FOUNDATION INVESTIGATION AND DESIGN REPORTS
BASTIEN CREEK CULVERT (C10)
STATION 18+175
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-128

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1211D
November 24, 2008

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FOUNDATION INVESTIGATION REPORT
BASTIEN CREEK CULVERT (C10)
STATION 18+175, HIGHWAY 17
CAMERON TOWNSHIP, MATTAWA, ONTARIO
W.P. 5079-05-01; AGREEMENT NO. 5006-E-0040

1. INTRODUCTION

The rehabilitation of Highway 17, from 9.5 km east of Highway 533 (Mattawa) easterly for 14.9 km will entail the rehabilitation of several existing culverts.

Shaheen & Peaker (S&P), A Division of Coffey Geotechnics Inc. was retained by D.M. Wills Associates Limited (WILLS) to carry out a foundation investigation at the site of the proposed rehabilitation of the existing Bastien Creek culvert (C10) under Highway 17 at Station 18+175 in Cameron Township, near Mattawa, Ontario.

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 PHYSIOGRAPHY

The site is located approximately 13.6 km east from the junction of Highway 533 with Highway 17 in Cameron Township as shown in Drawing No. 1.

According to the Physiography of Southern Ontario by L.J. Chapman and D.F. Putnam, 1984, the site is located within the Physiographic Region known as the Algonquin Highlands. Much of this region is underlain by Precambrian granitic bedrock. Locally, relief is rough, rounded knobs and ridges standing up, usually 15 to 60 m but occasionally up to 150 m high. The overburden is generally shallow but its thickness over the bedrock varies greatly over short distances. Many of the valleys are floored with outwashes of sand and gravel, with frequent swamp and bogs in the hollows. The northern part of the Algonquin Lake plain, that extends east to near Mattawa, shows the presence of silty clay, silt and sand deposits. In general, the highway in the project area appears to be built along spillways and shallow rock ridges, along with shallow till deposits.

According to Bedrock Geology of Ontario Map 2544 (Ministry of Northern Development and Mines, Ontario), the bedrock underlying the site consists of Mesoproterozoic Precambrian rocks (i.e. approximately 900 million years old), primarily felsic igneous tonalite, granodiorite, monzonite, granite, syenite and derived gneisses.

3. PROCEDURES

The fieldwork for this project was performed from May 14, 15, 21, 22, 23 and 27, 2008 and consisted of drilling and sampling eight boreholes (C2-1, C2-2, C2-3, C2-D1, C2-D2, C2-D3, C2-D4 and C2-D5) to depths of 2.1 to 20.0 m below existing grades. Boreholes C2-1 to C2-C3 were advanced adjacent to the existing culvert and Boreholes C2-D1, C2-D2, C2-D3, C2-D4 and C2-D5 were advanced for a possible road detour during the rehabilitation of the culvert. The locations of the boreholes at the site are given on the Borehole Location Plan Drawing No. 1.

The boreholes were advanced using a track-mounted drilling rig owned and operated by Landcore Drilling of Chelmsford, Ontario, under the full-time supervision of technical personnel from S&P. 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. The 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 granular (or cohesionless) soils (gravels, sands and silts) or the consistency of cohesive soils (clays and clayey soils).

In addition to SPT, where the consistency permitted, MTO field vane tests were performed to measure the undrained shear strength of the soil in situ.

In some cases, auger refusal was encountered within the borehole due to the presence of cobbles or boulders in the overburden. This necessitated wash boring with N-type casing. The bedrock was cored at two locations by NQ rock coring method.

A Dynamic Cone Penetration Test (DCPT) was performed adjacent to Borehole C2-1, after augering to a depth of 1.8 m. In this test, a 51 mm diameter, 60-degree apex cone, screw attached to the tip of an A-size rod, is driven into the ground, using the same driving energy as the SPT method. By recording the number of blows of the hammer to drive the cone/rod assembly, into the soil every 0.3 m, a qualitative record of soil compactness condition is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic force effects which in some cases affect the SPT results.

The borehole locations were established in the field by S&P 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.

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures. A standpipe piezometer was installed in Borehole C2-1 on completion.

A laboratory testing programme, consisting of natural moisture content determinations, Atterberg Limits and grain size analyses, were performed on selected samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets (Appendix A) and also in Appendix B.

4. SUMMARIZED SUBSURFACE CONDITIONS

The ground surface elevations range from 174.5 to 180.7 m at the boreholes. The existing top of highway embankment at the culvert location is at elevation 180.1 m, while the ground surface elevation at the boreholes advanced beyond the embankment range from 174.5 to 176.4m adjacent to the culvert.

The culvert boreholes were advanced from 17.0 m south (Borehole C2-1) and 21.0 m north (Borehole C2-3) of the existing road centerline of Highway 17. Borehole C2-2 was advanced from the existing Highway 17 shoulder. Boreholes C2-1 and C2-3 show, below a 0.1 to 0.2 m thick layer of topsoil, the presence of fine sand over stratified silt, and a silty sand deposit with till-like appearance, to depths/elevations of 16.8 m/159.6 m and 13.0 m/161.5 m, respectively. Below these elevations, the boreholes encountered gneissic bedrock. Borehole C2-2, drilled from the top of the highway embankment, encountered refusal to further augering at a depth of 19.8 m at El. 160.2 m, probably on the surface of the bedrock or close to it.

Boreholes C2-D1 through C2-D5, which were put down on the south side of the existing highway between Stations 18+096 and 18+270, were extended to practical auger refusal at depths ranging between 2.0 m and 15.5 m below the o.g. levels. Borehole C2-1 which was put down at the existing culvert location at Station 18+182 is also located on the south side of the road. This borehole was extended to practical auger refusal at 16.8 m below the o.g. (original ground) level, below which the bedrock was proven by diamond drilling and coring. In general, these boreholes show that the overburden consists of a surficial fine sand deposit, underlain by silt with thin clay interbeds. These deposits are in turn underlain by a silty sand deposit with traces of gravel and clay, which was identified as being probably of glacial origin (i.e. glacial till).

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A. An inferred stratigraphic section is shown in Drawings No. 1 and 2. The following description of the individual soil strata is to assist the designers of the project with an understanding of the anticipated subsurface conditions underlying the site. It should be noted that the surface conditions may vary in between and beyond borehole locations.

4.1 CULVERT SITE

Boreholes C2-1, C2-2 and C2-3 were advanced at Culvert C10 site (i.e. Bastien Creek Culvert).

4.1.1 TOPSOIL

Boreholes C2-1 and C2-3 which were advanced from the bottom of the existing highway embankment contacted 0.1 m and 0.2 m thick topsoil layer, respectively.

It should however be pointed out that in our experience the thickness of the topsoil and the topsoil and other organic rich soils frequently varies in between and beyond borehole locations and could be thicker in depressed areas and within watercourses.

4.1.2 EMBANKMENT FILL

Borehole C2-2 was drilled from the top of the existing highway embankment and thus contacted embankment fill extending to a depth/elevation of 6.6 m/173.4 m.

The upper 1.4 m of the fill was found to be granular pavement fill, consisting of gravelly sand with traces of silt. Standard Penetration tests performed in this pavement fill yielded N-values of 13 and 19 blows/0.3 m. These results indicate a compact relative density.

Underlying the pavement fill, below 1.4 m, the embankment fill was found to consist of sand with some gravel and frequent cobbles and boulders or rock fill, which necessitated intermittent coring to advance the borehole. N-values recorded in the fill ranged from 17 to 50 blows/0.3 m. Based on these test results, the relative density of the fill is described as compact to dense.

At a depth of 5.5 m/El. 174.5 m a layer/lense of peat and organic silt was contacted. This probably represents the original ground level. The soil below this level is likely to be the original soil which was disturbed by the construction activities.

4.1.3 FINE SAND

In Boreholes C2-1 and C2-3, put down from the original ground (o.g.) level, a surficial sand deposit was contacted immediately below the veneer of topsoil. This deposit was found to extend to a depth of 2.0 m and 3.7 m or to El. 172.5 m and 172.7 m in Boreholes C2-1 and C2-3, respectively.

The grain-size distribution of a sample from the deposit from Borehole C2-1 is given in Figure B-1 in Appendix B. The curve shows the following particle size distribution.

Gravel:	0%
Sand:	92%
Silt & clay:	8%

From the curve the soil is described as a relatively pervious material with a coefficient of permeability of the order of 10^{-2} cm/s.

4.1.4 SILT WITH THIN CLAY INTERBEDS

Underlying the upper sand (Boreholes C2-1 and C2-3) or the embankment fill (Borehole C2-2), all three boreholes contacted a stratified silt deposit with thin clay interbeds.

This deposit was encountered at depths of about 1.1 to 3.7 m below the o.g. levels or at elevations ranging from 173.4 m to 172.5 m. The thickness of the deposit at the borehole locations was found to range from 3.0 m in Borehole C2-1 to 4.4 m in Borehole C2-2 and the deposit was found to extend to El. 169.7 to 168.9 m.

The material consists of cohesive to non-cohesive silt size particles with thin cohesive clay interbeds. As such the behaviour of the soil ranges from cohesive to fine-grained granular (i.e. non-cohesive).

The grain-size distribution of four selected samples from the deposit is presented in Figure B-2, in an envelope format. The following grain-size distribution is indicated.

Gravel:	0-1%
Sand:	0-5%
Silt	71-81%
Clay:	17-29%

It should be pointed out that due to its layered nature, the grain-size distribution is representative of the over-all grain-size distribution and that the grain-size distribution of the layers would range from silt sizes (mostly) to clay sizes in the clay interbeds. As such the permeability of the soil would be higher in the horizontal direction (i.e. through the silt layers) to practically impervious through the thin clay layers. Nevertheless, the material is considered to be considerably less pervious than the overlying and the underlying granular overburden soils, both in the horizontal and vertical directions.

Atterberg limits tests were performed on the following samples

Borehole C2-1/SS8
Borehole C2-2/SS10
Borehole C2-3/SS7

These tests indicated a non-plastic material.

Standard Penetration tests performed in the deposit yielded N-values which range from 2 to 5 blows/0.3 m in Boreholes C2-1 and C2-3 while in Borehole C2-2 the recorded values range from 4 to 11 blows/0.3 m. The relatively higher penetration resistance values obtained in Borehole C2-2 are believed to reflect the strength gain under the embankment fill.

From the recorded N-values the relative density of the basically granular zones is described as very loose to compact (typically very loose) and the consistency of the cohesive interbeds as very soft to stiff but typically very soft to soft.

4.1.5 LOWER SAND

The stratified silt deposit is underlain by a major deposit of silty sand with traces of gravel and clay extending to the surface of bedrock. This basal granular deposit was encountered at depths of about 5.5 to 6.7 m below the o.g. levels or below El. 169.7-168.9 m and extended to the surface of the bedrock at El. 161.5 m in Borehole C2-3 and El. 159.6 m in Borehole C2-1, while Borehole C2-2 was terminated within this deposit at El. 160.2 m upon encountering refusal to augering possibly on the surface of the bedrock or close to it.

The grain-size distribution of four samples from this granular deposit was determined in the laboratory, giving the following grain-size distribution (also see Figure B-3 in Appendix B):

Gravel:	2-7%
Sand:	59-70%
Silt & clay:	28-37%

In general with increased depth the deposit attains a relatively coarser nature with increased gravel content, as well as containing some cobbles and boulders, especially immediately above the bedrock surface. The grain-size distribution of a sample from this relatively coarser zone is given in Figure B-4 in Appendix B, which indicates the following grain-size distribution.

Gravel: 33%
Sand: 47%
Silt & clay: 20%

Standard Penetration tests performed in the deposit yielded N-values which range widely from 3 to in excess of 50 blows/0.3 m. These results indicate a very loose to very dense relative density.

4.1.6 BEDROCK

Bedrock was proven by coring at Boreholes C2-1 and C2-3, as follows:

Borehole No.	Ground Elevation (m)	Overburden Depth to the Surface of Bedrock (m)	Elevation of the Surface of Bedrock (m)
C2-1	176.4	16.8	159.6
C2-3	174.5	13.0	161.5

Borehole C2-2 encountered auger refusal and refusal to coring at a depth of 19.8 m below the road surface, or Elevation at 160.2m.

The bedrock was identified as a gneiss. Its colour is generally pinkish grey. The formation belongs to the Pre-Cambrian Era.

The total core recovery (T.C.R.) in the bedrock was 100% for all rock core runs, with Rock Quality Designation (R.Q.D.) values between 55 and 62% in Borehole C2-1 and between 77 and 100% in Borehole C2-3. These results indicate a relatively sound rock with a fair rock quality in Borehole C2-1 and good to excellent in Borehole C2-3.

From the table presented above, the surface of the bedrock appears to be fairly flat, trending up towards the north. The difference in elevation in the surface of bedrock is 1.9 m in between Boreholes C2-1 and C2-3, over a horizontal distance of 38 m or about 5%.

4.1.7 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and at the completion of each borehole. One standpipe piezometer was installed at Borehole C2-1. The observations are shown on the individual Record of Borehole sheets.

However, due to the fact that wash boring and coring was used in the boreholes, these observations may not represent stabilized conditions.

Based on the moisture contents of the soil samples, it is our opinion that at the time of our investigation, the groundwater level at the site was generally about elevation 174.0 m.

It should, however, be pointed out that the groundwater would be subject to seasonal fluctuations as well as fluctuations due to weather events and the water level in the water course.

4.2 POSSIBLE DETOUR EMBANKMENT SITE

Boreholes C2-D1 through C2-D5 were advanced on the south side of the existing Highway 17 embankment between Stations 18+096 and 18+270, for the construction of a possible detour embankment, during the rehabilitation of Culvert C10 (see Drawing No. 1). In addition, Borehole C2-1 is also located on the south side of the highway, at Station 18+182.

The boreholes were put down from beyond the toe of the highway embankment from the o.g. levels where the ground surface elevations ranged from 180.7 m at the most westerly located borehole (Borehole C2-D1) dropping to El. 179.7 and 180.0 m at Boreholes C2-D2 and C2-D3, respectively with a further drop towards the Creek at Borehole C2-1 to El. 176.4 m. From thereon the grade rises easterly to El. 179.7 and 179.3 m at Boreholes C2-D4 and C2-D5, respectively.

4.2.1 TOPSOIL

The boreholes contacted a veneer of topsoil which ranges in thickness from 0.05 to 0.15 m.

It should however be pointed out that, in our experience, the thickness of organic rich soils frequently varies in between and beyond borehole locations and is typically thicker in depressed areas and in or near watercourses.

4.2.2 FINE SAND

Beneath the topsoil, all the boreholes contacted an upper granular soil consisting of fine sand with traces of silt and occasional fine gravel. This unit was found to extend to depths ranging between 0.7 m (Borehole C2-D1) and 4.5 m (Borehole C2-D5) below the ground surface or El. 180.0 m (Borehole C2-D1) to El. 172.7 m (Borehole C2-1).

The grain-size distribution of samples from the deposit is given in Figures B-1 (sample from Borehole C2-1) and B-5 (sample from Borehole C2-D4), Appendix B.

These curves indicate the following grain-size distribution

Gravel:	0-1%
Sand:	89-92%
Silt & clay:	8-10%

Standard Penetration tests performed in the deposit yielded N-values which range widely from 2 to 48 blows/0.3 m. These results indicate a very loose to dense relative density. The higher values showing the dense condition were recorded only in Borehole C2-D4, in the lower zone of the deposit. In the remaining areas the test results are typically between 4 and 16 blows/0.3 m, which indicate a very loose to compact condition.

4.2.3 SILT WITH THIN CLAY INTERBEDS

Beneath the surficial fine sand deposits, all the boreholes except for Borehole C2-D1 (most westerly borehole), contacted a stratified silt deposit with thin clay interbeds.

At the borehole locations this stratified deposit was contacted at depths of 1.4 m to 4.5 m or at El. 178.6-172.7 m and extended to 2.2 m (El. 177.5 m) to 10.0 m (El. 169.3 m). The thickness of the deposit at the borehole location was found to range from 0.8 m (at Borehole C2-D2) to 6.3 m (Borehole C2-D4).

As was mentioned in Section 4.1.3 of this report, the deposit consists of cohesive to non-cohesive silt size particles with thin cohesive clay interbeds. As such, the behaviour of the soil ranges from cohesive to fine grained granular (i.e. non-cohesive)material. Its overall characteristic can, however, be classified as non-cohesive.

The grain-size distribution of three samples, from Boreholes C2-1, C2-D4 and C2-D5, was determined in the laboratory, giving the following particle-size distribution (see Figures B-2 and B-6 in Appendix B).

Gravel:	0-3%
Sand:	2-17%
Silt:	53-76%
Clay:	18-27%

When analyzing the results, it should be kept in mind that the test results represent a combination of a number of interbeds (i.e. overall grain-size) and that the high percentage of clay size particles is primarily due to the clay interbeds in the deposit.

N-values recorded in the deposit range from 2 to 16 blows/0.3 m indicating a very loose to compact relative density or very soft to stiff consistency.

4.2.4 LOWER SAND

All the boreholes contacted a basal sand deposit at depths ranging from 0.7 m to 10.0 m below the ground surface or below El. 180.0 m (Borehole C2-D1) to 169.3 m (Borehole C2-D5). In Borehole C2-D1, the deposit was contacted underlying the upper fine sand deposit while in the remaining boreholes it was encountered beneath the stratified silt deposit. The boreholes were terminated within this deposit upon encountering practical refusal at depths/El. 2.0 m/178.7 m (Borehole C2-D1) to 16.8 m/El. 159.6 m (Borehole C2-1), probably on the surface of the bedrock or close to it, except in Borehole C2-1. This borehole was further advanced by coring and the refusal depth proved out to represent the surface of the bedrock.

The grain-size distribution of five samples from the deposit (from Boreholes C2-1, C2-D2, C2-D4 and C2-D5) is presented on Figures B-3 and B-7 in Appendix B. These curves indicate the following grain-size distribution.

Gravel:	2-13%
Sand:	52-63%
Silt:	15-34%
Clay:	3-13%

From the curves and a visual and tactile examination of the soil samples the deposit is believed to be of glacial till origin with some sand and gravelly sand layers.

Standard Penetration tests performed in the deposit show SPT resistance values of between 3 and 58 blows/0.3 m, indicating a very loose to very dense relative density. In general, however, the lower values were recorded in the upper zones of the deposit in Boreholes C2-1 and C2-D5, elsewhere the deposit is believed to be typically in a compact to dense condition.

4.2.5 INFERRED BEDROCK

As mentioned in the preceding section, practical refusal to further augering was contacted at depths 2.0 m/178.7 m (Borehole C2-D1) to 16.8 m/El. 159.6 m (Borehole C2-1), probably on the surface of the bedrock and close to it. In Borehole C2-1, after refusal, the borehole was further advanced by coring and the bedrock was found to consist of a Pre-Cambrian gneissic rock, as detailed in Section 4.1.6 of this report.

From these results, it can be surmised that the surface of the bedrock dips from the west at Borehole C2-D1 location from El. 178.7 m towards the creek location (to El. 159.6 m at Borehole C2-1). From thereon, it rises easterly to El. 164.4 m at Borehole C2-D4 and remains relatively level to Borehole C2-D5 at El. 163.7 m.

4.2.6 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were recorded during the drilling and at the completion of each borehole. As well, a piezometer was installed in Borehole C2-1.

Based on the observations made, it is our opinion that the groundwater table at the time of our investigation was typically at a depth of about 2 m below the ground surface at the site.

It should, however, be pointed out that the groundwater would be subject to seasonal fluctuations as well as fluctuations due to weather events and the water level in the water course.

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Drawings

METRIC

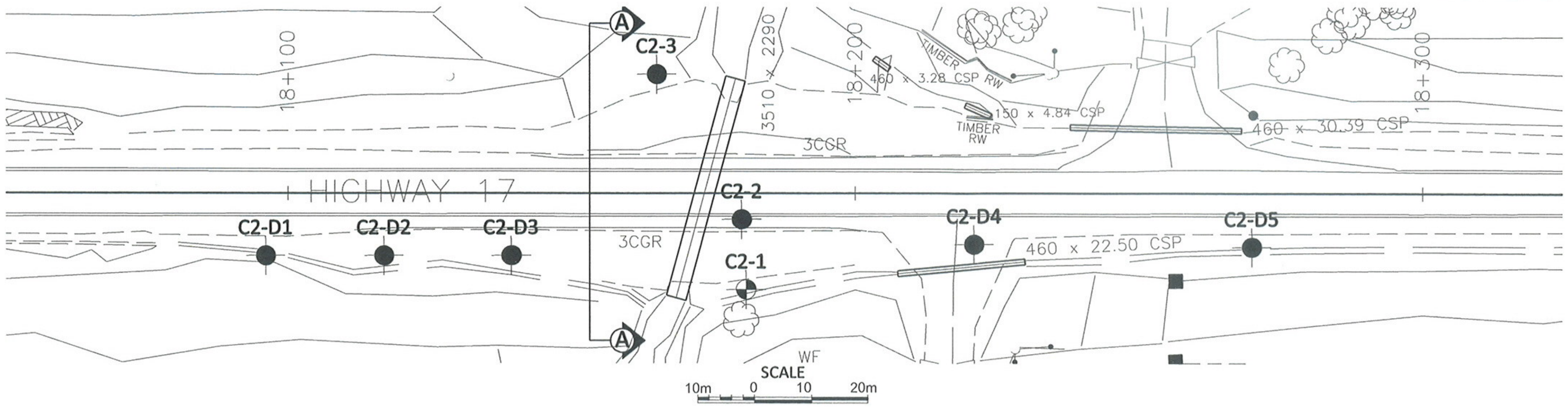
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No. 5006-E-0040
GWP: 173-98-00

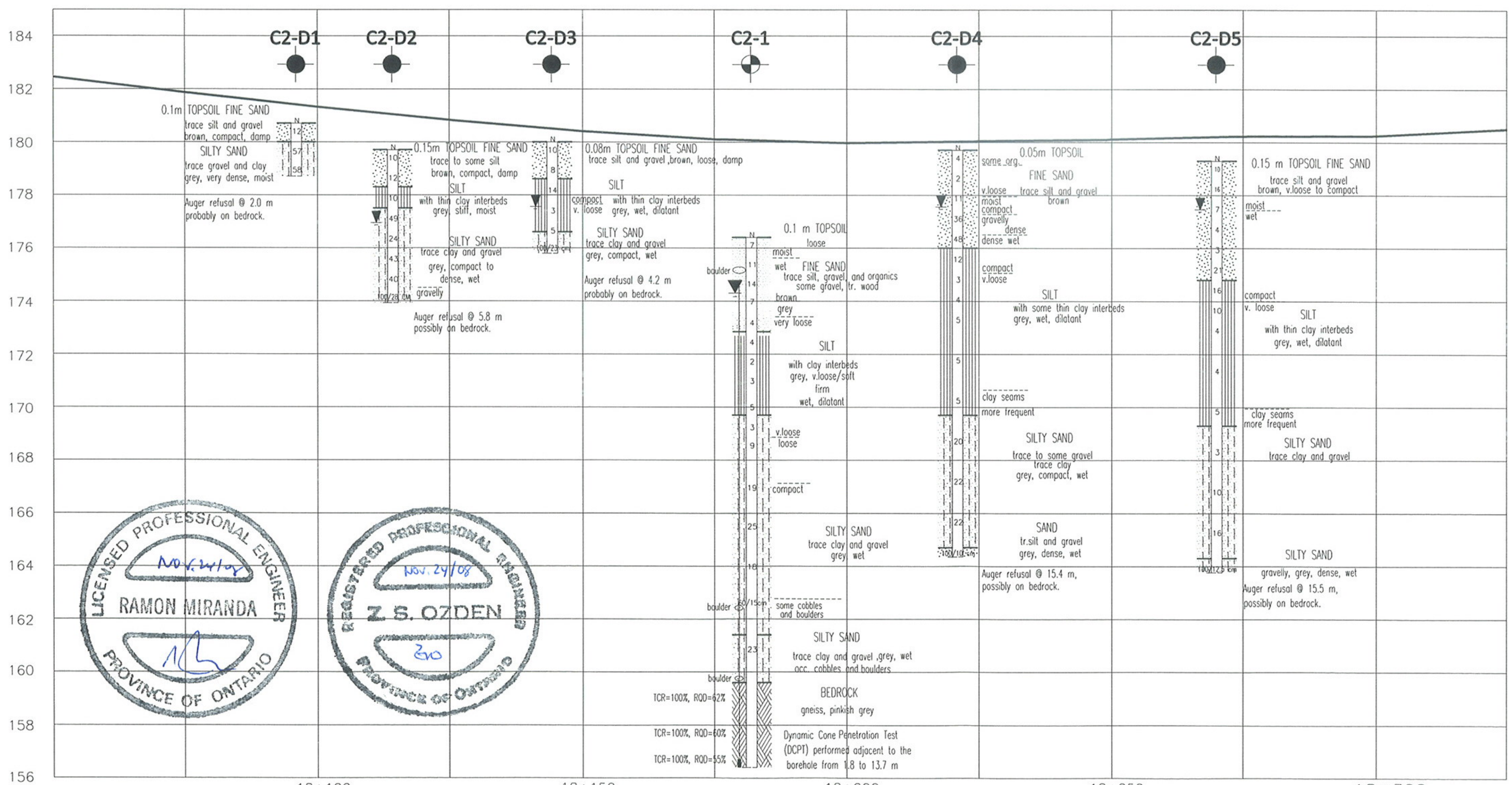
Highway 17 Mattawa
BOREHOLE LOCATION PLAN & STRATIGRAPHY (Culvert C10 @ 18+175)

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

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



KEY PLAN
N.T.S



Profile (along Highway 17 centerline)

LEGEND

- Borehole
- Borehole and Cone
- N — 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
C2-1	176.4	18+182	17.0 m Rt
C2-2	180.0	18+180	4.3 m Rt
C2-3	174.5	18+165	21.0 m Lt
C2-D1	180.7	18+096	11.0 m Rt
C2-D2	179.7	18+117	11.0 m Rt
C2-D3	180.0	18+140	11.0 m Rt
C2-D4	179.7	18+221	9.0 m Rt
C2-D5	179.3	18+270	9.5 m Rt

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. 31L-128

SPT 1211D			DIST
SUBM'D	CHECKED	DATE Oct. 2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 1

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

METRIC

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

CONT No. 5006-E-0040

GWP: 173-98-00

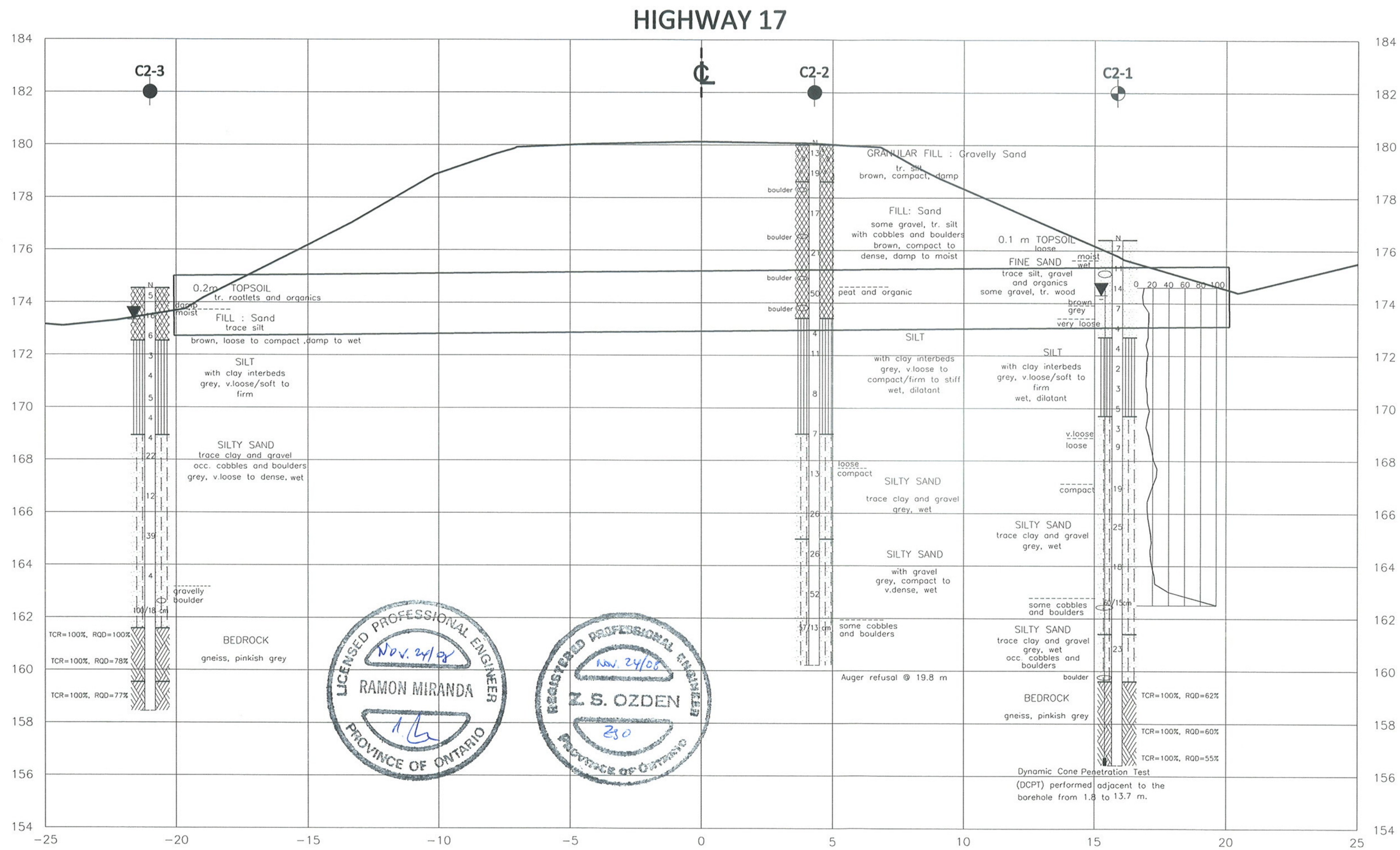
Highway 17 Mattawa

STRATIGRAPHY (Culvert C10 @ 18+175)

SHAHEEN & PEAKER

A Division of Coffey Geotechnics Inc.

HIGHWAY 17



18+173

Cross Section (A-A)

KEY PLAN
N.T.S

LEGEND

- Borehole
- Borehole and Cone
- N 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
C2−1	176.4	18+182	17.0m Rt
C2−2	180.0	18+180	4.3m Rt
C2−3	174.5	18+165	21.0m Lt

=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. 31L−128

SPT 1211D			DIST
SUBM'D	CHECKED	DATE Oct.2008	SITE
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

Appendix A

Records of Borehole Sheets

RECORD OF BOREHOLE No C2-1

METRIC

GWP 173-98-00	LOCATION Sta. 18+182 :17.0m Rt of C/L of Hwy 17 (D-5.5m)		ORIGINATED BY GH
DIST	HWY 17	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic	DATE 5/14/2008 5/15/2008		CHECKED BY ZO

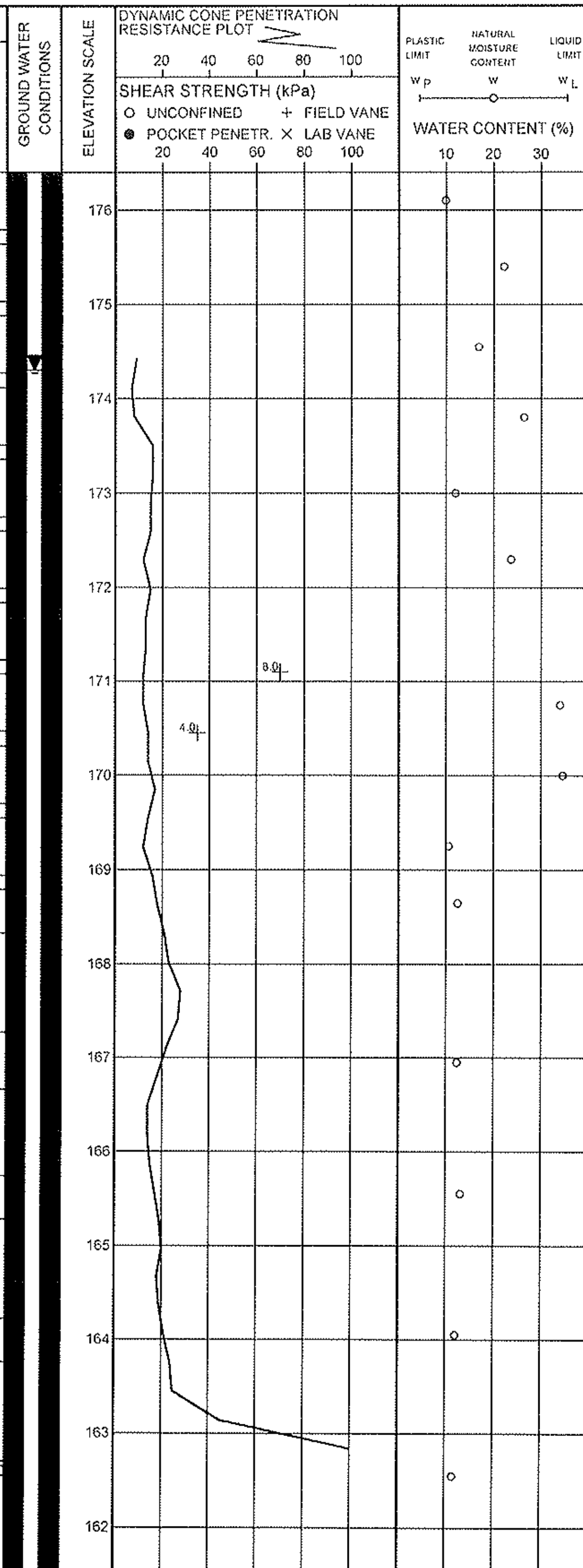
SOIL PROFILE

ELEV DEPTH	DESCRIPTION	STRAT PLOT
176.4	GROUND SURFACE	
0.0	0.1 m **TOPSOIL**	
	FINE SAND trace silt, gravel and organics; loose, moist; wet; boulder; compact, brown; grey; loose; very loose, some gravel, tr. wood	
172.7 / 3.7	**SILT** with clay interbeds, grey, v.loose/soft to firm, wet, dilatant	
169.7 / 6.7	**SILTY SAND** trace clay and gravel, grey, wet; v.loose; loose; compact; some cobbles and boulders	
161.4		

SAMPLES

NUMBER	TYPE	"N" VALUES	Remarks / Grain size (GR SA SI CL)
1	SS	7	
2	SS	11	
3	SS	14	Auger refusal at 1.2 m on a boulder. Borehole moved 2 m east. 0 92 (8)
4	SS	7	
5	SS	4	
6	SS	4	1 5 76 18
7	SS	2	
8	SS	3	Shear vanes taken in borehole 5 m away.
9	SS	5	
10	SS	3	
11	SS	9	
12	SS	19	4 59 34 3
13	SS	25	
14	SS	18	
15	SS	60/15cm	

GROUND WATER CONDITIONS | ELEVATION SCALE | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL



Continued Next Page

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

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

RECORD OF BOREHOLE No C2-1

METRIC

GWP 173-98-00 | LOCATION Sta. 18+182 17.0m Rt of C/L of Hwy 17 (D-5.5m) | ORIGINATED BY GH

DIST | HWY 17 | BOREHOLE TYPE Hollow Stem Auger | COMPILED BY SS

DATUM Geodetic | DATE 5/14/2008 5/15/2008 | CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
161.4 15.0	**SILTY SAND** trace clay and gravel grey, wet occ. cobbles and boulders	16	SS	23	161
					160
159.6 16.8	boulder / **BEDROCK** gneiss, pinkish grey	17	RC	TCR=100% RQD=62%	159
		18	RC	TCR=100% RQD=60%	158
					157
156.4 20.0		19	RC	TCR=100% RQD=55%	

End of Borehole.
Water level @ 2.1 m upon completion.
Dynamic Cone Penetration Test (DCPT) performed adjacent to the borehole from 1.8 to 13.7 m.
Piezometer installed to 17.6 m.
Water level in piezometer @ 2.0 m (El. 174.4 m) on May 16, 2008

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

RECORD OF BOREHOLE No C2-2

METRIC

GWP 173-98-00 LOCATION Sta. 18+180 :4.3m Rt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS

DATUM Geodetic DATE 5/27/2008 CHECKED BY ZO

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
180.0 0.0	GROUND SURFACE					
	GRANULAR FILL: Gravelly Sand tr. silt brown, compact, damp	1	SS	13		
		2	SS	19		
178.6 1.4	**FILL:** Sand some gravel, tr. silt with cobbles and boulders brown, compact to dense, damp to moist; boulder	3	RC			
		4	SS	17		
	boulder	5	RC			
		6	SS	21	Water level approx. el. 175.6	
	boulder	7	RC			
	peat and organic silt inclusions wet	8	SS	50		
	boulder	9	RC			
173.4 6.6	**SILT** with clay interbeds grey, v.loose to compact/firm to stiff wet, dilatant	10	SS	4		0 3 72 25
		11	SS	11		
		12	SS	8		0 0 71 29
169.0 11.0	**SILTY SAND** trace clay and gravel grey, wet; loose	13	SS	7		
	compact	14	SS	13		
		15	SS	26		
165.0						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

ELEVATION SCALE: 180, 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

Continued Next Page

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

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

RECORD OF BOREHOLE No C2-2

METRIC

GWP	173-98-00	LOCATION	Sta. 18+180 ;4.3m Rt of C/L of Hwy 17	ORIGINATED BY	SK
DIST	HWY 17	BOREHOLE TYPE	Hollow Stem Auger & NQ Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	5/27/2008	CHECKED BY	ZO

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR SA SI CL
165.0 / 15.0	**SILTY SAND** with gravel grey, compact to v. dense, wet	16	SS	26	165	~10	33 47 (20)
					164		
		17	SS	52	163	~10	
					162		
	some cobbles and boulders	18	SS	57/13 cm		~12	
					161		
160.2 / 19.8		19	AS			~9	
	End of Broehole. Auger refusal @ 19.8 m Wet Cave @ 4.3 m upon completion.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

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

RECORD OF BOREHOLE No C2-3

 METRIC

GWP 173-98-00 LOCATION Sta. 18+165 :21.0m Lt of C/L of Hwy 17 ORIGINATED BY SK
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS
DATUM Geodetic DATE 5/23/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
174.5	GROUND SURFACE								
0.0	0.2 m TOPSOIL, damp, tr. rootlets and organics	1	SS	5		174			
	FILL : Sand, trace silt, brown, loose to compact, damp to wet (moist)	2	SS	16	▽	173			
172.5 / 2.0	wet	3	SS	6					
	SILT, with clay interbeds, grey, v.loose/soft to firm	4	SS	3		172			0 2 81 17
		5	SS	4		171	8.0		
		6	SS	5		170	5.0		
168.9 / 5.6		7	SS	4		169	5.0	16	
	SILTY SAND, trace clay and gravel, occ. cobbles and boulders, grey, v.loose to dense, wet	8	SS	4					
		9	SS	22		168			
		10	SS	12		167			5 62 (33)
		11	SS	39		165			7 62 (31)
		12	SS	4		164			2 70 (28)
161.5 / 13.0	gravelly, boulder	13	SS	100/18 cm		162			Spoon bouncing on a boulder
	BEDROCK, gneiss, pinkish grey	14	RC	TCR=100% RQD=100%		161			
159.5		15	RC	TCR=100% RQD=78%		160			

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity; 15-⊕-5 (20/10) (%) STRAIN AT FAILURE

SPT 1211D : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C2-3

METRIC

GWP 173-98-00 LOCATION Sta. 18+165 :21.0m Lt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS

DATUM Geodetic DATE 5/23/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
159.5 15.0	**BEDROCK** gneiss, pinkish grey		16	RC	TCR=100% RQD=77%		159				
158.4 16.1	End of Borehole. Water level @ 1.2 m (not stabilized)* upon completion.										

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

SPT 1211D : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C2-D1

METRIC

GWP 173-98-00	LOCATION Sta. 18+096 :11.0m Rt of C/L of Hwy 17		ORIGINATED BY SK
DIST	HWY 17	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic	DATE 5/22/2008		CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
180.7	GROUND SURFACE						
0.0	0.1 m **TOPSOIL** **FINE SAND** trace silt and gravel brown, compact, damp	1	SS	12		180	
180.0 / 0.7	**SILTY SAND** trace gravel and clay grey, very dense, moist	2	SS	57			
		3	SS	58		179	
178.7 / 2.0	End of Borehole. Auger refusal @ 2.0 m probably on bedrock. Borehole dry on completion (not stabilized)						~~Spoon bouncing~~

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3; GR SA SI CL

+ ^{3}, X ^{3}: Numbers refer to Sensitivity

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

SPT 1211D : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C2-D2

METRIC

GWP 173-98-00 LOCATION Sta. 18+117 :11.0m Rt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 5/22/2008 CHECKED BY ZO

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT (kN/m³)	GR	SA	SI	CL
179.7	GROUND SURFACE											
0.0	0.15 m **TOPSOIL**											
	FINE SAND trace to some silt brown, compact, damp	1	SS	10		179	o					
		2	SS	12			o					
178.3 / 1.4	**SILT** with thin clay interbeds grey, stiff, moist	3	SS	10		178	o					
177.5 / 2.2	**SILTY SAND** trace clay and gravel grey, compact to dense, wet	4	SS	49	▽*	177	o					
		5	SS	24		176	o		2	55	30	13
		6	SS	43			o					
		7	SS	40		175	o					
	gravelly	8	SS	100/28 cm		174	o					
173.9 / 5.8	End of Borehole. Auger refusal @ 5.8 m possibly on bedrock Water level @ 2.7 m upon completion (not stabilized)*											

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 10 20 30

REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity; 20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C2-D3

METRIC

GWP 173-98-00	LOCATION Sta. 18+140 :11.0m Rt of C/L of Hwy 17		ORIGINATED BY SK
DIST	HWY 17	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic		DATE 5/22/2008	CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
180.0 /	GROUND SURFACE					180
0.0	0.08 m **TOPSOIL**					
	FINE SAND trace silt and gravel brown, loose, damp	1	SS	10		
		2	SS	8		179
178.6 / 1.4	**SILT** with thin clay interbeds grey, wet, dilatant; compact	3	SS	14		178
	v. loose	4	SS	3	▽*	177
176.6 / 3.4	**SILTY SAND** trace clay and gravel grey, compact, wet	5	SS	5		
175.8 / 4.2		6	SS	100/23 cm		176

End of Borehole.
Auger refusal @ 4.2 m upon completion, probably on bedrock.
Water level @ 2.4 m upon completion (not stabilized)*

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE (20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

RECORD OF BOREHOLE No C2-D4

METRIC

GWP 173-98-00	LOCATION Sta. 18+221 :9.0m Rt of C/L of Hwy 17		ORIGINATED BY SK
DIST	HWY 17	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic	DATE 5/21/2008		CHECKED BY ZO

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH (kPa) (O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10, 20, 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

ELEV / DEPTH	DESCRIPTION
179.7	GROUND SURFACE
0.0	**0.05m TOPSOIL**; **FINE SAND** trace silt and gravel brown; some org; v.loose; moist compact; gravelly wet; dense; dense wet
176.0 / 3.7	**SILT** with some thin clay interbeds grey, wet, dilatant; compact; v.loose; clay seams more frequent
169.7 / 10.0	**SILTY SAND** trace to some gravel trace clay grey, compact, wet
164.7	

NUMBER	TYPE	"N" VALUES	GR SA SI CL
1	SS	4	
2	SS	2	
3	SS	11	1 89 (10)
4	SS	36	
5	SS	48	
6	SS	12	
7	SS	3	
8	SS	4	
9	SS	5	
10	SS	5	
11	SS	5	3 17 53 27
12	SS	20	
13	SS	22	
14	SS	22	11 55 23 11

Elevation scale: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

Continued Next Page

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

20 / 15-⊕-5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C2-D4

METRIC

GWP 173-98-00	LOCATION Sta. 18+221 :9.0m Rt of C/L of Hwy 17		ORIGINATED BY SK
DIST	HWY 17	BOREHOLE TYPE Hollow Stem Auger	COMPILED BY SS
DATUM Geodetic	DATE 5/21/2008		CHECKED BY ZO

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.7 / 15.0											
164.4 / 15.4	**SAND** tr.silt and gravel grey, dense, wet		15	SS	100/10 cm				o		Spoon refusal
	End of Borehole. Auger refusal @ 15.4 m, possibly on bedrock. Water level @ 2.1 m (not stabilized)* and hole caved-in @ 8.5 m upon completion.						164				

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 10 20 30

+³, ×³: Numbers refer to Sensitivity

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

Ministry of Transportation
Ontario

Foundation Design

SPT 1211D : Highway 17 (Mattawa)

RECORD OF BOREHOLE No C2-D5

METRIC

GWP	173-98-00	LOCATION	Sta. 18+270 :9.5m Rt of C/L of Hwy 17
DIST		HWY	17
BOREHOLE TYPE	Hollow Stem Auger		
DATUM	Geodetic	DATE	5/21/2008
ORIGINATED BY	SK	COMPILED BY	SS
CHECKED BY	ZO		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
179.3	GROUND SURFACE
0.0	0.15 m **TOPSOIL**; **FINE SAND** trace silt and gravel brown, v.loose to compact; moist / wet
174.8 / 4.5	**SILT** with thin clay interbeds grey, wet, dilatant; compact / v. loose; clay seams more frequent
169.3 / 10.0	**SILTY SAND** trace clay and gravel
164.3	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	10	
2	SS	16	
3	SS	7	
4	SS	4	
5	SS	3	
6	SS	21	
7	SS	16	
8	SS	10	
9	SS	4	
10	SS	4	
11	SS	5	0 1 76 23
12	SS	3	
13	SS	10	10 52 25 13
14	SS	16	13 63 15 9

ELEVATION SCALE: 179, 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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

RECORD OF BOREHOLE No C2-D5

METRIC

GWP 173-98-00 LOCATION Sta. 18+270 :9.5m Rt of C/L of Hwy 17 ORIGINATED BY SK

DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS

DATUM Geodetic DATE 5/21/2008 CHECKED BY ZO

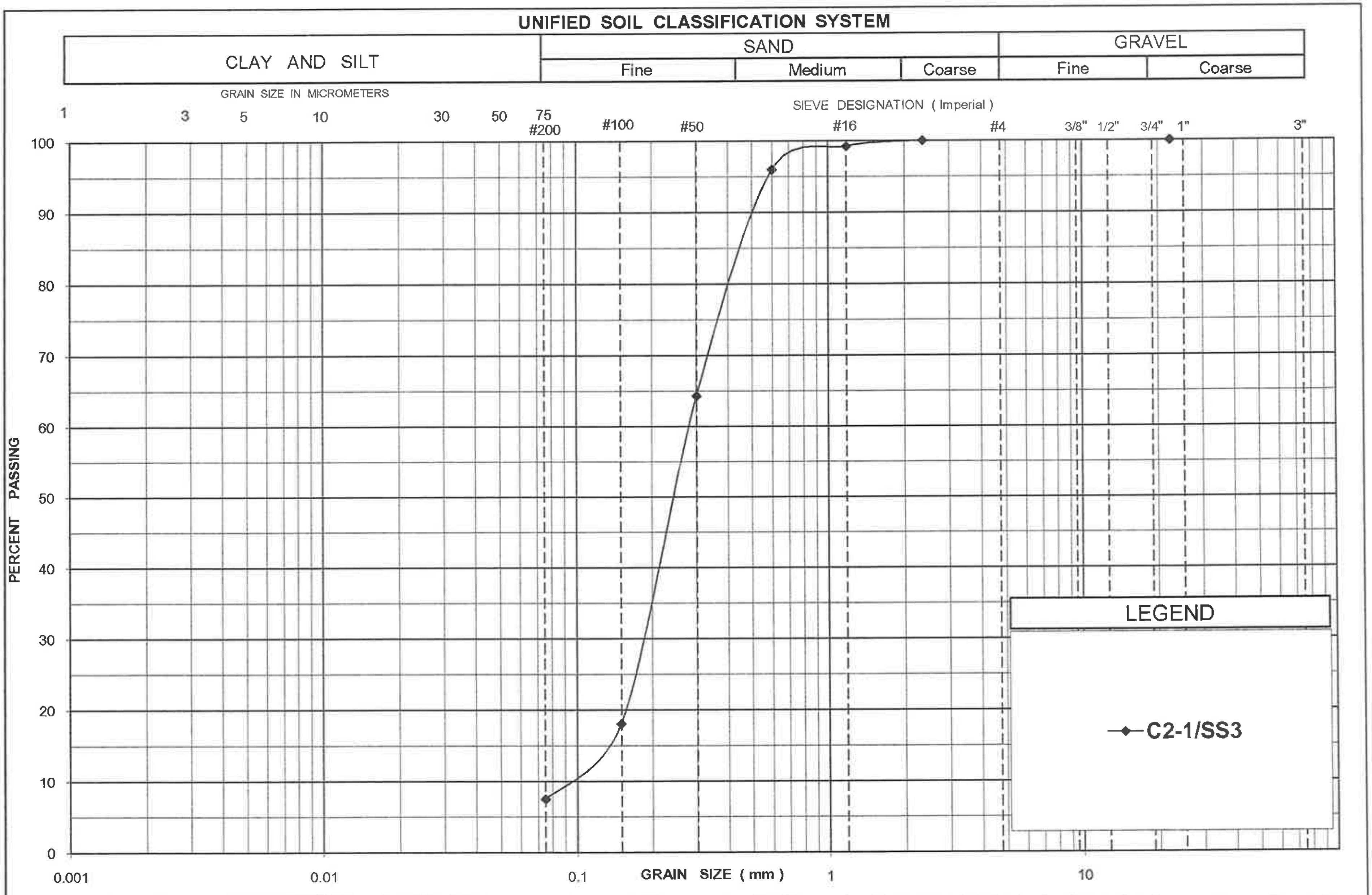
ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
164.3 15.0	**SILTY SAND** gravelly, grey, dense, wet						164				
163.7 15.5			15	SS	100/12.5 cm				o		Spoon refusal
	End of Borehole. Auger refusal @ 15.5 m, possibly on bedrock. Water level @ 1.8 m (not stabilized)* and hole caved-in @ 10.4 m upon completion.										

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

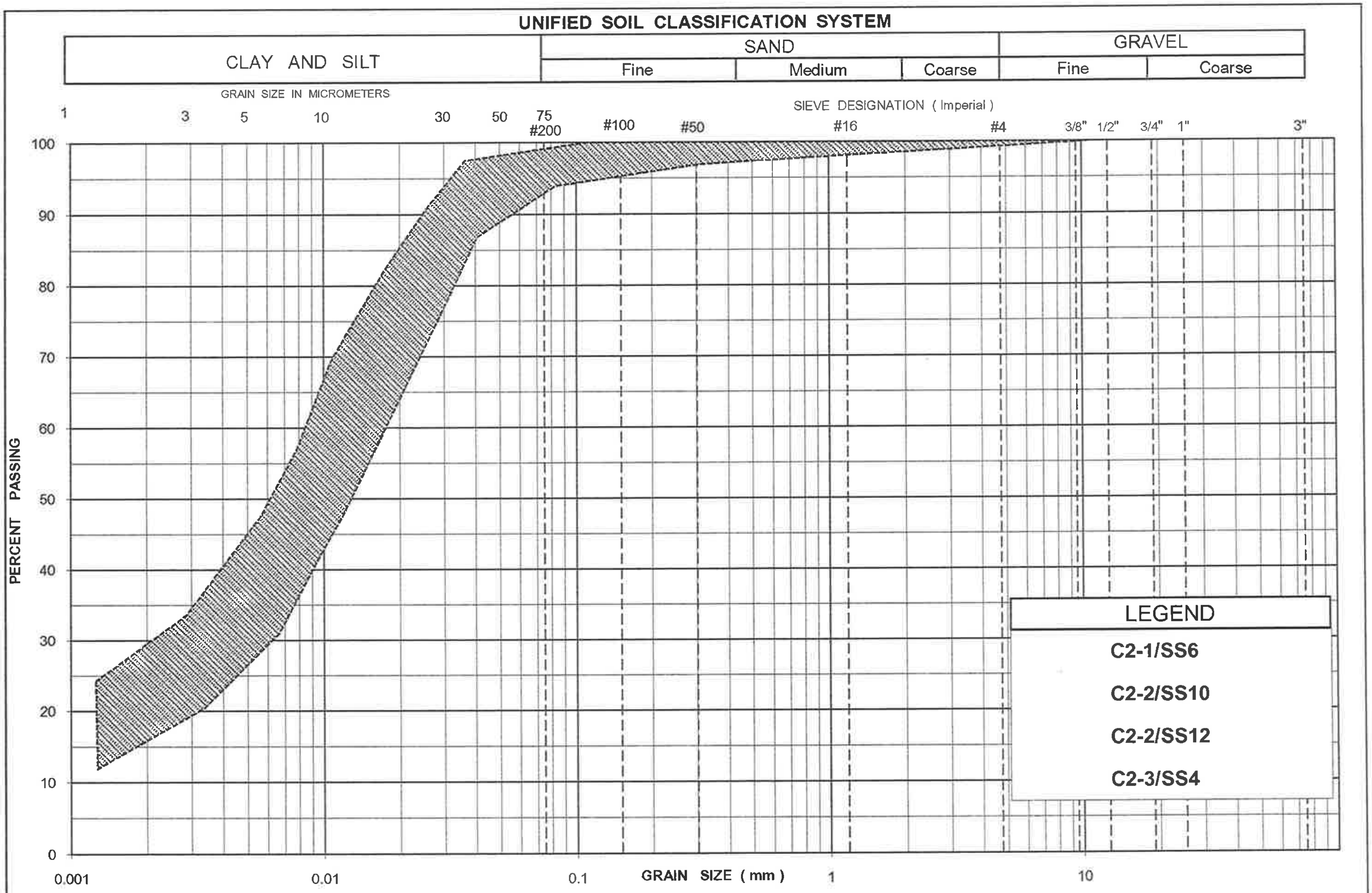
15-⌖-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Appendix B

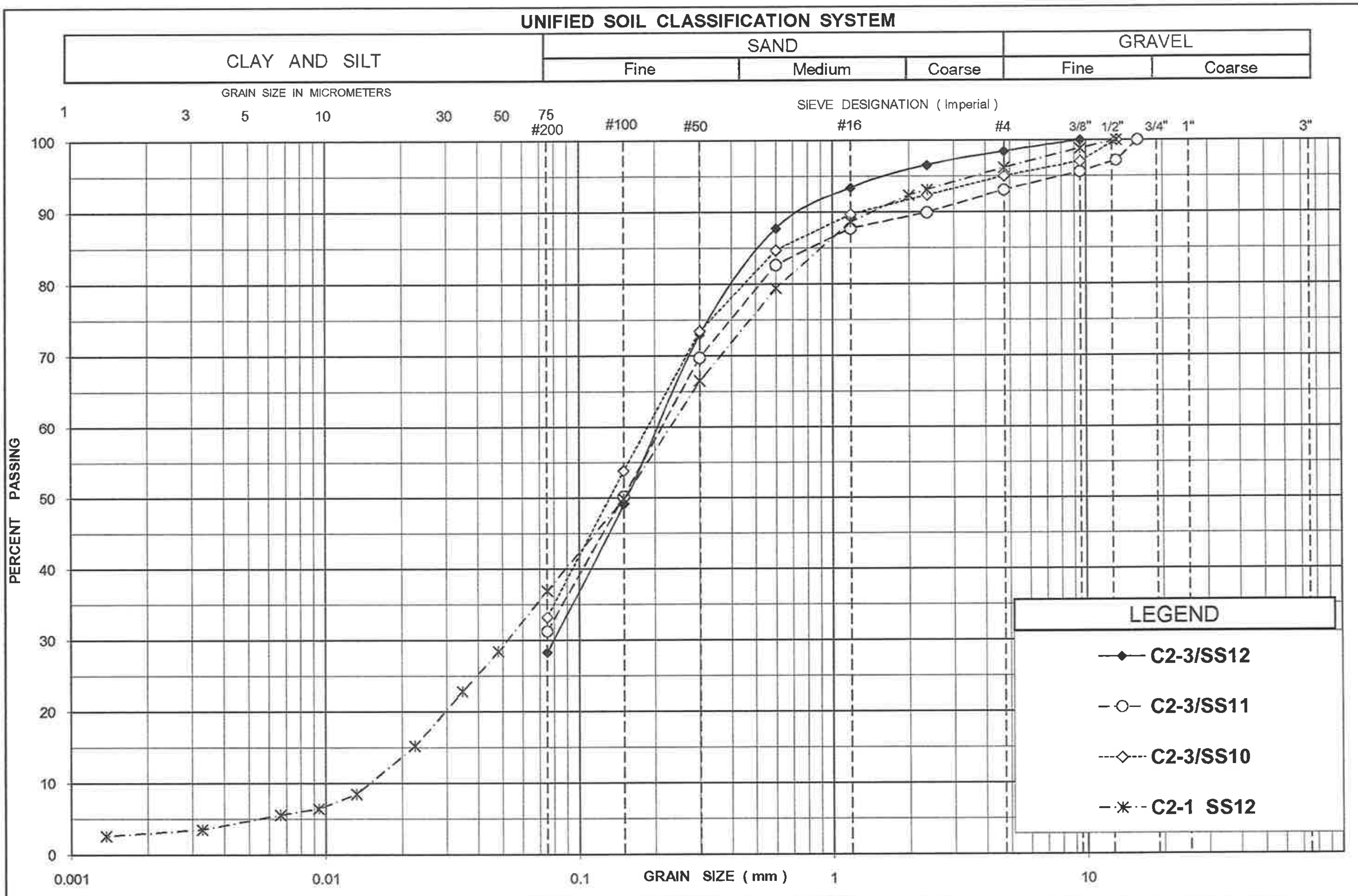
Laboratory Test Results



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION FINE SAND, trace silt	FIGURE No. B-1 REF. No. SPT 1211D DATE SEPTEMBER 2008



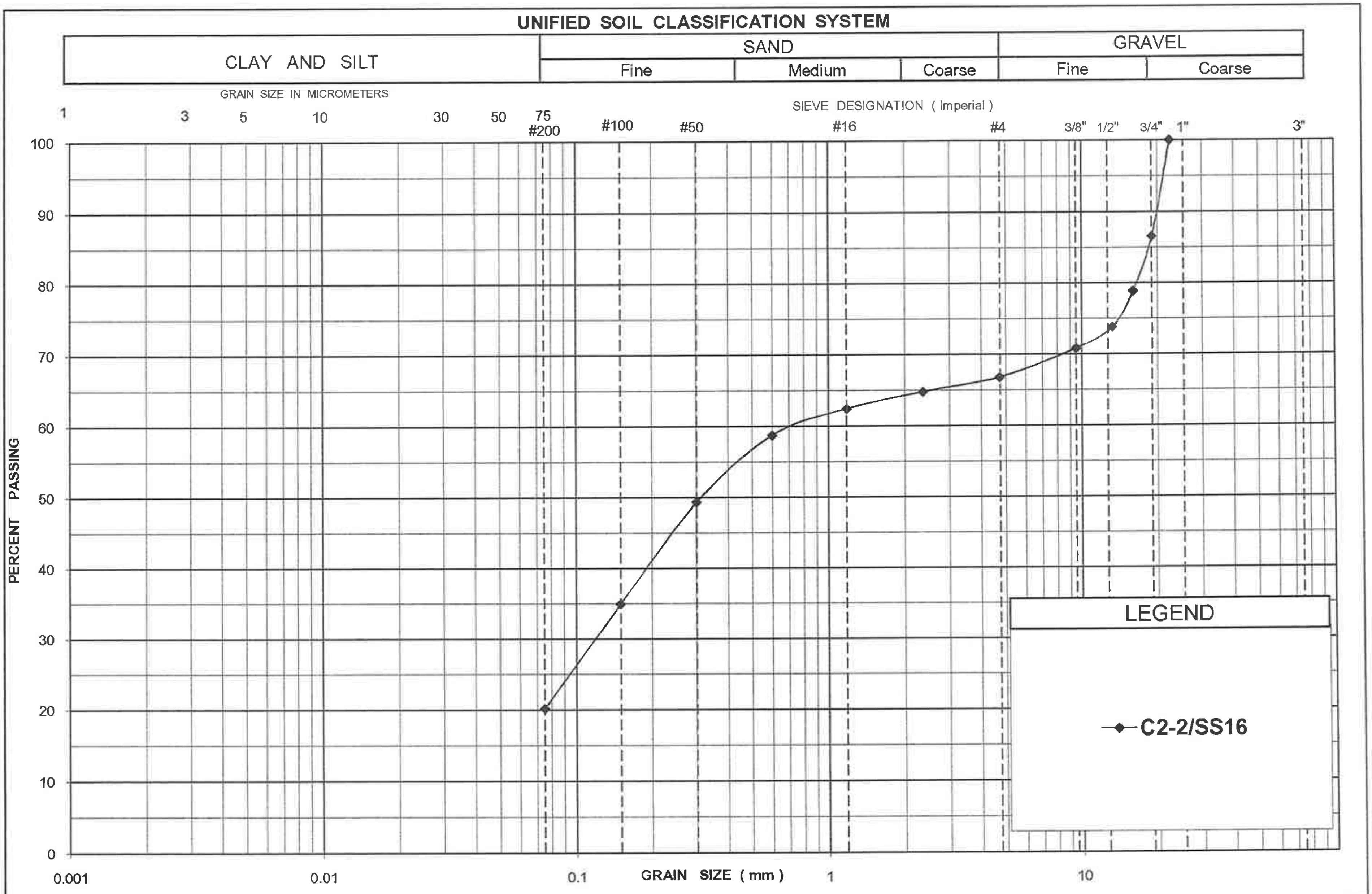
SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	**GRAIN SIZE DISTRIBUTION** SILT with thin clay interbeds	FIGURE No. B-2 REF. No. SPT 1211D DATE SEPTEMBER 2008



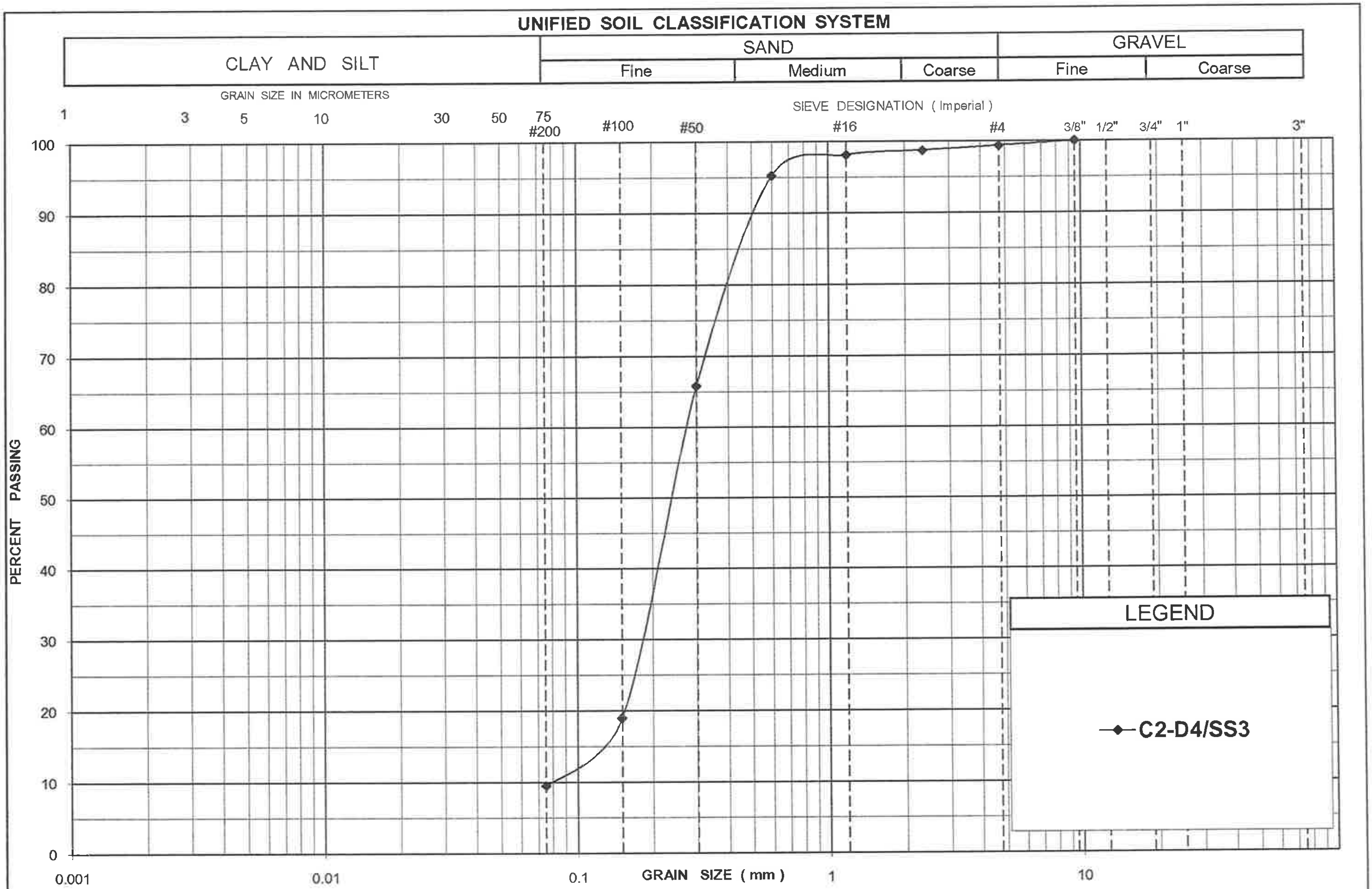
SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
SILTY SAND, trace gravel

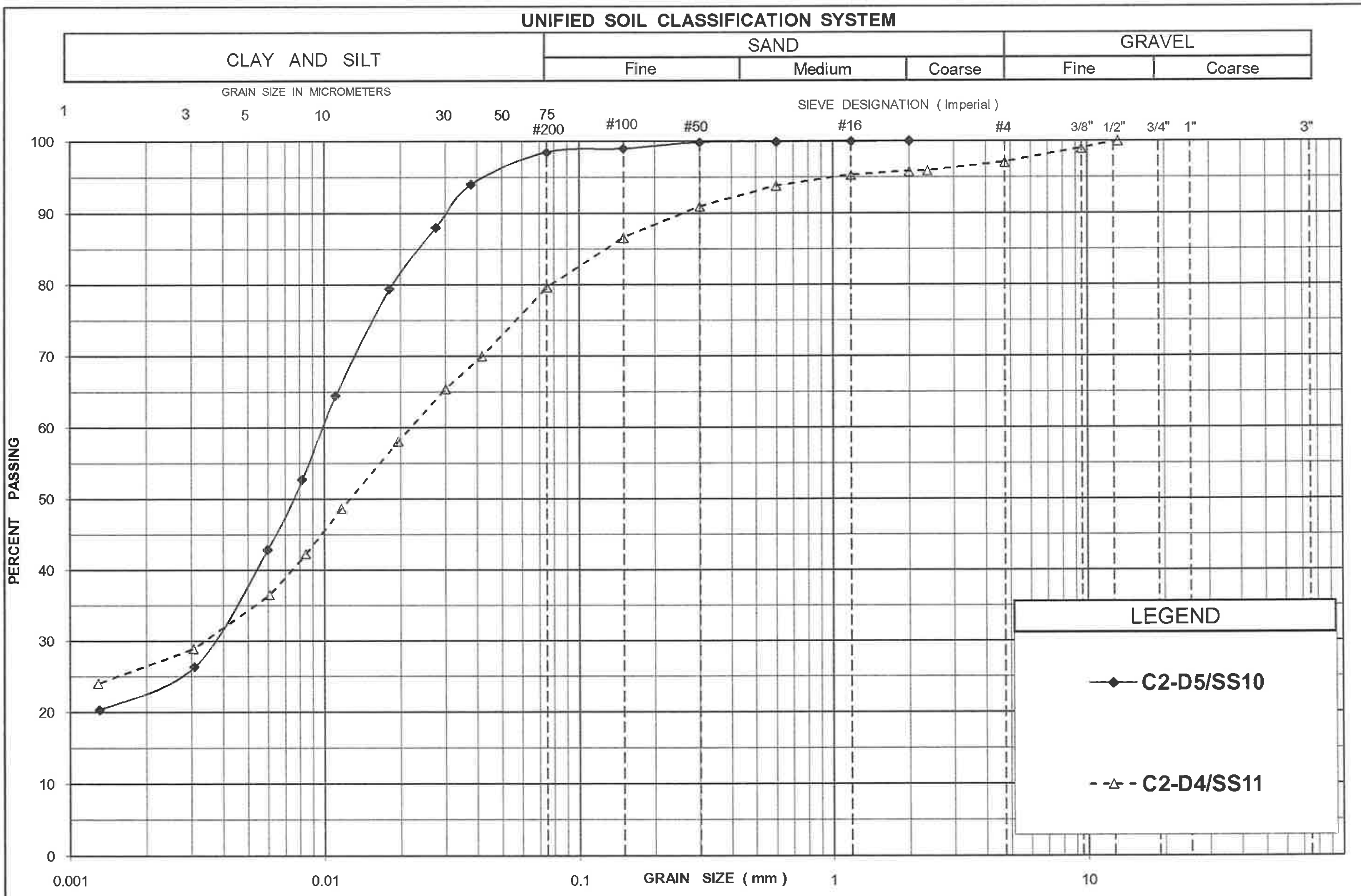
FIGURE No. B-3	
REF. No.	SPT 1211D
DATE	SEPTEMBER 2008



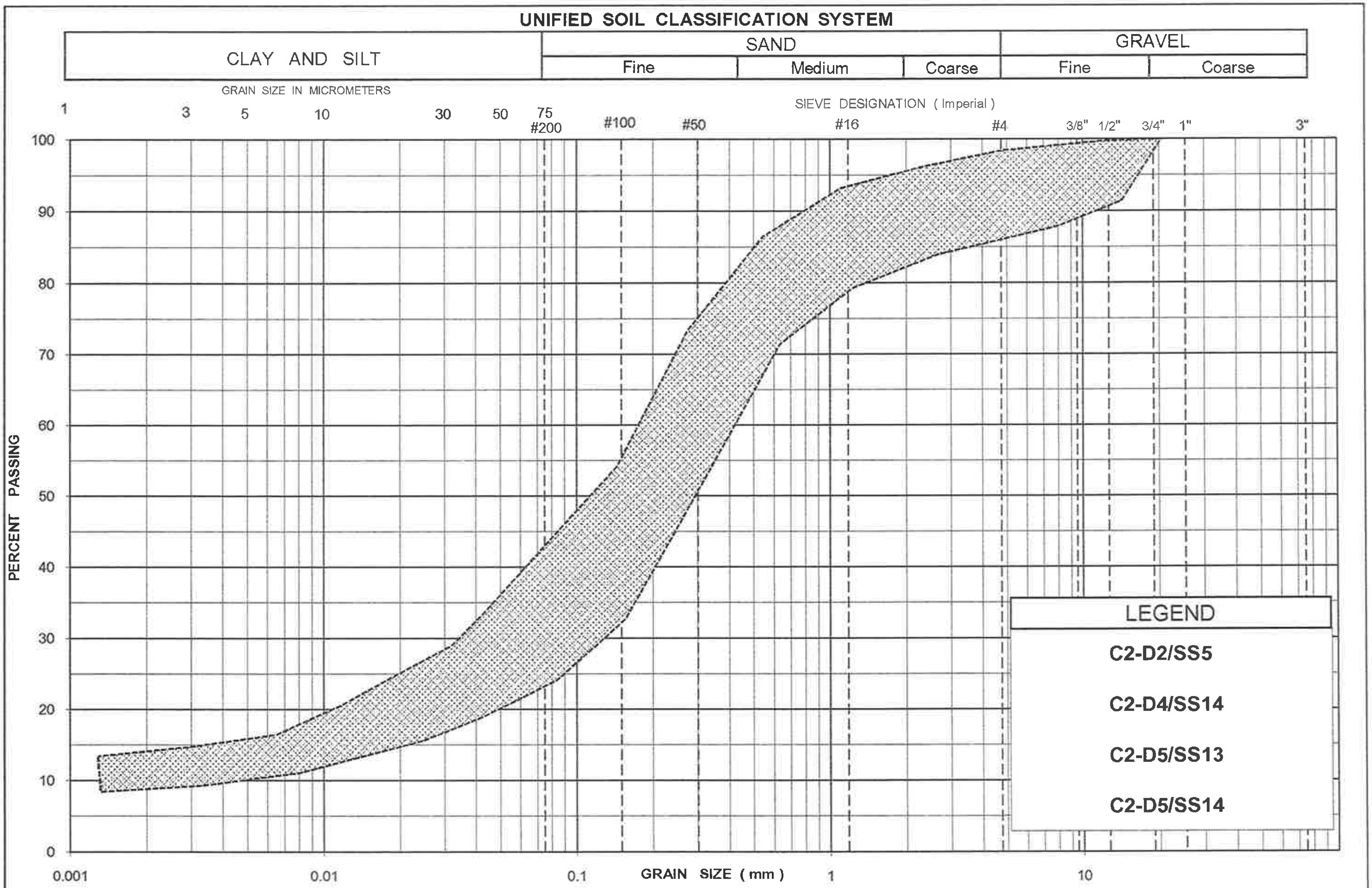
SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION GRAVELLY SAND, some silt	FIGURE No. B-4 REF. No. SPT 1211D DATE SEPTEMBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION FINE SAND, trace silt	FIGURE No. B-5 REF. No. SPT 1211D DATE OCTOBER 2008



SHAHEEN & PEAKER A Division of Coffey Geotechnics, Inc.	GRAIN SIZE DISTRIBUTION SILT with thin clay seams	FIGURE No. B-6 REF. No. SPT 1211D DATE OCTOBER 2008



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GRAIN SIZE DISTRIBUTION
SILTY SAND, trace clay & gravel

FIGURE No. B-7
REF. No. SPT 1211D
DATE OCTOBER 2008

Appendix C

Site Photographs

Photograph 1. Culvert C10, Station 18+175, south side (looking west)

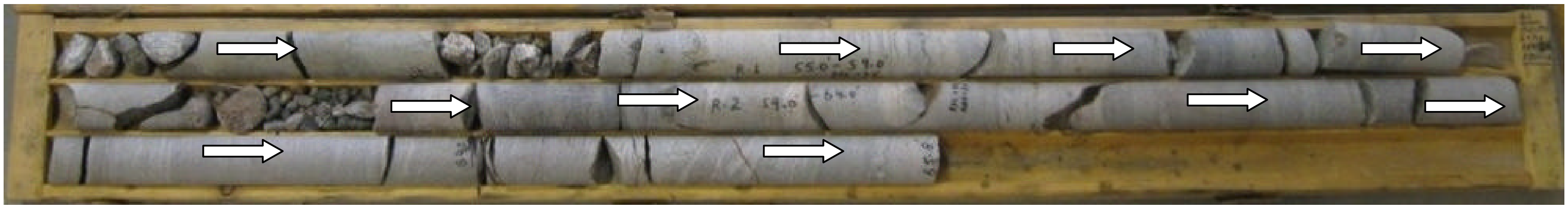
Photograph 2. Culvert C10, Station 18+175, south side

Photograph 3. Culvert C10, Station 18+175, north side

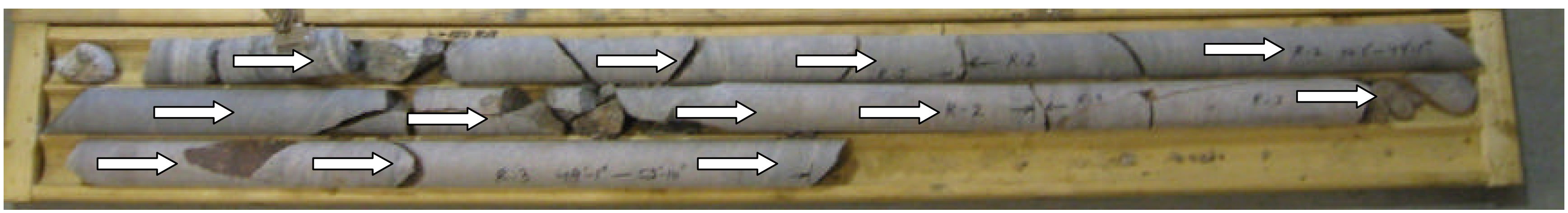
Photograph 4. Culvert C10, Station 18+175, north side(looking east)

Appendix D

Rock Core Photographs



BH C2-1



BH C2-3

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 $\overline{N}$.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

C_u (kPa)	0 – 12	12 – 25	25 – 50	50 – 100	100 – 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

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

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

ROCKS ARE DESCRIBED BY THEIR COMPOSION AND STRUCUTRAL FEATURES AND/OR STRENGTH.

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
σ_l, σ_2, σ_3	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
ε_1, ε_2, ε_3	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	$-^{o}$	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	$-^{o}$	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
i_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
i_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
i	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
i_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = (W_L – W_L)	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
i_{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
i'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION DESIGN REPORT
BASTIEN CREEK CULVERT (C10)
STATION 18+175
HIGHWAY 17, CAMERON TOWNSHIP
MATTAWA, ONTARIO
G.W.P. 173-98-00
AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-128

Prepared For:

D. M. WILLS ASSOCIATES LIMITED

Prepared by:

SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.

Project: SPT1211D
November 24, 2008

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FOUNDATION DESIGN REPORT
BASTIEN CREEK CULVERT (C10)
STATION 18+175, HIGHWAY 17
CAMERON TOWNSHIP, MATTAWA, ONTARIO
W.P. 5079-05-01; AGREEMENT NO. 5006-E-0040

5. DISCUSSION AND RECOMMENDATIONS

At Station 18+175, Bastien Creek flows through an existing culvert (designated as Culvert C10 for the purposes of this project) under Highway 17, some 13.6 km east of the junction of Highway 17 with Highway 533 near Mattawa. The culvert is a 40.2 m long, structural plate pipe arch (SPPA) culvert, with a width of 3510 mm and a rise of 2290 mm.

According to data supplied to us, the invert of the existing culvert is at El. 173.1 m at the inlet on the south side, dropping to El. 172.7 m at the outlet on the north side.

Boreholes C2-1, C2-2 and C2-3, which were drilled in the immediate vicinity of the existing culvert, show the presence of a stratified silt deposit at or a short distance below the granular bedding elevation (which should normally be present under the culvert). The deposit has thin clay interbeds. The unit is 3.0 to 4.4 m thick and N-values recorded range from 2 to 11 blows/0.3 m. The silt deposit is underlain by silty sand with traces of clay and gravel (probable till) which is further underlain by gneiss bedrock some 7.4 to 10.1 m below the silt deposit. N-values recorded in the basal silty sand deposit range from 3 to in excess of 50 blows/0.3 m.

5.1 REHABILITATION OF THE EXISTING CULVERT

To rehabilitate the existing culvert, it is proposed to pave the corroded invert of the culvert. We understand that this will be accomplished by installing cast-in-place concrete invert paving, reinforced with wire mesh. The thickness of the concrete paving will be about 150 mm. Based on the above, the additional stresses on the surface of the silt subgrade would be less than 6 kPa.

Based on these assumptions and the information obtained from the boreholes, the foundation settlements due to the additional stresses imposed by the rehabilitation should not exceed 8 mm. This amount of settlement would not adversely affect the performance of the SPPA culvert.

The water flowing will need to be diverted away from the existing culvert in order to effect the implementation of the ‘invert paving’ to rehabilitate the culvert. This can be accomplished by installing a temporary culvert under the highway. This would however either require traffic closure or, alternatively, shoring so that one half of the temporary culvert is first constructed (while diverting the traffic to the other half) and then the other half. Traffic closure will unlikely be acceptable to MTO, while the latter (i.e. staged construction by shoring) would not be cost-effective.

One alternative to temporary culvert construction would be to divert the water away from the existing culvert to a temporary holding area and pumping the collected water across the highway to the downstream side of the Creek. For such a scheme to be successful, however, the construction must be carried out during a dry season. It is generally up to the Contractor to devise a method to achieve a suitable diversion of the water away from the existing culvert and the subsequent disposal of the water to the downstream side.

In addition, dewatering of the subgrade may be necessary to effect the construction, since at the time of our investigation the groundwater level at the site was found to be above the existing culvert invert elevation. Depending on the site conditions, the dewatering may consist of gravity drainage and pumping from properly filtered and strategically placed deep and shallow sumps. However, this method is likely to be successful to lower the water level by only up to about 0.8 m. To effect deeper draw-down of the water level, more sophisticated methods such as well points would be required. Again, it is normally up to the contractor to choose a proper dewatering method to achieve suitable working conditions at the site. We recommend however that the contractor be asked to submit their method of diversion of water and that of dewatering to the CA for information purposes.

We also recommend that the contractor be made aware of the presence of cobbles and boulders (and possibly rock fill) in the embankment fill, as well as the presence of cobbles and boulders in the overburden. Furthermore, the contractor should be made aware that the silt deposit underlying the culvert is a dilatent material which can easily be disturbed, especially in the presence of water. The disturbed and dilated soil can undergo settlements. For this reason, the site should be properly dewatered and the vibrations which would disturb the silt soil (including foot-traffic) should be kept to a practical minimum during the construction.

5.1.1 EROSION PROTECTION

We recommend that the existing culvert be evaluated for the sufficiency of the existing erosion and scour measures and if observations show that they are deficient or if the concrete paving is expected to adversely affect erosion and scour potentials, further measures may be necessary. The following is a discussion of possible erosion measures.

Erosion and scour protection should be provided at the culvert inlet and outlet (including the slopes and sides). The erosion/scour protection should be designed by a specialist River Engineer/Scientist (as erosion and scour largely depend on the velocity of water in the watercourse and its regime) who is familiar with the findings of this report. The following are some general suggestions, considering that below some probable organic and alluvial deposits at the creek level, the boreholes indicate that the native soils can be expected to consist of fine sands and more likely silts. The fine sands and, particularly the silt, are erodible soil types.

We recommend that concrete cut-off (apron) wall be constructed both at the inlet and the outlet to prevent seepage beneath and around the culvert, especially through the granular bedding and granular backfill around the culvert, if these are not present beneath and around the existing culvert. Beneath the culvert, the concrete cut-off wall should extend to a suitable depth (e.g. below any possible scour depth). Consideration may also be given to an impervious seal at the inlet and outlet.

At the inlet, consideration may also be given to the use of a clay seal. The purpose of the clay seal is to ensure that water flow is channeled through the culvert and does not seep through the backfill around the structure and from beneath the structure. The clay seal should therefore be continuous and is typically 0.6 m thick. It should comply with the material specifications given in OPSS 1205. It should be extended around the culvert from at least 0.5 m above the high water level in the creek down to the channel bed and up the other side in a continuous manner. It should be ensured that it extends to cover all the granular backfill materials to prevent any seepage through them. Typically, the clay seal is protected by laying a 0.6 m thick rock protection over it. The clay seal would generally be extended at about 8 m beyond the inlet.

At the outlet as well as at the inlet (if clay seal is not used), in addition to the concrete cut-off wall and/or impervious seal or in conjunction with these, a 0.6 m thick rock protection, consisting typically of 300 mm size rock can be considered. As the subgrade may consist of silty sand soils, a layer of granular or man-made filter material should be used. This would generally be extended about 8 m along the channel and the sides (to at least 0.3 m above the high water). The granular filter material (where necessary) underlying the rock protection can consist of a suitable granular material such as Granular ‘A’. Alternatively, a suitable geotextile can be used underneath the rock fill, in lieu of the granular filter material.

Another reference for consideration is OPSD 810.010 Rip-Rap Treatment for Culvert Outlets.

5.2 DETOUR EMBANKMENT

We understand that a temporary (detour) embankment may possibly be built on the south side of the existing Highway 17, between Stations 18+100 and 18+270, to be used during the rehabilitation of the existing culvert.

The existing road cross sections supplied to us indicate that between Stations 18+100 and 18+140 and between Stations 18+190 and 18+270, the maximum height of the existing

Highway 17 embankment is 3 m. Based on the borehole data, up to 3 m high embankments for the detour embankments from Station 18+100 to Station 18+140 and from Station 18+190 to Station 18+270 should not cause foundation instability. This is based on the assumption that the detour embankment will be built as per standard MTO procedures and using suitable earth fill for the construction of the embankment with 2H:1V side slopes.

In between Stations 18+140 and 18+190, the height of the existing embankment rapidly increases to beyond 4 m, reaching a height of up to 7.0 m above the o.g. level at Station 18+169, near the existing culvert location.

We carried out stability analyses to explore the possibility of failure of the detour embankment due to inadequate foundation resistance. The stability analyses were carried out by means of limit equilibrium methods, utilizing the computer programme Slope/W. In our analyses, Bishop's simplified method was utilized, for both short-term (undrained) and long-term (drained) analyses calculations. Cross sections of the existing road, provided to us by D. M. Wills Associates Limited, were utilized for the analyses, as well as the borehole data. Normal 2H:1V side slopes and the use of earth fill were assumed for the make-up of the detour embankments.

Typical analyses results are given in Appendix G. The soil parameters used for the analyses are also shown on the same enclosures.

Based on the results, the maximum embankment height which can safely be sustained is 6.5 m. We therefore recommend that 6.5 m be the maximum embankment height over and above the (o.g.) existing grades, based on the available data.

Embankment stresses will likely cause a foundation settlement of about 150 mm for a 5.0 m high embankment and of the order of 200 mm for a 6.5 m high embankment. About one-third of this settlement can be expected to occur during construction and another one-third within the next four weeks after the completion of the embankment to its full height. The majority of the remaining settlements would take place within the next eight weeks.

In addition, the settlement of the new embankment fills under their own weight can be expected to occur. If the embankment is constructed to MTO standards, this should not exceed 40 mm. The time rate will depend on the material used for construction. However, if SSM or granular soils are used, about half of this settlement should be completed within one month and the remaining half substantially completed within one year.

Where the embankment height is in excess of 4.0 m we recommend that the detour be left unpaved, applying periodic grading, if and where required. As the detour embankment may cause distortions of the existing pavement on Highway 17, we recommend that rehabilitation of the existing Highway 17 pavement be carried out after the rehabilitation of the culvert and the removal of the detour embankment.

The construction of the detour embankment should be carried out in accordance with MTO standards. The boreholes show that after stripping the organic or otherwise unsuitable subgrade soils (about 0.1 to 0.2 m can be used for preliminary calculating purposes, but will likely be thicker near the existing Creek area), the exposed subgrade at most locations will likely consist of sandy soils (possibly silt at the creek location). If at the creek location thicker unsuitable soils are encountered, these should be removed in a manner so as not to cause an instability of the existing embankment (i.e. stripped in short sections and immediately backfilled).

All organic and otherwise unsuitable soils should be removed within an envelope given by an imaginary slope no steeper than 1:1 from the toe of the proposed embankment. After stripping, the exposed subgrade should be inspected and approved. It should then be compacted, where feasible, from the surface, using a suitable compactor.

Proper benching of the existing embankment slope should be implemented if and where abutting into existing embankments, as per MTO procedures and in accordance with OPSD 208.010.

The materials used for the construction of the embankment fills should consist of approved, acceptable earth fill. The embankment fill should be placed on the approved and properly rolled (where feasible) subgrade in lifts not exceeding 300 mm when loosely placed and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. Embankment construction should be carried out in conformance with SP206S03.

5.3 FROST PROTECTION

Design frost protection for the general area is 2.0 m. A permanent soil cover of at least 2.0 m or its thermal equivalent is therefore required for frost protection. In case of riprap (rock fill), only one half of the rock fill thickness should be assumed to be effective in providing frost protection.

6. CLOSURE

We recommend that once the details of the project are finalized, our recommendations be reviewed for their specific applicability.

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

Shaheen & Peaker
A Division of Coffey Geotechnics Inc.

Zuhtu Ozden, P.Eng.

R. Miranda, P.Eng.

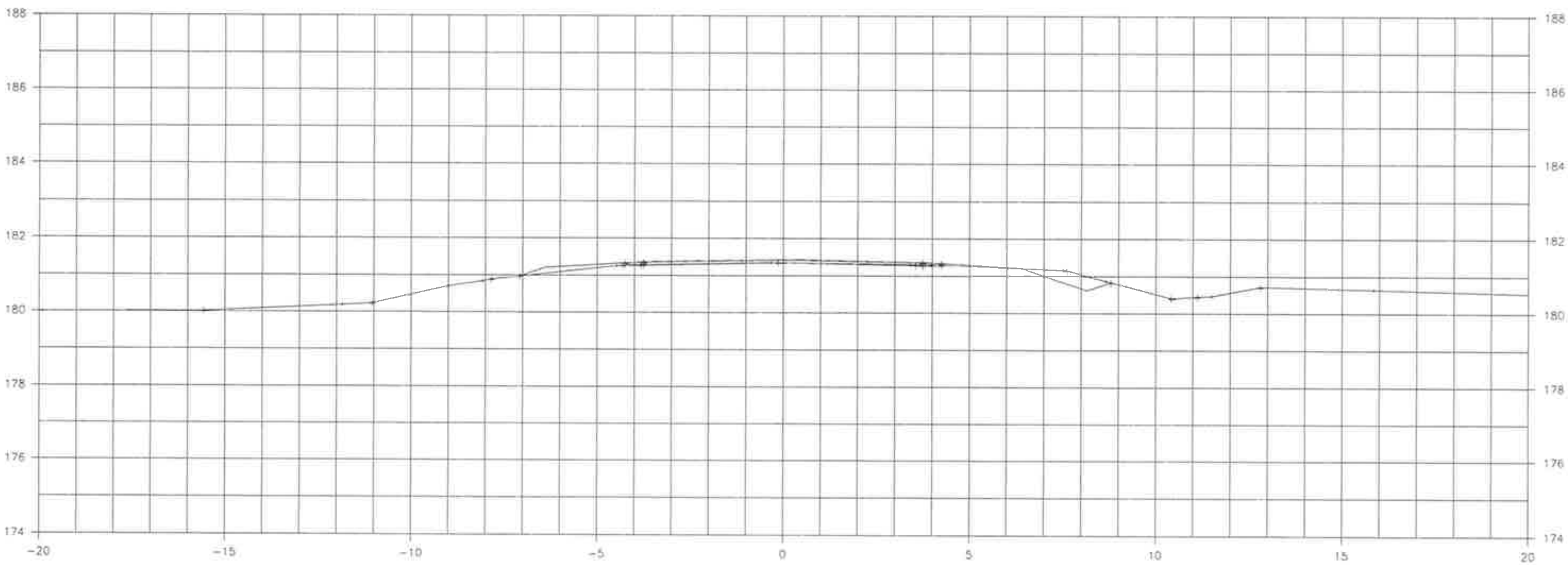
ZO:tr/idrive

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Appendix F

Cross Sections



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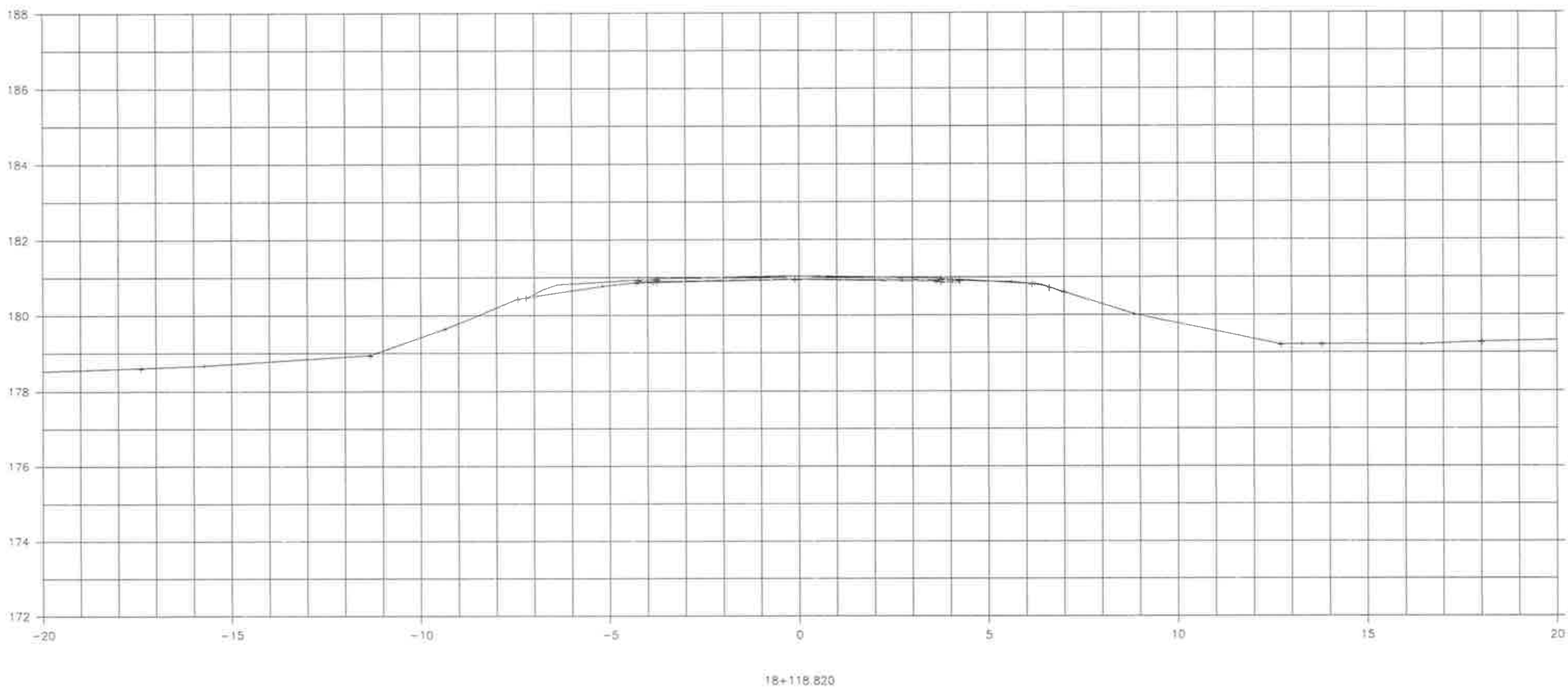

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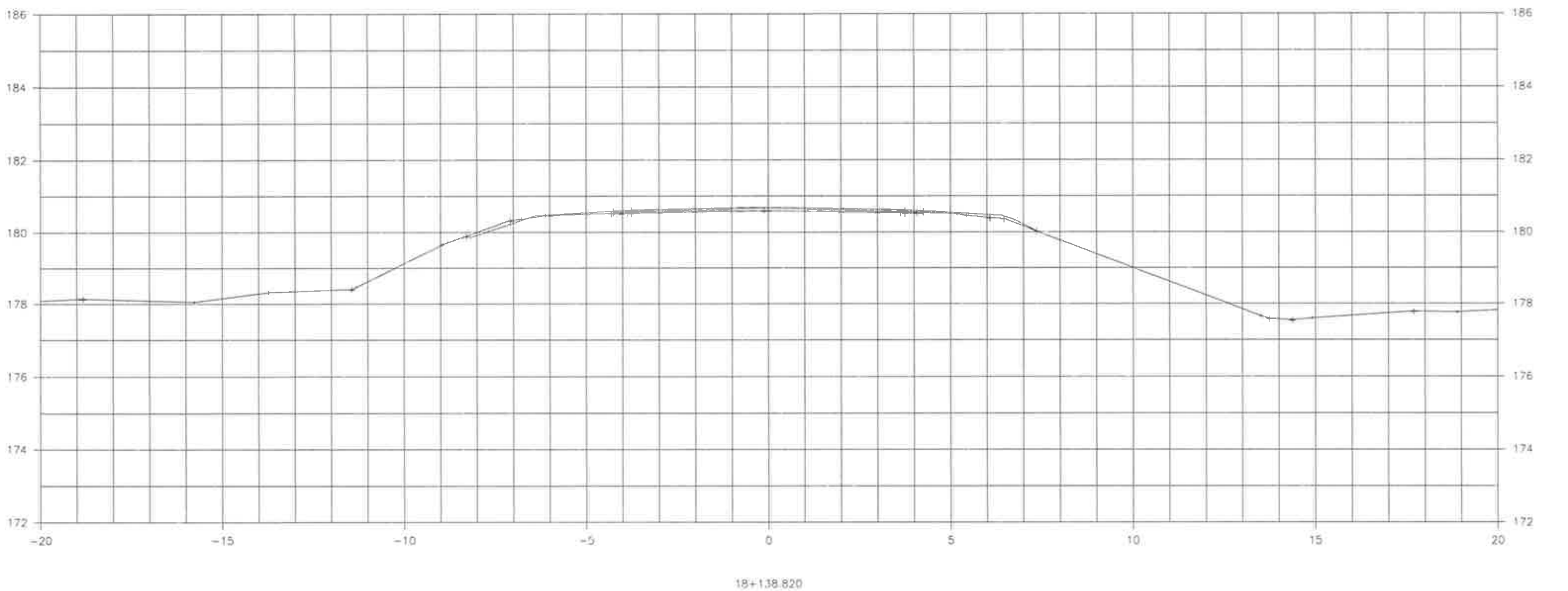
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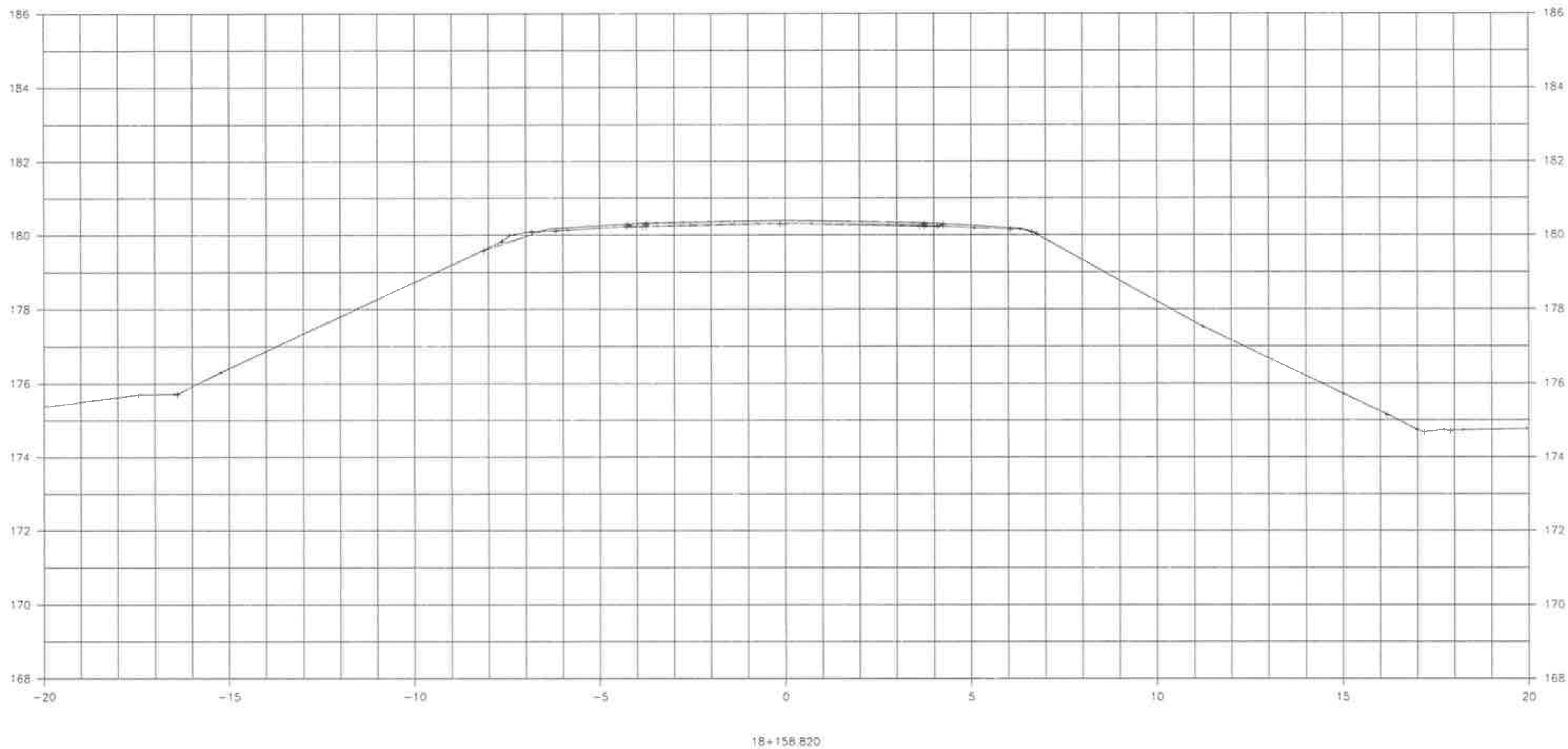
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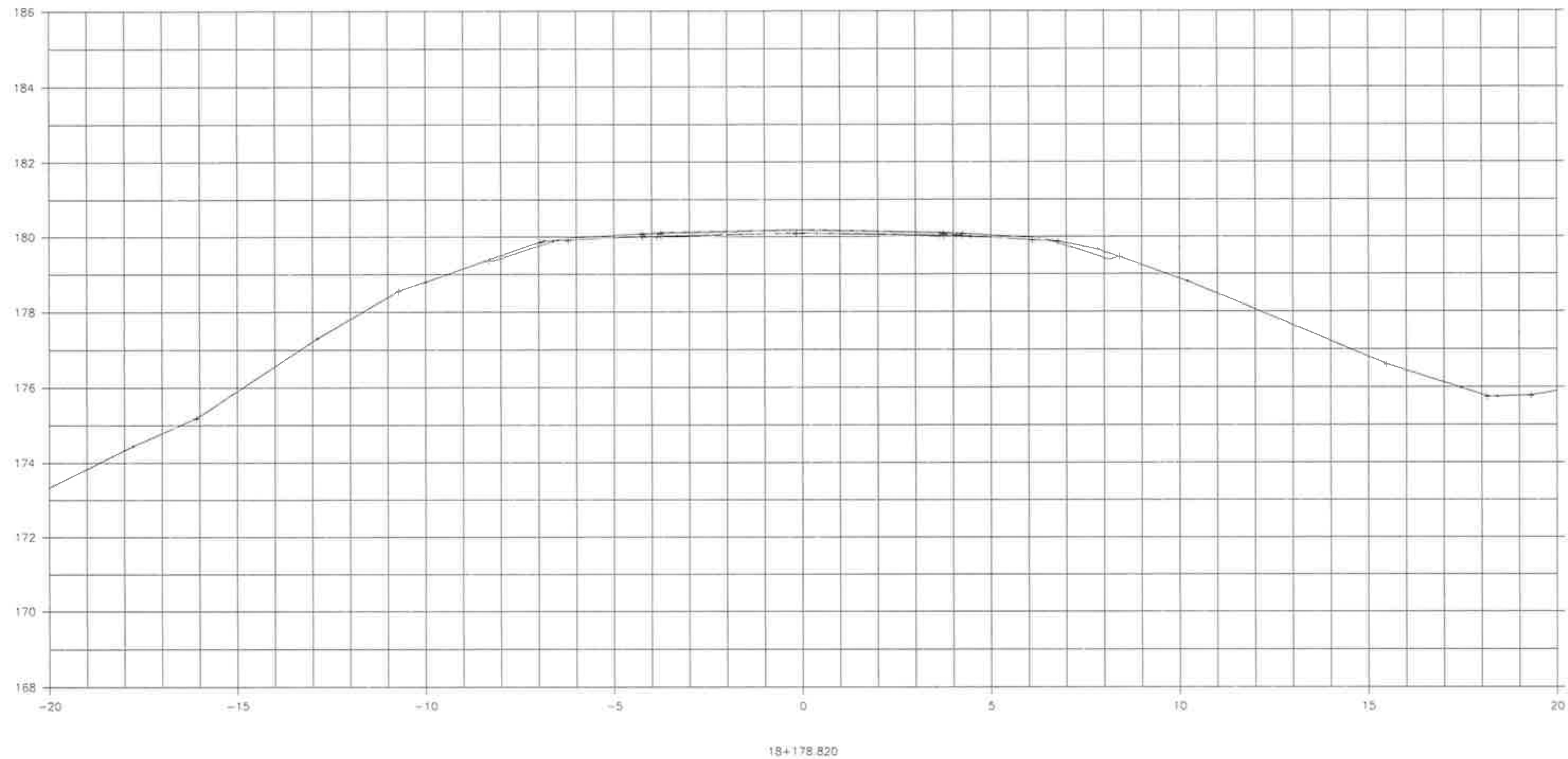
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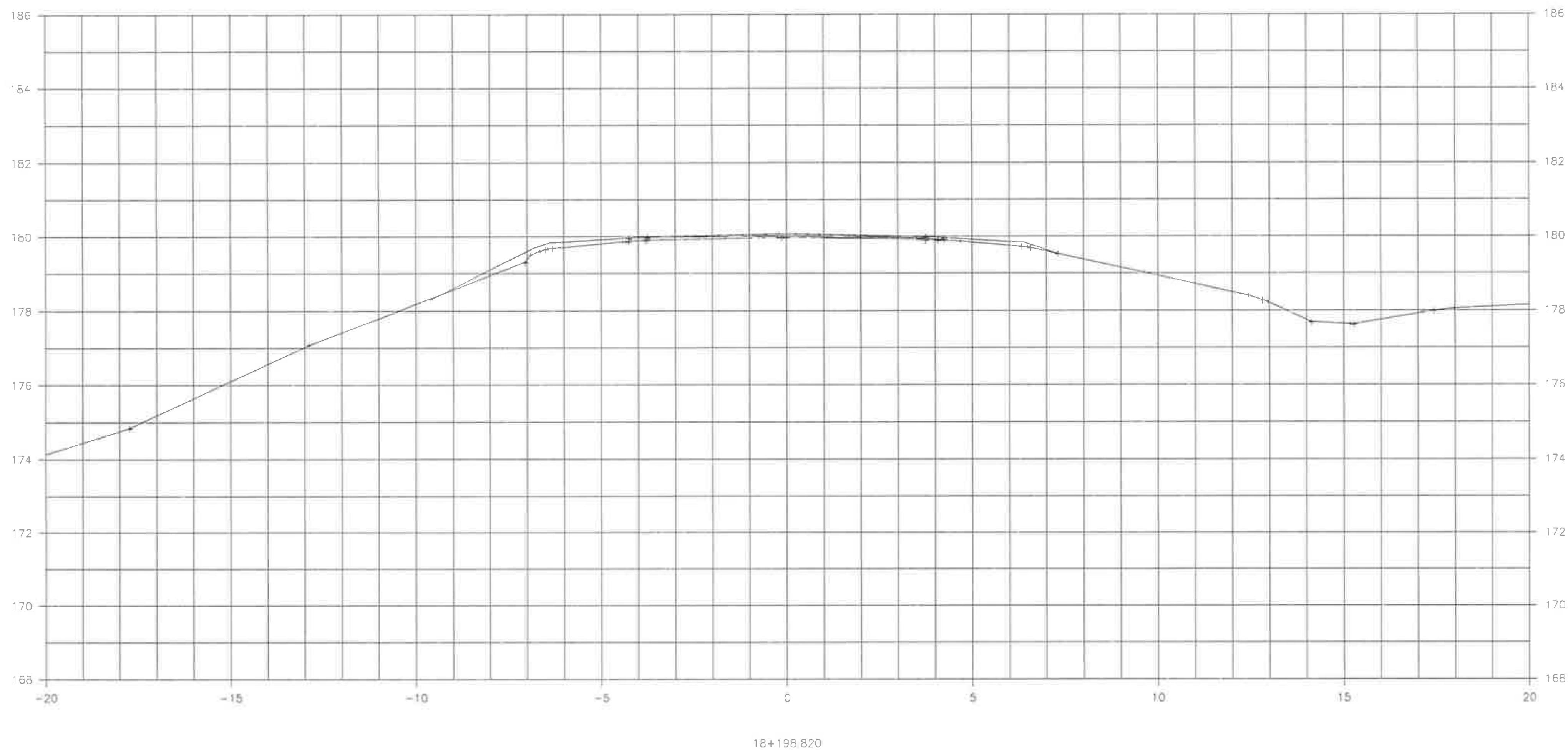
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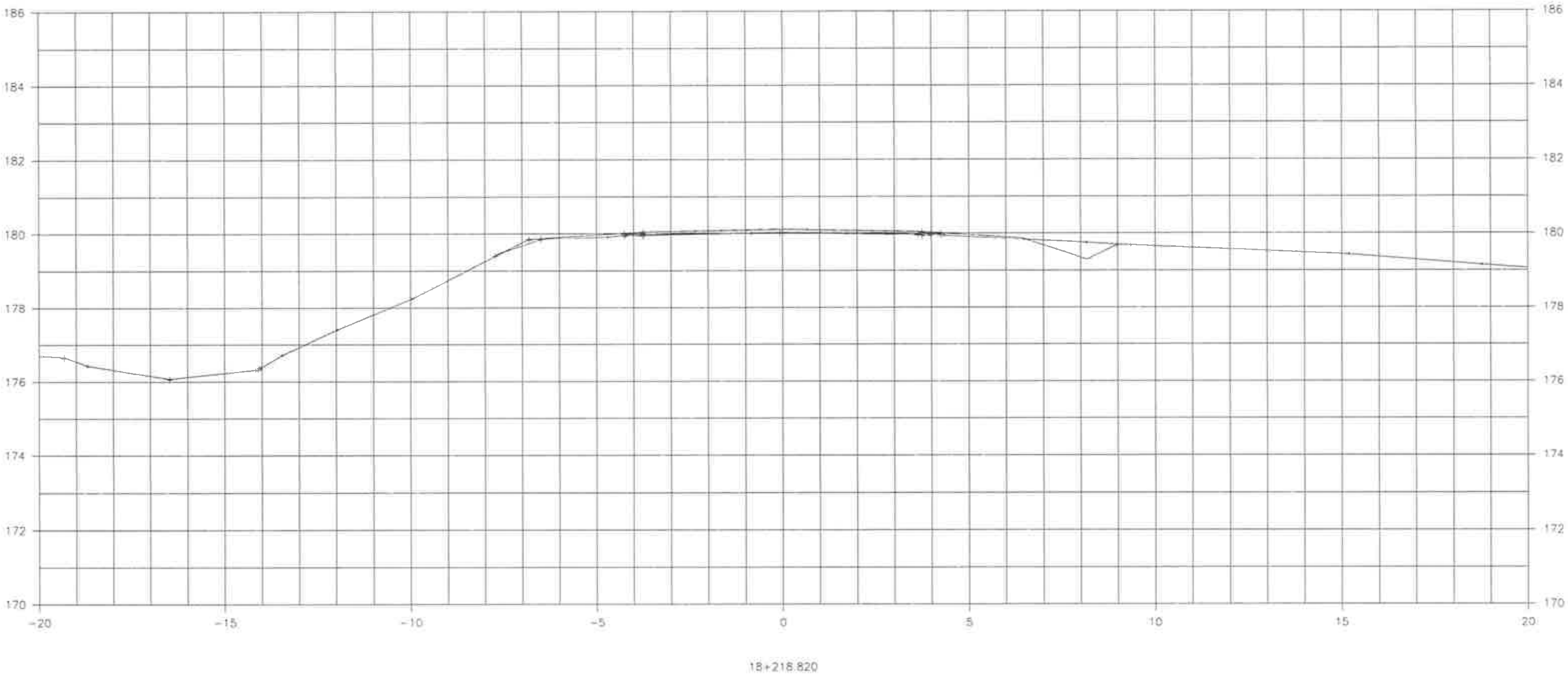
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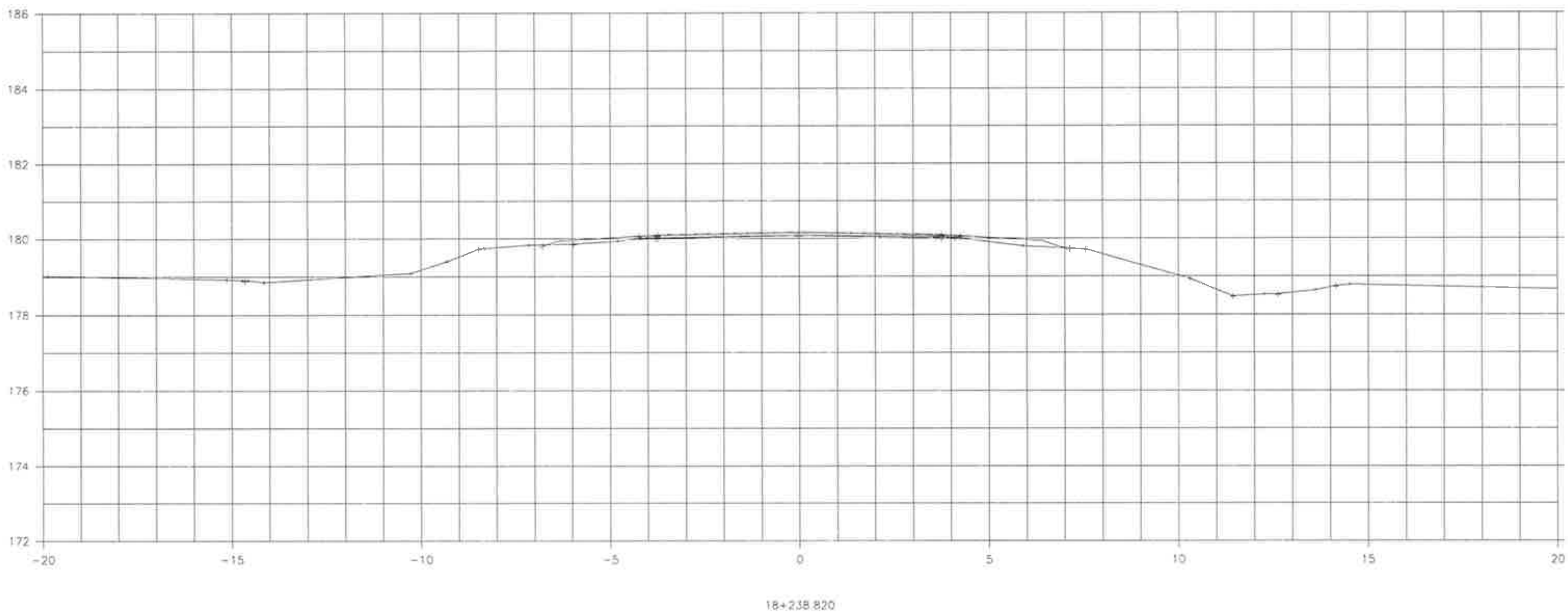
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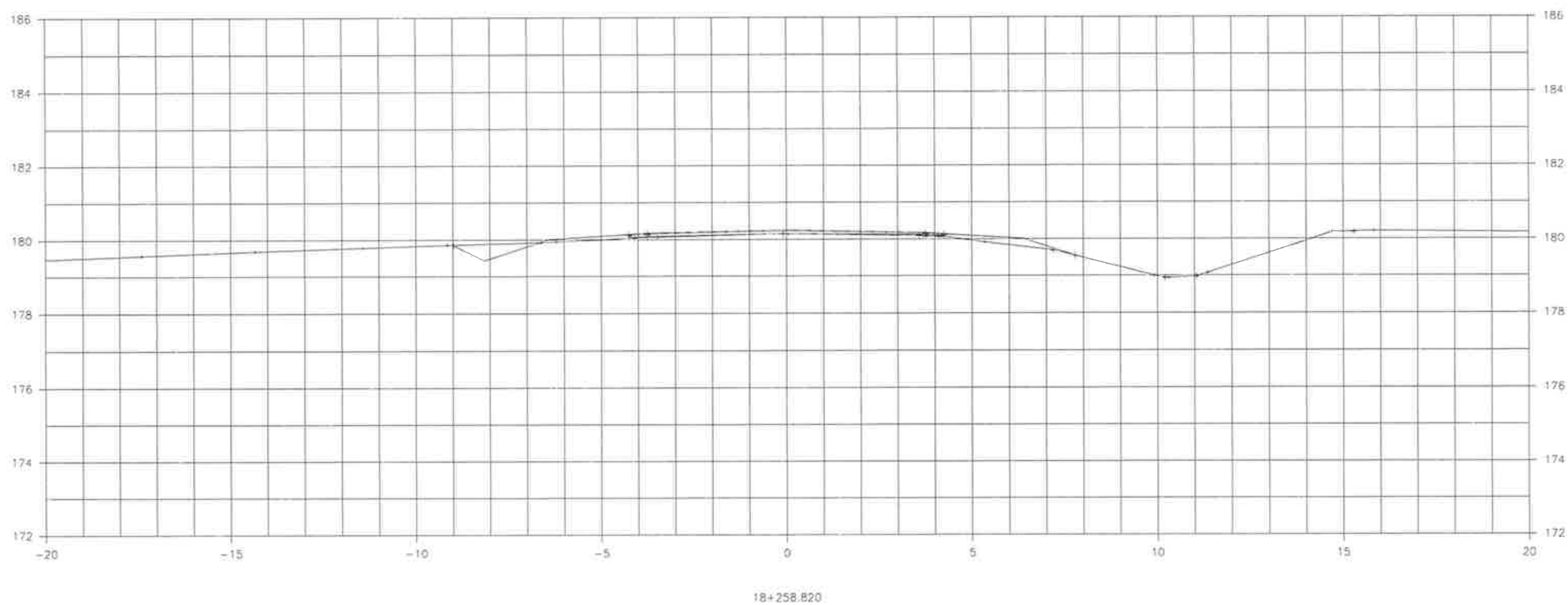
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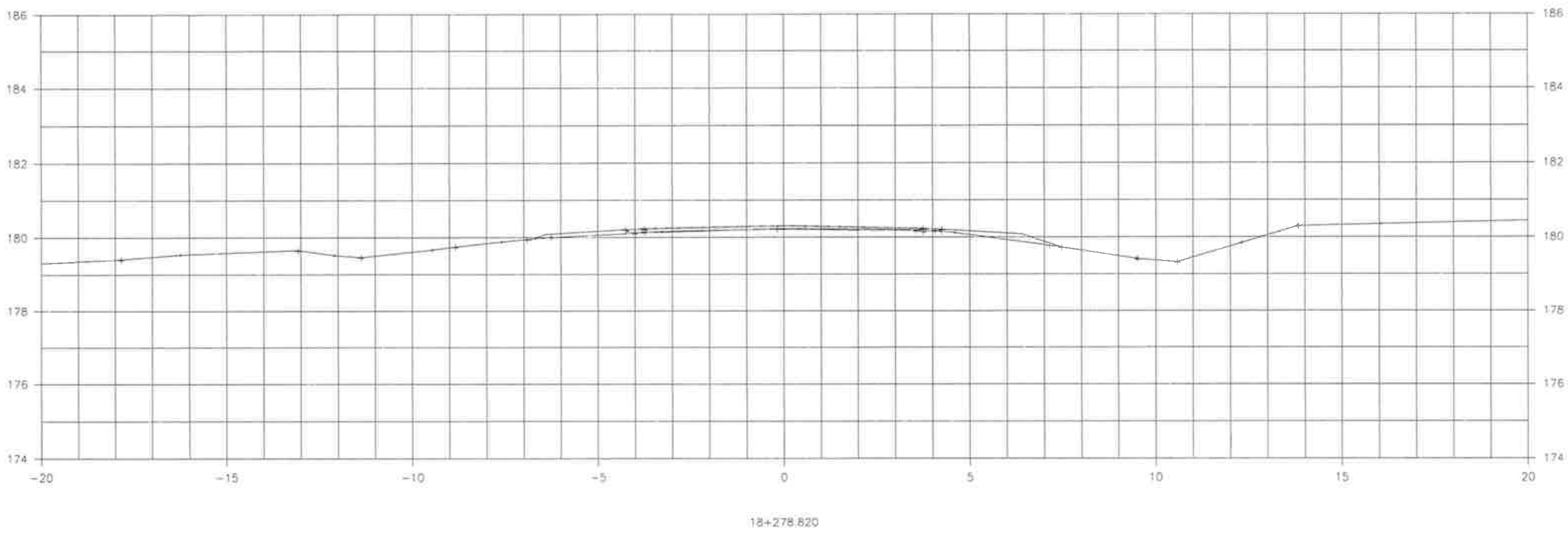
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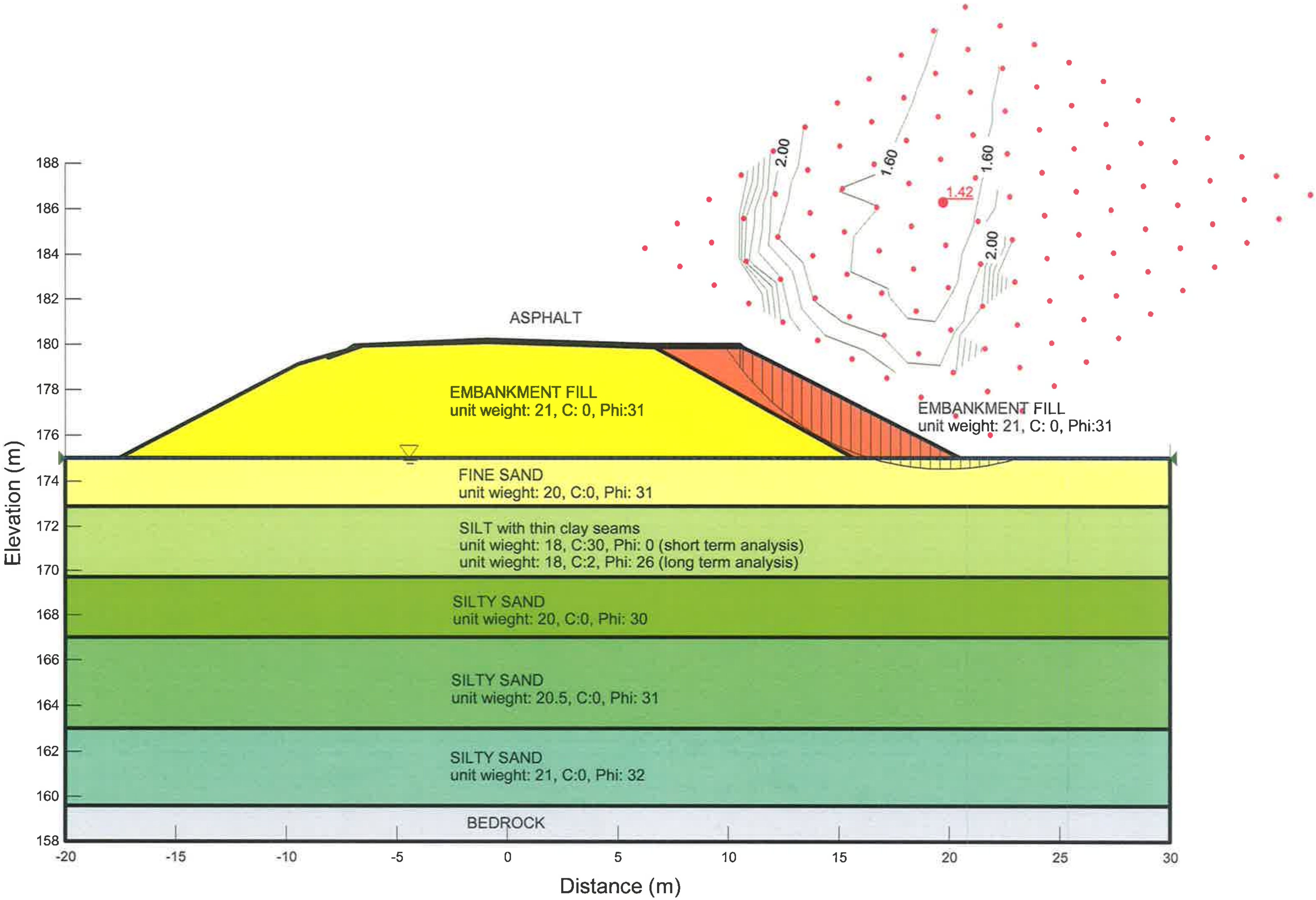
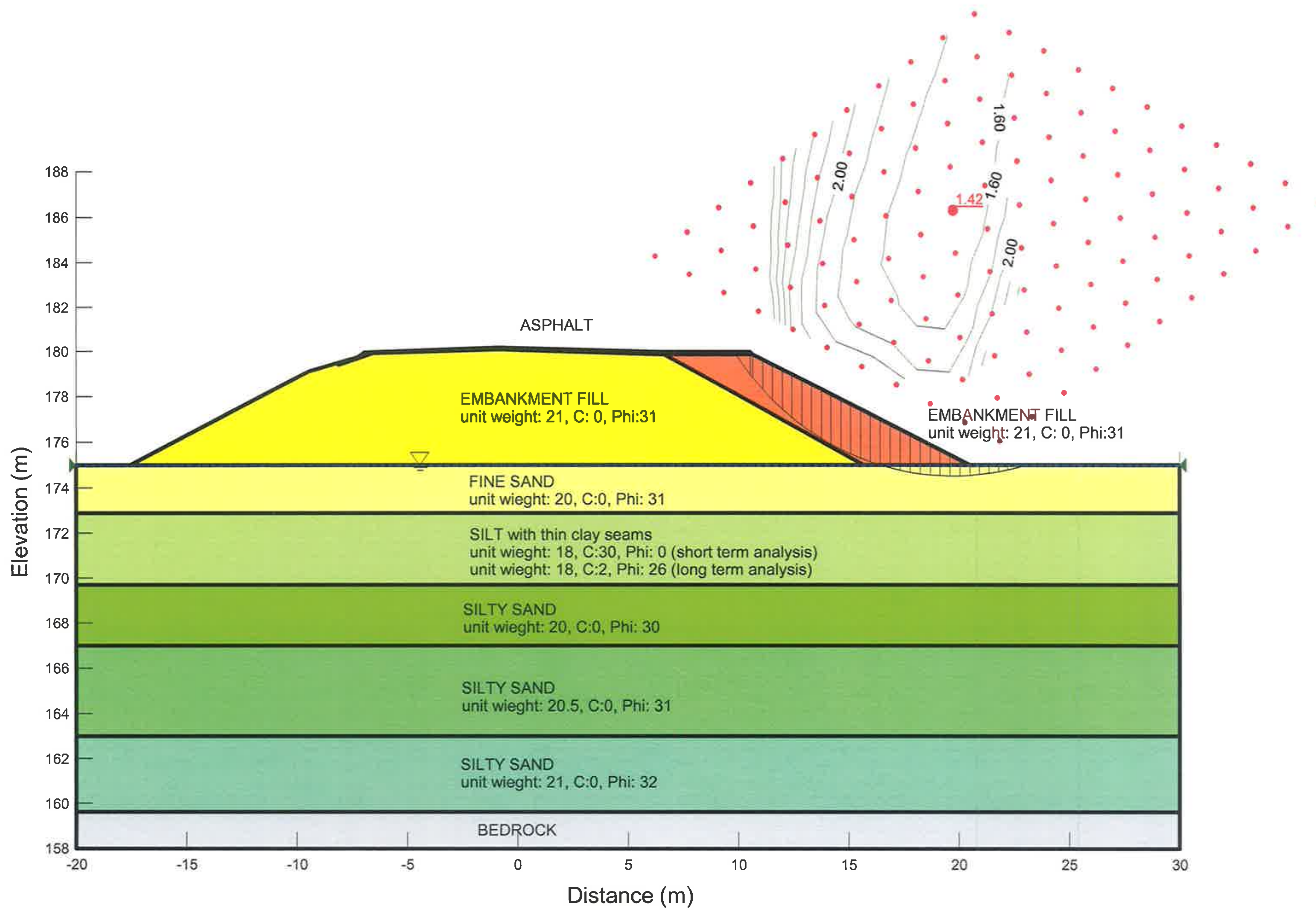
Appendix G

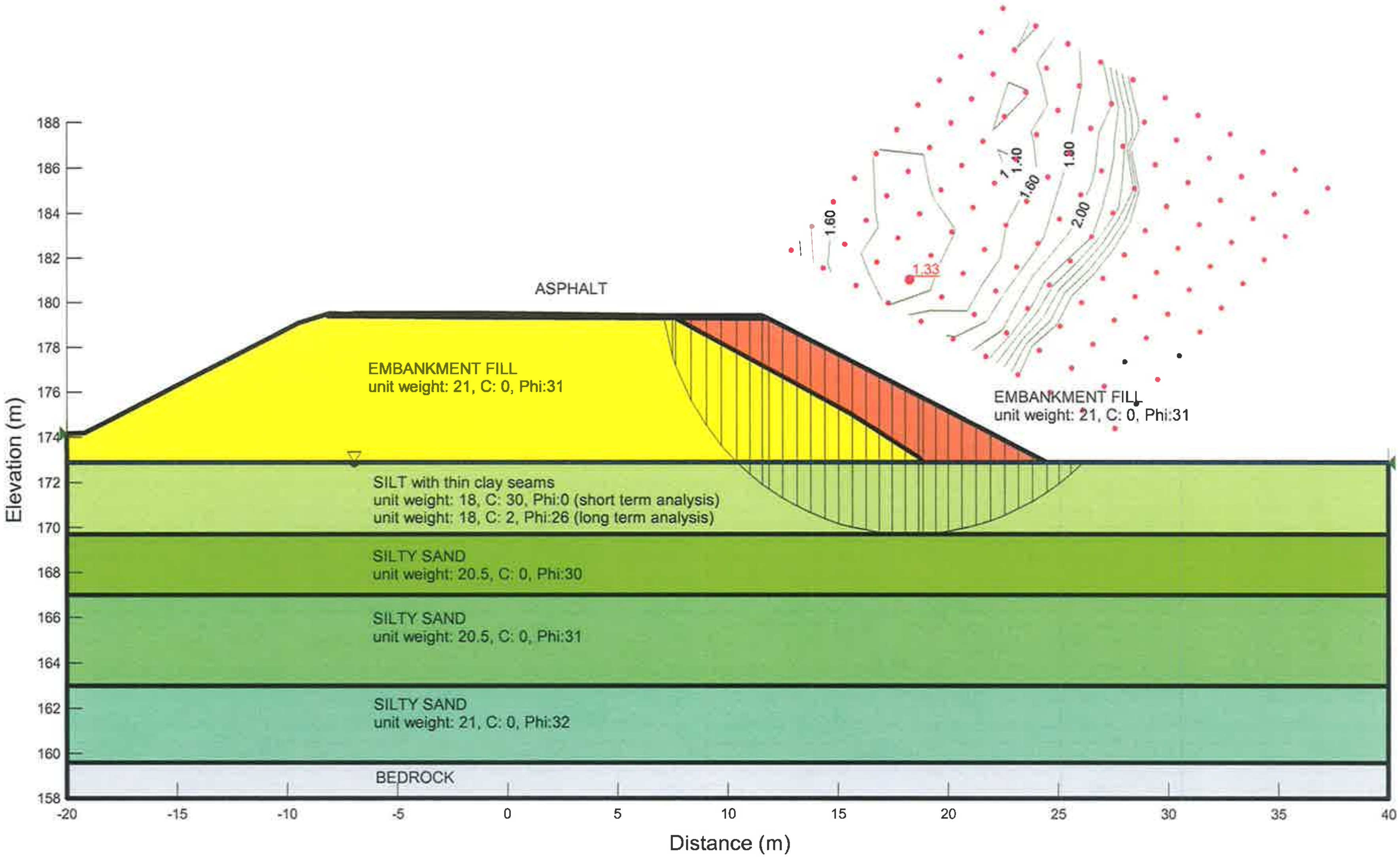
Slope Stability Analysis



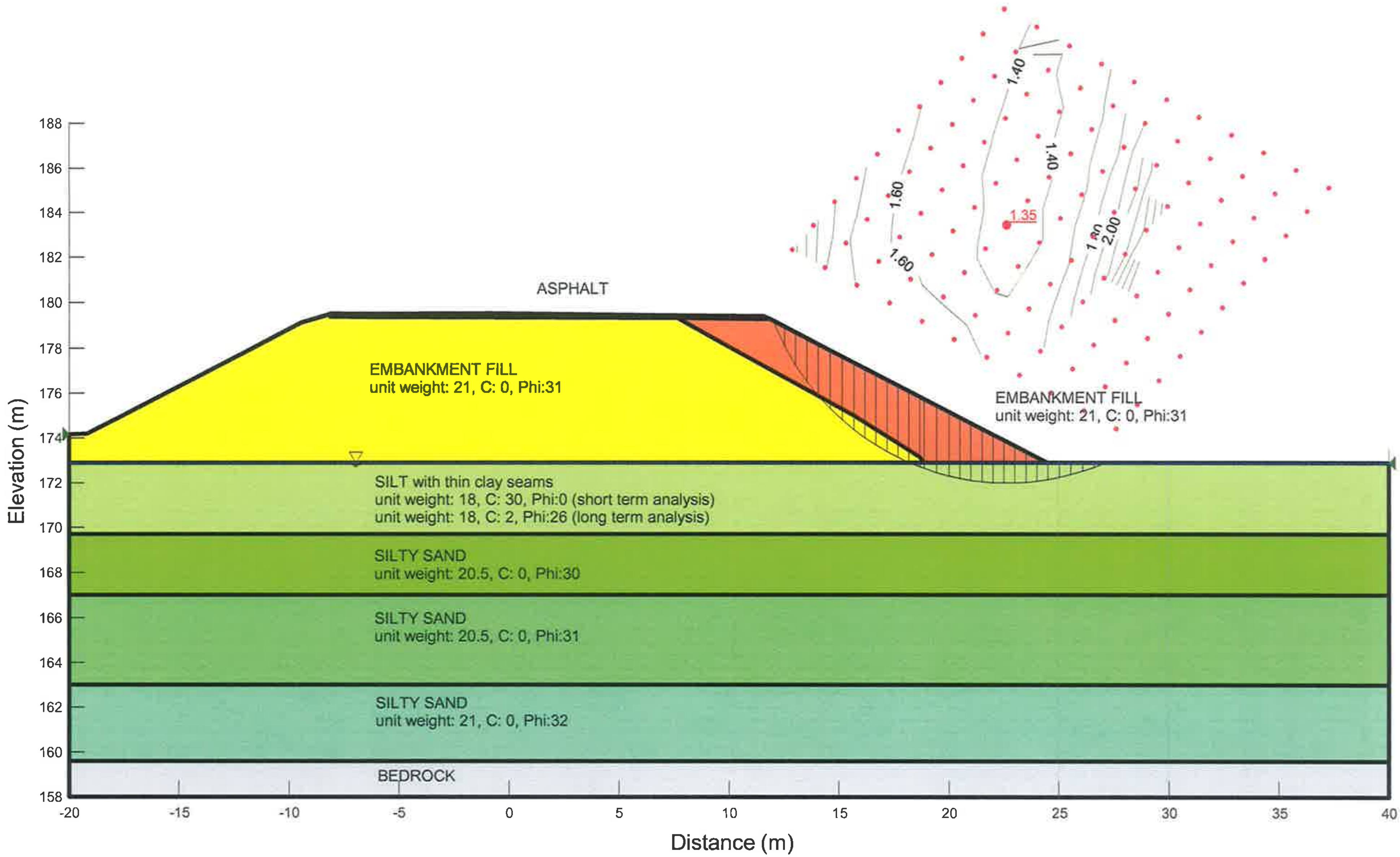
5m high embankment with detour (short term analysis)



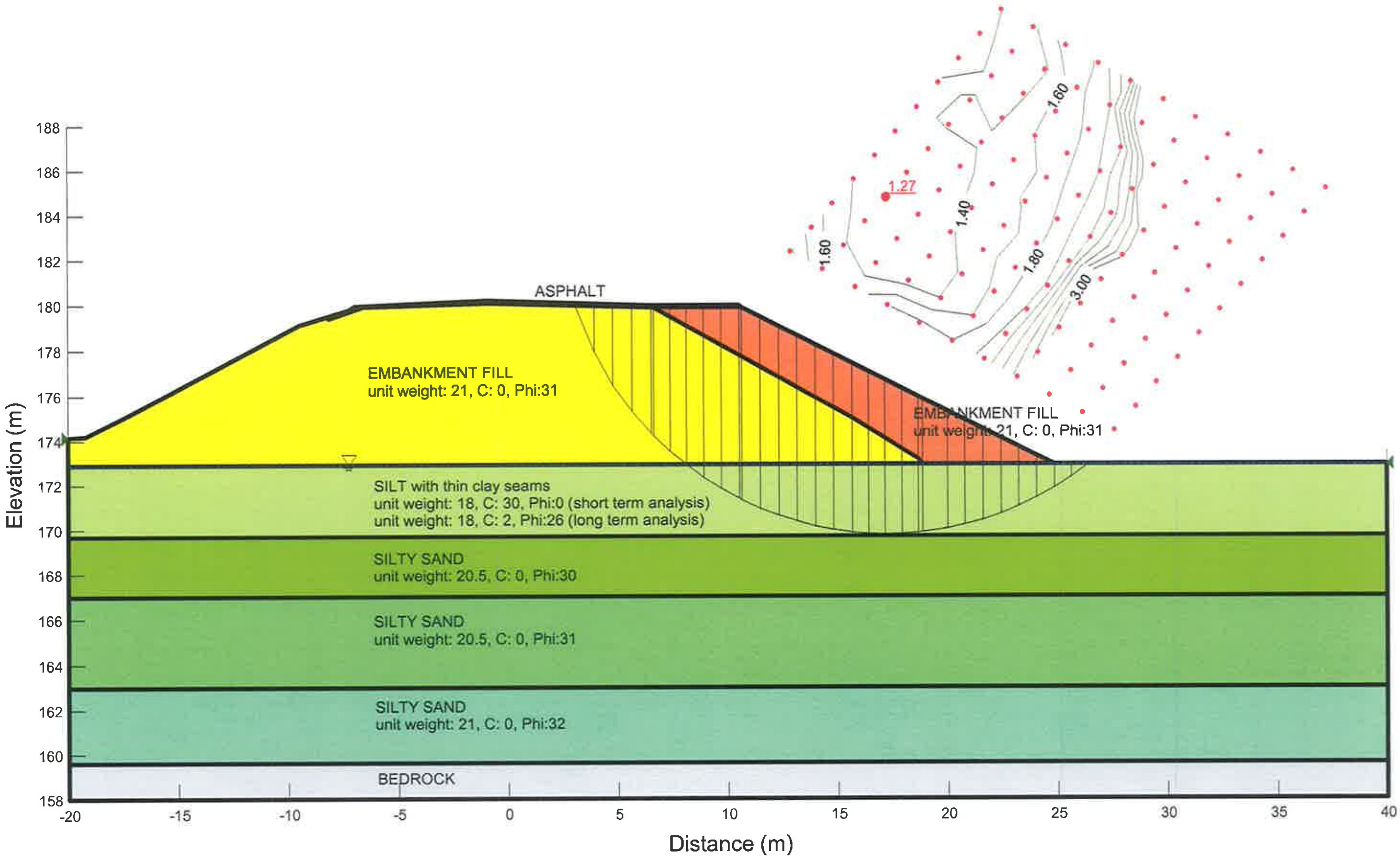
5m high embankment with detour (long term analysis)



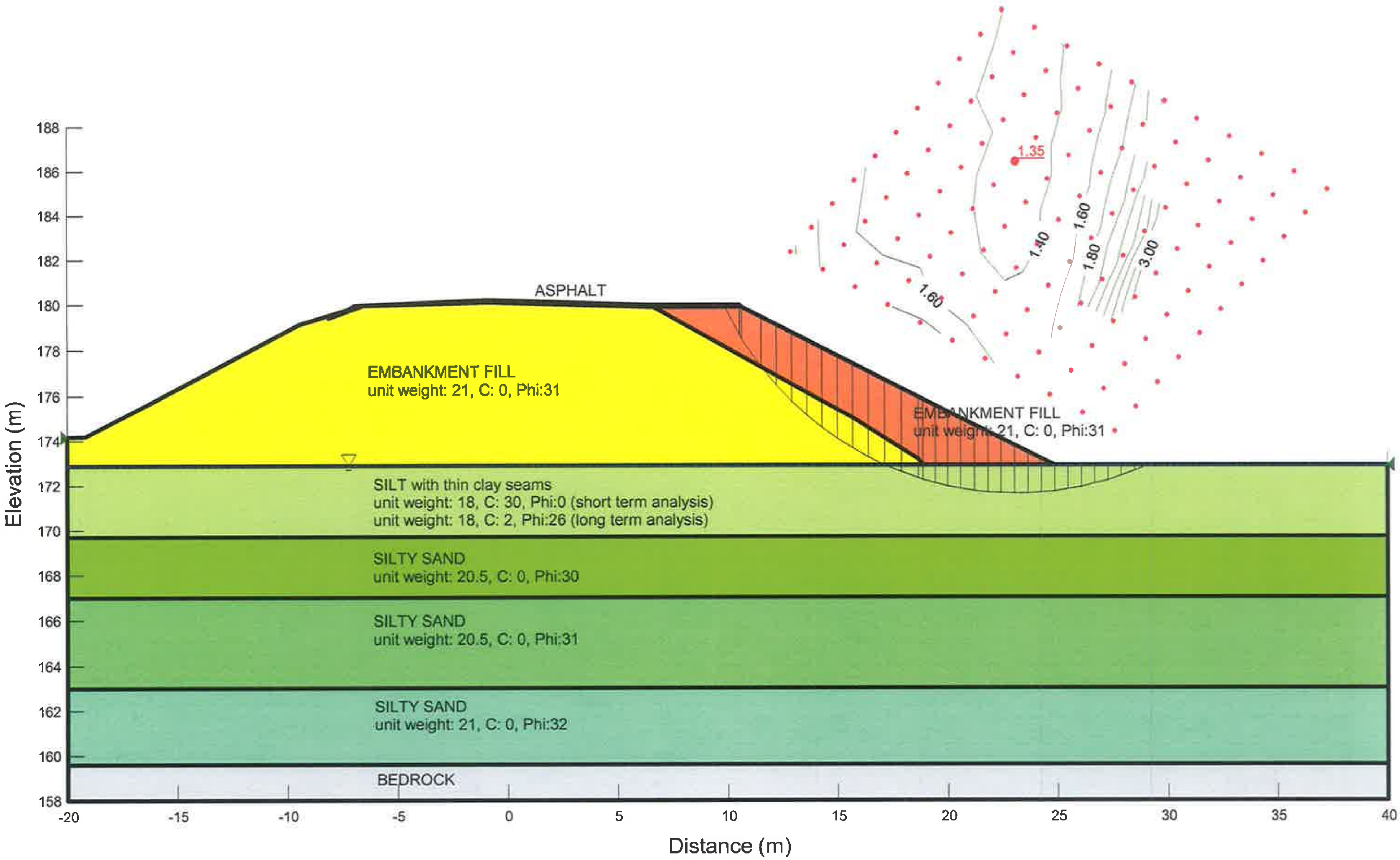
6.5 m high embankment with detour (short term analysis)



6.5 m high embankment with detour (long term analysis)



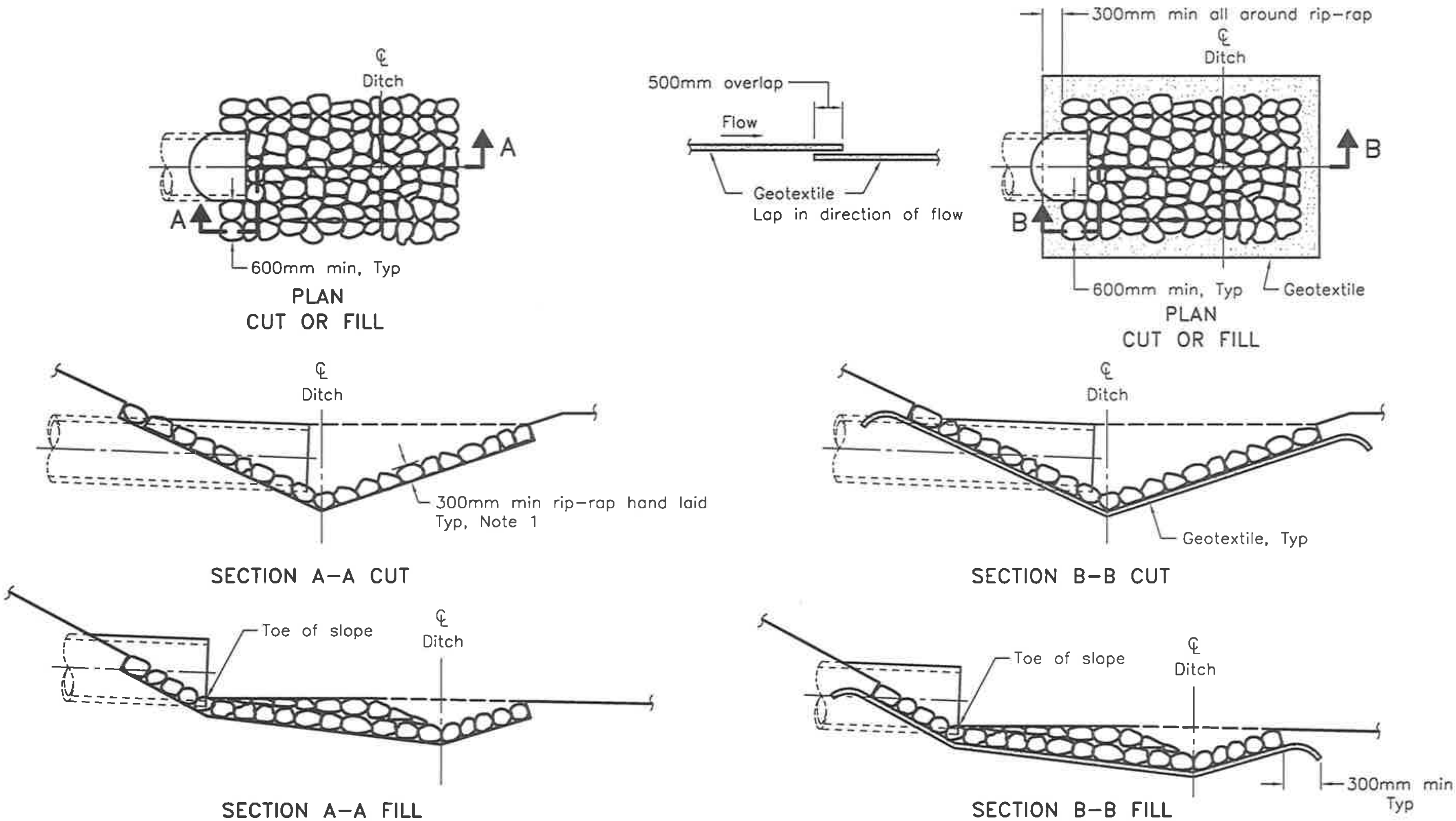
7m high embankment with detour (short term analysis)



7m high embankment with detour (long term analysis)

Appendix H

OPSD



TYPE A – WITHOUT GEOTEXTILE

TYPE B – WITH GEOTEXTILE

NOTES:

1 The thickness of the rip-rap layer shall be at least 1.5 times the rip-rap mean diameter.

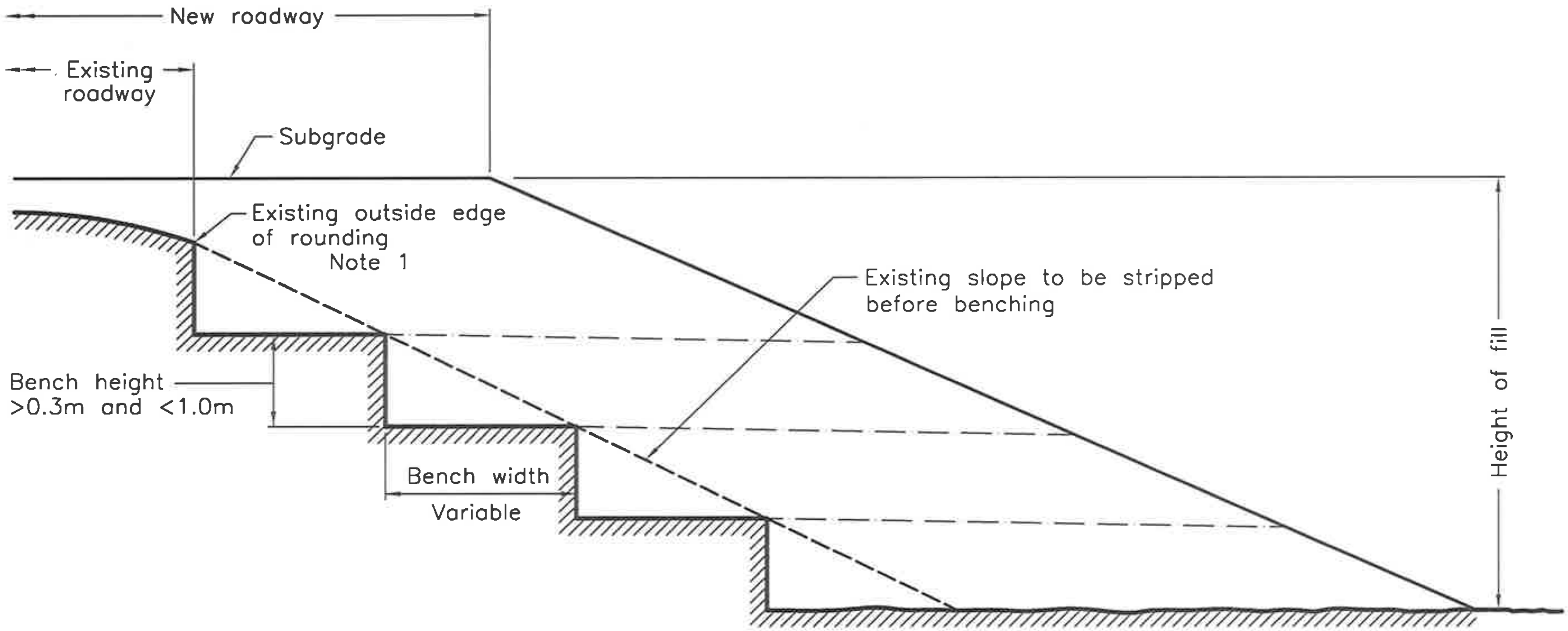
A All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2007 | Rev 1

RIP-RAP TREATMENT FOR SEWER AND CULVERT OUTLETS

OPSD 810.010



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	

Appendix I

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 Shaheen & Peaker, A Division of Coffey Geotechnics Inc. at the time of preparation. Unless otherwise agreed in writing by Shaheen & Peaker, 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. Shaheen & Peaker accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.