

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS
HIGHWAY 401 / BOUNDARY ROAD
BRIDGE, CITY OF CORNWALL, ONTARIO
W.P. #385-01-01, SITE # 31-215
GEOCRES NO. 31G-230**

AECOM

Project: SPT1223
May 25, 2009

**FOUNDATION INVESTIGATION REPORT
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BRIDGE, CITY OF CORNWALL, ONTARIO
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AECOM

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May 25, 2009

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AECOM
5081 Commerce Boulevard
Mississauga, Ontario
L4W 4P2

Attention: Mr. Bruce Dickey, P.Eng.

Dear Sirs:

**RE: Foundation Investigation Report, Highway 401/Boundary Road Bridge, City of Cornwall,
Ontario W.P. 385-01-01; Site # 31-215, GEOCRES No. 31G-230**

Please find attached the 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

Attachment A: Attachments

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**FOUNDATION INVESTIGATION REPORT
HIGHWAY 401/BOUNDARY ROAD BRIDGE
CITY OF CORNWALL, ONTARIO
W.P. # 385-01-01; SITE # 31-215**

1 INTRODUCTION

A new bridge which will replace the existing bridge is planned to be constructed to carry Boundary Road over Highway 401 in the City of Cornwall. Coffey Geotechnics Inc. (Coffey) was retained by AECOM to carry out a foundation investigation at the site of the proposed bridge (MTO Site Number #31-215).

The existing Bridge is a four-span bridge with a total length of about 77 m. The new bridge will be located about 2 m (clear distance) from the existing bridge, as shown on Drawing No. 1. It is our understanding that the existing bridge will be replaced by the proposed bridge with a similar length at the site and the existing bridge will retain two lanes traffic during the construction of the new bridge.

The purpose of the investigation was to obtain information about the subsurface conditions at the site by means of boreholes, and to determine the engineering characteristics of the subsurface soils by means of field and laboratory tests.

The findings of the investigation are presented in this report.

2 SITE DESCRIPTION AND GEOLOGY

The Project site is located at the intersection of Boundary Road with Highway 401 in the City of Cornwall.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, this project site is located within the Physiographic Region known as the Glengarry Till Plain.

The till has a medium texture and contains a high proportion of limestone mixed with materials derived from the Precambrian rocks to sandstone of the Nepean Formation. The outstanding characteristic of the region is stoniness. The till itself is very stony, and on the crests of the ridges and drumlins, which suffered wave action in the Champlain Sea, there are boulder pavements. The action of the wave also built numerous bars of sand and gravel deposits. There are, also, large undrained depressions in which peat and mucks are found.

According to Southern Ontario Geological Highway Map (Map 2418), the bedrock underlying this area consists of Middle Ordovician limestone and shale. A subsurface profile prepared for the construction of the existing bridge in 1961 show the presence of about 7 m overburden consisting of glacial till and silty fine sand with gravel, cobbles and boulders. The bedrock, which was contacted at about El. 50 m, is described as 'laminated limestone'. The existing approach embankments, which are approximately 5.0 to 5.5 m high, do not exhibit any apparent signs of slope instability or excessive erosion. As well, in the immediate vicinity of the existing bridge, there are no signs of excessive settlements/unusual cracking or deformations in the pavement.

3 INVESTIGATION PROCEDURES

The fieldwork for the proposed bridge was performed during the period of November 5, 2008 through November 20, 2008. As agreed with MTO, the fieldwork consisted of drilling and sampling of eight boreholes (Boreholes F1 through F8). The following table summarizes the borehole locations and drilling depths and the borehole locations are shown on Drawing No. 1.

Table 3.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Depth of Borehole Below Existing Ground Surface (m)	Piezometer
F1	10+054 (Approach-south)	8.4	No
F2	10+042 (Abutment-south)	10.5	Yes
F3	10+041 (Abutment-south)	16.0	No
F4	9+998 (Pier-centre)	9.9	No
F5	9+999 (Pier-centre)	10.0	No
F6	9+963 (Abutment-north)	10.2	Yes
F7	9+950 (Abutment-north)	16.8	No
F8	9+945 (Approach-north)	7.5	No

Marathon Drilling of Ottawa, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer from Coffey. The Boreholes were advanced using truck/track mounted drilling rigs, outfitted with tools and equipment for soil sampling and testing. The boreholes were advanced using three different methods (i.e. continuous flight hollow-stem augers, wash boring in the overburden and rock coring) depending on the ground conditions.

Samples in the boreholes were taken at frequent intervals of depth by the Standard Penetration Test method (SPT), in general accordance with ASTM D1586. This test consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm O.D. split barrel (SS – split-spoon) sampler into the ground. The number of blows of the hammer required to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the N-value of the soil which is indicative of the compactness condition of cohesionless granular soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in each of Boreholes F4 and F6 to enable groundwater level monitoring in the boreholes over a prolonged period of time without interference from surface water. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were established in the field by Coffey engineering staff, in relation to the existing features. The locations were then tied in and the geodetic elevations of the ground at the borehole locations were determined by the client's surveyors. This survey information was provided to us.

The soil and rock samples were transported to our geotechnical laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content,

grain size analyses, and Atterberg Limits tests, was performed on selected representative soil samples and unconfined compression tests was performed on selected rock cores. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4 SUBSURFACE CONDITIONS

The sub-surface conditions were explored at eight (8) boreholes (see Table 3.1 in Section 3) for this project. The plan locations of the boreholes are shown on Drawing No. 1 while a stratigraphic section and profiles are presented on Drawing Nos 2 and 3. Details of sub-surface conditions encountered at each borehole location for the investigation, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are enclosed in Appendix B.

Boreholes F3 and F7 were put down from the Boundary Road surface level from El. 63.3 and 63.0 m, respectively, while the remaining boreholes were drilled from the Highway 401 level or adjacent to it, from elevations ranging from 59.5 to 56.4 m.

Beneath various fill materials (including embankment fills) and/or a veneer of topsoil, in general, the boreholes show the presence of between 0.6 and 2.9 m thick surficial native soils which consist of clayey silt, sandy silt and gravelly sand, which are underlain by sandy silt to silty sand till. These overburden materials are in turn underlain by argillaceous limestone bedrock at about 6 to 8 m below the Highway 401 level, or at El. 49.4 to 50.8 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.

4.1 Asphalt

Boreholes F3 and F7, which were drilled from the surface of the existing Boundary Road, contacted 150 mm thick asphaltic concrete.

4.2 Topsoil

A 0.1 to 0.2 m thick surficial topsoil layer was contacted in Boreholes F1, F2, F4, F5, F6 and F8. As well, Borehole F1 contacted at 1.5 m depth (El. 56.8 m), another 1.2 m thick topsoil rich soil layer which is likely a possible fill material.

4.3 Fill

4.3.1 Pavement and Embankment Fill

Boreholes F3 and F7 drilled from the top of the Boundary Road embankment, contacted an about 2.5 m thick granular pavement fill (sand and gravel with some silt) followed by embankment fill which ranges from sandy silt to clayey silt at a depth of 2.6 m. These fill materials extend to a depth of 6.9 m (El. 56.4 m in Borehole F3 and 56.1 m in Borehole F7).

The grain-size distribution of two samples from the 2.5 m thick pavement fill is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel:	39-46%
Sand:	40-48%
Silt & Clay:	13-14%

The pavement fill is a granular (i.e. non cohesive) soil. Standard Penetration tests performed in granular pavement fill yielded N-values of 24 to 63 blows/0.3 m. These results indicate that the relative density of the pavement fill can be described as compact to very dense but typically compact.

The grain-size distribution of two samples from the embankment fill, underlying the granular pavement fill, is given in Figure B-2, in Appendix B. This indicates the following grain-size distribution.

Gravel:	11-20%
Sand:	36-37%
Silt:	33-37%
Clay:	11-15%

With the exception of the clays zones in Borehole F7, the embankment fill is considered to be a granular (i.e. non-cohesive) material.

Standard Penetration tests performed in the basically granular zones of the embankment yielded N-values of 8 to 75 blows/0.3 m. These results indicate that the relative density of the granular zones of the fill can be described as loose to very dense, while the consistency of cohesive zones of the fill material can be described as hard (i.e. well compacted).

These results indicate that the fill material received a systematic compaction when the fill was first placed, except for 1.0 m thick zone at a depth of about 2.5 m in Borehole F7.

4.3.2 Clayey Silt to Silty Sand Fill

Boreholes F1, F2, F4, F5 and F8 contacted fill materials which extended to depths of 0.8 m (F4 and F8) to 2.3 m below the ground surface to El. 58.0 to 54.2 m. The composition of the fill encountered in the boreholes ranges from clayey silt to silty sand but typically clayey silty with some topsoil inclusion and gravel size particles. The fill can be described as a basically cohesive soil with some non-cohesive (granular) zones.

Standard Penetration tests performed in the fill materials yielded N-values of 2 to 13 blows/0.3 m indicating a very soft to stiff consistency with some loose to compact zones.

The grain size characteristics of a sample from Borehole F-5 was determined in the laboratory which showed 33% gravel, 33% sand, 22% silt and 12% clay size particles (see figure B-3 in Appendix B)

4.4 Silt to Clayey Silt

Underlying the topsoil in Boreholes F6 and F8 and the fill in Boreholes F1, F3 and F7, a silt to clayey silt layer was contacted. The thickness of this deposit was found to range from 0.7 to 2.2 m and this unit extended to El. 55.7 to 54.5 m. The presence of organics was noted in the material.

The grain-size distribution of one sample from the deposit (from Borehole F6) is given in Figure B-4. The result of the test shows the following grain-size distribution:

Gravel:	5%
Sand:	14%
Silt:	42%
Clay:	39%

The Atterberg limits test performed on a sample from Borehole F6 is given in Figure B-5 in Appendix B. The test yielded the following index values:

Liquid Limit:	58%
Plastic Limit:	39%
Plasticity Index:	19

These results indicate an MH to OH material. OH designation reflects the presence of organics in the deposit. This deposit is a basically cohesive material and the silt is considered plastic.

In Borehole F1, F6 and F8 the recorded N-values range from 2 to 13 blows/0.3 m, which indicate a very soft to stiff consistency. In Boreholes F3 and F7, which was drilled from the top of the Boundary Road embankment the recorded N-values are 32 and 17 blows/0.3 m, respectively, which indicate a very stiff to hard consistency, probably due to the gain in strength under the weight of the embankment fill.

4.5 Silty Sand to Sandy Silt

Boreholes F4 and F6 contacted silty sand to sandy silt below the fill (in Borehole F4) and silt to clayey silt (in Borehole F6) at depths of 0.8 m (El. 55.6 m) and 2.3 m (El. 54.6 m), respectively. The thickness of the deposit was found to be 1.3 m in Borehole F4 and 0.8 m in Borehole F6. This is a typically fine-grained granular soil with some gravel and clay content.

The grain-size distribution of a sample from this granular deposit (from Borehole F4) is given in Figure B-6. As shown, the following grain-size distribution is indicated:

Gravel:	18%
Sand:	38%
Silt:	33%
Clay:	11%

N-values recorded in this deposit range from 1 blow/0.3 m (Borehole F6) to 11 blows/0.3 m (Borehole F4) which indicate very loose (Borehole F6) to compact (Borehole F4) condition.

4.6 Gravelly Sand to Sandy Gravel

Near the centre pier location, Boreholes F4 and F5 encountered a 0.6 to 1.6 m thick gravelly sand to sandy gravel layer at El. 54.3 and 52.7 m, respectively, below the surficial silty sand (Borehole F4) and the fill (Borehole F5). The deposit was found to extend to Elevations of 53.6 m to 52.7 m in Boreholes F4 and F5, respectively.

The grain-size distribution of three samples from the deposit is given in Figure B-7 which shows the following grain-size distribution:

Gravel:	30-46%
Sand:	39-44%
Silt & Clay:	15-27%

The results indicate a relatively coarse grained granular deposit.

Standard Penetration tests performed in the deposit yielded N-values which range from 15 to 26 blows/0.3 m indicating a compact relative density.

4.7 Silty Sand to Sandy Silt Till

Underlying the native surficial soil deposit (in Boreholes F1, F3, F4, F5, F6, F7 and F8) and fill material (Borehole F2), all the boreholes contacted a glacial till deposit at depths ranging from 1.5 m to 7.6 m. The following table summarizes the top and bottom elevations of the deposit, as encountered in the boreholes.

Table 4.7.1: Depth/elevation of glacial till deposit

Borehole No.	Depth Below Ground Surface/Elevation of the Top of the Deposit(m)	Depth of Below Ground Surface/Elevation of the Bottom of the Deposit (m)
F1	3.8 / 55.7	8.4 / 51.1*
F2	1.5 / 56.8	7.5 / 50.8
F3	7.6 / 55.7	12.9 / 50.4
F4	3.7 / 52.7	6.8 / 49.6
F5	2.9 / 53.6	6.8 / 49.7
F6	3.1 / 53.8	7.1 / 49.8
F7	7.6 / 55.4	13.6 / 49.4
F8	2.3 / 54.5	7.5 / 49.3*

*Auger refusal at that depth/Elevation-bottom of the deposit was not proven.

Boreholes F1 and F8 were terminated in this deposit upon encountering refusal while the others were extended into the underlying bedrock. The deposit consists of a heterogeneous mixture of sandy silt to silty sand with traces to some clay. The presence of silt and sand interbeds/lenses was also noted. The upper portion of the deposit (Boreholes F4 and F5) was found to contain cobbles and boulders which necessitated advancing holes in this deposit by coring. However, the presence of cobbles and boulders should be expected to occur throughout in the till deposits, due to their mode of deposition.

The grain-size distribution of nine samples from this granular (non-cohesive) deposit is given in Figure B-8 in an envelop form, which show the following grain-size distribution:

Gravel:	8-39%
Sand:	21-51%
Silt:	14-54%
Clay:	7-14%

Standard Penetration tests performed in this silty sand to sandy silt till deposit gave N-values which range from 4 blows/0.3 m to in excess of 100 blows per 0.1 m indicating a compact to very dense relative density with occasional, loose to very loose zones.

4.8 Bedrock

Grey to dark grey argillaceous limestone bedrock was encountered in Boreholes F2, F3, F4, F5, F6 and F7 and was proven by NQ coring as follow:

Table 4.8.1: Bedrock elevation and condition

Borehole No.	Ground Surface Elevation (m)	Depth Below Ground Surface/Elevation of the Bedrock Surface (m)	T.C.R. (%)*	R.Q.D. (%)**
F2	58.3	7.5 / 50.8	100	87-100
F3	63.3	12.9 / 50.4	91-100	42-100
F4	56.4	6.8 / 49.6	100	42-87
F5	56.5	6.8 / 49.7	100	38-100
F6	56.9	7.1 / 49.8	100	69-79
F7	63.0	13.6 / 49.4	78-100	18-52

* T.C.R.=Total Core Recovery

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

The Boreholes were advanced into the bedrock for a vertical distance of about 3.0 m by NQ coring. The percentage of recovery was 78 to 100% while the RQD values vary from 18% to 100%. These results indicate a various rock quality from very poor to excellent. In general, these results indicate that bedrock on south side of the bridge (Boreholes F2 and F3) is sound above El. 48.5 m but on north side of the bridge (Boreholes F6 and F7) the bedrock appears to be fractured. At the centre pier location at Boreholes F4 and F5, the bedrock appears to have more fractures in the top and bottom of the core section (i.e. 3.0 m) than the middle portion of the core. Unconfined compression tests were performed on selected rock samples near to the bedrock surface and the tests yielded unconfined compressive strength of between about 100 MPa (Borehole F5) and 170 MPa (Borehole F2). These results indicate that the rock can be classified as being generally very strong. The bedrock appeared to be weathered in the upper 1± m zone, particularly in Borehole F7.

At the borehole locations the surface of the bedrock was contacted at Elevations ranging from 50.8 m (Borehole F2) to 49.4 m (Borehole F7). From these results and from the regional geology the surface of the bedrock appears to be relatively flat in this project area.

4.9 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. In the deep boreholes, where NQ coring and wash boring were used (i.e. water introduced into the boreholes) the on-completion water levels may not be reliable. 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 4.9.1: Groundwater condition

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers
F1	59.5		1.9*/57.6	Nov/18/08	
F2	58.3	10.5/47.8	1.2/57.1	Dec/12/08	Yes
F3	63.3		3.7*/59.6	Nov/06/08	
F4	56.4		0.3*/56.1	Nov/20/08	
F5	56.5		0.0*/56.5	Nov/20/08	
F6	56.9	10.2/46.7	0.7/56.2	Dec/12/08	Yes
F7	63.0		3.8*/59.2	Nov/05/08	
F8	56.8		1.0*/55.8	Nov/13/08	

*groundwater table not stabilized

The piezometers were installed in the bedrock, because in our experience, sometimes an upward gradient occurs emanating from within the bedrock. From the measured values, it is our opinion that groundwater level below original grade at the time of investigation was at about El 56 to 57 m. While a perched water condition could possibly occur at the site due to the accumulation of the surface water in the fill materials, the measured water levels at the site do not represent a perched condition.

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

For and on behalf of Coffey Geotechnics Inc.



Gwangha Roh, Ph.D.



Ramon Miranda, P.Eng.



Zuhtu Ozden, P.Eng.



Drawings

NOTES:
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
ARE IN KILOMETRES + METRES.
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.



KEY PLAN
N.T.S.

LEGEND

Borehole

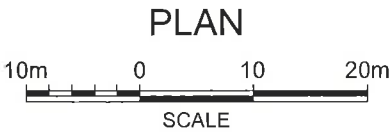
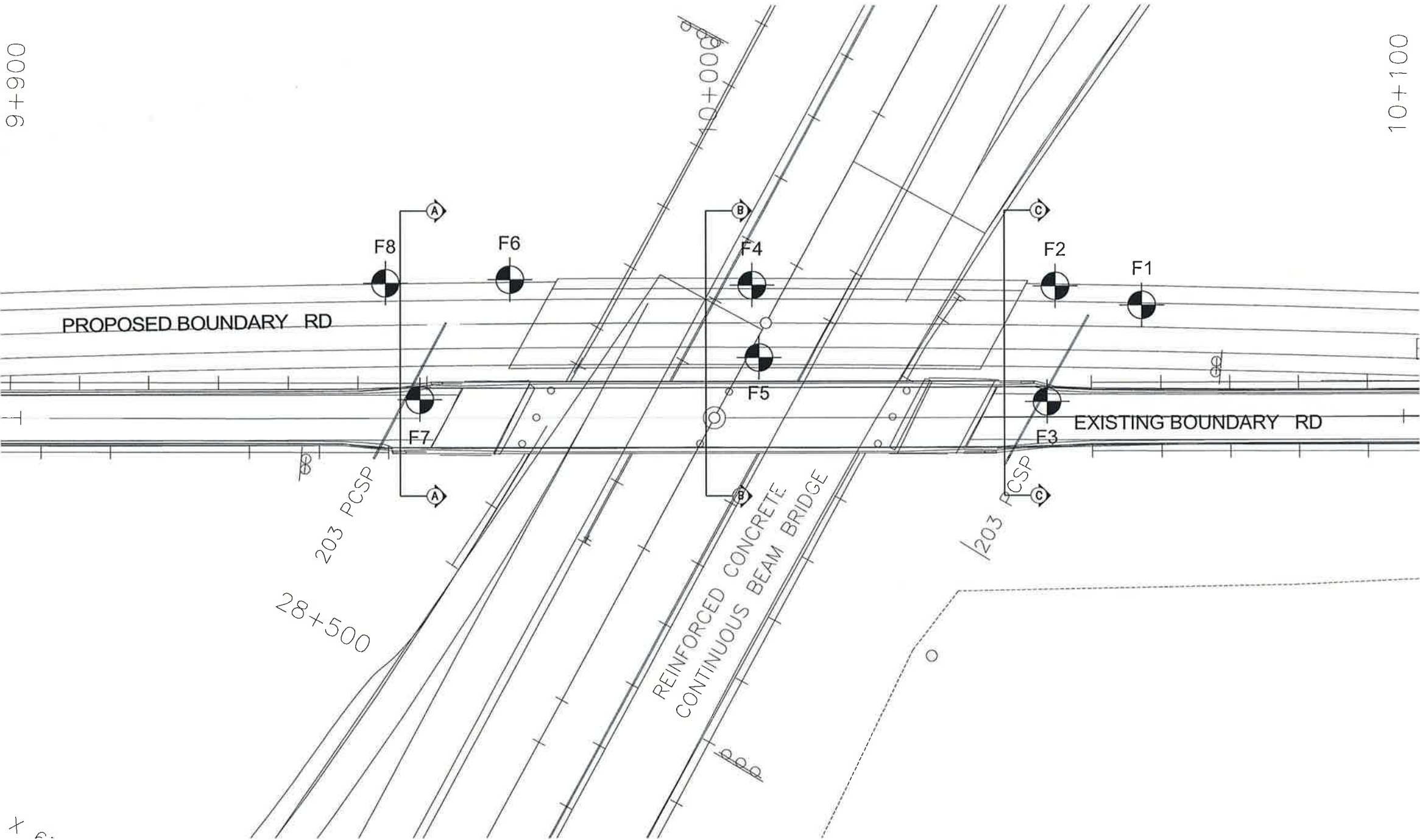
No.	ELEV.	STATION No.	OFFSET
F1	59.5	10+054.5	3.2m Lt C/L
F2	58.3	10+042	5.5m Lt C/L
F3	63.3	10+041	11.0m Rt C/L
F4	56.4	9+998	5.5m Lt C/L
F5	56.5	9+999	5.0m Rt C/L
F6	56.9	9+963	6.5m Lt C/L
F7	63.0	9+950	10.9m Rt C/L
F8	56.8	9+945	6.0m Lt C/L

NOTE

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REV.	DATE	BY	DESCRIPTION
Geocres No. 31G-230			
SPT 1223			DIST
SUBMD	CHECKED	DATE Jan. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1



NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION No.	OFFSET
F1	59.5	10+054.4	3.2m Lt C/L
F2	58.3	10+042	5.5m Lt C/L
F4	56.4	9+998	5.5m Lt C/L
F6	56.9	9+963	6.5m Lt C/L
F8	56.8	9+945	6.0m Lt C/L

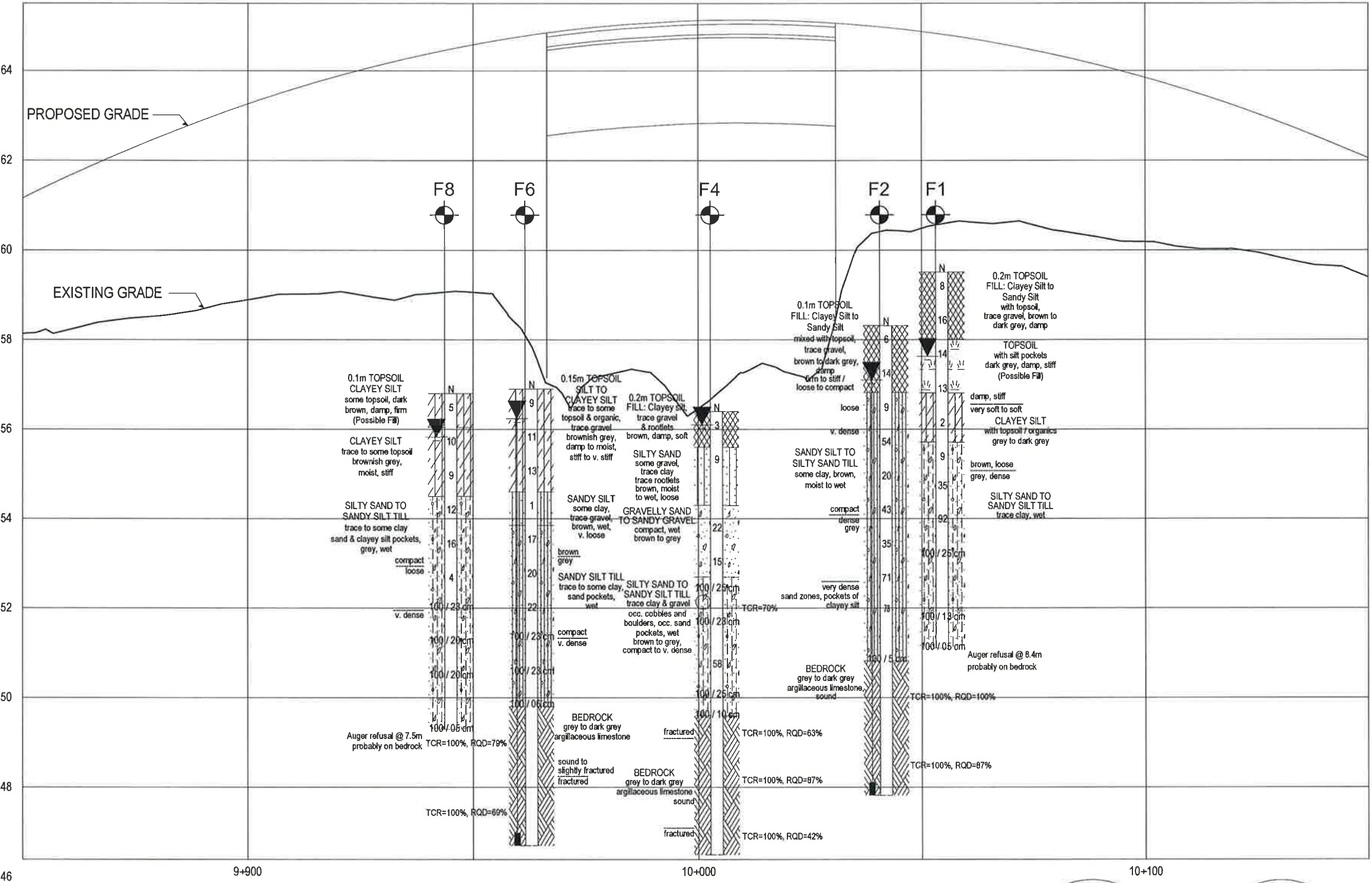
NOTE

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NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REV.	DATE	BY	DESCRIPTION
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Geocres No. 31G-230			
SPT 1223			DIST
SUBMD	CHECKED	DATE Jan. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2



NOTES:
FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation (W. L. NOT STABILIZED)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	STATION No.	OFFSET
F2	58.3	10+042	5.5m Lt C/L
F3	63.3	10+041	11.0m Rt C/L
F4	56.4	9+998	5.5m Lt C/L
F5	56.5	9+999	5.0m Rt C/L
F6	56.9	9+963	6.5m Lt C/L
F7	63.0	9+950	10.9m Rt C/L

NOTE

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

NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REV.	DATE	BY	DESCRIPTION
Geocres No. 31G-230			
SPT 1223			DIST
SUBMD	CHECKED	DATE Jan. 2009	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 3

SECTION A-A

SECTION B-B

SECTION C-C

SECTIONS



Appendix A

Record of Borehole Sheets

SPT 1223

RECORD OF BOREHOLE No F1

1 OF 1

METRIC

GWP 385-01-01 LOCATION Sta: 10+054.5; 3.2 m Lt. C/L of Boundary Road (D = -0.6m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 11/18/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI C
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE							
59.5 0.0	GROUND SURFACE						20	40	60	80	100	10	20	30	
	0.2 m TOPSOIL		1	SS	8										
	FILL: Clayey Silt to Sandy Silt with topsoil trace gravel, brown to dark grey, damp		2	SS	16										
58.0 1.5	TOPSOIL with silt pockets dark grey, damp, stiff (Possible Fill)		3	SS	14										
56.8 2.7	damp, stiff		4	SS	13										
	very soft to soft		5	SS	2										
55.7 3.8	CLAYEY SILT with topsoil / organics grey to dark grey		6	SS	9										spoon wet
	brown, loose		7	SS	35										8 48 33 11
	grey, dense		8	SS	92										
	SILTY SAND TO SANDY SILT TILL trace clay, wet		9	SS	100 / 25 cm										
			10	SS	100 / 13 cm										spoon bouncing
			11	SS	100 / 05 cm										
51.1 8.4	End of Borehole Auger refusal @ 8.4 m probably on bedrock Water level @ 1.9 m upon completion (not stabilized)* Hole caved in @ 3.8 m upon completion														

+³, x³: Numbers refer to Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F2

1 OF 1

METRIC

GWP 385-01-01 LOCATION Sta: 10+042; 5.5 m Lt. C/L of Boundary Road (D = -1.2m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers & NW Casing & NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/17/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)				
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR. X LAB VANE		
58.3 0.0	GROUND SURFACE						20 40 60 80 100	20 40 60 80 100	10 20 30		GR SA SI CL	
56.8 1.5	0.1 m TOPSOIL FILL: Clayey Silt to Sandy Silt mixed with topsoil, trace gravel, brown to dark grey, damp firm to stiff / loose to compact		1	SS	6							
			2	SS	14							
	loose		3	SS	9							15 40 31 14
	v. dense		4	SS	54							
	SILTY SAND TO SANDY SILT TILL some clay, brown, moist to wet		5	SS	20							26 45 22 7
	compact		6	SS	43							
	dense grey		7	SS	35							
	very dense		8	SS	71							
	sand zones, pockets of clayey silt		9	SS	78							
50.8 7.5			10	SS	100 / 5 cm							auger refusal **UCS=173 MPa
	BEDROCK grey to dark grey argillaceous limestone, sound		11	RCTCR=100% RQD=100%								
			12	RCTCR=100% RQD=87%								
47.8 10.5	End of Borehole Water level @ 0.9 m upon completion (not stabilized)* Hole caved in @ 4.5 m upon completion Piezometer installed to 10.5 m water level in piezometer 1.2 m - Nov 20, 2008 1.2 m - Dec 12, 2008 **UCS=Unconfined Compressive Strength											

+ 3, X 3: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F3

1 OF 2

METRIC

GWP 385-01-01 LOCATION Sta: 10+041; 11.0 m Rt. C/L of Boundary Road (D = +1.5m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers & NW Casing&NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/6/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)				
								○ UNCONFINED	+ FIELD VANE	● POCKET PENETR. X LAB VANE		
63.3 0.0	GROUND SURFACE						20 40 60 80 100	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L		GR SA SI CL
	150 mm ASPHALT		1	SS	63							
	FILL: Sand & Gravel with trace to some silt brown, damp, very dense to compact		2	SS	38							
			3	SS	24							39 48 11 2
60.7 2.6			4	SS	22							
	FILL: Sandy Silt trace to some gravel, trace clay brown to greyish brown damp to moist		5	SS	22							
		dense	6	SS	34							11 37 37 15
		compact	7	SS	25							
			8	16	18							
		v. dense grey	9	SS	75							gravel @spoon tip
56.4 6.9	CLAYEY SILT trace to some organics / topsoil zones of sandy silt, brownish grey, moist, hard		10	SS	32							
55.7 7.6			11	SS	72							39 31 23 7 gravel @spoon tip
	SILTY SAND TO SANDY SILT TILL greyish brown, moist											
		wet compact	12	SS	29							spoon wet
		v. dense grey	13	SS 100 / 29 cm								11 21 54 14
			14	SS	73							
50.4 12.9			15	SS 100 / 10 cm								
	BEDROCK grey to dark grey argillaceous limestone, sound		16	RC TCR=91% RQD=85%								auger refusal
			17	RC TCR=100% RQD=100%								
48.3												

Continued Next Page

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F3

2 OF 2

METRIC

GWP 385-01-01 LOCATION Sta: 10+041; 11.0 m Rt. C/L of Boundary Road (D = +1.5m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers & NW Casing&NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/6/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	WATER CONTENT (%)	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100						
48.3 15.0	BEDROCK grey to dark grey argillaceous limestone, fractured		18	RCTCR=100% RQD=42%			48											GR SA SI CL
47.3 16.0	End of Borehole Water level @ 3.7 m upon completion (not stabilized)* Hole caved in @ 7.5 m upon completion																	

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F4

1 OF 1

METRIC

GWP 385-01-01 LOCATION Sta: 9+998; 5.5 m Lt. C/L of Boundary Road (D = -0.3m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers, Wash Boring & NW Casing & NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/19/2008 11/20/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)				
							20 40 60 80 100					
56.4 0.0	GROUND SURFACE											
55.6 0.8	0.2 m TOPSOIL FILL: Clayey silt, trace gravel & rootlets brown, damp, soft		1	SS	3							
54.3 2.1	SILTY SAND some gravel, trace clay, trace rootlets brown, moist to wet, loose		2	SS	9							
52.7 3.7	GRAVELLY SAND TO SANDY GRAVEL some silt, trace clay, compact, wet brown to grey		3	SS	11							
			4	SS	22							
			5	SS	15							
			6	SS100 / 25 cm								
			7	RC TCR=70%								
			8	SS100 / 23 cm								
			9	SS	58							
			10	SS100 / 25 cm								
			11	SS100 / 10 cm								
			12	RC TCR=100% RQD=63%								
			13	RC TCR=100% RQD=87%								
			14	RC TCR=100% RQD=42%								
46.5 9.9	End of borehole Water level @ 0.3 m upon completion (not stabilized)* Hole caved in @ 2.3 m upon completion											

+ 3 x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F5

1 OF 1

METRIC

GWP 385-01-01 LOCATION Sta: 9+999; 5.0 m Rt. C/L of Boundary Road (D = -0.15m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers, Wash Boring & NW Casing&NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/20/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
56.5	GROUND SURFACE							20	40	60	80	100		
0.0	0.1 m TOPSOIL FILL: Silty Sand with some organics trace wooden pieces, brownish grey, wet, loose		1	SS	6		56							
55.7														
0.8	FILL: Clayey Silt, trace organics with sand & gravel, brownish grey moist to wet, firm to soft		2	SS	6		55							
			3	SS	2									
54.2														
2.3	GRAVELLY SAND some silt, brown, wet		4	SS	26		54							
53.6														
2.9	SILTY SAND TO SANDY SILT TILL trace clay, wet		5	SS	100 / 5 cm		53							
		Boulder	6	RC	TCR=33%									
			7	SS	22									
		compact												
		very dense	8	SS	100 / 25 cm		52							
			9	SS	58		51							
			10	SS	100 / 25 cm		50							
49.7														
6.8	highly fractured		11	RC	TCR=100% RQD=38%		49							
	BEDROCK grey to dark grey argillaceous limestone		12	RC	TCR=100% RQD=100%		48							
	sound													
	fractured		13	RC	TCR=100% RQD=62%		47							
46.5														
10.0	End of borehole Water level @ surface upon completion (not stabilized)* Hole caved in @ 2.0 m upon completion **UCS=Unconfined Compressive Strength													

+³ X³: Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F6

1 OF 1

METRIC

GWP 385-01-01 LOCATION Sta: 9+963; 6.5 m Lt. C/L of Boundary Road (D = -0.2m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers & NW Casing&NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/13/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
56.9	GROUND SURFACE						20	40	60	80	100				
0.0	0.15 m TOPSOIL		1	SS	9										
	SILT TO CLAYEY SILT trace to some topsoil and organic, trace gravel brownish grey, damp to moist, stiff to v. stiff		2	SS	11										
			3	SS	13										
54.6															
2.3	SANDY SILT some clay, trace gravel brown, wet, very loose		4	SS	1										
53.8															
3.1			5	SS	17										
	SILTY SAND TO SANDY SILT TILL trace to some clay, sand pockets wet		6	SS	20										
			7	SS	22										
			8	SS100 / 23											
			9	SS100 / 23											
49.8			10	SS 100 / 6-4 m											
7.1			11	RCTCR=100% RQD=79%											
	BEDROCK grey to dark grey argillaceous limestone		12	RCTCR=100% RQD=69%											
46.7															
10.2	End of borehole Water level @ 0.9 m upon completion (not stabilized)* Hole caved in @ 4.6 m upon completion Piezometer installed to 10.2 m Water level in piezometer 0.7 m - Nov 20, 2008 0.7 m - Dec 12, 2008														

+ 3 X 3: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F7

1 OF 2

METRIC

GWP 385-01-01 LOCATION Sta: 9+950: 10.9 m Rt. C/L of Boundary Road (D = +1.5m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers & NW Casing & NQ coring COMPILED BY SS
DATUM Geodetic DATE 11/5/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE					WATER CONTENT (%) W _p W W _L																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
63.0 0.0	GROUND SURFACE						20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1223

2 OF 2

METRIC

GWP	385-01-01	LOCATION	Sta: 9+950; 10.9 m RI. C/L of Boundary Road (D = +1.5m)	ORIGINATED BY	SK
DIST	HWY Hwy 401	BOREHOLE TYPE	Hollow Stem Augers & NW Casing&NQ coring	COMPILED BY	SS
DATUM	Geodetic	DATE	11/5/2008	CHECKED BY	ZSO

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

SPT 1223

RECORD OF BOREHOLE No F8

1 OF 1

METRIC

GWP 385-01-01 LOCATION Sta: 9+945; 6.0 m Lt. C/L of Boundary Road (D = -1.5m) ORIGINATED BY SK
DIST HWY Hwy 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SS
DATUM Geodetic DATE 11/13/2008 11/14/2008 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
56.8	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100					
0.0	0.1 m TOPSOIL CLAYEY SILT some topsoil, dark brown, damp, firm (possible fill)		1	SS	5									
56.0			2	SS	10									
0.8	CLAYEY SILT trace to some topsoil brownish grey, moist, stiff		3	SS	9									
54.5			4	SS	12									
2.3	SILTY SAND TO SANDY SILT TILL trace to some clay sand & clayey silt pockets, grey, wet		5	SS	16									
	compact		6	SS	4									
	loose		7	SS100 / 23 cm										
	v. dense		8	SS100 / 20 cm										
			9	SS100 / 20 cm										
49.3			10	SS100 / 05 cm										
7.5	End of Borehole Auger refusal @ 7.5 m probably on bedrock Water level @ 1.0 m upon completion (not stabilized)* Hole caved in @ 4.5 m upon completion													

+³, X³: Numbers refer to
Sensitivity

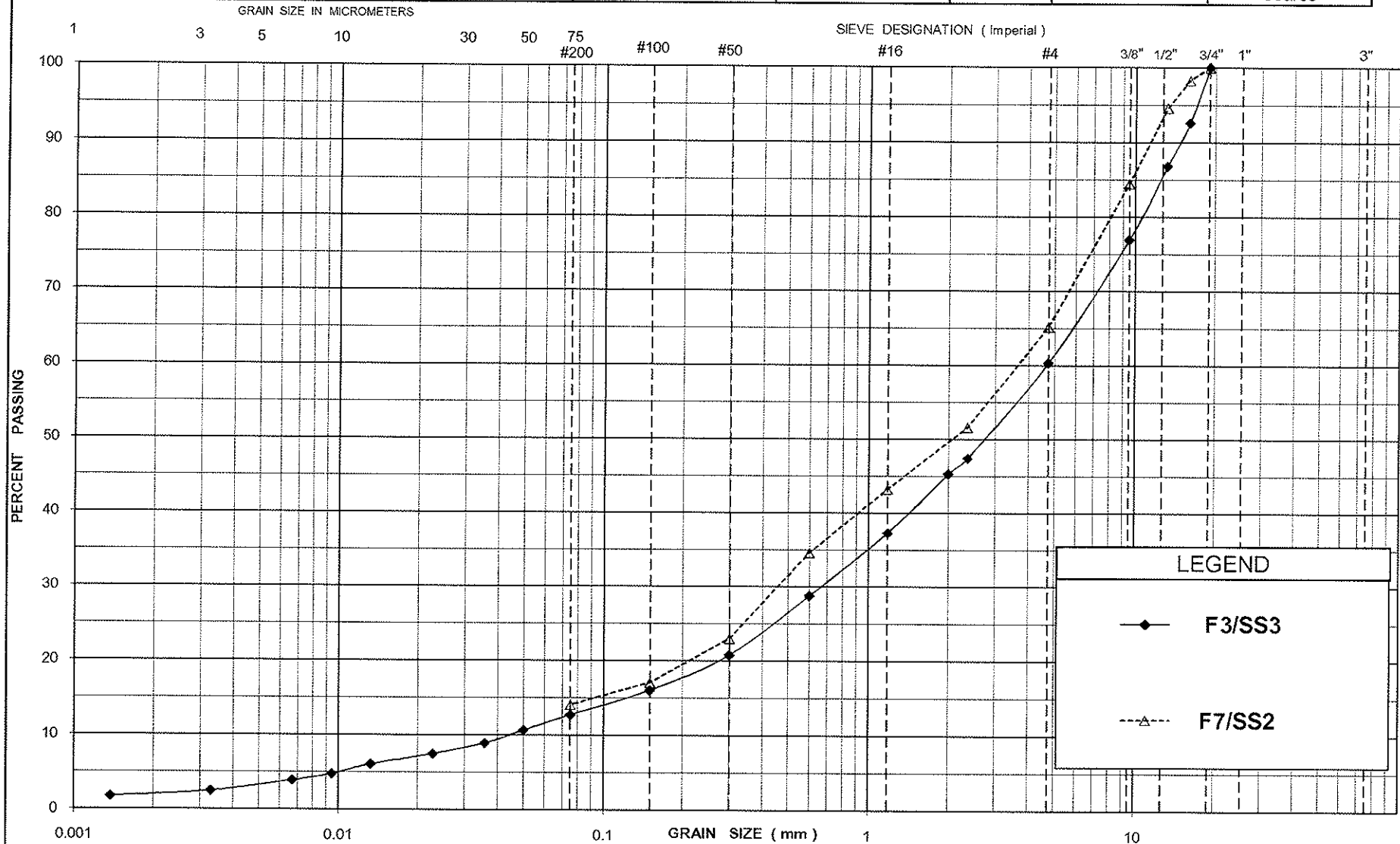
20
15 5
10 (%) STRAIN AT FAILURE

Appendix B

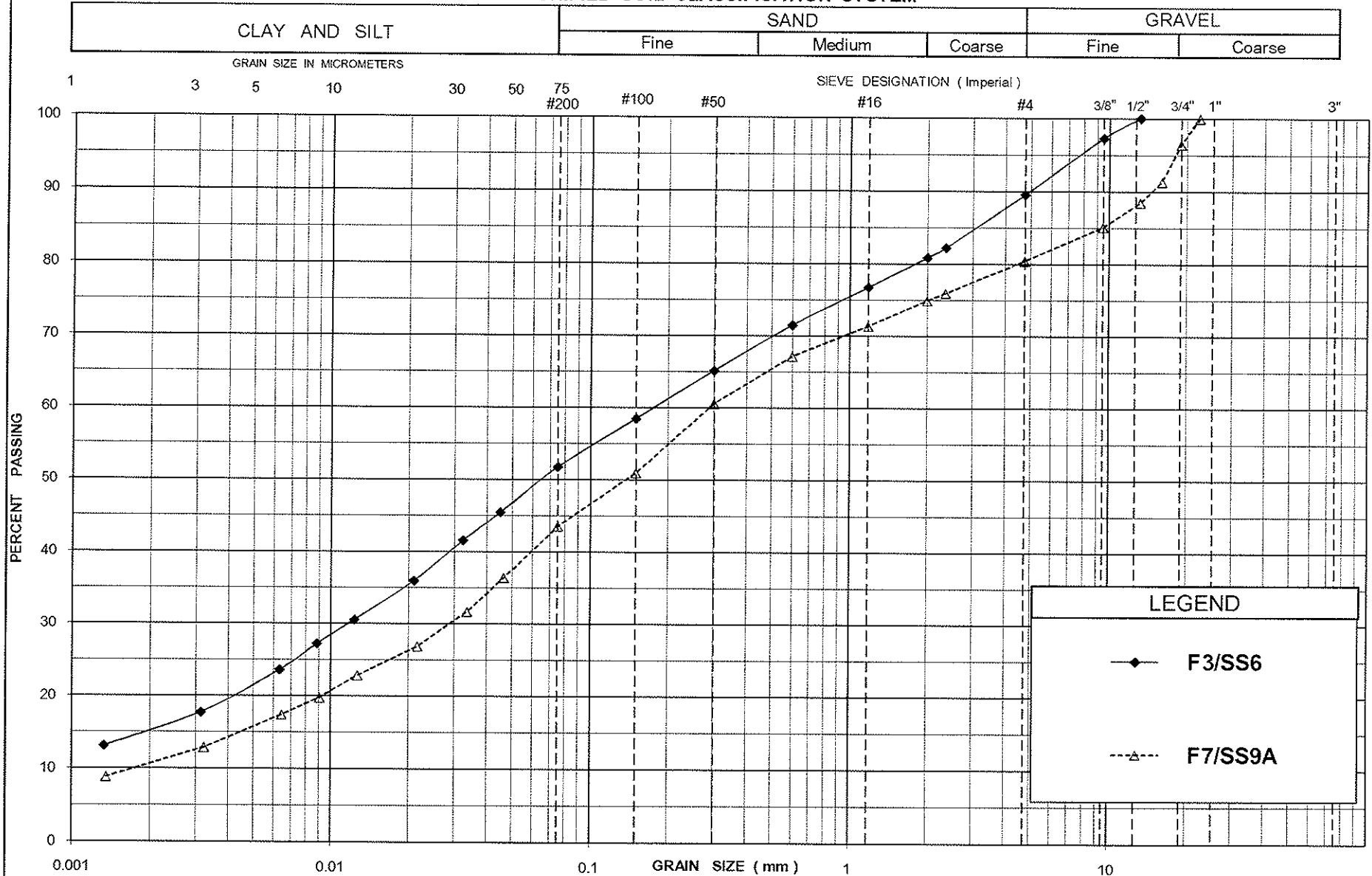
Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

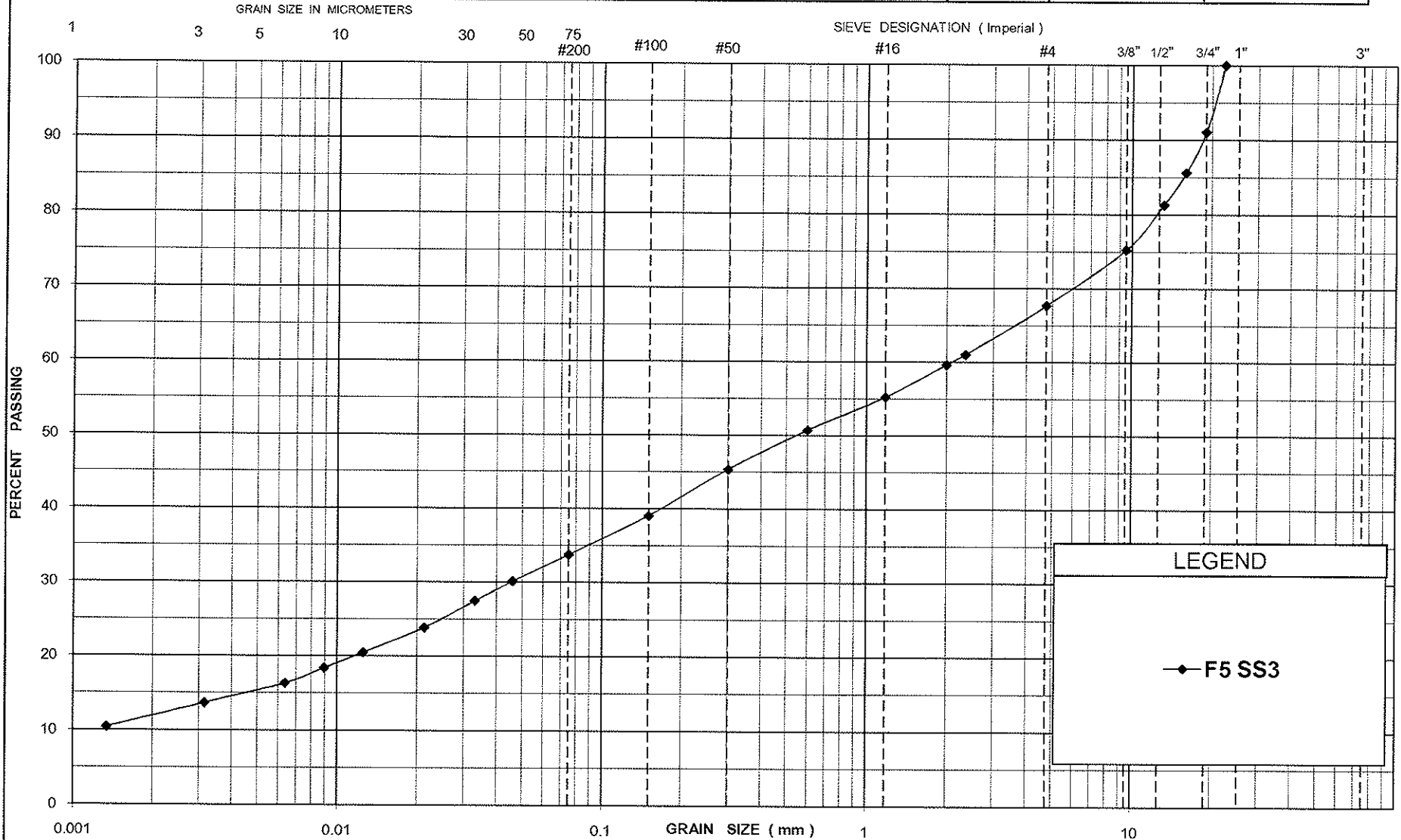


UNIFIED SOIL CLASSIFICATION SYSTEM

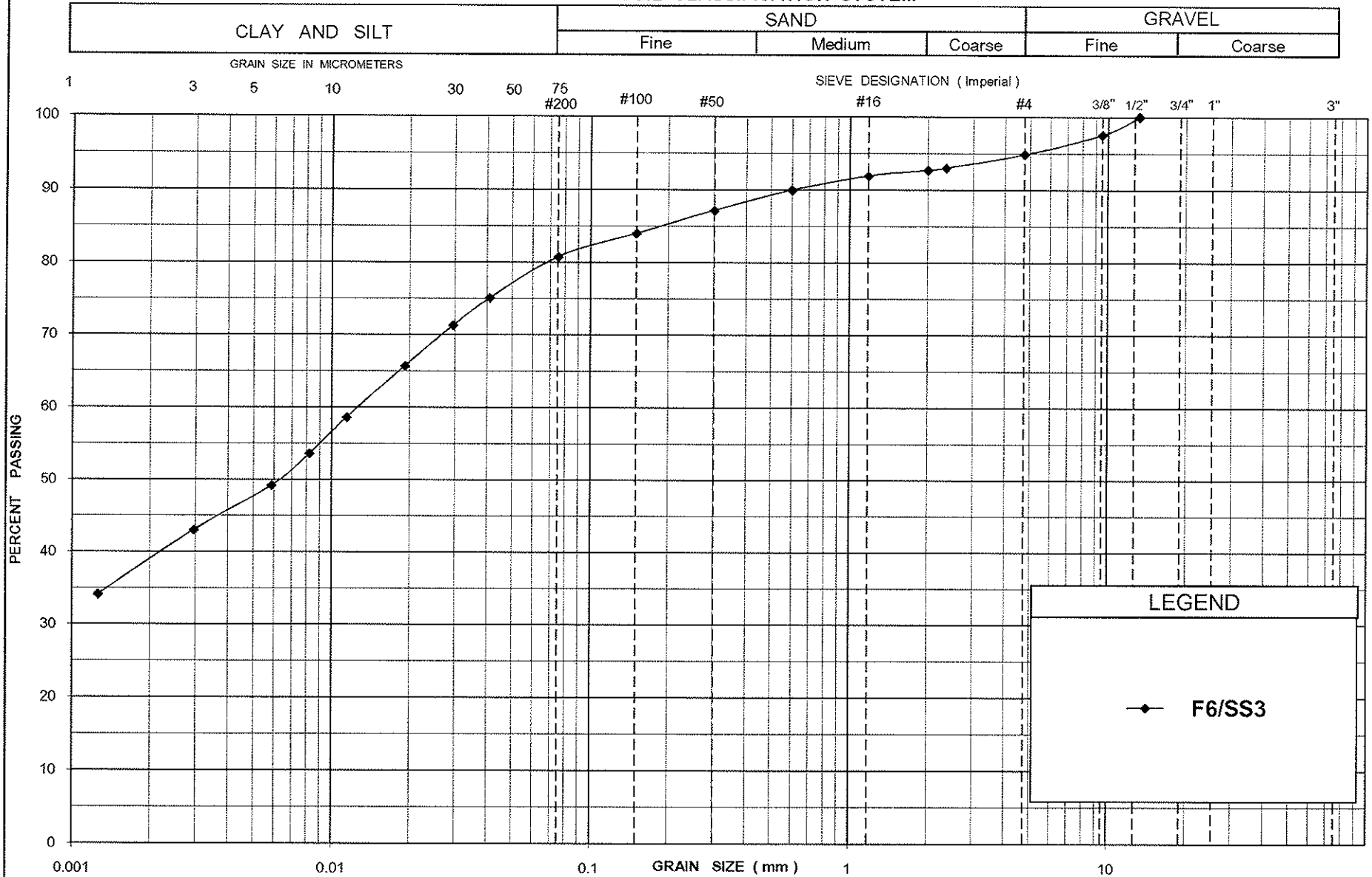


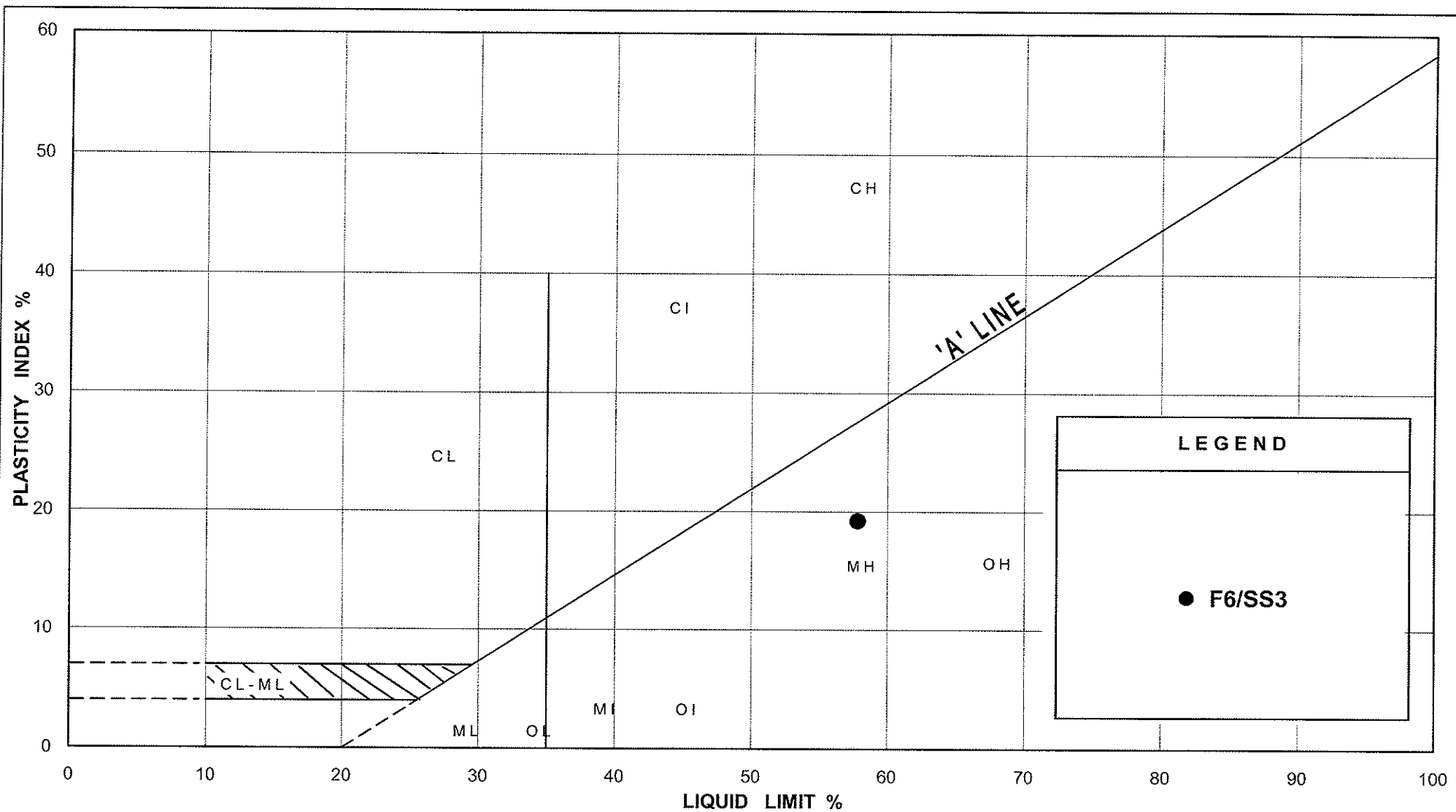
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



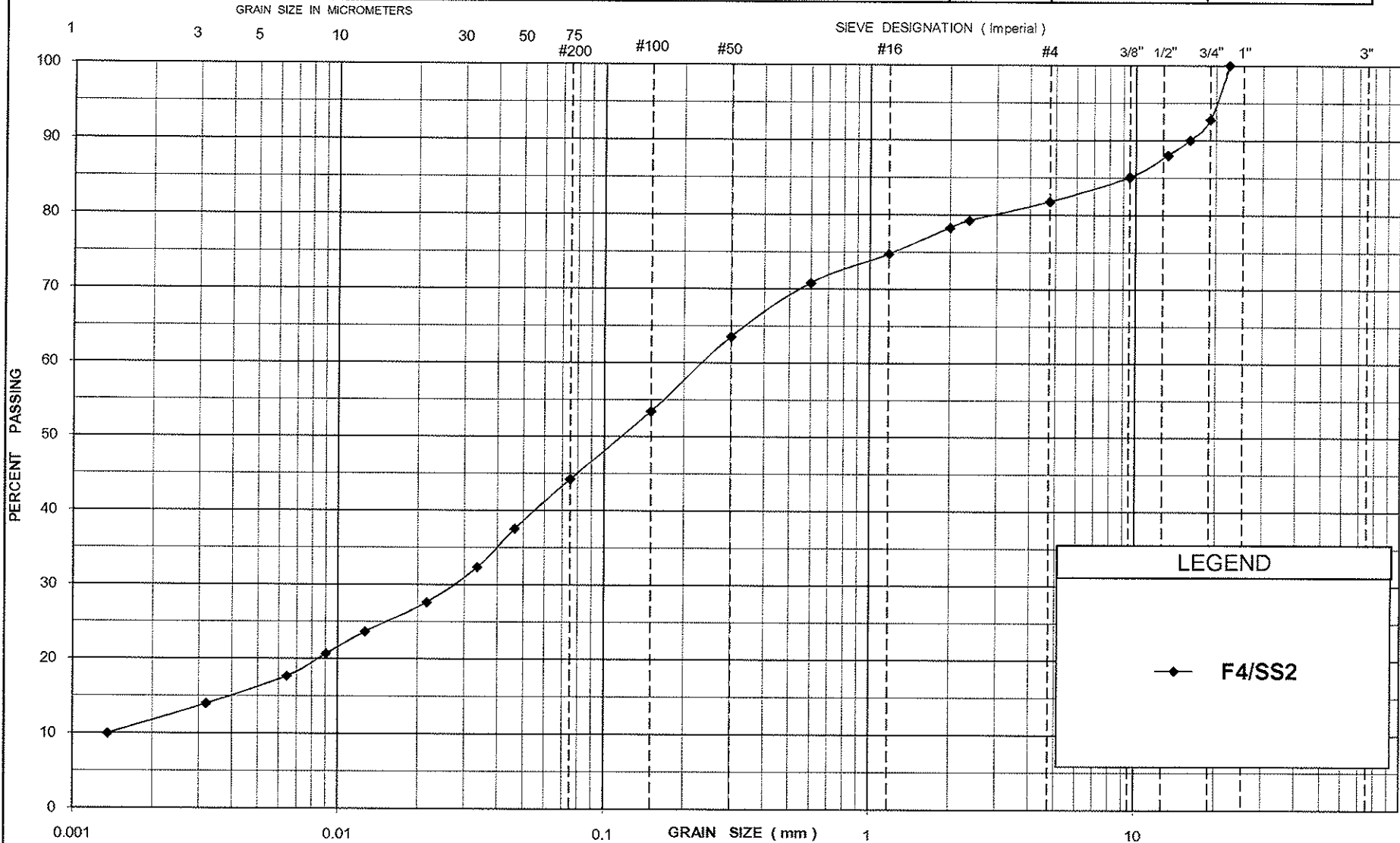
UNIFIED SOIL CLASSIFICATION SYSTEM



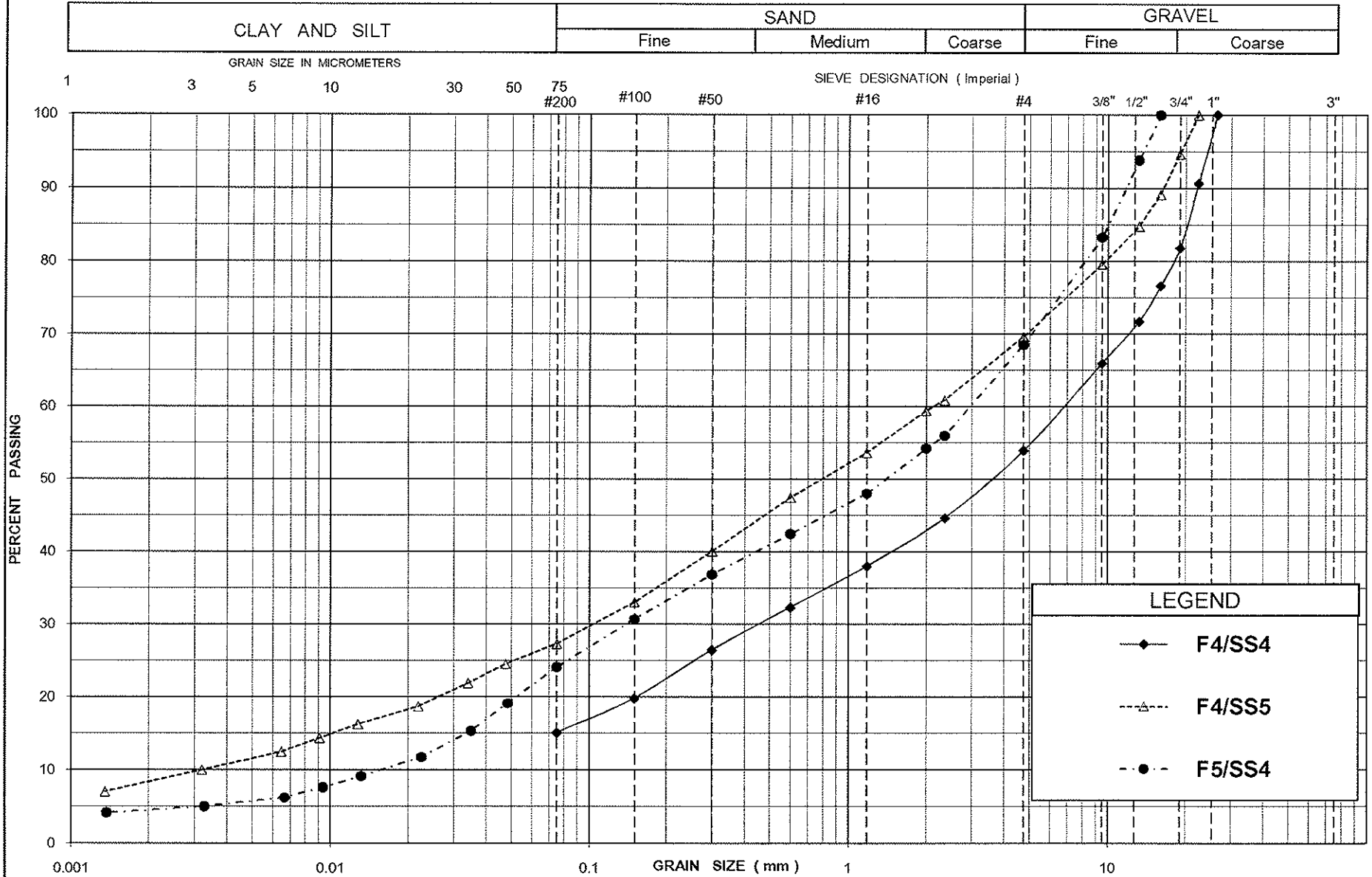


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL	
			Fine	Medium	Coarse	Fine	Coarse

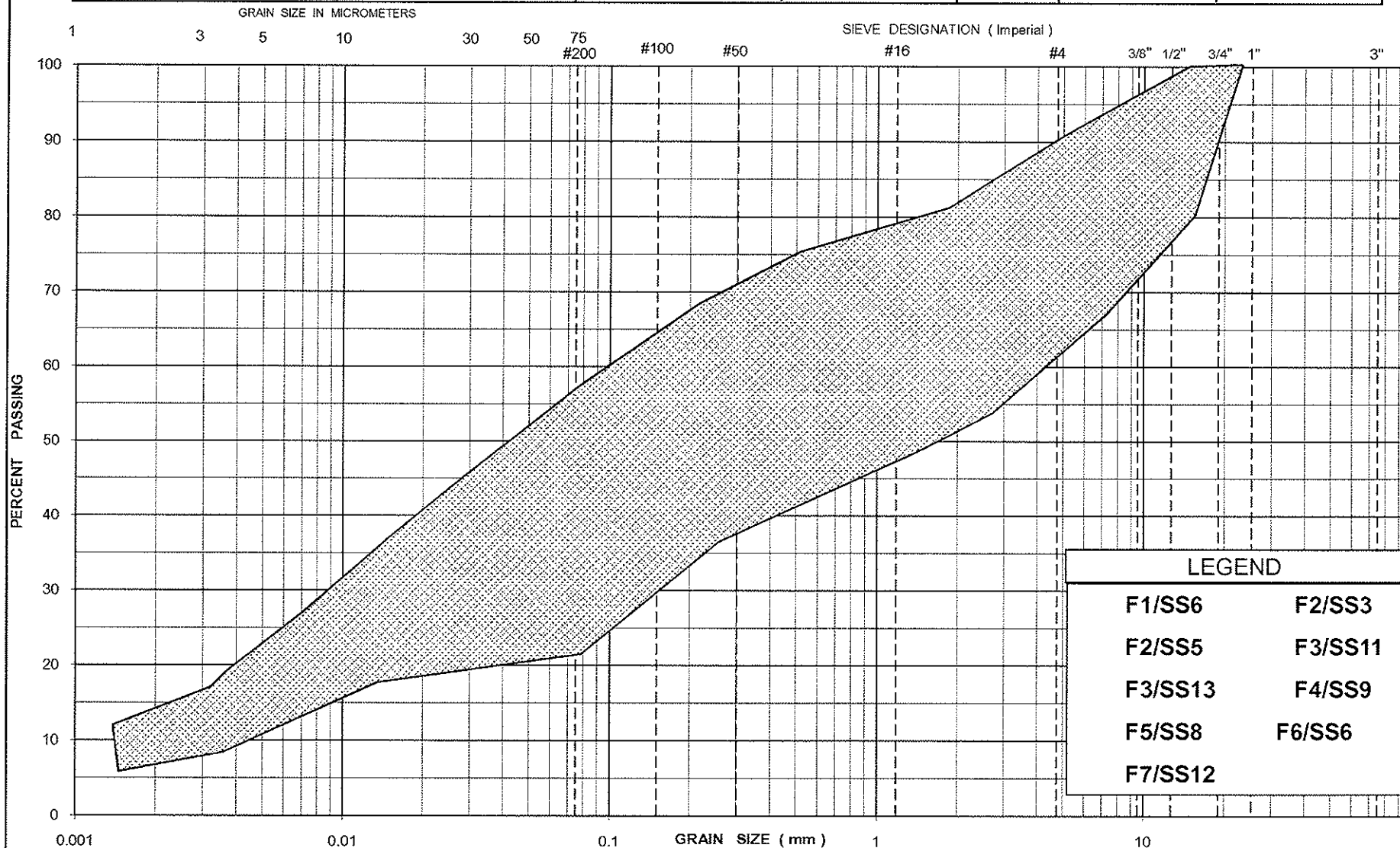


UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL	
			Fine	Medium	Coarse	Fine	Coarse



Appendix C

Site Photographs



Photograph 1. Existing Boundary Road Bridge over Highway 401 (looking east)



Photograph 2. Boreholes F1 and F2



Photograph 3. Borehole F3



Photograph 4. Borehole F4 (centre pier location)



Photograph 5. Centre pier area



Photograph 6. Boreholes F6 and F8



Photograph 7. Borehole F7



Photograph 8. Existing Boundary Road Bridge over Highway 401(looking south)

Appendix D

Rock Core Photographs

Top

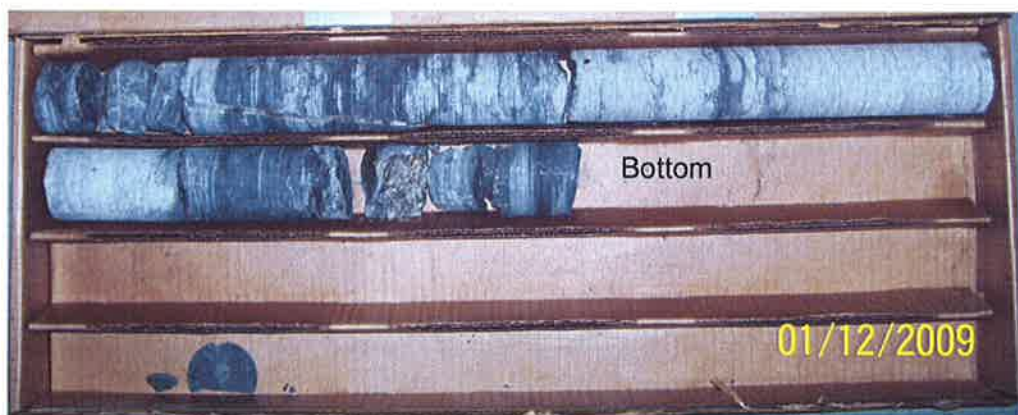


Borehole F-2

Top



Bottom

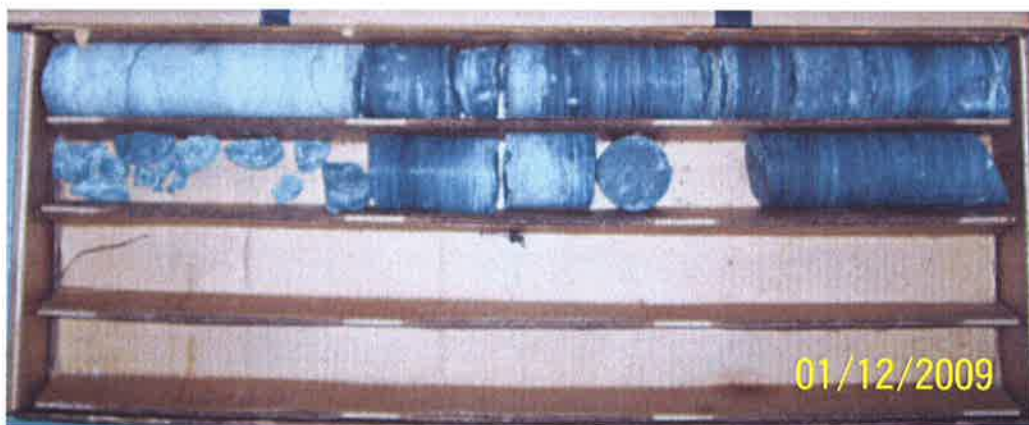


Borehole F-3

Top



Bottom



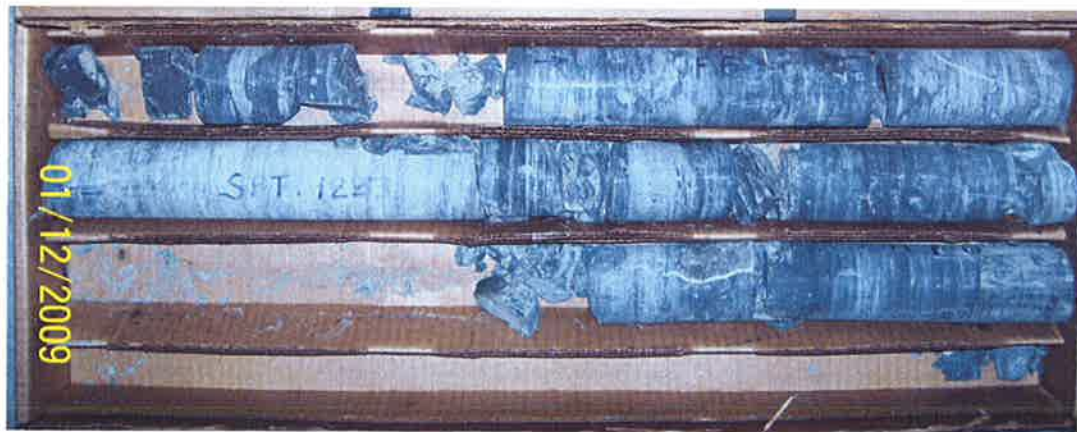
Borehole F-4

Top



Borehole F-5

Top



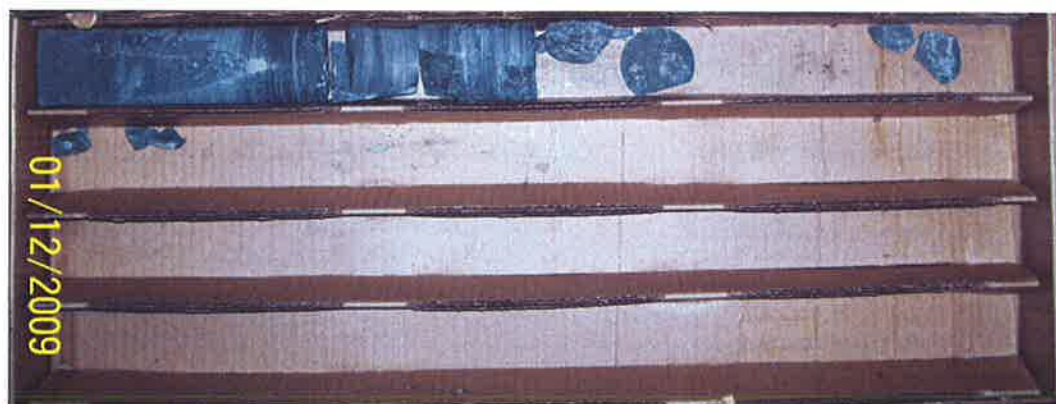
Bottom

Borehole F-6

Top



Bottom



Borehole F-7

Appendix E

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 \bar{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 COMPOSITION 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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	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_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

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
j_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
j_w	kN/m ³	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
j	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
j_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(W_L - W_p) / I_p$	v	m/s	DISCHARGE VELOCITY
P_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
j_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(W_L - W) / 1_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m ³	DENSITY OF SUBMERED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
j'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

**FOUNDATION DESIGN REPORT
HIGHWAY 401 / BOUNDARY ROAD
BRIDGE, CITY OF CORNWALL, ONTARIO
W.P. # 385-01-01, SITE # 31-215
GEOCRES NO. 31G-230**

AECOM

Project: SPT1223
May 25, 2009

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**FOUNDATION DESIGN REPORT
HIGHWAY 401/BOUNDARY ROAD BRIDGE
CITY OF CORNWALL, ONTARIO
W.P. # 385-01-01; SITE # 31-215**

5. DISCUSSION AND RECOMMENDATIONS

The proposed bridge which will carry Boundary Road over Highway 401 will be a two-span 71 m long structure; each span will be 35.5 m. It will incorporate a central pier in the unpaved median of Highway 401. The proposed bridge will carry two lanes traffic with a total bridge deck width of about 11.7 m. As shown in Drawing No.2, the anticipated height of the bridge over the existing Highway 401 grade ranges from about 7.5 m at the south abutment location to about 8.0 m at the north abutment location. From information supplied to us, the existing bridge is a four span structure with a total length of about 77 m and piers are supported on normal spread footing foundations, while the abutments are supported in driven steel H-piles. MTO drawing dated Feb 1961, indicates 10-inch H-piles @ 42 (probably HP 250 x 62) with a design working load of 36 tons (approximately 320 kN/pile). The piles were to be driven to the surface of the bedrock.

The sub-surface conditions were explored at eight (8) boreholes (see Table 3.1 in Section 3 of the foundation investigation section of this report) during this investigation. In general, beneath some embankment and other fill materials and a veneer of topsoil, the boreholes show the presence of surficial native soils which consist of clayey silt, sandy silt to silty sand and gravelly sand. These deposits are 0.6 to 2.9 m thick and are underlain by a major glacial deposit (silty sand to sandy silt till), which is in turn underlain by argillaceous limestone bedrock at Elevations of 49.4 to 50.8 m or about 6 to 8 m below the Highway 401 elevation. The elevations of the bedrock surface in the boreholes were found to be relatively flat.

The water level at the time of investigation was contacted at elevations in between 56 and 57 m, based on the measurements of water level in open boreholes and in the two piezometers installed. It should be noted that the groundwater table can be expected to be subject to seasonal fluctuations and in response to major weather events.

5.1 Foundations

We understand that the proposed bridge will be constructed adjacent to the existing bridge which will retain the two lanes traffic of Boundary Road during the construction of the new bridge. The clear distance between the existing and the new structure will be about 2 m. Based on the results of this investigation, we have considered a number of foundation options varying from normal spread footings to deep foundations which include drilled caissons, driven steel piles and micropiles.

The use of normal spread footings were considered for both the abutment locations and the central pier location. However, this option will necessitate extensive excavations which will extend below the groundwater table immediately adjacent to the existing bridge structure and as such it is not the preferred option. As well, integral type of abutment design is planned for this bridge which requires driven steel H-piles for the support of the abutments.

The presence of cobbles and boulders is always a possibility in the glacial till soils. As well in the 1961 Drawings by MTO, the presence of cobbles and boulders is mentioned. During the present investigation, boulders were encountered at relatively shallow depths in Boreholes F4 and F5 drilled at the proposed pier location (i.e. coring was necessary to advance the boreholes through boulders encountered at about El. 52 and 53 m, in Boreholes F4 and F5, respectively). Therefore the use driven piles at the central pier location is not recommended. At the boreholes drilled at the proposed abutment location, boulders were not encountered. For this reason, it is believed that there is less likelihood of encountering boulders at the proposed abutment locations, in comparison with the proposed central pier location. In addition the piles at the abutment locations will be longer and will enable the integral abutment design. As well, it appears that driven piles were used for the support of the abutments of the existing bridge, according to the information provided to us. For these three reasons, the use of driven H-piles to support the abutments is considered to be a feasible solution while, the lack of embedment in the overburden in Boreholes F4 and F5 lead us to conclude that the use of pile foundations at the central pier location is not a feasible solution. Consideration can be given to the use of micropiles for foundation support at the central pier but drilled caisson foundations, socketed into the very dense till or in the underlying bedrock are considered to be a more suitable option.

Driven piles should not be overdriven.

The relative merits and disadvantages of various foundation support types are summarized in Appendix F.

In summary the recommended foundation support for the abutments is driven H-piles while at the central pier, the use of drilled caissons is recommended.

The following is a further discussion of some of the more suitable foundation options for the proposed bridge.

5.1.1 Spread Footing Foundations on Natural Soil

The bridge can be supported on normal spread footing foundations placed on the undisturbed dense to very dense till stratum, as detailed in the following table.

Table 5.1.1.1: Spread Footing Foundations

Location	Recommended Highest Founding Elevation (m)	SLS (kPa)	ULS (kPa)	Subgrade Soils
South Abutment (Boreholes F1/F2/F3)	54.8	350	500	Dense to very dense silty sand to sandy silt till.
Central Pier (Boreholes F4/F5)	52.7	350	500	Dense to very dense silty sand to sandy silt till.
North Abutment (Boreholes F6/F7/F8)	52.0	350	500	Dense to very dense silty sand to sandy silt till.

The factored bearing resistance at ULS given in the above table incorporates a resistance factor of 0.5 as per Canadian Highway Bridge Design Code (CHBDC) S6-06. The serviceability condition is based on the premise that the maximum total and differential settlements will not exceed 25 mm and 20 mm,

respectively. This can be achieved provided that the founding subgrade is undisturbed during the construction.

Under inclined loading conditions, the bearing resistance at ULS should be reduced in accordance with CHBDC S6-06.

As can be seen from the above table, relatively deep excavations extending to below water table will be required. As the footings should be constructed in the dry, dewatering as well as a temporary shoring system will be required (due to the proximity to the existing structure).

For frost protection all footing should have a permanent earth cover of at least 1.7 m.

The actual founding elevation should also be chosen with due consideration for frost depth, as well as dewatering requirements.

Allowance should be made to place a 120 mm thick concrete mud mat (i.e. skim coat) in all footing excavations as soon as possible (not more than four hours) after excavation. All footing excavations should be inspected and approved by the geotechnical engineer prior to pouring the concrete mud mat.

Spread footing foundations are not recommended for the support units for this bridge due to the extensive excavation and dewatering requirements.

5.1.2 Steel H-Piles

The boreholes show that the geotechnical conditions at the site are suitable for the use of driven steel H-piles at the abutment locations to support the proposed Highway 401 underpass at Boundary Road near Cornwall, Ontario. The borehole data also show that with the prevailing subsurface conditions, the use of a low displacement pile, such as steel H-pile with a heavy section (e.g. HP 310 x 110) would be better suited than other pile types (e.g. steel tube piles, steel H-piles with lighter sections or precast concrete piles).

Steel H-piles (HP310 x 110) driven to refusal in the basal zone of the very dense silty sand to sandy silt till can be designed for MTO's standard values of 1800 kN/pile for U.L.S. and 1600 kN/pile for S.L.S., for very dense till soils. The following table summarizes the estimated pile tip elevations at the borehole locations.

Table 5.1.2.1: Estimated Tip Elevations for Steel H-Pile Foundation

Location	Estimated Pile Tip Depth/Elevations (m)
South Abutment (Boreholes F1/F2/F3)	51.2
North Abutment (Boreholes F6/F7/F8)	50.0

According to information given to us by AECOM, the top of the pile elevations are planned at El. 59 to 60 m and therefore the anticipated pile lengths at the south and north abutments are 8 to 9 m and 9 to 10 m, respectively.

At the pier location, the anticipated refusal elevations are about 50.5 m if boulders are not encountered. As the top of the pile elevation at this location is approximately 54.0-54.5 m, the pile lengths will be between 3.5 and 4.0 m (if boulders are not encountered), which are somewhat too short. Since in Boreholes F4 and F5 boulders were encountered (which necessitated advancing the boreholes by coring) at higher elevations refusal may be encountered at even higher elevations and in this event the piles will be even shorter.

Driven piles are therefore considered high risk and not feasible. As well, due to the presence of cobbles and boulders and the very dense nature of the till at the refusal depths hard driving conditions can be expected, creating vibrations immediately adjacent to the existing bridge footing. Therefore, while driven H-piles are the preferred option at the abutment locations, the use of driven piles at the pier location is not recommended.

The pile tip elevations provided in Table 5.1.2.1 are for estimating purposes only. Due to potentially variable soil conditions, the actual pile tip elevation may vary. The contract should allow for some variations in pile length and this aspect should be taken into consideration when ordering the piles. The piles should be driven into the competent lower glacial till deposit using a suitably heavy hammer capable of delivering a suitable rated energy. The possibility of piles encountering potential cobbles and boulders in the till should be anticipated and some piles may drive to the surface of the bedrock. In view of this, as well as the very dense nature of the till, the tips of the piles should be stiffened as per OPSD-3000.100, Type I to minimize damage to the piles in anticipation of heavy driving conditions. Care must be taken to avoid overdriving and damaging the pile tip (i.e., the structural capacity of the piles should not be exceeded).

The driving of the piles in the field should be controlled by a recognized pile driving formula, such as the Hiley Formula, in accordance with MTO Standard SS103-11. Normally, in accordance with MTO practice, the estimated ultimate resistance of the piles by the Hiley Formula can be calculated by multiplying the recommended axial resistance at U.L.S. by a factor of 2 (i.e., 1800×2), giving an ultimate geotechnical resistance of 3600 kN. In accordance with the above criterion, we recommend that the piles be driven to about 1.5 m above the estimated pile tip elevations, and driving should then be monitored and controlled by employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standard SS103-11, using an ultimate geotechnical resistance of 3600 kN per pile, subject to the approval of the QVE.

If the piles encounter refusal before sufficiently penetrating into the lower zones of the competent silty sand to sandy silt till deposit, then pile capacities may need to be revisited and alternative measures sought. Therefore, pile driving records should be kept and if refusal is met above the recommended bearing zone, a geotechnical engineer should review the driving records to assess the axial resistance. As well, the Structural Engineer should be consulted for minimum pile length requirements. It is also possible that the piles may be driven some distance below the estimated pile tip elevations to achieve the desired capacity.

All pile driving should be carried out in accordance with SP903S01. Re-striking should be done as per SP903S01. At least 10% of the piles (but not less than two piles) driven at each support element should be re-tapped not less than 24 hours after the driving of the pile, as per SP903S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped.

In addition, it may be necessary to stagger the driving of the piles, if heaving is observed. The use of light-weight (e.g. HP 310 x 79) piles is not recommended as lighter piles are more vulnerable to damage. If premature refusal is encountered, allowance may have to be made to resort to pre-augering, if necessary, as well as to reduce the axial resistance and uplift capacity of the piles. Any decision regarding pre-augering should be made in consultation with the Design Engineer, since pre-augering will lead to a loss in lateral resistances and also possibly in axial resistances. Consideration should also be given to provide an NSSP to alert the contractor of the possible presence of cobbles and boulders and possible heavy driving requirements through the very dense strata, as well as possible pre-augering.

For frost protection, all pile caps should have a permanent earth cover of at least 1.7m.

The piles should be provided with reinforced tips, as per OPSD 3000.100, Type I (see Appendix G). Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

Eccentric loading on piles and the required pile spacing should be considered as per the latest Canadian and Ontario Highway Bridge Design Code. Reference may be made to Section C6-8.7.1 of the Canadian Highway Bridge Design Code (S6-06), CHBDC, for assessing lateral pile resistance.

In cohesionless soils, the coefficient of horizontal subgrade reaction can be estimated from:

$$k_s = n_h z / d$$

Where k_s = coefficient of horizontal subgrade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 5.1.2.2.

Also as presented in the same table are estimated values for angle of internal friction and bulk unit weights.

Where the soil is primarily cohesive, the undrained shear strength of the soil is given. In this case,

$$k_s = 67 c_u / d$$

Where k_s = coefficient of horizontal subgrade reaction

c_u = undrained shear strength

d = width of pile

Table 5.1.2.2

Area Reference/ Borehole No.	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m ³)	Angle of Internal Friction (ϕ) Degrees	Recommended n_h Value (MN/m ³)	Recommended Undrained Shear Strength, C_u (kPa)	Groundwater Elevation (m)
South Abutment/ F2	58.3-56.8	Fill	18.0	-	-	25	57.1
	56.8-56.0	Silty sand to sandy silt till	20.0	30	1.3	-	
	56.0-50.8	Silty sand to sandy silt till	21.5	34	11.0	-	
South Abutment/ F3	60.7-59.5	Fill	20.5	30	6.6	-	57.3
	59.5-56.4	Fill	20.5	30	4.4	-	
	59.4-55.7	Clayey Silt	20.5	-	-	150	
	55.7-50.4	Silty sand to Sandy silt till	21.5	34	11.0	-	
North Abutment/ F6	56.7-54.6	Silt to clayey silt	19.0	-	-	75	56.2
	54.6-53.8	Sandy silt	17.0	27	1.0	-	
	53.8-51.6	Silty sand to sandy silt till	20.5	31	4.4	-	
	51.6-49.8	Silty sand to sandy silt till	22.0	35	11.0	-	
North Abutment/ F7	60.4-58.9	Fill	19.5	29	4.0	-	56.3
	58.9-58.4	Fill	19.5	-	-	-	
	58.4-57.7	Fill	20.5	-	-	150	
	57.7-56.1	Fill	20.0	-	-	200	
	56.1-55.4	Clayey silt	20.0	30	4.4	-	
	55.4-52.5	Silty Sand to Sandy silt till	20.5	-	-	-	
		Silty Sand to Sandy silt till	20.5	31	4.4	100	
	52.5-49.4	Silty Sand to Sandy silt till	22.0	35	11.0	-	

For preliminary design purposes, the recommended horizontal resistances for HP 310 x 110 steel H-piles are as follows:

Horizontal Resistance at ULS = 110 kN/pile

Horizontal Resistance at SLS = 40 kN/pile

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence, the current MTO standard for the flex zone consists of an annular space in between two concentric corrugated steel pipes (CSP's). One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. In accordance with current MTO practice, this space between the CSP's can be left void. After the pile is driven, the space between the H-pile and the inner CSP is filled with sand. This double CSP scheme is typically used for false abutments.

Alternatively, if a false abutment is not provided, in accordance with MTO structural office requirements (Report SO-96-01), the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and

filling with uniform sand. A special provision should be included in the contract specifying the gradation of the sand as follows:

Sieve Size	Percentage Passing
2 mm	100 %
600 µm	80-100 %
425 µm	40-80 %
250 µm	4-25 %
150 µm	0-6 %

A special provision should be provided in the contract for the supply and installation of the CSP's.

5.1.3 Caisson Foundations

Augered and cast-in-place concrete foundations (drilled caissons) can be considered for the central pier. At the central pier location, as mentioned before, spread footings and driven piles are not recommended.

We recommend caisson foundations for the pier using bearing resistances of 2000 kPa at SLS and 3000 kPa at ULS at El. 50.0 m on the very dense silty sand to sandy silt till, and 4000 kPa (ULS) by socketing at least 0.3 m into the relatively sound bedrock at El. 49.3 m, where SLS will not govern. This value can be increased to 5000 kPa (ULS) if the caisson is socketed at least 0.6 m into the relatively sound bedrock at about El 49.0 m. The recommended resistance values include both adhesion and end bearing components. This design value applies to commonly used caisson sizes in Ontario (i.e. between 0.76 and 2.1 m diameter). However, the use of smaller size caissons (i.e. between 0.76 and 1.2 m) is recommended, as these are easier and more efficient to install. For example a 0.90 m diameter caisson will have a base area of $r^2\pi = (0.9/2)^2 \times 3.1416 = 0.64 \text{ m}^2$. When designed for a SLS of 2000 kPa, the caisson should be capable of carrying an axial load of $0.64 \times 2000 = 1280 \text{ kN}$ at SLS. It is anticipated that if this size caisson is used, for example, about a dozen caissons will likely be installed to support the pier in the overburden, while significantly less number of caisson units would be required if socketed into the bedrock. The recommended clear distance between any two adjacent caissons is two diameters (edge to edge). If this presents problems, we would be placed to further discuss this requirement.

The minimum caisson diameter should be 0.76 m to enable the cleaning and inspection of the base of the caisson.

Difficulties may be encountered during the installation of the caissons due to the presence of granular overburden with cobbles and boulders below water table. This can be discussed with a specialist contractor in relation to cost vs. caisson diameter. Some dewatering may be required especially in the gravelly sand to sandy gravel layer overlying the glacial till, during the installation of the caissons. These aspects should be flagged in the contract documents to minimize construction claims. Temporary steel casing will need to be installed during the construction of the caisson holes to prevent caving. The casing will be withdrawn as the concrete is poured, ensuring a sufficient head of concrete in the casing to prevent 'necking.' To prevent the disturbance of the base of the caisson (if the base is in the overburden) due to hydrostatic uplift, the concrete must be poured without delay after cleaning the base and its inspection and approval. For this reason, it may be more expedient to extend the caissons to the surface of the bedrock or to socket into the bedrock (i.e. to avoid basal heave issues within the till deposit). Although these are

standard aspects of caisson installation, we recommend that they be mentioned in the contract documents to avoid possible claims of 'extras.' If disturbance appears to be a problem, the caissons can be extended to the surface of the bedrock. Tremie concreting of the caisson can be used to reduce dewatering requirements for the installation of the caissons. This however is in our opinion not warranted.

5.1.4 Micropiles

Another alternative which may be considered is the use of micropiles to support the central pier.

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. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects. Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to enhance the support of existing structures. Micropile structural capacities, by comparison, rely on high capacity steel elements to resist most or all of the applied loads. These steel elements have been reported to occupy as much as one-half of the whole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout&round interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter (typically 160 to 260 mm), any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

Axial resistances of up to about 900 kN/micropile are available (at ULS and SLS will typically not govern). In this present case, up to a similar resistance would be available depending on the diameter and penetration into the sound bedrock. The lateral resistances would also depend on the diameter, as well as to a lesser extent on the socket length into the bedrock.

As mentioned before, the use of micropiles may be less economical than caissons due to the fact that the installation requires a more specialized installer for the micropiles than the many contractors who are able to routinely install caissons.

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

5.2 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') and the provision of drains 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, $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$ $K_b = 0.35$

$K_o = 0.43$ $K^* = 0.45$

Compacted Granular 'B' Type I

Angle of Internal Friction, $\phi = 32^\circ$ (unfactored)

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$ $K_b = 0.41$

$K_o = 0.47$ $K^* = 0.57$

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure. This case occurs when some movement (yield) of the structure takes place but not in a sufficient magnitude to fully mobilize an active condition (as such it is an intermediate condition between K_o and K_a).

K^* is the earth pressure coefficient for a soil loading a fully-restrained structure, including compaction surcharge effects.

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.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with CHBDC S6-06. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Section 6.9 of CHBDC S6-06.

For unrestrained wing walls (if any), the intermediate earth pressure coefficient K_b may be adopted. In the determination of degree of wall displacement or rotation to mobilize the fully active earth pressure state, Section C6.9 of the CHBDC S6-06 Commentary should be referenced.

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

5.2.1 Seismic Design Data

5.2.1.1 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-S6-00). For seismic design, therefore, in accordance with Clause 4.4.6.1 site coefficient, S , for the site is 1.0.

5.2.1.2 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 Cornwall. 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 $A=0.20$.

5.2.1.3 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 $k_h=0.20$. 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' ($\phi = 35^\circ$ - unfactored)	Granular 'B' Type II ($\phi = 32^\circ$ - unfactored)
Non-Seismic, K_a	0.27	0.31
Seismic, K_{AE}	0.42	0.47

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

5.2.1.4 Liquefaction Potential

The proposed structures will be supported by deep foundations (driven piles/ micropiles/ cassetions) founded in/on dense tills and/or bedrock. The founding soils are considered not liquefiable.

The liquefaction potential of the soils below the approach embankments under seismic loading has been considered using the empirical method outlined in Section C4.6.2 of the CHBDC Commentary, which correlates the cyclic resistance ratio of the soils with their normalized penetration resistance and fines content. Based on this assessment, and assuming a ground surface acceleration of 0.20 g, a factor of

safety of greater than 1.0 against liquefaction is obtained for magnitude 7.5 earthquake events under the approach embankment.

5.3 Approach Embankments

Based on the information provided to us by AECOM, the grade at the north and south abutment locations will be raised to about El. 64.5 and 65.0 m, respectively and that this will involve a grade raise of up to about 6 to 7 m over the existing grades (o.g.).

Based on the available borehole data, foundation failures are not anticipated for approach embankments of the height constructed with normal 2H:1V side slopes or flatter, provided that all organic, soft/very loose or otherwise unsuitable materials will be removed as per MTO standards, prior to placing the embankment fills. The anticipated stripping depths/elevations at the borehole locations are as follows:

Borehole No.	Existing Ground Elevation at the Borehole Location (m)	Recommended Stripping Elevation/Depth (m)
F1	59.5	56.0/3.5*
F2	58.3	57.0/1.3
F6	56.9	56.6/0.3
F8	56.8	56.2/0.6

*At Borehole F1 location the stripping depth is excessive. This is because in this borehole between El. 58.0 and 56.8 m, the soil was found to consist of topsoil and below El. 56.8 m it was found to consist of clayey silt with topsoil and organic inclusions, as well as being very soft (i.e. 'N' = 2 blows/0.3 m) to El. 55.7 m. In view of this, the stripping may need to be implemented to El. 56.0 m. This can however be a local condition (i.e. a pocket of poor soil placed locally). We recommend however that in this area, test pitting be implemented to further examine the actual soil conditions and stripping be implemented only if and where required locally. As well if found suitable, the inorganic soil between El. 59.3 and 58.0 m can be reused, if found suitable.

After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a heavy compactor. If necessary, the groundwater table should be lowered to at least 0.7 m in below the subgrade level, before any proofrolling and the application of significant compaction effort. This dewatering can be achieved by gravity drainage and pumping from strategically placed sumps and, if necessary, ditches.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized, 2 horizontal to 1 vertical side slopes can be used for the construction of the approach fills. Proper erosion control measures should be implemented by seed and cover (OPSS 572) or sodding (OPSS 571).

The existing embankments side slopes should be properly benched as per MTO standards (OPSD 208.010) if and where the new embankment fills are to abut into the existing.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill (e.g. Select Subgrade Materials - OPSS1010). Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of SP 105S01 and OPSS 206. Construction should be in accordance with SP 206S03. In general, the fills should be placed in suitable lift thickness, each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. Quality assurance should be provided as per MTO standard 501.08 as given in OPSS 501.

Based on the findings of Borehole, the anticipated foundation settlement under the stresses generated by the approximately 6 to 7 m grade raise is approximately 50 mm on the north side and about 30 mm on the south side, while another 30 mm of settlement can occur due to settlement of the new embankment fill under its own weight. The anticipated total settlements are therefore not more than 60 mm, which, in our opinion, necessitate neither surcharging nor preloading, especially since some of these settlements would take place during and immediately after construction. The foundation settlements should be substantially completed within a period of about three months while the settlement due to the own weight of the embankment will depend on the type of soil used to build the embankment (e.g. the settlement of granular soils will be relatively rapid while clayey soils will settle more slowly). Assuming an average SSM type soil, the settlement of the embankment under its own weight should also be substantially completed within about three months. We recommend that in order to minimize differential settlements immediately adjacent to the new bridge structure, the approach embankments be constructed to the subgrade elevation (i.e. bottom of granular pavement fill) prior to driving the piles. The grade in the area of the pile driving can then be lowered to the desired elevation for pile driving. This will effect some of the settlements prior to paving of the road

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.

SP 105S19 – Protection Systems

SP 902S01 – Excavation and Backfilling to Structures.

The boreholes show that the excavations can be expected to extend through some fill material and surficial clayey silt, sandy silt and gravelly sand deposits which are underlain by silty sand to sandy silt till layers. These deposits are underlain by bedrock. These soils can be classified as follows:

Granular Embankment (Pavement) Fill	Type 3 soil above water level Type 4 soil below water level
Other Fill (clayey silt to sandy silt)	Type 3 soil above water level Type 4 soil below water level
Clayey Silt to Sandy Silt	Type 3 soil above water level Type 4 soil below water level
Gravelly Sand	Type 3 soil above water table Type 4 soil below water table
Glacial Till (dense to very dense)	Type 2 soil above water table Type 3 soil below water table (if the soil was not dewatered)
Glacial Till (loose to compact)	Type 3 soil

At the time of our investigations, the groundwater table at the site was encountered between El. 57 and 56 m and therefore excavations extending below these elevations can be expected to require aggressive dewatering, depending on the depth of excavation below these elevations and on the groundwater level at

the time of construction. Again depending on the same parameters, the type of dewatering may range from pumping from filtered deep sumps along with perimeter ditches to pumping from deep wells. As was mentioned before, coarser granular soils with relatively high permeability were encountered in Boreholes F4 and F5 below El. 55.6 and 54.2 m extending to El. 52.7 and 53.6 m, respectively. As was also mentioned before, excavations in this particular area extending below the depths mentioned can be expected to require more rigorous pumping due to groundwater emanating from these relatively coarser soil deposits. This aspect should be taken into account, if spread footings are to be used. As well, an assessment will need to be made if dewatering will cause detrimental settlement of the central pier of the existing bridge, since this bridge will maintain the Boundary Road traffic over the highway during the construction period.

Temporary support may be necessary to retain the existing embankment fills. The contractor will probably choose to slope the ground rather than shore it for the duration of the construction, where feasible. Where support is necessary, the shoring should be designed so that the lateral movement of any portion of the roadway protection system will not exceed the established criterion for the structure performance level. In this case, the Performance Level should be 2.

Shoring will also be required if the central pier is supported on a spread footing foundation, as the new bridge foundation is expected to be located very close to the existing foundation. 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 1. The shoring system should be designed by a Professional Engineer, experienced in this type of work. All shoring should be in accordance with SP 105S19.

It is anticipated that shoring will likely consist of soldier pile and thicker logging which is common in Ontario. At the central pier location consideration can be given to the use of tight interlocking sheeting. The advantage of the latter is that dewatering effort with tight interlocking system will be minimized. However, it suffers from the disadvantage that the sheet piles may be damaged during driving due to boulders, as well. Whatever generated during driving may be detrimental to the existing pier footing.

Table 5.4.1: Recommended Unfactored Parameters for Temporary Shoring Design

Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Granular Embankment Fill (i.e. upper 2.6 m)	0.32	0.49	3.1	21.0
Lower Embankment Fill	0.33	0.50	3.0	20.5
Other Fill	0.38	0.55	2.7	18.0
Organic Rich Fill & Topsoil	0.49	0.70	1.4	16.0
Clayey Silt	0.45	0.62	2.2	17.0
Sandy Silt	0.40	0.58	2.4	17.0
Gravelly Sand/Sandy Gravel	0.31	0.47	3.2	21.0
Silty Sand to Sandy Silt Till (loose to compact)	0.33	0.50	3.0	20.5
Silty Sand to Sandy Silt Till (dense to very dense)	0.29	0.45	3.4	22.0
Bedrock (upper 0.6 m)	0.24	0.32	3.6	23.0
Soil Type	K_a	K_o	K_p	γ (kN/m ³)
Bedrock (below 0.6 m)	0.12	0.15	5.0	24.0

It should be pointed out that the presence of cobbles and boulders can be expected within the overburden, as well possibly in the embankment fill. These can be expected to cause problems during the installation of shoring units. As well, there may be difficulty in advancing these units into the rather hard bedrock, if this is required to effect shoring.

As was mentioned before, materials that may impede the driving of the piles should not be used in the affected areas.

It is also recommended that as a precaution, it would be prudent to monitor the vibrations during the driving of the piles at the abutment locations.

5.5 Frost Protection

Design frost protection depth for the general area is 1.7 m. Therefore, a permanent soil cover of 1.7 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.

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.


Gwangha Roh, Ph.D.


Ramon Miranda, P.Eng.




Zuhtu Ozden, P.Eng.



Appendix F

Summary of Foundation Alternatives

Appendix F

Summary of Foundation Alternatives

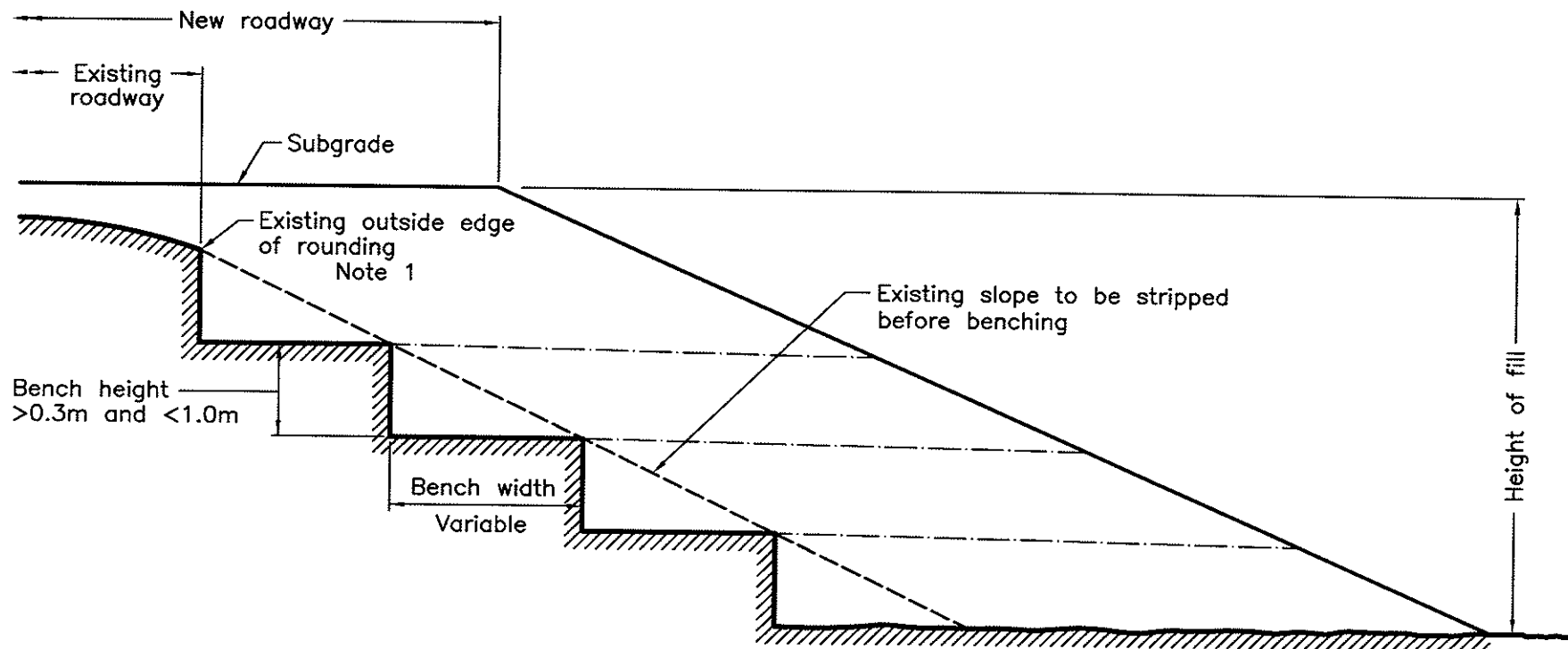
Foundation Type	Advantage/ Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Normal Spread Footings	Not suitable for integral abutment design at abutment locations. Extensive excavation will be required. Significant dewatering requirement at the central pier location.	High risk due to excavations immediately adjacent to existing bridge foundations	Least costly for the central pier	Not recommended due to extensive excavation (partially extending below the groundwater table), close to the existing structure.
Spread footings on Engineered Fill (i.e. compacted Granular 'A' pad) for abutment support	Impractical to implement considering the closeness of the existing bridge structure. Will require extensive shoring. Not suitable for integral abutments.	Moderate bearing resistance and moderate settlement potential can be expected for footings founded on compacted Granular 'A' pad resting on suitable till at between El. 53.5 and 56.0 m.	Relatively Expensive	Unlikely to be feasible for the presently proposed scheme and is not recommended based on constructability and economics
Footings on expanded base (Franki-type) concrete piles	Not suitable with the prevailing surface and subsurface conditions.	Very risky due to vibrations generated during its construction.	Expensive	Not recommended
Auger press piles	Not suitable with the prevailing subsurface condition	Not reliable.	Expensive	Not recommended
Timber piles	Short piles will not provide adequate axial and lateral resistance.	Not suitable and risky. Piles may be damaged during the driving due to the presence of cobbles and boulders and dense layers in the till	Economical but not suitable	Not recommended along a major highway based on relativity

Foundation Type	Advantage/Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Driven Concrete piles	High displacement piles.	May be damaged during the driving due to boulders	Uneconomical	Not recommended based on cost and relativity.
Steel H-piles	Low displacement piles; suitable for integral abutments. High axial resistance are available.	Boulders represent risk; very short length at pier location. Will create some vibrations	Moderate cost	Not recommended at the central pier due to reliability. Considered the best choice for the abutment foundations based on reliability and suitability, although some problems may arise during the piles if boulders are encountered.
Steel Tube Piles	Relatively less suitable than H-piles; unsuitable for integral abutments	Similar to H-pile but less reliable	Moderate cost	Less suitable than steel H-piles. Not recommended.
Drilled and cost-in-place Concrete piles (drilled caissons)	Boulders may create problems and dewatering may be required during installation. High axial resistance are available.	Boulders may create problems and some dewatering may required	Moderate cost	Considered unsuitable at the abutment locations since integral abutment type is preferred. As well steel H-piles are better suited for the support of abutment foundation elements, with the prevailing subsurface conditions Preferred alternative at the central pier location

Foundation Type	Advantage/Disadvantage	Risks/Consequences	Relative Costs	Recommendations
Micropile Foundations	Less vibration but less economical. Not suitable for integral abutments can be socketed into the bedrock		High Cost due to less competitive pricing.	<p>Will likely be less economical than drilled caissons since at present less widely used in the province.</p> <p>Can be considered as an alternative to drilled caissons at the central pier location.</p>

Appendix G

OPSD



NOTES:

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
- A Benching is not required on existing slopes flatter than 3H:1V.

- B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

ONTARIO PROVINCIAL STANDARD DRAWING

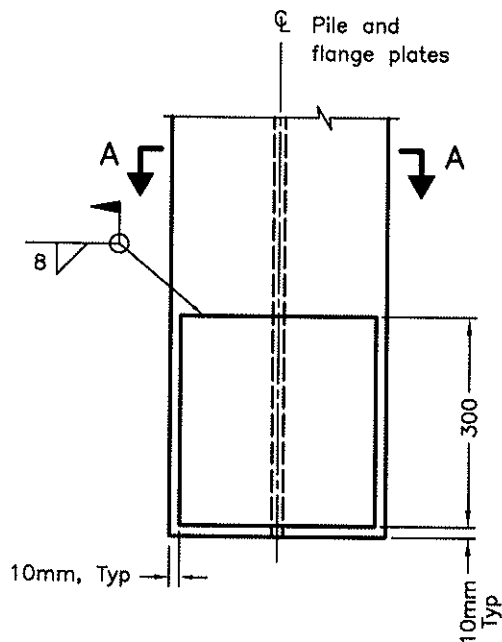
Nov 2003

Rev 1

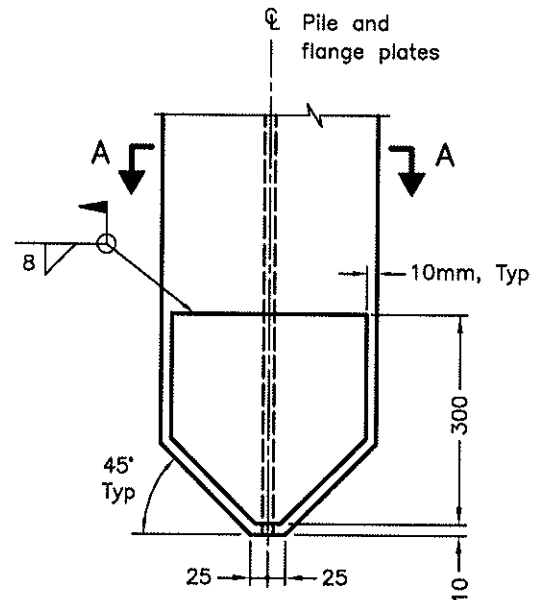
BENCHING OF EARTH SLOPES



OPSD - 208.010

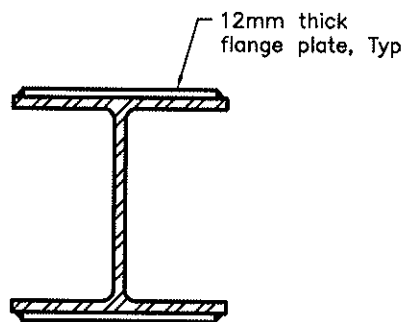


TYPE I



TYPE II

ELEVATION



PILE DRIVING SHOE
SECTION A-A

NOTES:

- A Flange plates shall be according to CSA-G40.20/G40.21, Grade 300W.
- B Welding shall be according to CSA-W59.
- C Driving shoe Type I shall be used unless Type II is specified.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

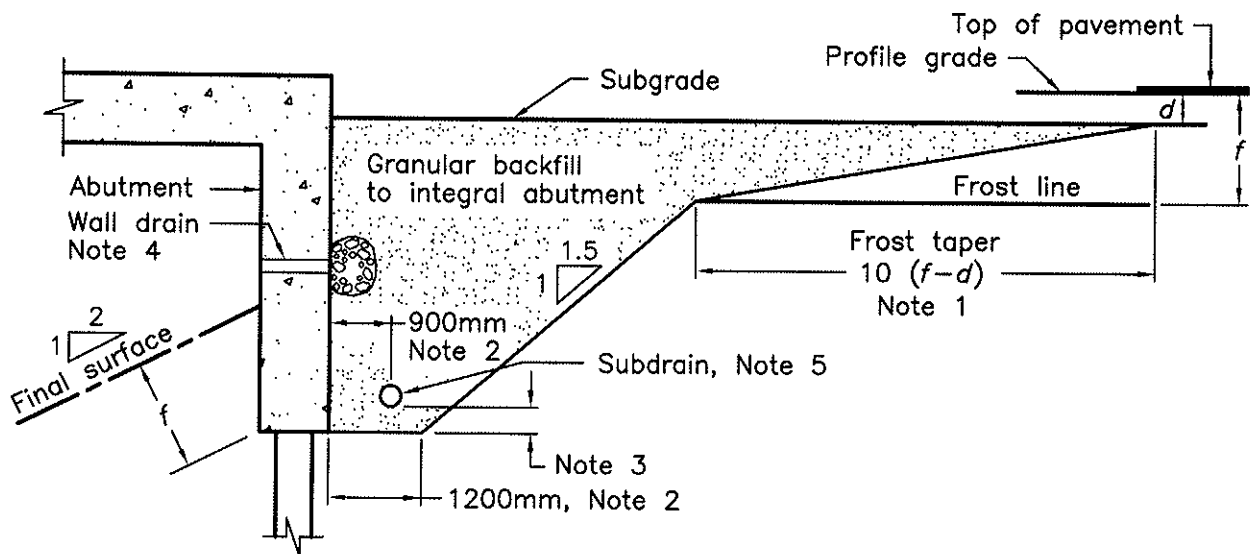
Rev 1

FOUNDATION
PILES

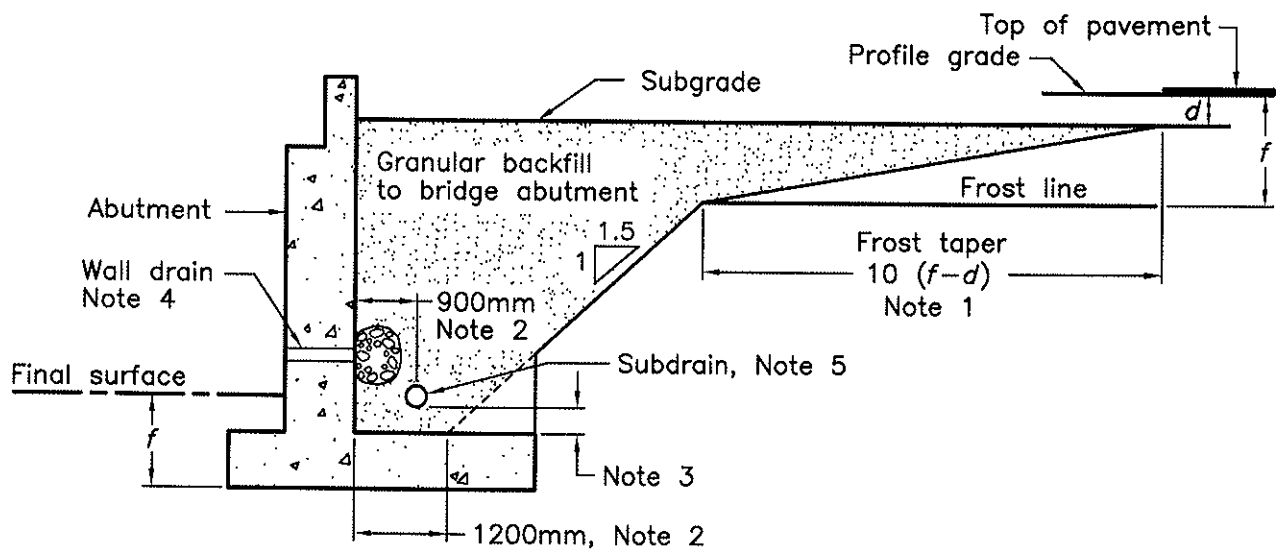
STEEL H-PILE DRIVING SHOE



OPSD - 3000.100



INTEGRAL ABUTMENT



ABUTMENT

NOTES:

- 1 d = depth of combined base and subbase courses.
 f = roadbed depth of frost penetration as specified.
- 2 Dimensions perpendicular to back face of abutment.
- 3 Height to be consistent with positive drainage of subdrain as specified.
- 4 Where specified, wall drains shall be installed according to OPSD-3190.100.
- 5 150mm dia perforated pipe subdrain wrapped with geotextile.
- A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.
- B Sections shown are parallel to centreline of roadway.
- C Subdrain to be installed with a 2% gradient behind wall.
- D All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005

Rev 0

WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT



OPSD - 3101.150

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.