coffey geotechnics
SPECIALISTS MANAGING THE EARTH

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORTS, HIGHWAY 417 / ST. LAURENT BLVD. BRIDGE, CITY OF OTTAWA, ONTARIO, G.W.P. 4011-06-00, GEOCRES NO. 31G5-239

AECOM

Project: TRANETOB01226AB
March 24, 2011

Coffey Geotechnics Inc.
20 Meteor Drive Etobicoke Ontario M9W 1A4 Canada

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SPECIALISTS MANAGING THE EARTH

March 24, 2011

AECOM
5080 Commerce Boulevard
Mississauga, Ontario
L4W 4P2

Attention: Ms. Peggy Baleka

Dear Madam:

RE: Preliminary Foundation Investigation and Design Reports, Highway 417/St. Laurent Blvd. Bridge, City of Ottawa, Ontario, G.W.P. 4011-06-00

Please find attached the Preliminary Foundation Investigation and Design Reports relating to the above noted site.

For and on behalf of Coffey Geotechnics Inc.

Ramon Miranda, P.Eng.
Manager, Transportation division

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SPECIALISTS MANAGING THE EARTH

PRELIMINARY FOUNDATION INVESTIGATION REPORT HIGHWAY 417 / ST. LAUREN BLVD. BRIDGE, CITY OF OTTAWA, ONTARIO G.W.P. 4011-06-00, GEOCRES NO. 31G5-239

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CONTENTS

CONTENTS

Appendices

PRELIMINARY
FOUNDATION INVESTIGATION REPORT
HIGHWAY 417/ST. LAURENT BLVD. BRIDGE
CITY OF OTTAWA, ONTARIO
G.W.P. 4011-06-00

1 INTRODUCTION

The existing Highway 417 overpass at St. Laurent Boulevard in the City of Ottawa is to be widened to support future widening of Highway 417. Coffey Geotechnics Inc. (Coffey) was retained by AECOM to prepare a preliminary foundation investigation and design report for the proposed bridge widening based on available information. No field work was required as a part of this project.

To obtain available subsurface information at the proposed overpass widening site, MTO GEOCRES information system was searched. Two previous foundation investigations in 1957 and 1984 were found in MTO GEOCRES information system, at the intersection of current Highway 417 (formerly Queensway) and St. Laurent Blvd, as follows:

- *'Foundation Investigation – St. Laurent Blvd. at Queensway'* - MTO GEOCRES No. 31G05-015, prepared by McRostie & Associates, Ottawa, 1957
- *'Foundation Investigation Report for St. Laurent Blvd. Overpass'* - MTO GEOCRES No. 31G5-137, prepared by Engineering Materials Office, Foundation Design Section, 1984

These foundation reports are included in this report as Appendices A and B.

The existing structure is a two-span bridge with a total length of about 38 m. A widening of approximately 2.5 m is anticipated to the north and 1.25 m to the south. A new retaining wall is proposed at each quadrant of the proposed bridge.

This preliminary foundation investigation and design report is prepared based only on the above mentioned available information.

2 SITE DESCRIPTION AND GEOLOGY

The project Site is located at the intersection of St. Laurent Blvd. with Highway 417, about 0.8 km west of Cyrville Road, in the City of Ottawa, Ontario.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the project site is located within the Physiographic Region known as the Russell and Prescott Sand Plains.

The Site lies on a glacial till plain characterized by glacial tills and silt/sand deposits. In addition, silty clay layers are not uncommon at the Site. Topography across the site is generally flat.

According to the Southern Ontario Geological Highway Map (Map 2418), the bedrock underlying this area consists of a dark grey to black shale of the Billings Formation and is found to be considerably weathered and fractured. The geological explanation for rock in this condition is that at the time glaciation, frost penetrated to great depths and the softer shale layers were disturbed by frost action. This explanation is also advanced for the presence of shale fragments in the overburden above the parent rock.

The existing approach embankments are approximately 6 m high close to the bridge abutments.

3 AVAILABLE SITE INVESTIGATION INFORMATION

3.1 Compiled Subsurface Information from 1957 Investigation

It is our understanding that foundation investigation which was carried out in 1957 was performed to supplement a previous investigation (not available in MTO GEOCRES information system). It appears that this previous investigation consisted of a single borehole and was put down for the then proposed construction of the single span Queensway (now Highway 417) overpass at St. Laurent Blvd., in the City of Ottawa. The details of that borehole (prior to 1957 investigation) data and details of the bridge foundation were not available to us. The details of the three boreholes drilled in 1957 by McRostie and Associates, Consulting Engineers, are given in Appendix A.

The following table summarizes the borehole locations and drilling depths. The borehole locations are shown on Plate No. 1 in Appendix A.

Table 3.1.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Depth of Borehole Below the Ground Surface Existing at that time (m)	Piezometer
1	North-West quadrant area	6.5 m	No
2	North-East quadrant area	4.8 m	No
3	South-West quadrant area	5.1 m	No

Based on the foundation investigation report, diamond drilling was performed into the shale until about 3 m of core was recovered with a percentage of recovery above 75 %, since the proposed structure would be founded on shale.

Standard Penetration Tests (SPT) was also carried out above the coring depths.

Groundwater conditions in the boreholes were observed in the open boreholes between one hour and a few hours (overnight) after completion, as presented on the individual borehole logs in Appendix A.

3.1.1 Subsurface Conditions

The subsurface conditions were explored at three (3) boreholes (see Table 3.1.1 above) for the Queensway (currently Highway 417) overpass at St. Laurent Blvd. The plan location of the boreholes is shown on Plate No. 1 in Appendix A. The Record of Borehole Sheets of this foundation investigation is also included in Appendix A.

Boreholes 1, 2 and 3 were put down in the vicinity of the intersection area of St. Laurent Blvd. and Queensway (currently Highway 417) between El. 65.9 and 66.0 m.

Beneath a 0.3 m thick topsoil, all boreholes contacted an about 0.5 m to 0.9 m thick organic soil layer. Below the organic soil, Borehole 3 encountered about 0.5 m thick well-graded sand. Underneath the organic soil in Boreholes 1 and 2, and the sand in Borehole 3, an about 0.3 m to 0.6 m thick till deposit was contacted and this till deposit was in turn underlain by a shale in all boreholes at depths of about 1.5 m to 1.9 m or at El. 64.4 to 64.0 m.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. The following paragraphs are only meant to amplify and complement these data.

3.1.1.1 Topsoil

A 0.3 m thick topsoil was encountered at the original grade at all borehole locations.

3.1.1.2 Organic Soil

A 0.5 m to 0.9 m thick organic soil was contacted below the topsoil at all borehole locations. This basically granular soil was found to extend to depths ranging between 0.8 m and 1.2 m below the ground surface or to El. 65.2 to 64.6 m.

Standard Penetration tests performed in Boreholes 1 and 2 in this layer yielded N-values of 5 and 11 blows/0.3 m. Based on the borehole information (see Appendix A), the relative density of this organic soil in Boreholes 1 and 2 was described as loose.

3.1.1.3 Sand

Borehole 3 contacted a 0.5 m thick well graded sand layer/lense below the organic soil at a depth of 0.8 m or at El. 65.1 m. Based on the borehole information (see Appendix A), the relative density of this sand was found to be loose to medium dense (i.e. compact). This deposit is considered to be a granular (non-cohesive) soil type.

3.1.1.4 Till

Below the sand layer in Borehole 3 and the organic soil deposit in Boreholes 1 and 2, a 0.3 m to 0.6 m thick glacial till deposit was encountered, at a depth of 1.2 m below the grades that existed at the time of the investigation in 1957 or at El. 64.6 to 64.8 m.

Standard Penetration tests performed in this glacial till deposit yielded N-values ranging from 14 blows/0.15 m to 75 blows/0.28 m. On the Record of Borehole sheets (see Appendix A), the till in Boreholes 1 and 2 is described to have a medium dense (i.e. compact) to very dense relative density. From this description the deposit is considered to be a granular soil type.

Due to their mode of their deposition, the presence of cobbles and boulders should be always anticipated in the glacial till deposits.

3.1.1.5 Bedrock

The boreholes were advanced into the bedrock by diamond drilling. Bedrock condition and elevations are summarized in the table below.

Table 3.1.1.5.1: Bedrock Elevation and Conditions

Borehole No.	Ground Surface Elevation (m)	Inferred Top of Bedrock Elevation (m)	Rock Core Elevation (Depth), m	Core Recovery (%)	Remark
1	65.9	64.0	64.0-62.8 (1.9-3.1)	69	Broken Shale
			62.8-61.7 (3.1-4.2)	90	Shale
			61.7-59.4 (4.2-6.5)	100	Shale
2	66.0	64.2	64.2-63.4 (1.8-2.6)	83	Shale
			63.4-61.8 (2.6-4.2)	97	Shale
			61.8-61.2 (4.2-4.8)	91	Shale
3	65.9	64.4	64.4-63.8 (1.5-2.1)	75	Shale
			63.8-62.3 (2.1-3.6)	88	Shale
			62.3-60.8 (3.6-5.1)	100	Shale

As shown in the above table and on the individual record of borehole sheets, the percentage of rock core recovery was 69 to 100%.

3.1.2 Groundwater Conditions

Groundwater conditions were observed in the open boreholes after the completion of the boreholes. The observations made in the boreholes are shown on the individual Record of Borehole Sheets in Appendix A and are summarized in the following table.

Table 3.1.2.1: Groundwater conditions

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Time Elapsed	Piezometer
1	65.9	-	0.5/65.4	Overnight	No
2	66.0	-	0.3/65.7	One hour	No
3	65.9	-	0.5/65.4	Overnight	No

It should be pointed out that the observed water levels represent the conditions at the time of investigation and that they would be subject to fluctuations, both seasonally and in response to major weather events.

3.2 Compiled Subsurface Information from 1984 Investigation

It is our understanding that foundation investigation was performed in 1984 (with 10 boreholes plus three DCPT) to replace the then existing single span reinforced concrete rigid frame overpass bridge with an about 40± m, two span precast concrete box girder structure with retaining walls at each quadrant. The following table summarizes the borehole locations and drilling depths. The borehole locations are shown on Drawing No. 628201-A in Appendix B.

Table 3.2.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Depth of Borehole Below Existing Ground Surface in 1984(m)	Piezometer
1	Central Pier	4.7	No
2	Central Pier	2.6	No
3*	West Abutment	10.1	No
4	East Abutment	4.4	No
5	East Retaining Wall	3.0	No
6	West Abutment	4.8	No
7	East Abutment	5.0	No
8	East Retaining Wall	7.2	No
9	West Retaining Wall	6.2	No
10	West Retaining Wall	7.3	No
11	West Abutment	4.6	No
12**	West Abutment	2.5	No
13***	East Abutment	9.1	No

*Boreholes 3 and 13 augering and DCPT without sampling
**Borehole 12 (DCPT only), supplementary to Borehole 11
*** Borehole 13 no overburden sampling

The bedrock was proven in 5 of 13 boreholes by obtaining up to 2 m of BX rock core.

Standard Penetration Tests (SPT) were performed in all of the boreholes except for Boreholes 3, 12 and 13. Dynamic Cone Penetration Tests (DCPT) were advanced in or adjacent to Boreholes 3, 4, 5, 7, 8, 9, 10, 11 and 13.

Overnight water level readings in open boreholes were recorded as presented in the individual Record of Borehole Sheet in Appendix B.

3.2.1 Subsurface Conditions

The subsurface conditions were explored at ten (10) boreholes and three (3) DCPT (see Table 3.2.1 above) for the Highway 417 overpass at St. Laurent Blvd. The plan location of the boreholes is shown on Drawing No. 628201-A in Appendix B. The Record of Borehole Sheets of this foundation investigation are included in Appendix B.

The subsurface conditions are considered variable at the site. In general, from the o.g. level, under the topsoil, non-cohesive or cohesive organic soils were contacted. Below the organic soils, native granular soils (sand and silt) and cohesive soils (silty clay) were encountered. A heterogeneous mixture of cohesive and non-cohesive glacial till was found below the native non-cohesive and cohesive soils. The till deposit was in turn underlain by a shale bedrock in all boreholes at El. 63.9 to 65.7 m (including inferred bedrock depths for Boreholes 8 and 9 based on refusals on probable bedrock). About 5 m to 6 m fill used for the existing structure approach was found to be consisting of clay to silty clay.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix B. The following paragraphs are only meant to amplify and complement these data.

3.2.1.1 Topsoil

Boreholes 9, 10 and 11 and 12 contacted topsoil at the existing grade.

3.2.1.2 Pavement

Boreholes 1, 2, 3 and 13 were drilled on the existing pavement and contacted an asphalt pavement.

Below the asphalt pavement, Boreholes 1 and 2 contacted sand and gravel (pavement fill) materials which were found to extend to a depth of 0.8 m.

3.2.1.3 Peat

A dark brown to black peat was found in Boreholes 4, 5 and 6 at the existing grade. In Boreholes 4 and 5, the thickness of the peat layer was recorded to be 1.4 m and 1.1 m, respectively. In Borehole 6, the peat deposit was found to be about 3 m thick (including about 0.4 m thick silty clay layer within the peat) and was found to extend to a depth of 3.0 m or to El. 63.9 m.

The report prepared by MTO personnel indicate that the peat found at the site appeared to be "at an intermediate stage of decomposition, as it did not have a totally fibrous texture". Root fibres and pieces of decomposed wood were however, still evident. The organic deposit had a spongy consistency and was described as "quite compressible".

Natural moisture contents of 98% and 120% were measured in the laboratory on two samples from this predominantly cohesive organic soil.

Based on Standard Penetration test results, which yielded N-values ranging from 4 to 16 blows/0.3 m, the consistency of this peat is described as soft to very stiff.

3.2.1.4 Silty Clay

A 0.3 m to 0.7 m thick silty clay layer was contacted below the peat in Boreholes 4 and 5 and was found to extend to depths of 1.7 m to 1.8 m or El. 65.2 to 65.0 m. In Borehole 6, a 0.4 m thick silty clay layer was found interbeded within the peat at a depth of 1.0 m. Borehole 8 also contacted 0.6 m thick silty clay below the organic silty clay at a depth of 5.8 m or El. 66.1 m. This silty clay contains traces of gravel and sand. Testing for organic content indicated 1.8% to 6.1% organics in this silty clay (i.e. organic intrusions).

The grain-size distribution of a sample from the silty clay indicates the following grain-size distribution.

Gravel:	0%
Sand:	5%.
Silt:	55%
Clay:	40%

The Atterberg limits tests performed on four samples from the silty clay is given in Figure 1 in Appendix B. The tests yielded the following index values:

Liquid Limit:	19-48%
Plastic Limit:	12-21%
Plasticity Index:	7-27

The results are representative of cohesive soils of typically low to medium plasticity.

Standard penetration test N-values ranging from 10 blows/0.3 m to in excess of 77 blows/0.2 m were recorded. Higher N value was recorded due to the very dense glacial till deposit immediately below the silty clay. Based on these field test results, the silty clay can typically be described of stiff to very stiff consistency.

3.2.1.5 Embankment Fill

Below the topsoil in Boreholes 9 and 10 and from the existing grade at Borehole 8, an about 5.3 m to 5.9 m thick embankment fill material (mainly clay to silty clay) was encountered. This embankment fill was found to extend to depths of 5.3 to 6.1 m or El. 65.2 to 66.6 m.

The grain-size distribution of five samples from the fill indicates the following grain-size distribution.

Gravel:	0%
Sand:	2-24%.
Silt:	18-32%
Clay:	56-70%

The results of Atterberg limits tests performed on five samples from the cohesive fill are given in Figure 4 (only 4 test results are shown) in Appendix B. The tests yielded the following index values:

Liquid Limit:	47-72%
Plastic Limit:	20-29%
Plasticity Index:	25-45

These results are representative of cohesive soils of medium to high plasticity.

Field vane tests indicate that the undrained shear strength of the fill ranges from 44 to in excess of 100 kPa. Sensitivity of the fill varies from 5 to 12. Standard penetration test N-values ranging from 3 to 13 blows/0.3 m were recorded. Based on these field test results, the fill can be described as firm to very stiff in consistency and does not appear to have received a high degree of compaction when it was first placed.

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

3.2.1.6 Organic Soils

3.2.1.6.1 Organic Clay, Organic Silty Clay

About 2.3 m thick organic clay to silty clay deposit was contacted at the existing grade at Borehole 7. Below the fill materials, Boreholes 8, 9 and 10 contacted about 0.3 m to 0.5 m thick organic silty clay at depths of 5.3 m to 6.1 m or El. 65.2 to 66.6 m. Borehole 9 was terminated at the interface of this deposit to the probable bedrock at a depth of 6.2 m or El. 64.9 m, due to the auger refusal.

The results of Atterberg limits tests performed on two samples from the organic clay are given in Figure 2 in Appendix B. The tests yielded the following index values:

Liquid Limit:	42-105%
Plastic Limit:	30-97%
Plasticity Index:	8-12

These results are representative of organic cohesive soils of intermediate (OI) to high (OH) plasticity.

Standard Penetration test N-values ranging from 9 blows/0.3 m to in excess of 30 blows/0.1 m were recorded. Higher blow counts were recorded due to the bedrock immediately below this organic soil layer. Based on these field test results together with the interpretation of results of the tests, these cohesive organic soils can typically be described as having soft to stiff consistency.

3.2.1.6.2 Organic Silt

Borehole 11 contacted about 1.7 m thick organic silt at a depth of 0.9 m or El. 65.7 m. This deposit contains some sand and traces of gravel.

Standard Penetration test N-values ranging from 3 blows/0.3 m to in excess of 100 blows/0.15 m were recorded in this deposit. Higher N-values were recorded due to the bedrock immediately below this organic soil layer. Based on these field test results, this organic soil (basically a granular material) can typically be described to have a very loose to loose compactness condition.

3.2.1.7 Glacial Till

3.2.1.7.1 Cohesive Till

A glacial deposit, consisting of heterogeneous mixture of silty clay, sand and gravel, was encountered below the organic clay in Borehole 7 at a depth of 2.3 m or El. 65.0 m and below the silty clay in Borehole 8 at a depth of 6.4 m or El. 65.5 m. This till is further underlain by a bedrock at a depth of 3.0 m or El. 64.3 in Borehole 7 while Borehole 8 was terminated at the interface of this till deposit and the probable bedrock at a depth of 7.2 m or El. 64.7 m, due to the auger refusal.

The results of Atterberg limits tests performed on two samples from the cohesive till are given in Figure 3 in Appendix B. The tests yielded the following index values:

Liquid Limit:	17-22%
Plastic Limit:	10-13%
Plasticity Index:	7-9

The results are representative of cohesive soils of low plasticity and the fact that the measured natural moisture contents are generally below the measured plastic limit values indicates that this deposit is somewhat over-consolidated. Due to the mode of their deposition, the presence of cobbles and boulders should always be anticipated in glacial till deposits.

Based on Standard Penetration test results, which yielded N-values ranging from 21 to 29 blows/ 0.3 m, the consistency of this till can be described as very stiff.

3.2.1.7.2 Non-cohesive Till

Boreholes 4 and 5 contacted a 0.5 m to 1.0 m thick silty sand till deposit at depths of 1.7 m and 1.8 m or at El. 65.2 to 65.0 m, below the silty clay.

The grain-size distribution of two samples from this non-cohesive (i.e. granular) till indicates the following grain-size distribution.

Gravel:	18-22%
Sand:	40-48%.
Silt:	29-33%
Clay:	5%

Due to the mode of their deposition, the presence of cobbles and boulders should always be anticipated in these non-cohesive till deposits.

Based on Standard Penetration test results, which yielded N-values ranging from 10 blows/0.3 m to 70 blows/about 0.15 m, the relative density of this till can be described as compact to very dense, but typically very dense.

3.2.1.8 Silty Sand

Below the pavement structure in Boreholes 1 and 2 and below a shallow veneer of topsoil in Borehole 11, about 0.7 m to 1.8 m thick layer of silty sand was encountered at depths of 0.2 to 0.8 m or at El. 66.4 to 65.9 m.

The grain-size distribution of four samples from this non-cohesive deposit indicates the following grain-size distribution.

Gravel:	6-10%
Sand:	56-74%.
Silt:	13-30%
Clay:	4-6%

Based on Standard Penetration test results, which yielded N-values ranging from 14 blows/0.3 m to 95 blows/about 0.25 m, the relative density of this basically granular deposit can be described as compact to very dense. Some higher blow counts were recorded at the interface with bedrock but these high values are believed to reflect the influence of the bedrock.

3.2.1.9 Bedrock

Bedrock at the Site was proven in 5 boreholes by obtaining up to 2 m of rock cores. In the remaining boreholes, split spoon samples of the weathered bedrock were recovered or augering was advanced to refusal.

The bedrock underlying this area of Ottawa is known to consist of grey and black shales of the Billings Formation. The formation belongs to the Upper Ordovician Period and is approximately 460 million years old.

Table 3.2.1.9.1: Inferred Bedrock Elevation and Conditions

Borehole No.	Ground Surface Elevation (m)	Bedrock surface depth/ elevation (m)	Penetration into Bedrock by augering Depth (m)	Length of coring (m)	Recovery (%)	R.Q.D. (%)**
1	66.8	2.6/64.2	-	1.5	90	50
2	66.7	2.3/64.4	2.3-2.6	-	-	-
3	72.8	8.4/64.4	8.4-10.1	-	-	-
4	66.9	2.7/64.2	-	1.3	38	0
			-	0.4	88	88
5	66.8	2.3/64.5	2.3-3.0	-	-	-
6	66.9	3.0/63.9	3.0-4.4	0.4	68	68
7	67.3	3.0/64.3	-	1.3	90	57
			-	0.7	100	80
8	71.9	7.2/64.7*	-	-	-	-
9	71.1	6.2/64.9*	-	-	-	-
10	72.2	6.5/65.7	6.5-7.3	-	-	-
11&12	66.6	2.6/64.0	2.6-3.0	1.6	83	70
13	72.9	8.2/64.7	8.2-9.1	-	-	-

*inferred bedrock surface (borehole was terminated within the overburden due to the auger refusal)

**R.Q.D. = Rock Quality Designation

As shown in the above table and on the individual record of borehole sheets, the percentage of rock core recovery was 38% to 100% while the RQD values vary from 0% to 88%. These results indicate a range rock quality values from very poor to good.

Based on Standard Penetration test results, which yielded N-values ranging from 21 blows/0.3 m to in excess of 100 blows/0.3 m, and recorded auger refusal depths, it is our opinion that the upper 0.3 m to 1.7 m of the bedrock at the Site is extremely to highly weathered.

In general, the bedrock appears to be extremely to highly weathered in this area due to the frost penetration into the bedrock during the glacial period.

3.2.2 Groundwater Conditions

Groundwater conditions were observed in the open boreholes. The observations made in the boreholes are shown on the individual Record of Borehole Sheets in Appendix B and are summarized in the following table.

Table 3.1.2.1: Groundwater Conditions

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Time	Piezometer
1	66.8	-	1.3/65.5	Overnight	No
2	66.7	-	1.0/65.7	Overnight	No
3	72.8	-	5.6/67.2	Overnight	No
4	66.9	-	0.7/66.3	Overnight	No
5	66.8	-	0.5/66.3	Overnight	No
6	66.9	-	0.9/66.0	Overnight	No
7	67.3	-	1.0/66.3	Overnight	No
8	71.9	-	5.6/66.3	Overnight	No
9	71.1	-	Water level not established	Overnight	No
10	72.2	-	5.5/66.7	Overnight	No
11&12	66.6	-	0.5/66.1	Overnight	No
13	72.9	-	6.7/66.2	Overnight	No

It should be pointed out that the observed water levels represent the conditions at the time of investigation and that they would be subject to fluctuations, both seasonally and in response to major weather events. A perched water condition can be encountered within the approach fills or surficial deposits.

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

Appendix A

Foundation Investigation Report

(By McRostie & Associates)

BA 677

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA 1
CANADA

393 BELL STREET
TELEPHONE CE. 2-5334

(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.

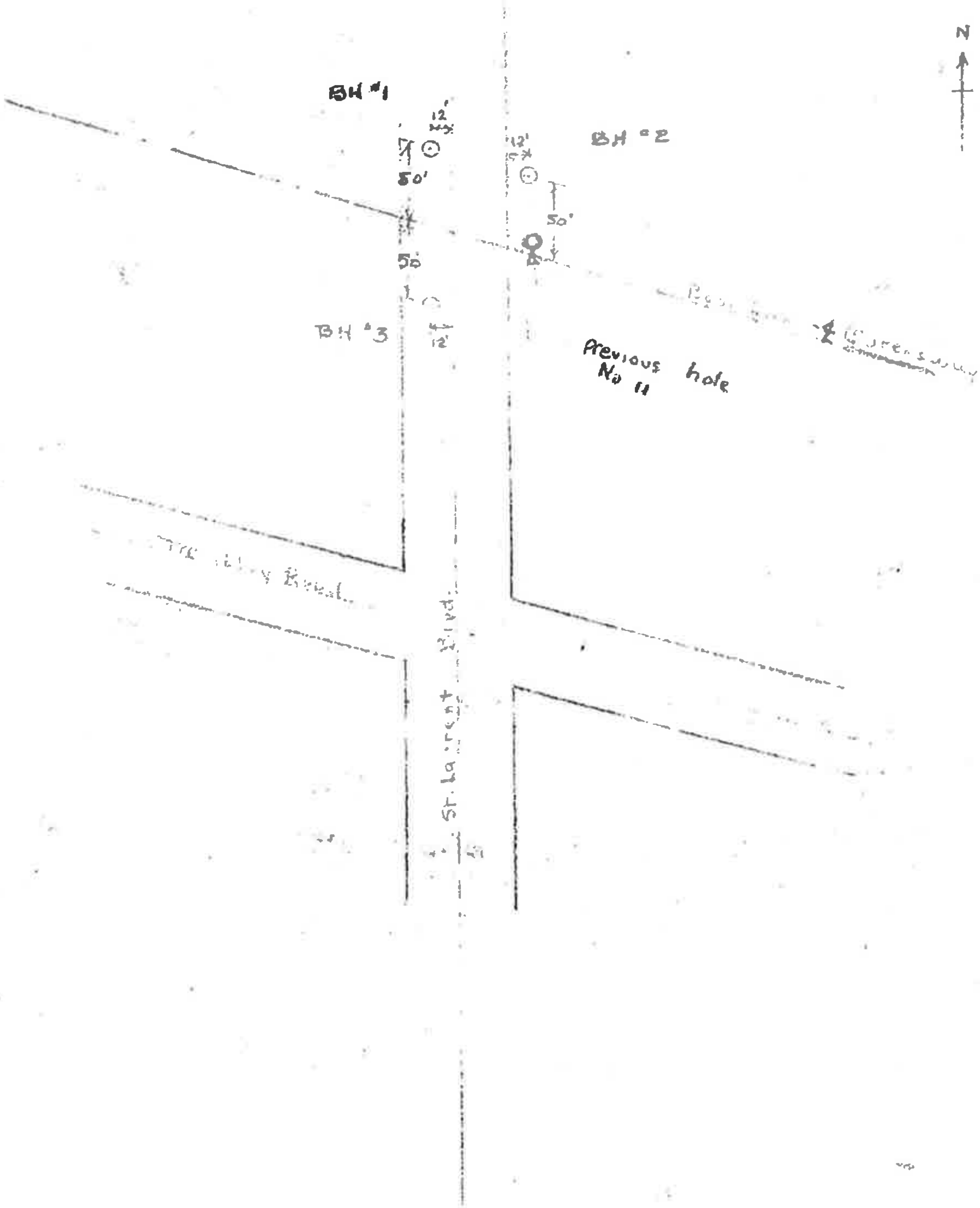
Cores recovered from diamond drilling were examined in detail for the slope and thickness of the bedding planes since a non-uniform slope of the planes indicates a broken condition.

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.



McROSTIE & ASSOCIATES
CONSULTING ENGINEERS

BOREHOLE LOCATIONS
ST. LAURENT AT QUEENSWAY

SCALE 1" = 100'

PLATE 1

FORM S-10-2

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

ST. LAURENT BLVD. & QUEENSWAY.

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 216.1

REMARKS GEODETIC DATUM

DATE JUNE 3-4 '57

HOLE No. 1

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
				GROUND SURFACE	0	216.1
				TOP SOIL (to 1.0)		
				LOOSE ORGANIC SOIL (to 4.0)	2	
		11	1-1		4	212.1
				MEDIUM DENSE TILL (to 5.5)		
		14 for 6" + 40 for 8"	1-2	VERY DENSE TILL (to 6.1)	6	210.0
				BROKEN SHALE ROCK (DRILLED 67% CORE RECOVERY) (to 10.1)	8	
					10	206.0
				SHALE ROCK (DRILLED – 50% CORE RECOVERY) (to 13.7)	12	
					14	
				SHALE ROCK (DRILLED – 100% CORE RECOVERY)	16	
					18	
					20	
				21.2 BOTTOM OF HOLE		184.9

PENETRATION TEST

.......LB. HAMMER .... INCH DROP — NO CASING .... INCH DIA. ROD

BLOWS PER FOOT

← OVERNIGHT WATER LEVEL 1.5'

% WATER CONTENT

PLATE 2

L. M. CO. A19146

FORM S-10-2

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

ST. LAURENT BLVD. & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 216.5

REMARKS GEODETIC DATUM

DATE JUNE 5, 1957

HOLE NO. 2

Unconfined Compressive Strength Kips/Ft.²	Small Scale Penetrometer Kips/Ft.²	Standard Penetration Blows/Ft.	Sample Number	Description of Soil (Ground Surface)	Depth in Feet	Elevation	Penetration Test (....LB. Hammer ....Inch Drop; No Casing ....Inch Dia. Rod) Blows per Foot
				TOP SOIL 1.0	0	216.5	← WATER LEVEL AFTER 1 HR.
				LOOSE ORGANIC SOIL	2		
		5	2-1	4.0	4	212.5	
				MEDIUM DENSE TILL 5.0			
		75 [illegible] 11	2-2	VERY DENSE TILL 5.9	6	210.6	
				SHALE ROCK (DRILLED - CORE RECOVERY 83%) 8.4	8		
					10		
				SHALE ROCK (DRILLED - CORE RECOVERY 77%)	12		
				13.9	14		
				SHALE ROCK (DRILLED CORE RECOVERY 91%) 15.9	16	200.6	
				BOTTOM OF HOLE			

% WATER CONTENT

PLATE 3

FORM S-10-2

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

ST. LAURENT BLVD. & QUEENSWAY

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 216.3

REMARKS GEODETIC DATUM

HOLE No. 3

DATE JUNE 4 1967

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT.²	SMALL SCALE PENETROMETER KIPS/FT.²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL (GROUND SURFACE)	DEPTH IN FEET	ELEVATION	PENETRATION TEST (....LB. HAMMER, ...INCH DROP; NO CASING, ...INCH DIA. ROD; BLOWS PER FOOT)
				TOP SOIL	0	216.3	
				1.0			
				ORGANIC SOIL	2		← OVERNIGHT WATER LEVEL 1.5'
				2.5			
			3.1	LOOSE WELL GRADED SAND		213.3	
				3.0 MEDIUM DENSE WELL GRADED SAND			
				4.0	4		
				TILL			
				5.0		211.3	
				SHALE ROCK (DRILLED) (CORE RECOVERY 75%)	6		
				7.0		209.3	
				SHALE ROCK (DRILLED) (CORE RECOVERY 88%)	8		
					10		
				12.0	12	204.3	
				SHALE ROCK (DRILLED) (CORE RECOVERY 100%)	14		
					16		
				16.8		199.5	
				BOTTOM OF HOLE	18		

% WATER CONTENT

PLATE 4

Appendix B

Foundation Investigation Report

(By Engineering Materials Office-Foundation Design Section)

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

WP 62-82-~~01~~ 02 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.

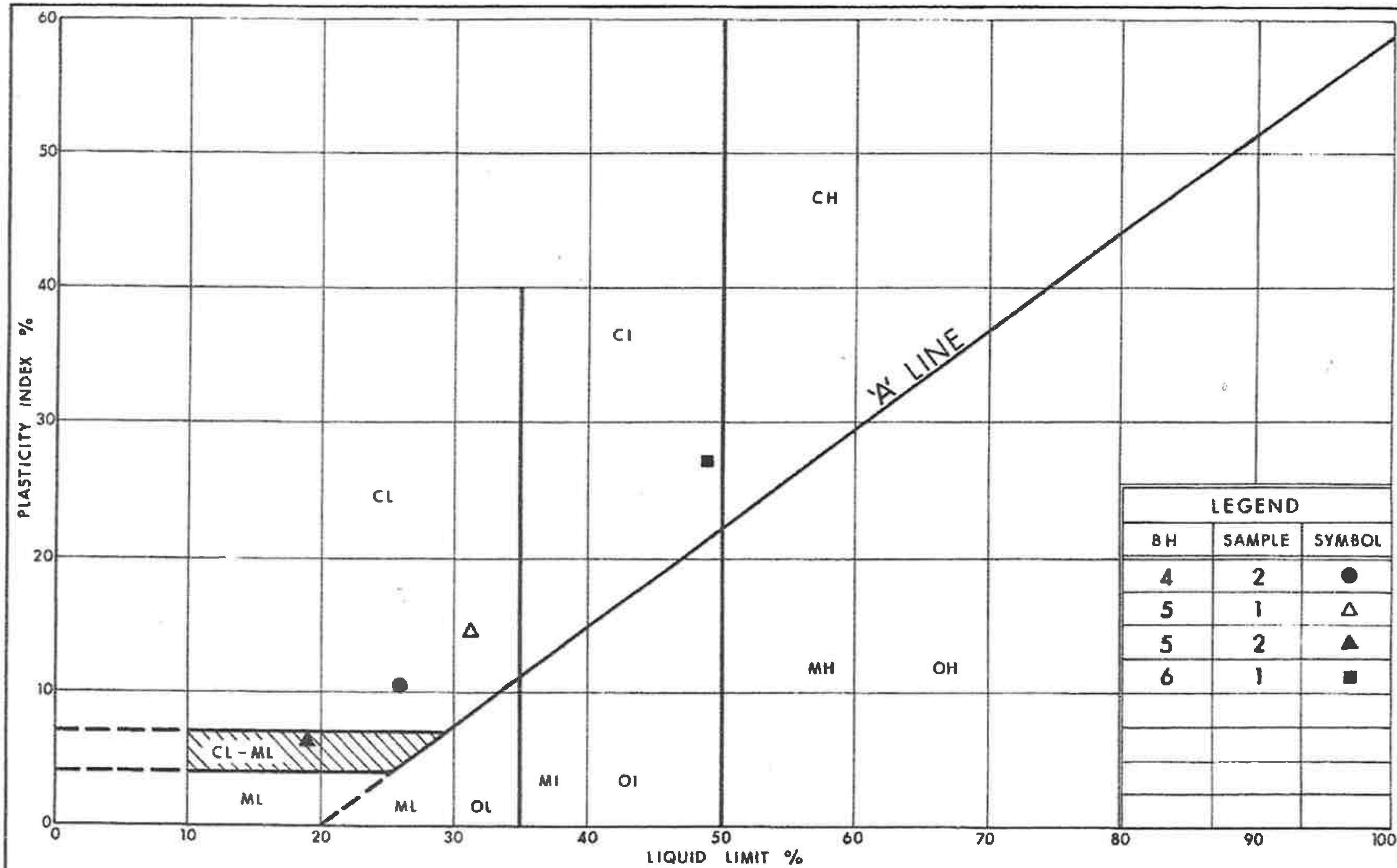
Rock Quality Designation (RQD) values for the weathered bedrock is as low as 0%, and from 50-89% for the unweathered shale. Based on the RQD, the unweathered shale is considered to be of fair to good quality.

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.

APPENDIX

Oct 75, FF-5-21



LEGEND		
BH	SAMPLE	SYMBOL
4	2	●
5	1	△
5	2	▲
6	1	■

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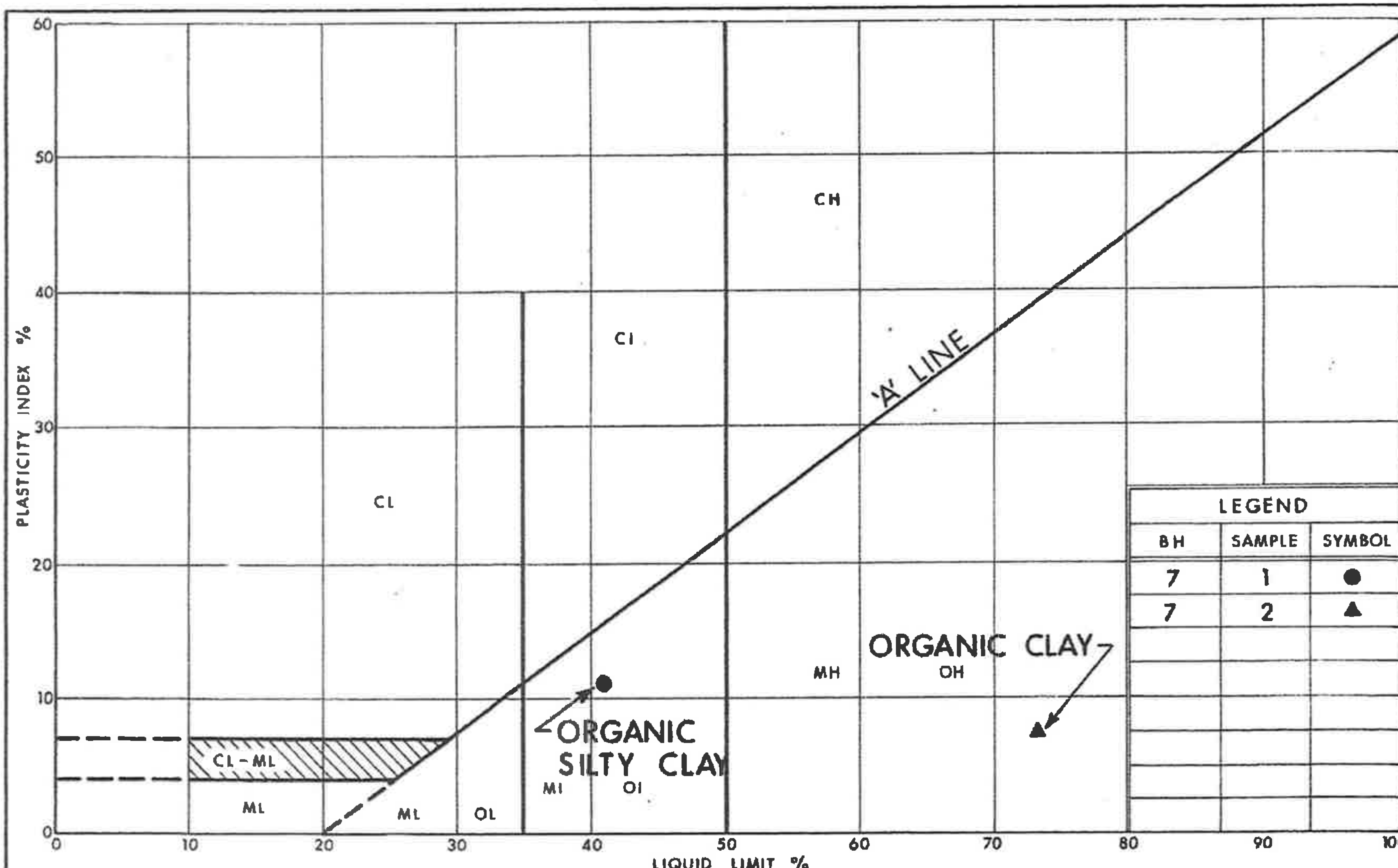
PLASTICITY CHART

SILTY CLAY, TRACE OF SAND AND GRAVEL

FIG No 1

W P 62-82-01

Oct 75, FF-5-21



LEGEND		
BH	SAMPLE	SYMBOL
7	1	●
7	2	▲

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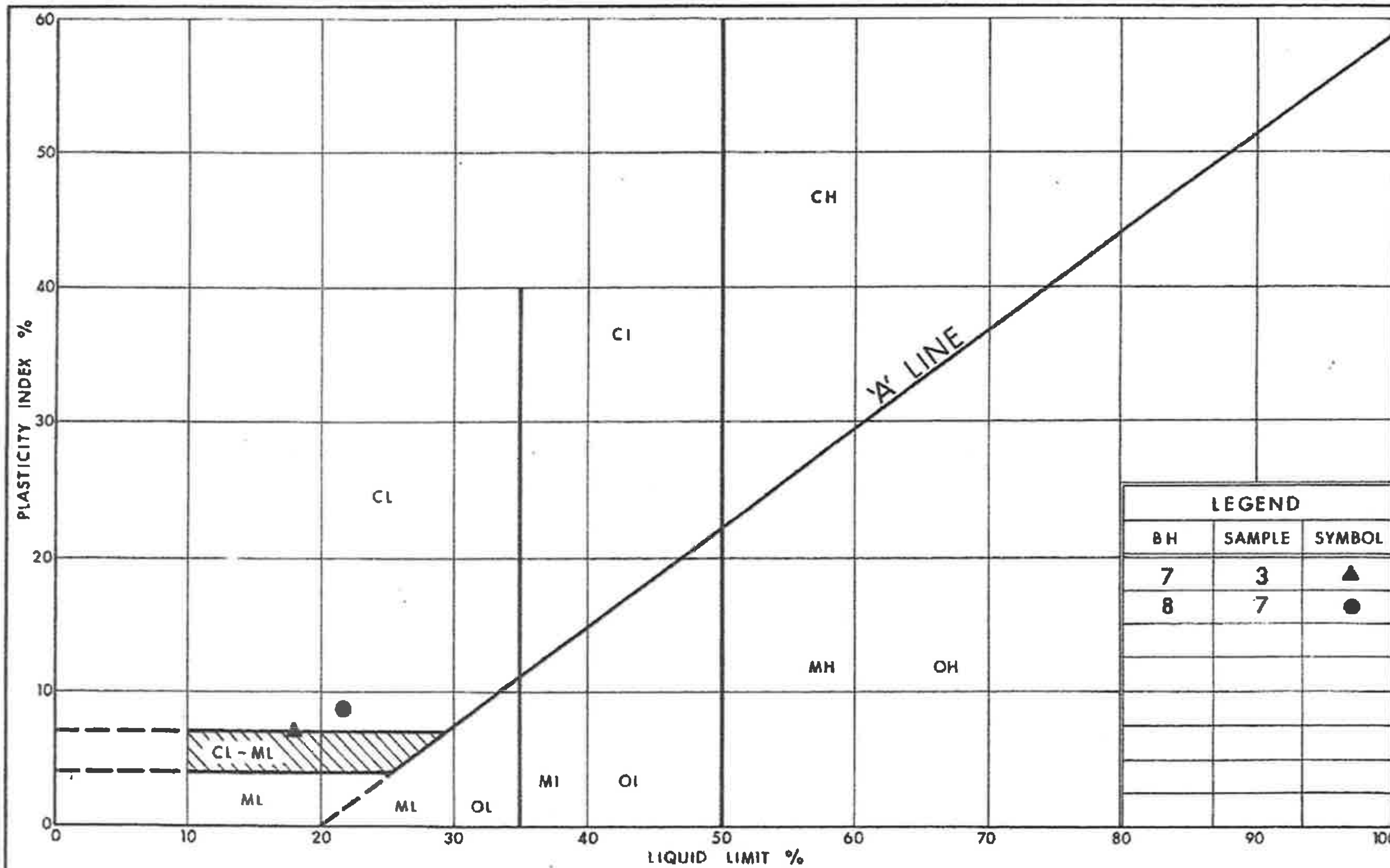
PLASTICITY CHART

FIG No 2

W P 62-82-01

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
7	3	▲
8	7	●

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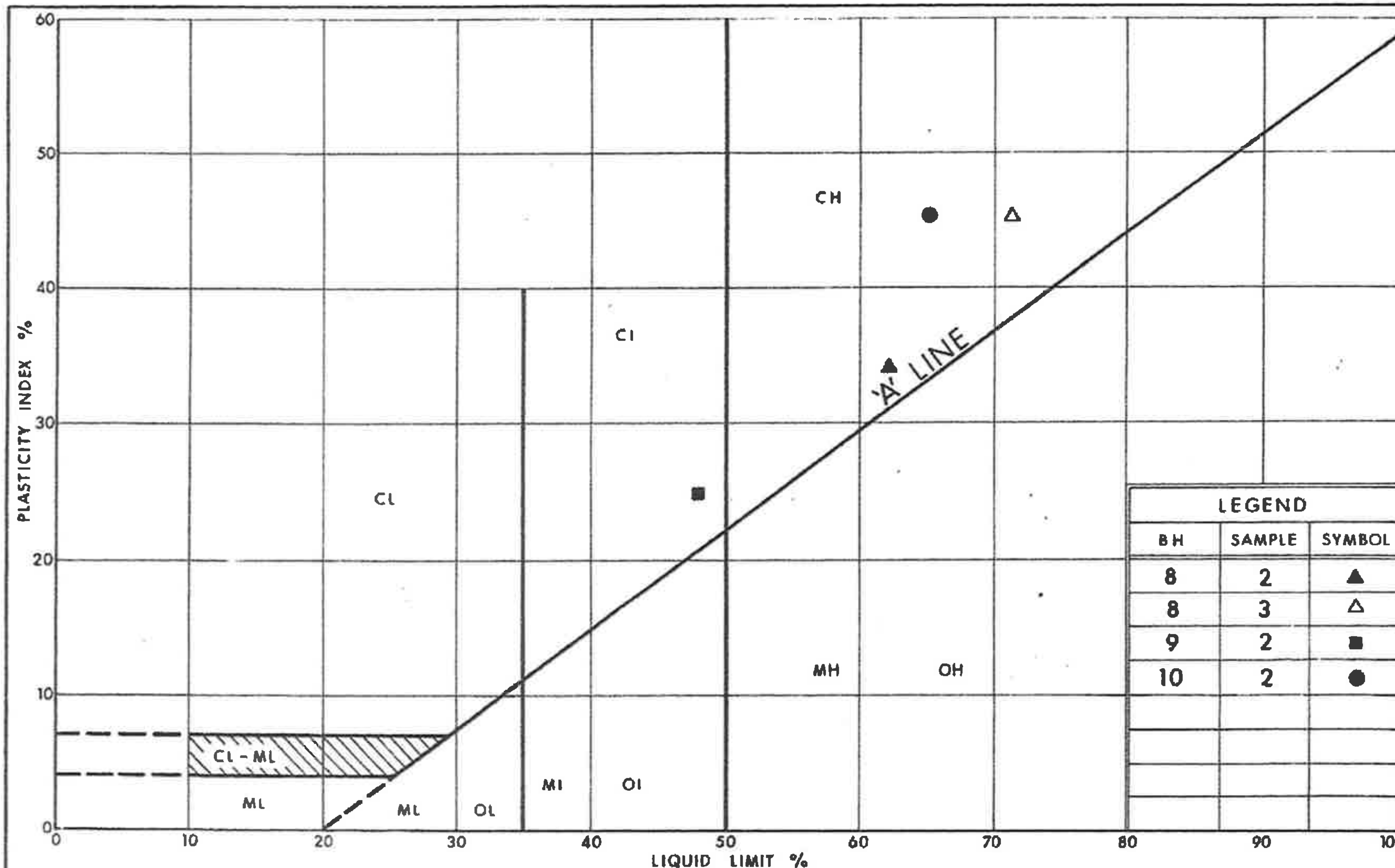
PLASTICITY CHART
HET MIXTURE OF
SILTY CLAY, SAND & GRAVEL (Glacial Till)

FIG No 3
W P 62-82-01

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
8	2	▲
8	3	△
9	2	■
10	2	●

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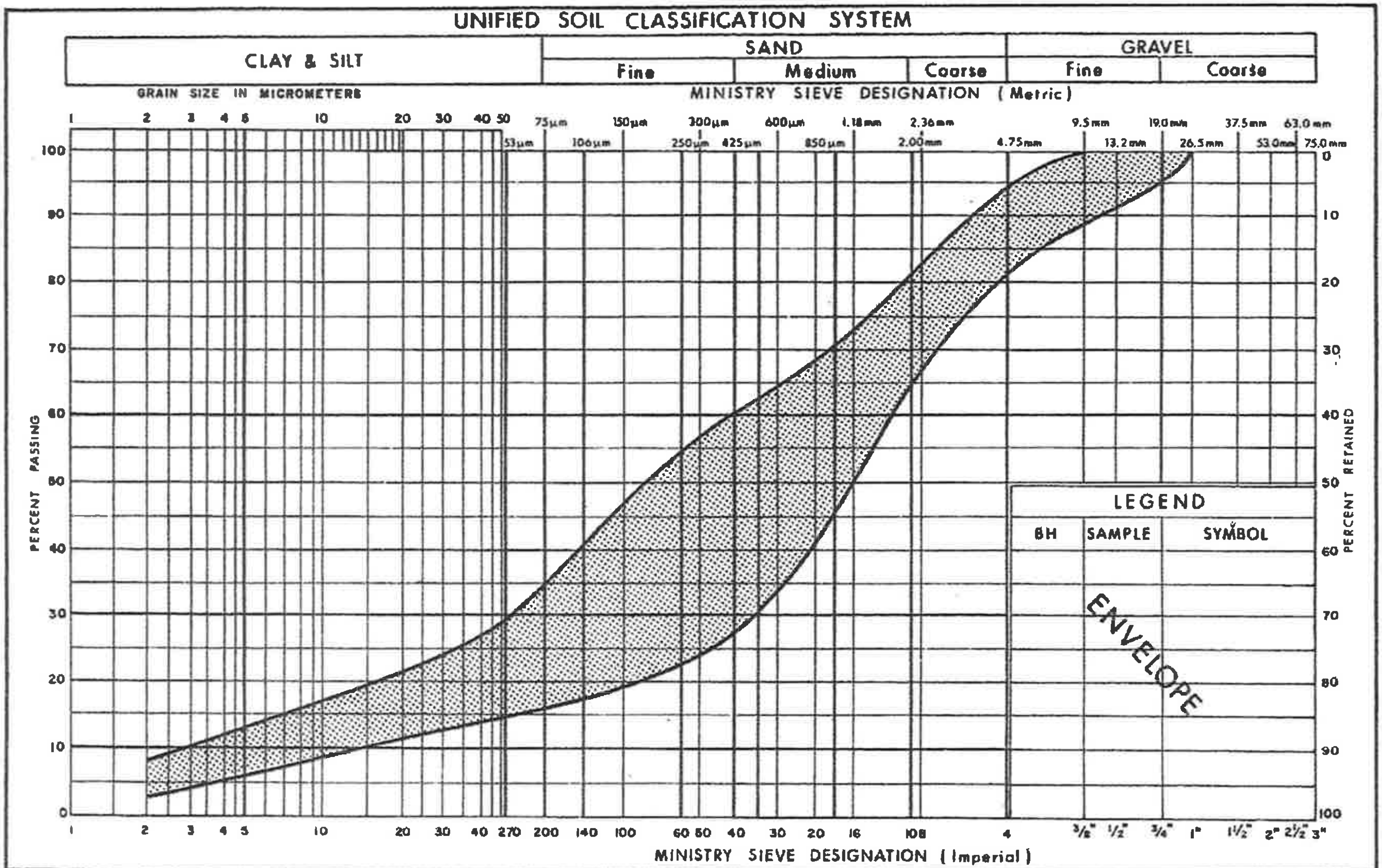
PLASTICITY CHART
FILL
(SILTY CLAY TO CLAY, TRACE TO WITH SAND)

FIG No 4
WP 62-82-01

78 12 M



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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 (R Q D), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	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
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1, %	VOID RATIO
n	1, %	POROSITY
w	1, %	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX $= w_L - w_p$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	1, %	VOID RATIO IN LOOSEST STATE

e_{min}	1, %	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_U	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

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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

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.8	Asphalt Surface							
0.0	Asphalt Pavement							
	Sand and Gravel Subbase Very Dense	1	SS	90/	15 cm	66		
66.0 / 0.8	Silty Sand trace gravel Very Dense	2	SS	95/	23 cm			
		3	SS	74		65	o	7 69 18 6
		4	SS	15/	10 cm		o	6 66 22 6
64.2 / 2.6	Black Shale Bedrock Weathered Unweathered					64		
		5	BXL RC	REC 90%		63		RQD = 50%
62.1 / 4.7	End of Borehole Note: Water Table Not Stabilized							

+3, x5 : Numbers refer to Sensitivity

20 / 15 ◇ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

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 [signature]

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × 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						
66.7	Asphalt Surface										
0.0	Asphalt Pavement										
65.9 / 0.8	Sand and Gravel Subbase						66				
	Silty Sand trace gravel Very Dense to Compact		1	SS	60	▽					
			2	SS	18		65		o		9 74 13 4
64.4 / 2.3	Black Shale Bedrock Weathered		3	SS	85/20 cm						
64.1 / 2.6	End of Borehole Refusal to Auger Note: Water Table Not Stabilized										

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

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 3

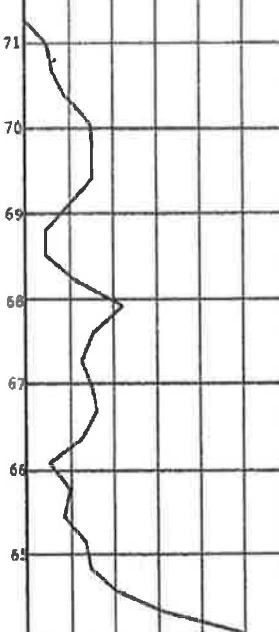
METRIC

W P 62-82-01
DIST 9 HWY 417
DATUM Geodetic

LOCATION Sta. 33 + 426.1; O/S 1.4 m RT ℄ Hwy. 417
BOREHOLE TYPE Solid Stem Auger & Cone Test
DATE 1984 04 19

ORIGINATED BY DT
COMPILED BY DT
CHECKED BY SP

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
72.8	Asphalt Surface										
0.0	Asphalt Pavement						72				Auger
							71				
							70				
							69				
							68				Cone
						▽	67				
							66				
							65				
64.4 / 8.4	Black Shale Bedrock						64				Auger
62.7	Weathered						63				
10.1	End of Borehole Refusal to Auger										



Notes:
1. Water Table Not Stabilized
2. No Samples Taken from this Borehole

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

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

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

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ / Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.9	Ground Surface								
0.0	Peat, with Layer of Organic Clay Stiff				▼	66			
		1	SS	9			W = 98%		
65.5 / 1.4	Silty Clay								
65.2 / 1.7	Silty Sand, Trace Clay some gravel (Glacial Till) Very Dense	2	SS	10		65		2.9%	
		3	SS	73					22 40 33 5
64.2 / 2.7	Black Shale Bedrock					64			
	Weathered / Unweathered	4	BXL RC	REC 38%					RQD = 0%
		5	BXL RC	REC 88%		63			RQD = 89%
62.5 / 4.4	End of Borehole								

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 20 40 60

OFFICE REPORT ON SOIL EXPLORATION

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

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 5

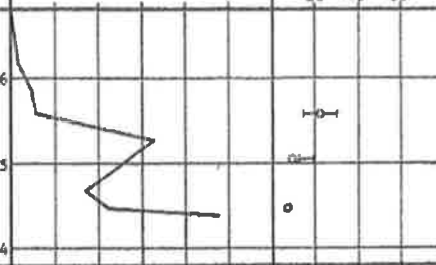
METRIC

W P 62-82-01 LOCATION Sta. 13 + 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

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	WATER CONTENT (%) (PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L) 20 40 60	UNIT WEIGHT γ / Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.8	Ground Surface										
0.0	Peat						66				
65.7 / 1.1	Silty Clay, Trace Sand Stiff to Hard		1	SS	15						0 5 55 40
65.0			2	SS	77/20 cm		65			1.8%	
1.8	Silty Sand, Trace Clay some gravel (Glacial Till) Very Dense										
64.5 / 2.3	Black Shale Bedrock Weathered		3	SS	70/15 cm						18 48 29 5
63.8			4	SS	60/8 cm		64				
3.0	End of Borehole Refusal to Auger										



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

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 6

METRIC

W P 62-82-01 LOCATION Sta. 33 + 435.0; O/S 19.0 m RT ℄ 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

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE	PLASTIC LIMIT Wp, NATURAL MOISTURE CONTENT W, LIQUID LIMIT WL; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.9	Ground Surface										
0.0	Peat					▽	66				
65.9 / 1.0	Silty Clay, trace sand, gravel		1	SS	20					6.1%	
65.5 / 1.4	Peat, traces of fibres, roots Very Stiff to Soft		2	SS	16		65				
			3	SS	4				W=119.5%		
63.9 / 3.0	Black Shale Bedrock		4	SS	73		64				
			5	SS	21		63				
	Weathered		6	SS	50/3 cm						
	Unweathered		7	BXL RC	REC 68%						RQD = 68%
62.1 / 4.8	End of Borehole Note: Water Table Not Stabilized										

+3, x5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 7

METRIC

W P 62-82-01 LOCATION Sta. 33 + 468.6; O/S 22.5 m RT ℄ 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 [illegible]

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH	WATER CONTENT (%) (W_p W W_L)	UNIT WEIGHT γ / Organic Matter	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
67.3	Ground Surface									
0.0	Organic Clay to Silty Clay trace sand, fibres Stiff				▽	67				
		1	SS	11		66				
		2	SS	9				W_p = 97 % W_L = 105 %		
65.0 / 2.3	Heterogeneous Mixture of Silty Clay, Sand, Gravel, trace organics (Glacial Till) V. Stiff	3	SS	21		65	80/15 cm		2.1%	
64.3 / 3.0	Weathered / Unweathered Black Shale Bedrock	4	BXL RC	REC 90%		64				RQD = 57%
		5	BXL RC	REC 100%		63				RQD = 80%
62.3 / 5.0	End of Borehole									

OFFICE REPORT ON SOIL EXPLORATION

SHEAR STRENGTH: O UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE

+3, ×5 : Numbers refer to Sensitivity

20 15 ◇ 5 (%) STRAIN AT FAILURE 10

Ministry of Transportation and Communications
Ontario

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 (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 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						
71.9	Ground Surface										
0.0	Fill Organic Silty Clay						71				
	Clay of High Plasticity Trace to some sand with silt Firm to Stiff		1	SS	5						
			2	SS	5		70	+12			0 12 32 56
			3	SS	6		69	+7			0 2 28 70
			4	SS	13		68 67	30/5 cm			
66.6 5.3	Organic Silty Clay, trace of Fibrous Material, Wood		5	SS	20	▼					
66.1 5.8	Silty Clay, trace sand, gravel Very Stiff		6	SS	18		66				
65.5 6.4	Heterogeneous Mixture of Silty Clay, Sand, Gravel trace organics (Glacial Till) V. Stiff		7	SS	29		65			1.4%	
64.7 7.2	End of Borehole Refusal to Auger Probable Bedrock										

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

20 / 15 ⌖ 5 / 10 (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

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

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%) Wp W Wl	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
71.1	Ground Surface										
0.0	Topsoil					*	71				
							70				
	Fill		1	SS	3						
	Silty Clay						69	+5			
	some sand										
	Stiff to Very Stiff		2	SS	4		68	+7			0 24 18 58
			3	SS	12		67				0 14 30 56
							66	+5			
65.2								75/5 cm			
64.9	Organic Silty Clay		4	SS	30/8 cm		65				
6.2	End of Borehole Refusal to Auger Probable Bedrock										

* Note:
Water Level
Not Established

+3, ×5 : Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 10

METRIC

W P 62-82-01 LOCATION Sta. 33 + 402.2; O/S 18.9 m LT ℄ 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 [signature]

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
72.2	Ground Surface					72	
0.0	Topsoil						
	Fill Clay of High Plasticity some sand silt Very Stiff	1	SS	5		71, 70	
		2	SS	3		69	0 14 20 66
		3	SS	6	▼	68, 67	
66.1 6.1	Organic Silty Clay trace sand Stiff	4	SS	35		66	
65.7 6.5	Black Shale Bedrock Weathered					65	
64.9 7.3	End of Borehole Refusal to Auger						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100 — 100/ 23 cm

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

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ Organic Matter

$+^3, \times^5$: Numbers refer to Sensitivity   15 ⌖ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

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

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
66.6	Ground Surface						
0.0	Topsoil					66	
	Silty Sand some gravel Compact				▽		10 56 30 4
65.7 / 0.9	Organic Silt some sand trace gravel Loose	1	SS	14			
		2	SS	3		65	
		3	SS	6			
64.0 / 2.6		4	SS	100/15 cm		64	
	Black Shale Bedrock Weathered / Unweathered	5	BXL RC	REC 83%		63	RQD = 70%
62.0 / 4.6	End of Borehole						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100 — 50/3 cm

SHEAR STRENGTH: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ

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

15 ✧ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 13

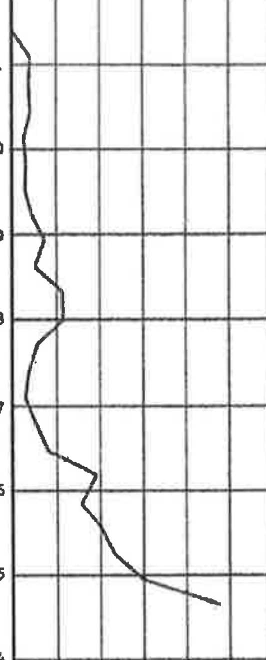
METRIC

W P 62-82-01 LOCATION Sta. 33 + 466.4; O/S 1.2 m RT ℄ Hwy. 417 ORIGINATED BY DT

DIST 9 HWY 417 BOREHOLE TYPE Solid Stem Auger & Cone Test COMPILED BY DT

DATUM Geodetic DATE 84 04 19 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, × LAB VANE	PLASTIC LIMIT W_p, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
72.9	Asphalt Surface										
0.0	Asphalt Pavement										Auger
							72				
							71				Cone
							70				
							69				
							68				
							67				
						▽	66				
64.7							65				
8.2	Black Shale Bedrock Weathered						64				Auger
63.8											
9.1	End of Borehole Refusal to Auger Notes: 1. Water Table Not Stabilized 2. No Samples taken from this Borehole										



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

OFFICE REPORT ON SOIL EXPLORATION

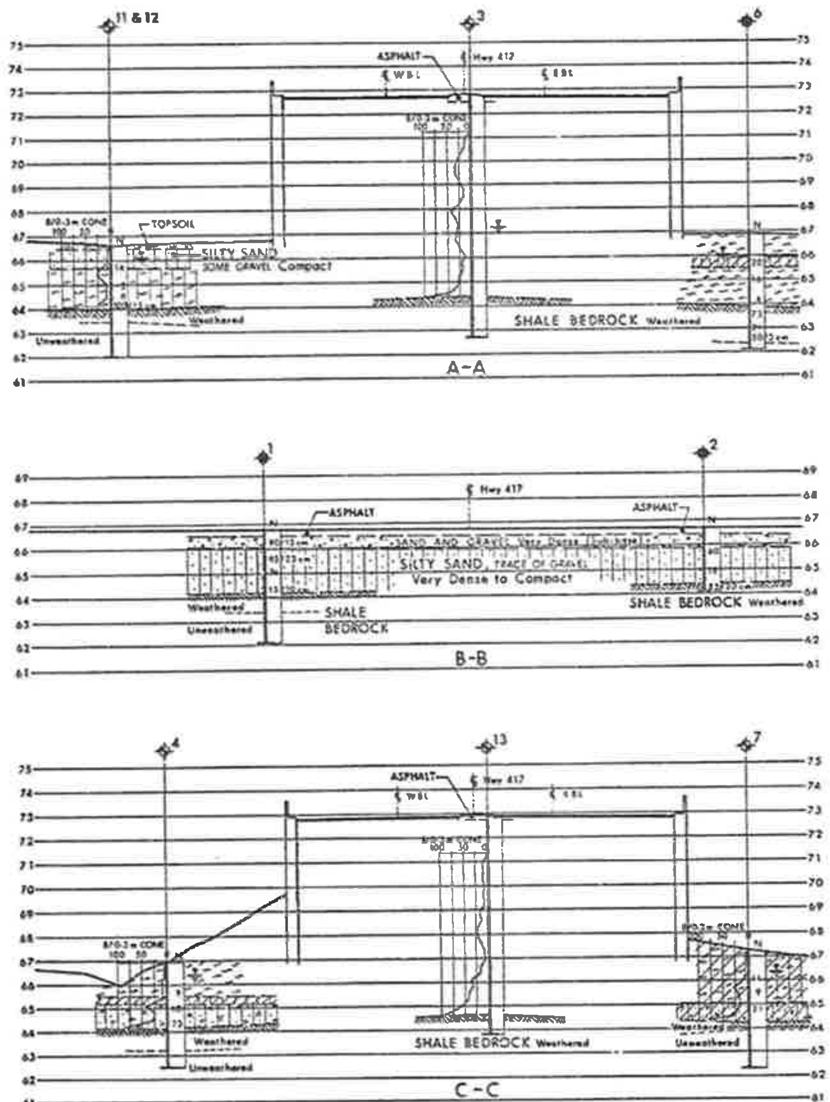
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
WP No 62-82-01

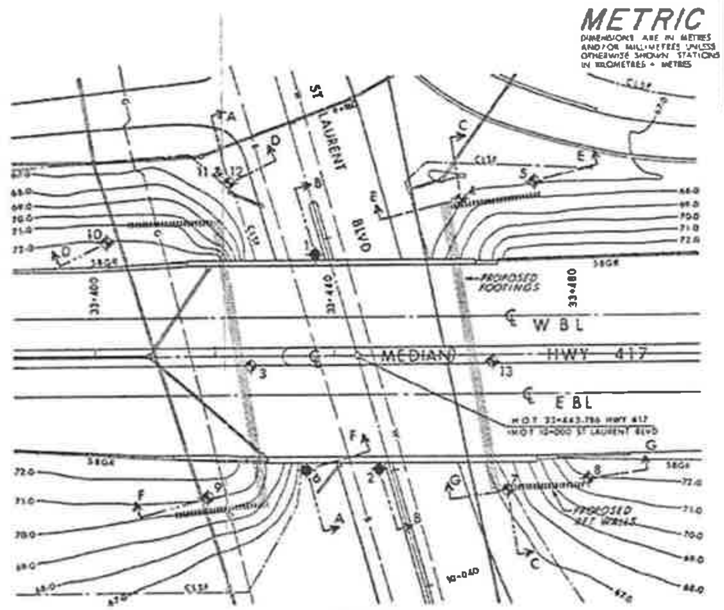
ST LAURENT BLVD OVERPASS

BORE HOLE LOCATIONS & SOIL STRATA

SHEET

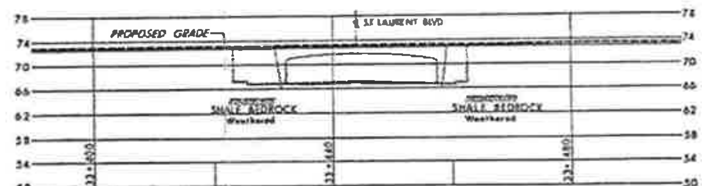


SECTIONS
SCALE
HOR 4m 2 0 4m
VERT 2m 1 0 2m



PLAN
SCALE
8m 4 0 4 8m

NOTE:
For Sections D-D, E-E, F-F and G-G Refer to Dwg 628201-B

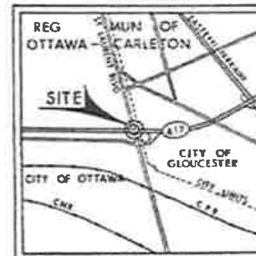


℄ PROFILE MEDIAN HWY 417
SCALE
8m 4 0 8m

SOIL STRATIGRAPHY LEGEND

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

REF No F-7009-1 / 1984 01



KEY PLAN
SCALE
0.5km 0 0.5 1km

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL 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	18.6m Rt
3	72.8	33+426.1	1.4m Rt
4	66.9	33+461.6	25.4m 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	72.7	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

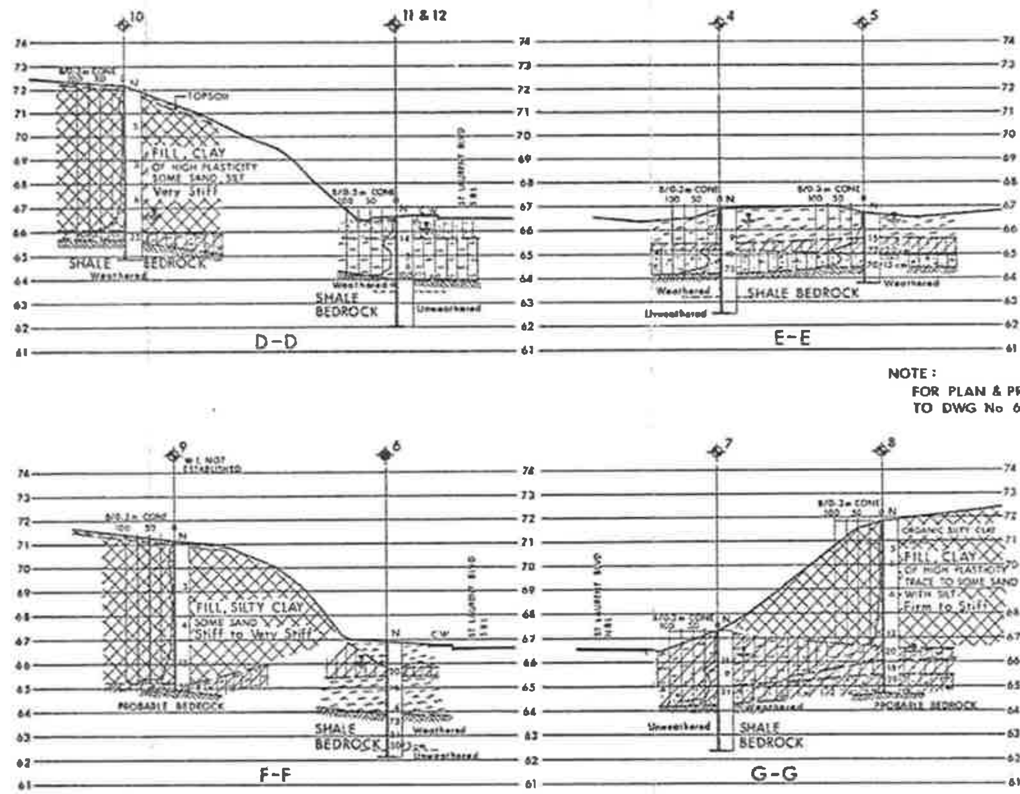
REV	DATE	BY	DESCRIPTION

Geocres No 31G5-137

HWY No 417			DIST 9
SUBM'D LP	CHECKED	DATE 1984 06 08	SITE 3-72
DRAWN	CHECKED	APPROVED	DWG 628201-A

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



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

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

SEE DWG 628201-A

KEY PLAN
SCALE

SOIL STRATIGRAPHY LEGEND

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

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 B.H. 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	22.5m 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
SUBM'D L.P.	CHECKED	DATE 1984 08 12	SITE 3-72
DRAWN	CHECKED	APPROVED	DWG 628201-B

Appendix C

Site Photographs

Photograph 1. Highway 417 Overpass at St. Laurent Blvd., Looking North

Photograph 2. Highway 417 Overpass at St. Laurent Blvd., Looking South

Appendix D

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J 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 COMPOSION AND STRUCUTRAL 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*

JOINT 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
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	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 = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

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

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SPECIALISTS MANAGING THE EARTH

PRELIMINARY FOUNDATION DESIGN REPORT, HIGHWAY 417 / ST. LAURENT BLVD. BRIDGE, CITY OF OTTAWA, ONTARIO, G.W.P. 4011-06-00, GEOCRES NO. 31G5-239

AECOM

Project: TRANETOB01226AB
March 24, 2011

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CONTENTS

PRELIMINARY
FOUNDATION DESIGN REPORT
HIGHWAY 417/ST. LAURENT BLVD. BRIDGE
CITY OF OTTAWA, ONTARIO
G.W.P. 4011-06-00

5 DISCUSSION AND RECOMMENDATIONS

The existing Highway 417 overpass structure at St. Laurent Blvd. is to be widened. The existing structure is a two-span bridge with a total length of about 38 m. It is proposed to widen both the west and east abutments and also the central pier on both the north and south sides. A widening of approximately 2.5 m is anticipated to the north and 1.25 m to the south. New retaining walls are also proposed at each approach quadrant and as such temporary roadway protection will be required to facilitate the removal of the existing retaining walls and for new retaining wall construction, as well as for the construction of the foundations of the widened sections of the bridge.

The sub-surface conditions were previously explored in 1957 at three (3) borehole locations (see Table 3.1.1 in Section 3 of the foundation investigation section of this report) and in 1984 at ten (10) boreholes and three (3) DCPT's (see Table 3.2.1 in Section 3 of the foundation investigation section of this report). This preliminary foundation design report is prepared based on the available subsurface information only. No additional borehole was drilled for this preliminary foundation design report, and thus the existing subsurface conditions at the Site may vary from the previously obtained information. The overburden subsurface conditions at the Site are considered variable, even though the native overburden is relatively shallow (about 3 m below o.g.) overlying the bedrock. In general, from the o.g. level, under the topsoil, the boreholes contacted non-cohesive or cohesive organic soils. Below the organic soils, the boreholes show the presence of native granular soils (sand and silt) and cohesive soils (silty clay). Cohesive and non-cohesive glacial till deposits were found below the natural non-cohesive and cohesive soils and these till deposits were in turn underlain by a shale bedrock in all boreholes at El. 65.7 to 63.9 m. The fill used for the existing structure approach was found to be mainly consisting of clay to silty clay (i.e. cohesive materials).

The water level at the time of investigation (1957 and 1984) was contacted between Elevations 67.2 m and 65.4 m. It should be noted that the groundwater table can expected to be subject to seasonal fluctuations and in response to major weather events. It is also pointed out that a perched water condition can be encountered within the approach fills and in the surficial native soils.

5.1 Foundations

We understand that the existing approximately 38 m long bridge is supported on spread footings based on the drawing (see Appendix E) provided to us by AECOM. Details of the existing retaining wall foundation and the proposed construction method (i.e. full road closure, lane closure or staged construction) and the construction sequence were not available at the time of preparing this report.

Based on the available subsurface information, we have considered a number of foundation options varying from normal spread footings to deep foundations which include drilled caissons, micropiles and driven steel piles.

5.1.1 Abutments and Central Pier

We understand that the existing abutments are supported on spread footings founded on unweathered shale bedrock at about El. 62.5 to 63.0 m (west abutment) and at about 63.5 m (east abutment) based on the General Arrangement (GA) drawing (see Appendix E) provided to us by AECOM, as well as an old GA drawing (given in Appendix E and which is also available in MTO GEOCRES information system, along with several other drawings).

It is our understanding that the existing overpass structure central pier is supported by spread footings placed on bedrock at about El. 62.8 m, based on the general arrangement drawing provided to us by AECOM (see Appendix E).

The use of same foundation type is generally recommended for most bridge structure widening for better performance of the structure. If proper dewatering and inspection of footing excavation can be provided, the use of spread footing foundation can be a feasible option for this project. It should be pointed out that care should be taken for the anticipated deep excavation adjacent to or near the existing structure with the observed high groundwater table (excavation will extend to considerable depths below the groundwater table) and the nature of underlying bedrock (susceptible to slaking and degradation when exposed to atmosphere and possible heaving with time, especially in the presence of water at the excavation bottom). As well, considerable shoring effort may be required to retain the existing embankment (if the existing retaining walls need to be demolished prior to excavation) and the native soils and the highly weathered shale, overlying the unweathered bedrock, in order to effect the excavation for the abutment foundation. The new retaining wall construction can be carried out with the abutment foundation construction, if spread footing can be used. For the central pier, temporary shoring will be also required to effect the excavation for the spread footing foundation widening along the centreline of St. Laurent Blvd.

The use of augered and cast-in-place concrete foundations (drilled caissons) can also be considered as a feasible foundation option for this project. If drilled caisson foundation is selected, the new proposed retaining walls can also be founded on drilled caisson foundations.

The use of driven piles is considered impractical due to the anticipated short pile lengths, especially at the central pier location.

The use of micropiles is also considered and discussed in Section 5.1.1.4.

When designing foundations it must be remembered that total settlements experienced by the new foundations will translate into differential settlements between the existing bridge structure and the widened section.

The advantages and disadvantages of various foundation types at the support locations are summarized in Appendix F.

The following paragraphs present a further discussion on these options.

5.1.1.1 Spread Footing Foundations

The abutments and central pier can be supported on spread footing foundations similar to the existing bridge foundation typically placed on the shale bedrock. If higher bearing resistances are required, spread footing foundations at greater depths can also be considered. Placing footings of widening deeper than the existing foundations, however, is not recommended, as this approach may lead to disturbing the existing footings. It should therefore only be considered if higher resistances are necessary. The following table summarizes the recommended resistances and highest footing elevations at the borehole locations:

Table 5.1.1.1.1 Spread Footing Foundations for Abutments

Location	Applicable Boreholes (year of investigation)	Recommended Highest Footing Elevation (Bottom of Footing) (m)	Recommended SLS ♦(kPa)	Recommended Factored ULS (kPa)	Subgrade Material
East Abutment	2 (1957)*	64.4/64.2 63.4*	400 Will not govern	700 1500	Very Dense Till / Weathered Shale Shale**
	4 (1984)	64.0 63.0	400 Will not govern	700 1500	Weathered Shale Shale**
	5 (1984)****	64.0 63.0***	400 Will not govern	700 1500	Weathered Shale Shale**
	7 (1984)	64.0 63.0	400 Will not govern	700 1500	Weathered Shale Shale**
	8 (1984)****	64.5*** Insufficient information	400 -	700 -	Very Dense Till or Weathered Shale -
	13 (1984)	64.0 63.0***	400 Will not govern	700 1500	Weathered Shale Shale**
West Abutment	1 (1957)*	64.2/63.9 62.5*	400 Will not govern	700 1500	Very Dense Till / Weathered Shale Shale**
	3 (1957)*	64.2 62.5*	400 Will not govern	700 1500	Weathered Shale Shale**
	3 (1984)	64.0 62.5***	400 Will not govern	700 1500	Weathered Shale Shale**
	6 (1984)	62.8 62.2	400 Will not govern	700 1500	Weathered Shale Shale**
	9 (1984)****	64.7 *** Insufficient information	400 -	700 -	Weathered Shale -
	10 (1984)****	65.4 Insufficient information	400 -	700 -	Weathered Shale -
	11&12 (1984)	64.0 63.0	400 Will not govern	700 1500	Weathered Shale Shale**

* Founding elevation need to be confirmed at the site during the construction because R.Q.D was not recorded in 1957 investigation
** Relatively sound shale
*** Presumed elevation (borehole terminated in weathered shale or overlying glacial till)
**** Retaining wall footing
* SLS for 25 mm in total settlement, 19 mm in differential settlement

The retaining walls can be designed using the resistances given in the above table.

Table 5.1.1.1.2 Spread Footing Foundations for Central Pier

Location	Applicable Boreholes (year of investigation)	Recommended Highest Footing Elevation (Bottom of Footing) (m)	Recommended SLS** (kPa)	Recommended Factored ULS (kPa)	Subgrade Material
Central Pier	1 (1984)	64.0 63.0	400 N/A	700 1500	Weathered Shale Shale*
	2 (1984)	64.0 63.0***	400 N/A	700 1500	Weathered Shale Shale*

* Relatively sound shale
** SLS for 25 mm in total settlement, 19 mm in differential settlement
*** Presumed elevation (borehole terminated in weathered shale)

In any event, allowance should be made to place a 150 mm thick concrete mud mat (i.e. skim coat) in the footing excavation as soon as possible (not more than four hours) after excavating to the bearing grade. The footing excavation must be inspected, evaluated and approved by the Geotechnical Engineer prior to pouring the concrete mud mat.

It should be noted that in between and beyond borehole locations, the bedrock surface and the depth to the suitable founding surface may vary.

For the use of spread footing foundations on the bedrock, all loose or weathered rock under the footprint of the footing should be removed and replaced with mass concrete. All footing excavations and bearing surfaces must be inspected, evaluated and approved by a Geologist or Geotechnical Engineer who is familiar with the findings of this investigation. This is important for this site, since the bedrock appears to be extremely to highly weathered to variable depths. Mass concrete may be placed to raise the grade to the founding level, where necessary. Probing may be necessary to ensure that a highly weathered soft rock layer(s) does (do) not underlie the sufficiently sound rock layer beneath the footing. Probing or star drilling may be considered for this purpose.

Under the inclined loading conditions the Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6.7.4 of C.H.B.D.C.

Sliding resistance can be provided by utilizing the sliding resistance between concrete and clean bedrock surface. For the evaluation of the sliding resistance of the foundation (C.H.B.D.C. 6.7.5) the ultimate angle of friction between the underside of the foundations and the clean bedrock surface (or between concrete surfaces) can be taken as 28°, while the ultimate angle of friction (unfactored) between the underside of the foundations and the glacial till can be taken as 26°. If additional horizontal resistance is required or if the rock surface is not sufficiently level, dowelling or keying-in into the bedrock can be considered. Such measures would be required if the rock surface is smooth and/or inclined.

If there are net uplift forces which are to be resisted by rock anchors, the factored rock/grout bond resistance at U.L.S. can be taken as 600 kPa and at the S.L.S. it can be taken as 300 kPa. The upper 1.0 m of the rock should, however, not be included in calculating the resistance and the minimum embedment depth should be 2.0 m into the sound rock (embedded length in the sufficiently sound rock). The anchors should also be checked for rock wedge pull-out assuming a 60 degree apex cone/wedge and the anchor group resistance should also be checked.

For frost protection the footing should have a permanent earth cover of at least 1.8 m.

As can be seen from above table, deep excavations will be required, which extend below the water table. These will necessitate shoring and dewatering. In addition, the excavations can be expected to extend to variable depths.

5.1.1.2 Drilled Caisson Foundations

The use of augered and cast-in-place concrete foundations (drilled caissons) can be considered as a feasible foundation option for the abutments and the central pier. From the reliability viewpoint, drilled caisson foundation is a favourable option for this project. As it will require little or no shoring for the central pier this may be an attractive alternative, provided that sufficient overhead space is available to install the caisson under the existing bridge.

Caissons extended at least 1.5 m into the relatively sound shale bedrock can be designed for a vertical geotechnical resistance of 2000 kPa at ULS and SLS need not be considered. This resistance value can be increased to 2500 kPa at ULS with 2.0 m socketing into the sound rock. Higher resistance of caisson is not recommended at this site due to the presence of soft zones in the shale bedrock. The following table summarizes the anticipated caisson bottom elevations at the borehole locations.

Table 5.1.1.2.1: Caisson Foundations for Abutments

Location	Applicable Boreholes (year of investigation)	Recommended bottom of caisson elevation (m)	Recommended SLS (kPa)	Recommended Factored ULS (kPa)	Subgrade material
East Abutment	2 (1957), 4 (1984)* and 7 (1984)*	61.7 61.2	N/A N/A	2000 2500	Sound Shale Sound Shale
West Abutment	1 (1957), 3 (1957)*, 6 (1984)* and 11&12 (1984)*	61.0 60.5	N/A N/A	2000 2500	Sound Shale Sound Shale

* Presumed elevation-borehole not deep enough

The retaining wall foundations can be designed using the resistance given in Table 5.1.1.2.1.

Table 5.1.1.2.2: Caisson Foundations for Central Pier

Location	Applicable Boreholes (year of investigation)	Recommended bottom of caisson elevation (m)	Recommended SLS (kPa)	Recommended Factored ULS (kPa)	Subgrade material
Central Pier	1 (1984)* and 2 (1984)*	61.7 61.2	N/A N/A	2000 2500	Sound Shale Sound Shale

* Presumed elevation-borehole not deep enough

These design values are applicable to commonly used caisson sizes in Ontario (i.e. between 0.76 m and 1.8 m diameter) provided the minimum caisson length is 4.0 m below the bottom of the pile cap. However, the use of relatively smaller caisson sizes (i.e. between 0.76 m and 1.35 m) would be preferable as these are relatively easier and more efficient to install. For example, a 0.9 m diameter caisson will have a base area of $r^2\pi=(0.9/2)^2x3.1416=0.64\ m^2$. When designed for a ULS value of 2000 kPa, the caisson would be capable of carrying an axial load of 0.64 m^2 x 2000 kN/m^2 = 1280 kN/caisson at ULS. Similarly, if a 1.2 m diameter caisson is used, then the caisson resistance at ULS would be $(1.2/2)^2$ x3.1416 x2000 kN/m^2 =2260 kN/caisson.

As was mentioned before, these resistance values assume a minimum of 1.5 m to 2.0 m socket into the sound shale bedrock, depending on the design value used. Proper penetration into the sound shale bedrock must be verified during the installation of the caissons by the Geotechnical Engineer appointed by the QVE, who would also inspect the base of the caissons and approve them. We recommend that an NSSP be issued to cover this requirement.

The minimum caisson diameter is 0.76 m to enable the cleaning and inspection of the base of the caisson. The clear distance between any two adjacent caissons should be at least two diameters (edge to edge).

Difficulties may arise during the installation of the caissons due to the presence of granular (non-cohesive) soil types below the groundwater table, as well as the possible presence of cobbles, boulders and shale fragments in the till along with possible hard layers in the shale bedrock. Some dewatering is expected to be necessary to intercept and remove surface water and to pump out any perched water. Dewatering of the glacial till deposit may also be necessary. Temporary steel casing will be required during the construction of the caisson holes to prevent caving. The casing/liner would be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking'. Concrete must be poured expeditiously after the preparation and approval of the base of the caisson to prevent the deterioration of the shale bedrock. Even though these are standard aspects of caisson installation operations, we recommend that they be 'red-flagged' in the contract documents to reduce the possibility of claims for 'extras' by the contractor, including the possible presence of cobbles, boulders and shale fragments in the glacial till deposit. An NSSP should be issued to alert the contractor of cobbles, boulders, shale fragments in the overburden and the possible hard zones in the shale bedrock, as well as possible dewatering requirements.

The tremie concrete method can be used, if desired or required to reduce the degree of dewatering during the installation of the caisson foundations. Based on the borehole data, however, the use of the tremie concereting method is unlikely to be necessary.

The anticipated caisson elevations at the borehole locations, as given in the tables above, can be used for preliminary design purposes, with interpolation in between and beyond the borehole locations. Actual caisson depths in the field would be decided during their installation, ensuring at least 1.5 m to 2.0 m socket into the relatively sound shale bedrock. This is important for this project since the bedrock appears to be extremely to highly weathered to variable depths below the rock/overburden interface. The sockets may have to be advanced by rock coring or churn drilling since the shale bedrock at the site may contain medium strong layers.

As shown in Tables 5.1.1.2.1 and 5.1.1.2.2, most of the boreholes were not deep enough for estimating caisson depths and as such, presumed caisson elevations were used. This aspect will need to be verified for detail design purposes by drilling deeper boreholes (i.e. rock coring). At that time the recommended resistance values will need to be revisited and may need to be reused (possibly increased).

5.1.1.3 Driven Pile Foundations

Driven pile foundations, including steel-H-piles is not recommended due to the anticipated short lengths of the piles, as well as vibrations induced by pile driving close to the existing structure. The short pile lengths are expected to be an especially significant factor at the proposed central pier location. It may be possible to utilize driven H-piles at the abutment locations, where the pile caps can be set at higher elevations in comparison with the central pier location. Nevertheless, unless there is a compelling use for their use, it is our opinion that the use of driven piles, including steel H-piles, is not a good choice from reliability point of view.

5.1.1.4 Micropile Foundations

An alternative which may be considered is the use of micropiles to support the abutments, central pier and retaining walls. Under normal circumstances, micropiles are less cost effective than caissons or spread footings, but this case it may be an attractive solution if overhead restriction under the existing bridge present problems for the construction of the caissons and due to shoring requirements for spread footings.

A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access-restrictive environments and in most soil and rock types and ground conditions. Due to the small pile diameter (typically 160 mm to 260 mm), the end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the surrounding soil or rock and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

The axial resistance of micropile for this project would depend on the diameter, penetration length into the sound bedrock at this site and the type of reinforcement. The lateral resistances would also depend on the diameter, as well as, to a lesser extent, on the socket length into the bedrock.

The use of micropiles is generally less economical than spread footing foundations and caissons due to the required numbers of micropile to achieve similar geotechnical resistance to conventional foundations. However, it is advantageous if low overhead is necessity and/or interference of new foundation support with the existing pile foundations is a concern. As was mentioned before, geotechnical resistances will also depend on such factors as diameter, method of installation, socket lengths, etc. Typically, the geotechnical resistance is calculated by multiplying the circumferential area (i.e. circumference x length) by bond strength. For preliminary estimating purposes, the bond strength between the micropile and the sound shale bedrock can be taken as 300 kPa at SLS and 600 kPa at ULS, but the contribution from the relatively fractured upper 1.5 m of the bedrock should be disregarded. A special provision will need to be developed for this project.

The axial and horizontal resistances of micropiles and other details regarding the design of micropiles can be discussed with specialist contractor and we will be pleased to expand on this further should you wish to pursue this option.

5.2 Retaining Walls

It is our understanding that retaining walls at each quadrant of the existing overpass need to be replaced, as a part of the proposed widening. As well, all side slopes outside of new retaining walls are required to be maintained similar to the existing slope configuration (2H:1V). Details of the existing and proposed retaining wall and side slopes are not available at the time of preparing this report. We assume that the existing retaining walls are supported on strip footings placed on the bedrock, similar to the bridge foundations. It is our opinion that if desired the retaining wall foundations can be placed on the bedrock to match the foundations of the widened portion of the bridge. Alternatively, if the widening is to be supported on deep foundations, then consideration can be given to deep foundation options to support the retaining walls, matching the foundations for the widening. It should be pointed out that the existing retaining wall foundations may found within the excavation limit of the proposed widening. It may be prudent to point this possibility to the Contractor. The recommended geotechnical resistances were given in Section 5.1.

For the strip footing foundation construction at the site, relatively deep excavations will be required, which will extend below the water table. These will necessitate dewatering and considerable shoring, to retain the existing approach embankment (if the existing walls need to be demolished prior to construction) and native overburden for proposed foundation excavation and the possibly the extremely weathered upper zone of the shale bedrock. For the construction of retaining wall foundations placed on the bedrock, allowance should be made to place a 150 mm thick concrete mud mat (i.e. skim coat) in the footing excavation as soon as possible (not more than four hours) after excavating to the bearing grade. The foundation excavation should be inspected and approved by the Geotechnical Engineer prior to pouring the concrete mud mat. All loose or weathered rock under the footprint of the foundation should be removed and replaced with concrete. This is important for this Site, since the bedrock appears to be extremely to highly weathered to variable depths. Mass concrete may be placed to raise the grade to the founding level, where necessary. If spread footing foundations are to be used for the abutment supports, the proposed retaining wall foundations can be preferably installed at the same time and at about similar elevations as the proposed spread footing founding elevations.

For frost protection, the footing should have a permanent earth cover of at least 1.8 m (or equivalent artificial insulation), including pile caps.

5.3 Lateral Earth Pressures

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150, as given in Appendix G.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B', OPSS 1010) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with CHBDC S6-06. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A' and Granular 'B' Type II

Angle of Internal Friction, ϕ = 35° (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$ $K_o = 0.43$

Compacted Granular 'B' Type I

Angle of Internal Friction, ϕ = 32° (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$ $K_o = 0.47$

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

5.3.1 Seismic Design Data

5.3.1.1 Site Coefficient

The subsurface conditions encountered at the site are represented by Soil Profile Type I (see Clause 4.4.6.2 of CHBDC CAN/CSA-S6-06). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S, for the site is 1.0.

5.3.1.2 Seismic Zone and Zonal Acceleration Ratio (A)

Table A3.1.1 of the CHBDC provides a zonal Acceleration Ratio (A) of 0.20 and Velocity Related Seismic Zone (Z_v) of 2 for Ottawa. As site coefficient (S) is 1.0, and the zonal acceleration is 0.20, the design zonal acceleration ratio for the site can be taken as A=0.20.

5.3.1.3 Seismic Earth Pressures

Seismic (earthquake) loading should be taken into account in the design in accordance with Section 4.6 of the CHBDC.

In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as k_h=0.30. The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration coefficient, k_v. Three discrete values of vertical acceleration coefficient are typically selected analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

The following seismic active pressure coefficients (K_{AE}) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h, and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

Seismic Active Pressure Coefficients

Active Earth Pressure Coefficient	Granular 'A' (ϕ = 35° - unfactored)	Granular 'B' Type II (ϕ = 32° - unfactored)
Non-Seismic, K_a	0.27	0.31
Seismic, K_{AE}	0.55	0.61

In the calculation of K_{AE}, the effect of the friction between the wall and the soil is not considered (i.e.δ=0).

5.3.1.4 Liquefaction Potential

If the proposed structure is supported by deep foundations (caissons) or spread footings founded in/on the shale bedrock, the bedrock is considered not liquefiable.

The stiff cohesive fill material within the approach fill is unlikely to liquefy under earthquake loading. However, soft portions of the cohesive fill may settle under earthquake loading. These aspects need to be confirmed in the detail design stage. As well, different materials may have been used for the construction of the approach fills for the existing bridge.

5.4 Construction Comments

All excavations, shoring and backfilling should be carried out in conformance with the Occupational Health and Safety Act (OHSA), Regulation 213/91, as well as the following specifications.

OPSS539 – Protection Systems

SP902S01 – Excavation and Backfilling to Structures.

The boreholes show that the excavations can be expected to extend through some fill materials topsoil, non-cohesive and cohesive organic soils, native granular soils (sand and silt) and cohesive soils (silty clay).

These are generally underlain by cohesive and non-cohesive glacial tills and/or shale bedrock. The overburden materials can be classified as follows:

Granular Embankment (Pavement) Fill	Type 1 soil
Embankment Fill	Type 3 soil (stiff to very stiff)
	Type 4 soil (soft to firm)
Topsoil & Peat	Type 4 soil
Organic Soils	Type 4 soil
Silty Clay	Type 3 soil
Sand / Silty Sand	Type 3 soil above water level
	Type 4 soil below water level, if the soil was not dewatered
Glacial Till (granular)	Type 2 soil above water table
	Type 4 soil below water table, if the soil was not dewatered
Glacial Till (cohesive)	Type 2 soil above water table
	Type 3 soil below water table, if the soil was not dewatered

Dewatering will be required during the construction since the groundwater table at the time of MTO investigation was typically about 1 m below the then existing grade of St. Laurent Blvd. (about El. 65.5 to 66 m). It should be noted that the groundwater table can expected to be subject to seasonal fluctuations and in response to major weather events). Based on the available data, as excavations must be carried out in the dry, dewatering will be required. This may consist of deep wells/deep filtered sumps along with perimeter ditches (to intercept and dispose of surface/perched water). Due to the extent of excavation, shoring will be required for the spread footing and retaining wall construction. The shoring system should be designed by a Professional Engineer, experienced in this type of Work. All shoring should be in accordance with OPSS539.

In Ontario, shoring typically consists of soldier pile and timber lagging or sheet piling (with or without bracing / rakers). In this instance, tiebacks will also likely be required. The soldier piles can be expected to extend into the shale bedrock. Tiebacks would also extend into the shale bedrock, depending on the depth of shoring / height of the soils to be retained. We will be pleased to discuss such a system, if you wish us to do so.

The shoring system should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structural performance level. In this case, the required performance level is considered 2. The shoring system should be designed by a Professional Engineer, experienced in this type of work. As mentioned before, all shoring should be in accordance with OPSS539.

Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m^3)
Topsoil	0.55	0.72	1.8	14.0
Peat	0.61	0.78	1.4	12.0
Granular Pavement Fill	0.32	0.49	3.1	21.0
Approach Embankment Fill	0.41	0.60	2.2	17.0
Silty Clay	0.38	0.55	2.7	18.0
Sand / Silty Sand	0.30	0.50	3.0	21.0
Cohesive Till	0.33	0.50	3.0	20.0
Granular Till	0.29	0.45	3.4	21.5
Weathered Shale	0.26	0.42	3.6	22.0

It should be pointed out that the presence of cobbles, boulders and shale fragments can be expected within the overburden, as well possibly in the embankment and other fill layers. These can be expected to cause problems during the installation of shoring units. This aspect should be 'red-flagged' in the contract documents.

It should also be pointed out that monitoring of the existing bridge including retaining walls and side slopes need to be considered due to the expected dewatering requirement immediately beside the existing structures.

As well, the presence of the existing sanitary sewer adjacent to the existing east abutment of Highway 417 overpass at St. Laurent Blvd. may need to be considered. This aspect will need to be confirmed at the detail design stage.

5.5 Embankment Widening

Based on the available data, foundation failures are not anticipated for widened approach embankments with side slopes of 2H:1V or flatter, provided that all unsuitable soils are removed as per MTO standards, including all topsoil and other unsuitable, weak or very loose soils. These include the existing organic deposits discussed in Section 3.2.1.3 Peat and Section 3.2.1.6 Organics Soils, where applicable. This aspect should be looked into in greater detail during the detailed investigation phase.

Settlement analyses and precautions are required to minimize differential settlements which may lead to cracking of the pavement (asphaltic concrete), especially since the previous embankment fill (i.e. Circa 1950's) appears to consist of rather erratically compacted and/or inadequately compacted clayey soils, which were typically described as being of "high plasticity".

The widening can be expected to cause some settlement of the existing embankment fills, depending on the details of widening, as well as the nature of the existing embankment materials and whether all the unsuitable soils (including naturally occurring organic, rather deep organic deposits) were stripped under the full footprint of the existing embankment. These aspects should be looked into, including the geotechnical investigation that may be necessary.

The existing embankment side slopes will need to be properly benched as per MTO standards (OPSD 208.010) where the embankment is to be widened. The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. select subgrade materials-OPSS 1010). Fill used for construction of the widening should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. Construction should be in accordance with OPSS 206. Quality assurance should be provided as per MTO Standard 501.08 (OPSS 501).

5.6 Frost Protection

Design frost protection depth for the general area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rip-rap (rock fill), only one-half of the rock fill thickness should be assumed to be effective in providing frost protection.

5.7 Further Geotechnical Investigation

An additional investigation may be required once the details of the widening are better known. The investigation will also need to consider deeper penetration into the bedrock to further check rock quality, especially if deep foundations are to be used. As well, the embankment fill widening used for the present bridge may be different than the fill used for the original bridge. This aspect may need to be verified.

6. CLOSURE

The Limitations of Report, as quoted in Appendix H, are an integral part of this report.

For and on behalf of Coffey Geotechnics Inc.

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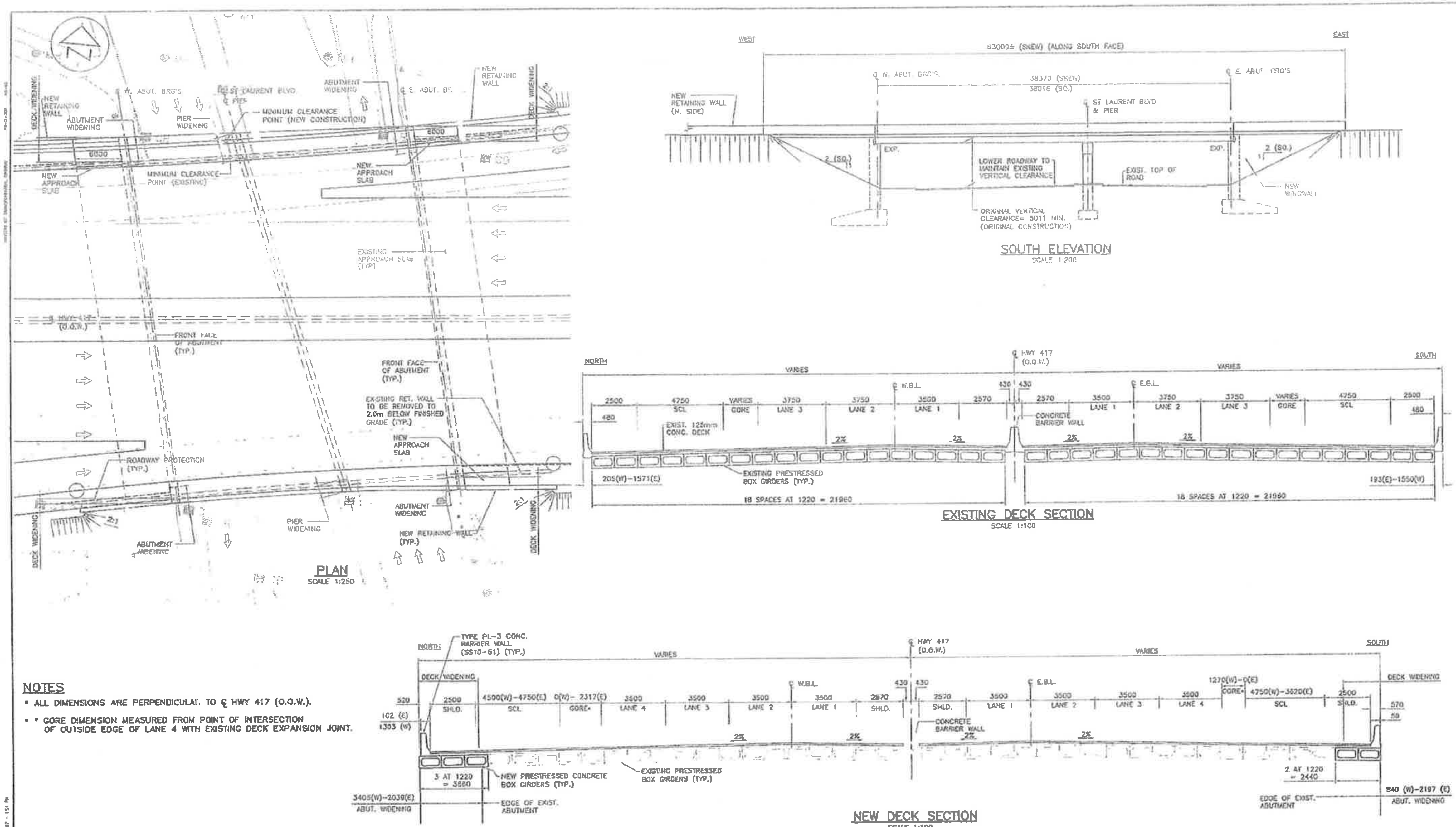
Appendix E

General Arrangement Drawings



NOTES

- ALL DIMENSIONS ARE PERPENDICULAR TO ℄ HWY 417 (O.O.W.).
- * CORE DIMENSION MEASURED FROM POINT OF INTERSECTION OF OUTSIDE EDGE OF LANE 4 WITH EXISTING DECK EXPANSION JOINT.

Ministry of Transportation Ontario / TSH engineers architects planners	SCALE AS NOTED		HIGHWAY 417, FROM HIGHWAY 416 TO ANDERSON ROAD PRELIMINARY DESIGN STUDY AND ENVIRONMENTAL ASSESSMENT G.W.P. 663-93-00	ST-LAURENT BOULEVARD OVERPASS WIDENING GENERAL ARRANGEMENT	PLATE D-15

Appendix F

Summary of Foundation Alternatives

Summary of Foundation Alternatives for Abutment Widening

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Cost effective Moderate cost May require extensive excavations. Will require shoring and dewatering. Retaining wall construction can be done at the same time	Some risk due to extensive excavation and dewatering which will be required. Monitoring during the construction may be required due to the closeness to the existing structure.	Moderate cost	Can be a preferred option, if proper dewatering and inspection of excavation can be provided.
Steel H-piles	Low displacement piles. Short pile length. Vibration monitoring is essential.	Cobbles, boulders and shale fragments may be encountered during the installation, which may present problems. Care must be taken to drive the piles into the bedrock	Moderate cost	May not be feasible due to the anticipated short pile length. Not recommended based on reliability.
Drilled and cast-in-place Concrete piles (drilled caissons)	Less vibration created than driven piles.	The presence of cobbles, boulders and shale fragments may present problems during the installation of drilled caisson foundations. Low overhead under the existing bridge as well as existing embankment fills (requiring shoring) may present problems for caisson installation. Should be discussed with a specialist contractor once the details of the widening are known.	Moderate cost	A feasible option for both bridge widening and retaining walls. Preferred option for the bridge widening from reliability point of view, provided possible low overhead conditions will not create problems during the construction.
Micropile Foundations	Minimizes vibrations and dewatering. Can be installed in low overhead conditions	Cost effectiveness is a main concern	Expensive due to special equipment / material and specialist contractor	A feasible option but more expensive than drilled caissons.

Summary of Foundation Alternatives for Central Pier Widening

Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Cost effective Moderate cost Will require extensive excavations. Will require shoring and dewatering.	Relatively high risk due to extensive excavation and dewatering. Monitoring during the construction may be required due to the closeness to the existing structures.	Moderate cost	Can be a preferred option, if proper dewatering and inspection of excavation can be provided.
Steel H-piles	Low displacement piles. Short pile length. Vibration monitoring is essential.	Cobbles, boulders and shale fragments may be encountered during the installation, which may present problems. Care must be taken to drive the piles into the bedrock	Moderate cost	Not feasible due to the anticipated short pile length. Not recommended.
Drilled and cast-in-place Concrete piles (drilled caissons)	Less vibration created than driven piles.	The presence of cobbles, boulders and shale fragments may present problems during the installation of drilled caisson foundations. Low overhead conditions under the existing bridge may present some problems for caisson installation. This aspect should be discussed once the details of the widening are known.	Moderate cost	A feasible option. Preferred option for the bridge widening from reliability point of view, provided possible low overhead conditions will not create problems during the construction.
Micropile Foundations	Minimizes vibrations and dewatering. Can be installed in low overhead conditions.	Cost effectiveness is a main concern	Expensive due to special equipment / material and specialist contractor	A feasible option but more expensive than drilled caissons.

Appendix G

List of Standard Specifications

List of Standard Specifications

OPSD

- 3101.150 WALLS, ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT
- 208.010 BENCHING OF EARTH SLOPES

OPSS

- 539 TEMPORARY PROTECTION
- 1010 MATERIAL SPECIFICATION FOR AGGREGATES - BASE, SUBBASE, SELECT SUBGRADE, AND BACKFILL MATERIAL
- 206 CONSTRUCTION SPECIFICATION FOR GRADING
- 212 CONSTRUCTION SPECIFICATION FOR. BORROW
- 501 CONSTRUCTION SPECIFICATION FOR COMPACTING

SP

- 902S01 EXCAVATION AND BACKFILLING TO STRUCTURES

Appendix H

Limitations of Report

LIMITATIONS OF REPORT

This report is intended solely for the Client named. The material in it reflects our best judgment in light of the information available to Coffey Geotechnics Inc. (Coffey) at the time of preparation. Unless otherwise agreed in writing by Coffey, it shall not be used to express or imply warranty as to the fitness of the property for a particular purpose. No portion of this report may be used as a separate entity, it is written to be read in its entirety.

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environment aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Coffey accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.