6	28.7m Lt
6	66.9	33+435.0	19.0m Rt
7	67.3	33+468.6	22.5m Rt
8	71.9	33+482.3	20.8m Rt
9	71.1	33+418.4	23.5m Rt
10	72.2	33+402.2	18.9m Lt
11 & 12	66.6	33+422.7	28.7m Lt

NOTE

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

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

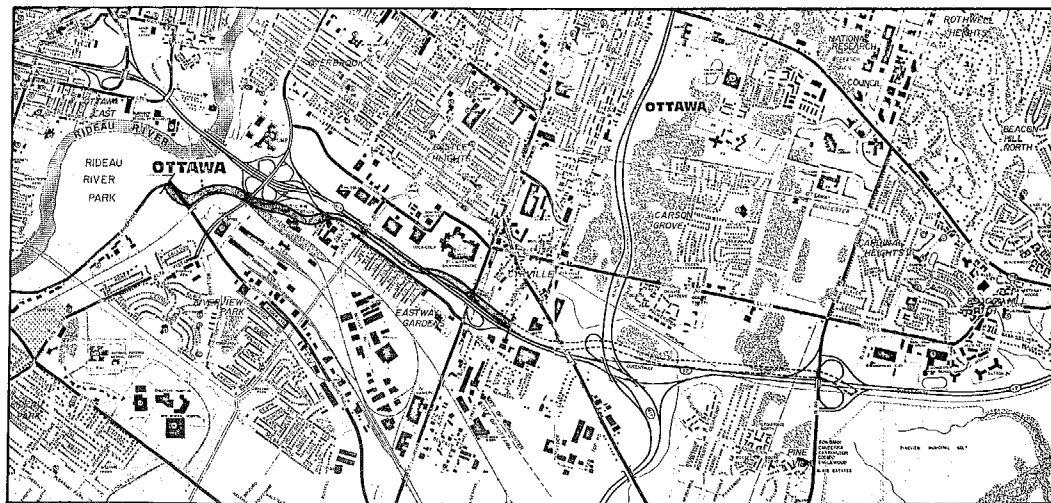
REV.	DATE	BY	DESCRIPTION

Geocres No 31G5-137

HWY No 417	DIST 9
SUBMIT L.P. CHECKED	DATE 1984 06 12
DRAWN BY CHECKED	SITE 3-72
	DWG 628201-B

KEY PLAN

FIGURE 1



-  — PROPOSED EAST TRANSITWAY ALIGNMENT - STAGE 1
-  — EAST TRANSITWAY FUNCTIONAL ALIGNMENT - STAGE 2

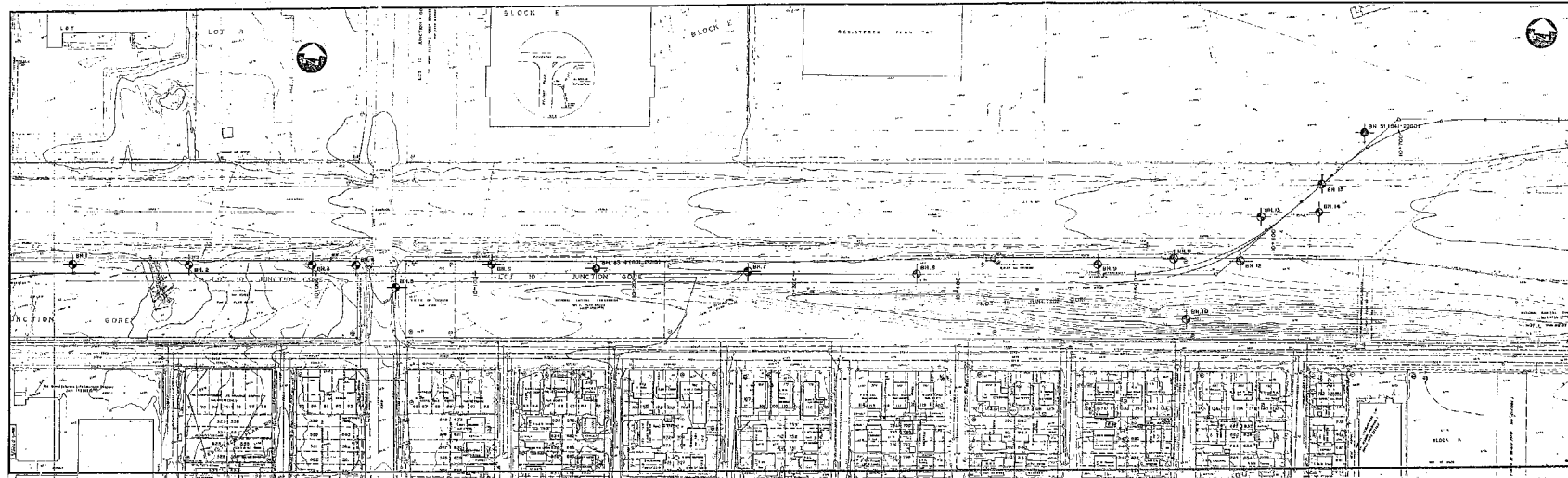
SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

SCALE
1" = 18,500'

JUNE 7, 1985
Date of Issue
Project: 83-1016

Golder Associates

Drawn by: J.C.
Checked by: J.C.



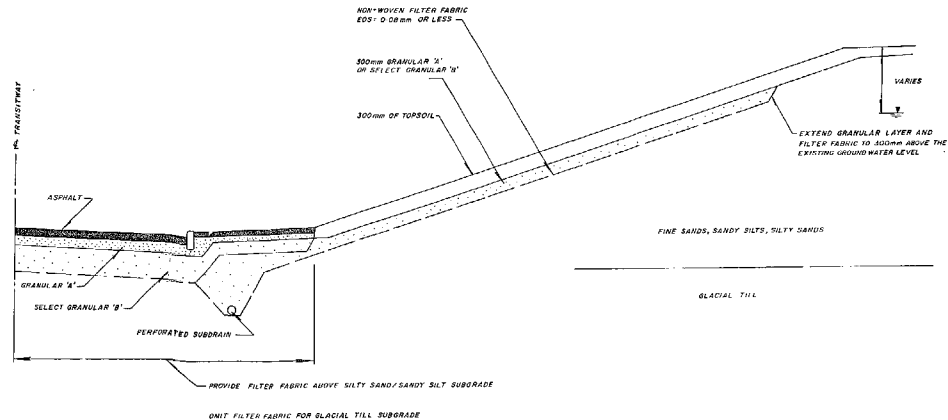
- LEGEND**
- BORING LOCATION IN PLAN
 - BOREHOLE LOCATION IN PLAN, PREVIOUS GOLDER ASSOCIATED REPORT NO 914-2000 & 931-2016
 - PREVIOUS CENTRELINE OF TRANSITWAY
BOREHOLE STATIONING REFER TO THIS LINE
 - REVISED CENTRELINE OF TRANSITWAY (GRAVITY 1985)

REFERENCE PLAN SUPPLIED BY M.M. DILLON LTD.

SCALE
1:1000

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACCOMPANYING REPORT.

TYPICAL OPEN CUT SECTIONS BELOW THE
EXISTING GROUND WATER LEVEL



SCALE
1:50

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

FEB. 9, 1985
DATE: 8.31.2295.2

Golder Associates

Drawn: 15
CNS: 15

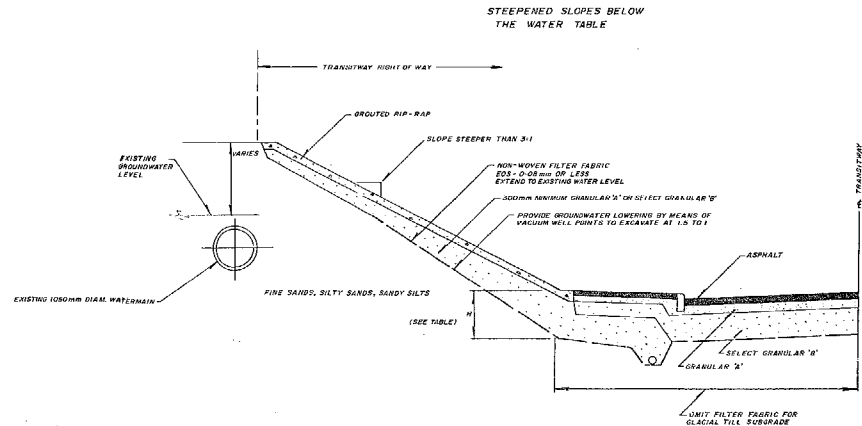
SLOPE PROTECTION AND ROADWAY DRAINAGE

FIGURE 5

TYPICAL GRANULAR DRAINAGE LAYER DEPTH (H)

STATION	DESIGN SLOPE	H (mm)
9+950	2.5:1	800
9+960	2.0:1	900
10+000	2.0:1	1100
10+020	2.0:1	1100

NOTE: EXISTING GROUNDWATER LEVEL TO BE LOWERED TO AT LEAST THE GLACIAL TILL BEFORE EXCAVATION AND MAINTAINED UNTIL SLOPE CONSTRUCTION IS COMPLETED.



SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

SCALE
1:50

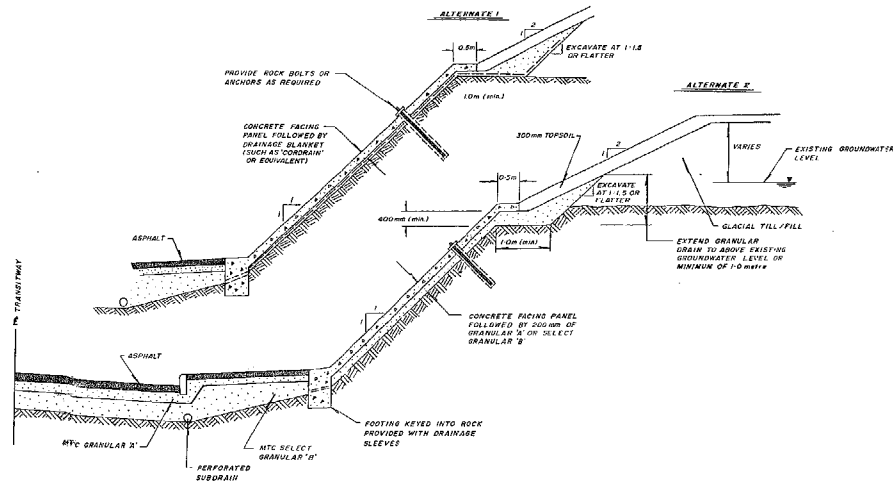
Date: FEB. 11, 1998
Revised: 03/17/2005

Golder Associates

Drawn by: J. G. Golder
Checked by: J. G. Golder

ROCK SLOPE PROTECTION
AND DRAINAGE

FIGURE 6



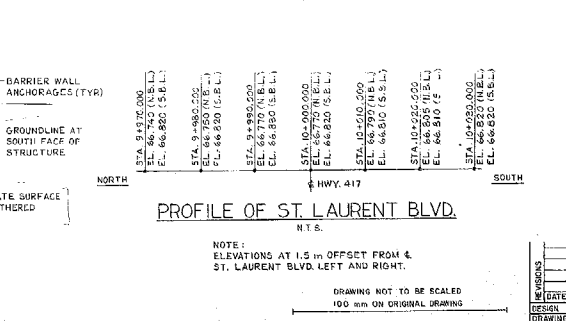
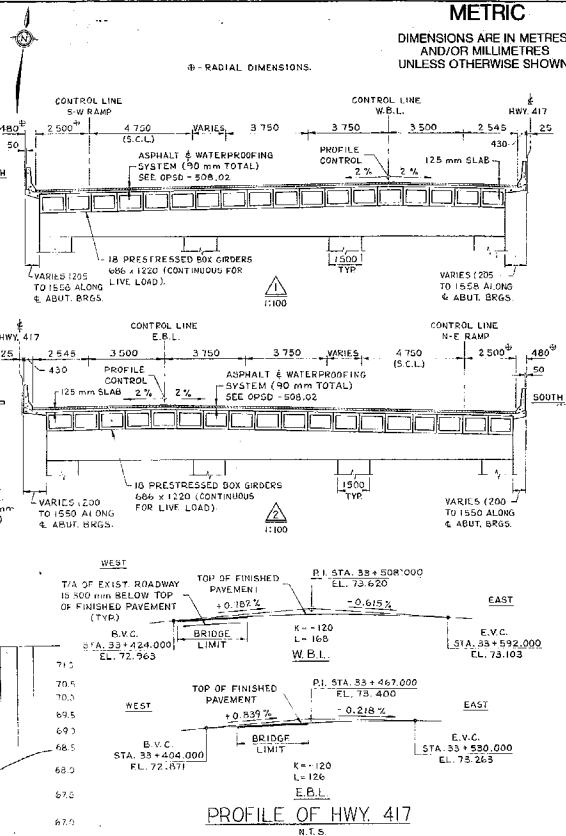
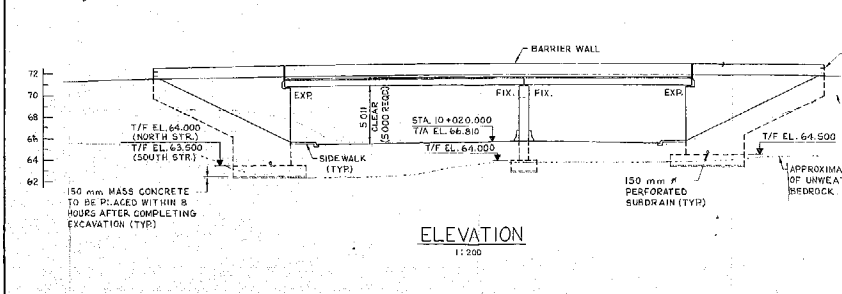
SCALE
1:50

SPECIAL NOTE
THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

Date: FEB. 8, 1984
Project: 84-1-22-2-2

Golder Associates

Drawn: J.S.
Checked: J.S.



DIST. 9	
CONT No	
WP No 62-82-02	
<u>ST. LAURENT BOULEVARD</u>	
<u>INTERCHANGE OVERPASS</u>	
GENERAL ARRANGEMENT	
	SHEET

NOTES

CLASS OF CONCRETE

PRESTRESSED BOX BEAMS	35 MPa
FOOTINGS	20 MPa
PIERS	30 MPa
SUPPORTS AND WINGWALLS	30 MPa
DECK	30 MPa
BARRIER WALLS	30 MPa
REMAINDER	20 MPa

CLEAR COVER TO REINFORCING STEEL

FOOTINGS	100 ± 25
ABUTMENTS & WINGWALLS:	
FRONT FACE	80 ± 20
BACK FACE	70 ± 20
PIERS	80 ± 20
DECK:	
TOP	70 ± 20
BOTTOM	40 ± 10
REMAINDER UNLESS OTHERWISE NOTED	70 ± 20

REINFORCING STEEL

REINFORCING STEEL SHALL BE GRADE 400 UNLESS OTHERWISE SPECIFIED. BAR MARKS WITH SUFFIX 'C' DENOTE COATED BARS.

CONSTRUCTION NOTES

THE CONTRACTOR SHALL FINISH THE BEARING SEATS TO THE SPECIFIED ELEVATIONS TO A TOLERANCE OF ± 3 mm.
CONSTRUCTION STAGING IS TO BE AS INDICATED ON DWG. P1-2.

