coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION AND DESIGN REPORTS HARMONY BEACH ROAD BRIDGE REPLACEMENT, HIGHWAY 7090, TOWNSHIP OF HAVILLAND, DISTRICT OF ALGOMA, ONTARIO G.W.P. 5430-06-00, SITE 38S-345 GEOCRES 41K-83

D.M. Wills Associates Limited

Project: TRANETOB01240AA
May 12, 2010

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

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

May 12, 2010

D.M. Wills Associates Limited
452 Charlotte Street
Peterborough, Ontario
K9J 2W3

Attention: Mr. Andy Staszak

Dear Sir:

RE: Foundation Investigation and Design Reports, Harmony Beach Road Bridge Replacement, Highway 7090, Township of Havilland, District of Algoma, Ontario, G.W.P.5430-06-00, Site 38S-345, GEOCRES No. 41K-83

Please find the attached Foundation Investigation and Design Reports for the above noted site.

For and on behalf of Coffey Geotechnics Inc.

Ramon Miranda, P.Eng.
Manager, Transportation Division

coffey geotechnics
SPECIALISTS MANAGING THE EARTH

FOUNDATION INVESTIGATION REPORT HARMONY BEACH ROAD BRIDGE REPLACEMENT, HIGHWAY 7090, TOWNSHIP OF HAVILLAND, DISTRICT OF ALGOMA, ONTARIO G.W.P. 5430-06-00, SITE 38S-345 GEOCRES 41K-83

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FOUNDATION INVESTIGATION REPORT
HARMONY BEACH ROAD BRIDGE REPLACEMENT, HIGHWAY 7045
TOWNSHIP OF HAVILLAND, DISTRICT OF ALGOMA, ONTARIO
G.W.P. 5430-06-00, SITE 38S-345

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by D.M. Wills Associates Limited (Wills) to carry out a foundation investigation for the proposed replacement of the Harmony Beach Road Bridge over the Harmony River on Highway 7045, in the Township of Havilland, approximately 1.3 km north east of Highway 17 junction with Harmon Beach Road (Highway 7045). The site is located within the District of Algoma and has MTO Site Number 38S-345.

The existing Harmony Beach Road Bridge is a five-span structure with a total length of about 19 m, which is supported by four sets of timber piers and timber abutments. It is our understanding that the original bridge was built as a two lane bridge and it was narrowed down to 4.1 m single lane centred on the structure after the rehabilitation in 2006.

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 site is located some 45 km north of Sault Ste. Marie at Harmony Beach on the Bachawana Bay (Lake Superior). The Harmony River flows into the Bachawana Bay which is about 120 m west of the proposed bridge replacement site (see site photographs, presented in Appendix D). A sand bar formation has developed at the mouth of the river discharging into the Bachawana Bay. The channel is clean, sand lined, and straight at the bridge site. North of the site the terrain is relatively flat. Based on the profile drawings supplied to us by Wills, to the south the land rises about 6 m within 150 m distance at a gradual slope. Water levels in the river would be subject to some fluctuations, both but would be largely regulated by the water level in Lake Superior.

According to the Quaternary Geology of Ontario (Map 2556), the site is located within a sand, gravelly sand and gravel glaciolacustrine deposit. Underlying the surficial granular deposit, thick silty clay to clay deposit is commonly encountered in this area. Bedrock underlying this area is low silicate basic volcanic rock according to Ontario Geologic Map 2108 (See Appendix E). As well, exposures of bedrock in this area are plentiful (as shown in Appendix D) and deposits of drift occur as thin, irregularly distributed patches.

The existing approach embankments, which are approximately up to 2.5 m high close to the bridge abutment, 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.

* Highway 7045 has been changed to Highway 7090.

3 INVESTIGATION PROCEDURES

The fieldwork for the proposed replacement of Harmony Beach Road Bridge was performed during the period of July 21 through July 27, 2009. As agreed with MTO, the fieldwork consisted of drilling and sampling four boreholes (Boreholes B1 through B4) for the bridge structure, two boreholes for the approach embankments (Boreholes B5 and B6) and four boreholes (Boreholes B7 through B10) for determination of stripping depth at the toe area of the existing embankment, as well as performing field vane and Dynamic Cone Penetration tests (DCPT). The plan location of the boreholes is shown in Drawing No. 2. The following table summarizes the borehole locations and drilling depths.

Table 3.1: Borehole Locations and Drilling Depths

Borehole No.	Location	Drilling Depth Below Existing Ground Surface (m)	Dynamic Cone Penetration Tests	Piezometer
B1	10+180 (1.2 m Lt C/L)	18.5	No	No
B2	10+183 (6.3 m Rt C/L)	18.4	Yes	No
B3	10+213 (1.0 m Lt C/L)	15.5 (including 3.2 m rock coring)	Yes	No
B4	10+210 (5.1 m Rt C/L)	16.0 (including 3.0 m rock coring)	No	Yes
B5	10+169 (3.0 m Rt C/L)	11.1	No	No
B6	10+230 (1.0 m Lt C/L)	9.6	No	No
B7	10+189 (7.0 m Rt C/L)	1.2	No	No
B8	10+190 (11.5 m Lt C/L)	1.0	No	No
B9	10+206 (7.0 m Rt C/L)	1.0	No	No
B10	10+208 (6.5 m Lt C/L)	1.2	No	No

Landcore Drilling of Chelmsford, Ontario, carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer (Mr. Raid Khamis, P.Eng.) from Coffey. Deep boreholes (Boreholes B1 through B6) were advanced using a track-mounted drilling rig, outfitted with tools and equipment for soil sampling and testing. Drilling was effected using hollow-stem augers, however, in Boreholes B1, B2 and B3 wash boring methods were also utilized below a depth of 9 m. Coring was utilized to advance the borehole through the cobbles and boulders in Borehole B3. As well, bedrock was proven by rock coring in Boreholes B3 and B4. Shallow boreholes (Boreholes B7 through B10) were advanced by hand augering to figure out the extent of stripping at the toe area of the existing embankment.

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).

In cohesive (clayey) deposits, where the consistency of the soil permitted, relatively undisturbed samples (TW) were taken with 50 mm (2") or 75 mm (3") diameter thin-walled (Shelby) tubes which were pushed into the bottom of the borehole by the application of static weight or using hydraulic pressure. The undrained shear strength of the soil was also measured in-situ by field vane tests. Where the consistency of clay permitted, a standard MTO type field vane was used to conduct the tests.

As mentioned above, in Boreholes B2 and B3, Dynamic Cone Penetration tests were performed. In Dynamic Cone Penetration Test (DCPT), a 51 mm diameter, 60 deg. apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples can be obtained by the DCPT method 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 effects which in many cases affect the SPT values, especially when fine-grained granular soils or cobbles/boulders are encountered.

Groundwater conditions in the boreholes were observed during drilling and upon completion in the open boreholes. In addition, a piezometer was installed in Borehole B4 to enable groundwater level monitoring in the borehole 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 and unit weight determinations, grain size analyses, one dimensional odeometer (consolidation) and Atterberg Limits tests, was performed on selected representative soil samples. Two rock core samples from Boreholes B3 and B4 were forwarded to the laboratory of Golder Associates where the samples were tested for their unconfined compressive strength (UCS), bulk and dry densities. 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 ten boreholes (see Table 3.1 in Section 3) at the site. The plan locations of the boreholes are shown on Drawing No. 2 and details of sub-surface conditions encountered at each borehole location, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. A stratigraphic profile and sections at the bridge location are shown on Drawing Nos. 2 and 3. Detailed laboratory test results are enclosed in Appendix B. Rock core photographs are shown in Appendix E.

In general, the sub-surface stratigraphy comprises pavement (asphaltic concrete), granular pavement fill and embankment fill materials overlying a surficial sand deposit, which are in turn underlain by a 2.5 to 9.4 m thick deposit of silty clay to clay. This silty clay to clay is further underlain by lower granular soil deposits such as silty sand, sand, gravelly sand and gravel, followed by basic volcanic bedrock. Bedrock was proven by NQ coring in Boreholes B3 and B4.

4.1 Asphalt

A 40 to 80 mm thick asphalt layer was contacted in Boreholes B1, B2, B3 and B6. Boreholes B1 and B3 were drilled from the existing road pavement while Borehole B2 and B4 were drilled on the paved shoulder.

4.2 Embankment Fill

Boreholes B1, B2, B3, B4, B5 and B6 which were drilled from the existing paved road and shoulder area contacted a 0.2 to 0.3 m thick sand and gravel layer followed by gravelly sand, sand some gravel and sand fill materials extending to a depth of 1.1 to 3.5 m below the existing ground surface.

The grain-size distribution of a sample from the deposit from Borehole B3 is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel:	61 %
Sand:	33 %
Silt & Clay:	6 %

Based on a recorded N-value of 5 to 30 blows/0.3 m, this basically granular pavement and embankment fill is considered very loose to compact but typically loose to compact.

4.3 Surficial Granular Soils

Underlying the fill, a surficial granular soil deposit consisting of sand to gravelly sand was contacted in Boreholes B1, B2, B5 and B6. This granular (non-cohesive) soil cap was found to extend to a depth of 1.8 to 4.0 m below the ground surface or to El. 183.4 to 181.1 m (i.e. 0.7 to 2.3 m thick).

The grain-size distribution of four samples from the deposit from Boreholes B1, B2 and B5 is given in Figure B-2, in Appendix B. This indicates the following grain-size distribution.

Gravel:	5-46%
Sand:	51-89%
Silt & Clay:	3-6%

From the grain-size distribution, the material is considered to be more pervious than the underlying silty clay to clay deposits. Also from the grain-size distribution curves, the estimated coefficient of permeability (k) of the samples tested is of the order of 1×10^{-2} to 1 cm/sec.

Standard Penetration tests performed in this granular (non-cohesive) soil deposit gave N-values which range from 3 to 12 blows/0.3 m indicating a very loose to compact relative density.

Boreholes B7 through B10 put down by hand augering to 1.0 to 1.2 m below the o.g. levels, near the toe of the existing embankments, to estimate stripping depths, contacted sand to gravelly sand within the depths explored.

It is believed that these surficial granular soils encountered at the site have been deposited by the River shortly before discharging into the Lake. These soils were not contacted in Boreholes B3 and B4. It is believed that this is due to disturbance and mixing of the existing surficial sand at these locations with the embankment fill, during the construction of the existing bridge structure.

4.4 Silty Clay to Clay

Underlying the non-cohesive deposits described in the previous sections, all the deep boreholes contacted a major cohesive deposit at depths ranging from 1.8 m to 4.0 m or El. 183.4 to 181.1 m. The following table summarizes the top and bottom elevations of the deposit, as encountered in the deep boreholes.

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)
B1	4.0/181.4	13.4/172.0
B2	3.5/181.1	11.4/173.2
B3	3.5/182.0	8.5/177.0
B4	2.3/182.4	9.9/174.8
B5	1.8/183.4	11.1*/174.1*
B6	3.8/181.7	6.3/179.2

*End of borehole

The cohesive soil deposit consists of a reddish grey silty clay to clay and its thickness increases from south to north or from 2.5 m at Borehole B6 to more than 9.6 m at Borehole B5.

The grain-size distribution of seven samples from the deposit (from Boreholes B1 through B6) is given in Figure B-3. The results of the tests on the samples show the following grain-size distribution:

Gravel:	0 %
Sand:	0-2 %
Silt:	28-46 %
Clay:	52-72 %

Figure B-4 present the results of a grain-size analysis carried out on more silty zone (i.e. silty clay zone) in the deposit.

The results of Atterberg limits tests performed on nine samples recovered from the deposit are given in Figure B-5 in Appendix B. These tests yielded the following index values:

Liquid Limit:	54-71 %

Plastic Limit:	21-33 %
Plasticity Index:	29-42

These results indicate clay soils of high plasticity. As shown on the individual Record of Borehole Sheets, the measured natural moisture contents are near or typically in excess of the measured liquid limits which indicate the likelihood of a normally consolidated soil deposit.

The Atterberg Limits test results performed on two of the more silty zone in the clay deposit are presented in Figure B-6 in Appendix B. These indicate clayey soils of intermediate plasticity.

Standard Penetration tests conducted in the silty clay to clay deposit gave N-values which range from 0 to 4 blows/0.3 m but typically zero (i.e. sampler sank under own weight of the sampler and the drilling rods) which indicate a very soft to soft consistency. The undrained in-situ shear strengths of the deposit were measured in the field by means of field vane tests, using MTO type field vanes. The measured values range from 20 to 55 kPa, indicating a soft to firm consistency but typically soft. It should be kept in mind when analysing these results that the tests were performed in boreholes drilled from the top of the roadway. The undrained, in-situ shear strengths of the deposit beyond the influence of the embankment can be expected to be lower.

Figure C1 in Appendix C presents the measured undrained in-situ shear strengths versus elevation.

In Figures C2 through C5 in Appendix C, the variation of the measured in-situ vane strength values (i.e. in-situ undrained shear strengths) versus elevation is presented, for each of Boreholes B1, B2, B3 and B4. Also plotted on each figure is the effective overburden stress (P'_o), as well as the plot of 0.23 P'_o with elevation. It is commonly acknowledged that with Ontario clays if the measured undrained shear strengths are in excess of 0.23 P'_o line, the deposit may be somewhat over-consolidated. In this respect, about top 3 m portion of the silty clay to clay layer at north abutment location (Boreholes B1 and B2) and silty clay to clay at south abutment location (Boreholes B3 and B4) appear to be slightly over-consolidated. The silty clay to clay in Boreholes B1 and B2 appears to be normally consolidated below the upper±3 m zone.

Two oedometer (one dimensional consolidation) tests were performed in the laboratory on 76 mm (3") diameter Shelby tube (TW) samples. The results are presented in Figure B-7 and B-8 in Appendix B. These results show a possible pre-consolidation pressure that is similar to the existing overburden pressure which means this silty clay to clay deposit is probably normally consolidated. Compression index (C_c) of about 0.8 and recompression index (C_r) of about 0.15 are obtained.

There is some evidence that the lower zones of the deposit where it is thicker may still be consolidating under its own weight (i.e. may be underconsolidated).

The measured bulk unit weight of the TW samples range from 15.3 to 15.5 kN/m^3.

4.5 Basal Granular Soils

Underlying the silty clay to clay deposit, Boreholes B1, B2, B3, B4 and B6 encountered basal granular soils consisting of silty sand, sand, gravelly sand and gravel, with some cobbles and boulders. These lower granular soils were contacted at depths ranging from 6.3 m (Borehole B6) to 13.4 m (Borehole B1) below the ground surface or at Elevations 179.2 m (Borehole B6) to 172.0 m (Borehole B2). Borehole B5 was terminated within the clay deposit at 11.1 m depth or El. 174.1 m.

Boreholes B1, B2 and B6 were terminated in these lower granular deposits at depths ranging from 9.6 to 18.5 m below the ground surface or at El. 175.9 to 166.2 m. In Borehole B3 and B4, the boreholes were extended to the underlying bedrock at depths of 12.3 and 13.0 m or at El. 173.2 and 171.7 m, respectively.

The composition of these granular soils range from relatively finer silty sand to gravelly sand to coarse grained materials consisting of gravel with frequent cobble and boulder size particles (e.g. Boreholes B2 and B3).

The grain-size distribution of a sample from the relatively finer sand is given in Figure B-9 in Appendix B, while Figure B-10 shows the grain-size distribution of a layer of sand within a gravelly zone. These indicate the following grain-size distribution:

Gravel:	0 -2%
Sand:	86-87 %
Silt & Clay:	11-14 %

Figure B-11 shows the grain-size distribution of fine samples from the more prominent, relatively well-graded sand in the basal granular deposits. These indicate the following grain-size distribution:

Gravel:	12-28%
Sand:	40-61 %
Silt & Clay:	11-40 %

Figure B-12 shows the grain-size distribution of a relatively coarser gravelly sand layer within the basal granular deposits. The following grain-size distribution is indicated.

Gravel:	85 %
Sand:	12 %
Silt:	3 %

It should be pointed out that the presence of cobbles and boulders was noted in these deposits. In particular, boulders were contacted in Borehole B1, B2 and B3. Borehole B3 was advanced near the bedrock surface in between 10.0 and 12.3 m depths (El. 175.5 to 173.2 m) by coring through several boulders.

N-values recorded in these deposits range widely from 7 to in excess of 100 blows/0.3 m, indicating a loose to very dense compactness condition.

These basal granular deposits are water bearing and appeared to be under excess hydrostatic pressure.

4.6 Bedrock

In Boreholes B3 and B4, which were put down at the south abutment location, a reddish grey coloured basic volcanic rock (See the project site on Ontario Geologic Map 2108 in Appendix E) was contacted at depths of 12.3 m and 13.0 m or El. 173.2 and 171.7 m, respectively.

The percentage of rock core recovery was 72 to 100 % while the RQD values vary from 38 to 85 %. These results indicate a rock quality ranging from poor to good, but typically fair to good.

Unconfined compression tests were performed on selected intact rock samples and the tests yielded unconfined compressive strengths of about 161 to 188 MPa. These results indicate that the rock samples tested can be classified as being "very strong".

4.7 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. In the deep boreholes (i.e. Borehole B1 through B6), where wash boring and NQ coring 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 and are summarized in the following table.

Table 4.7.1 Summary of Groundwater Level Measurements

Borehole No	Ground Surface Elevation (m)	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers
B1	185.4	-	3.7/181.7*	Upon completion	No
B2	184.6	-	2.3/182.3*	Upon completion	No
B3	185.5	-	0.9/184.6*	Upon completion	No
B4	184.7	12.2/172.5	2.4/182.3	2 days after completion	Yes
B5	185.2	-	Dry*	Upon completion	No
B6	185.5	-	2.4/183.1*	Upon completion	No
B7	183.3	-	0.4/182.9*	Upon completion	No
B8	183.1	-	0.3/182.8*	Upon completion	No
B9	182.9	-	0.5/182.4*	Upon completion	No
B10	183.1	-	0.4/182.7*	Upon completion	No

* not stabilized

As shown in the above table a piezometer was installed in the basal granular deposit above the bedrock. The water level in the piezometer was measured at El. 182.3 m or close to the o.g. level. A previous investigation (1959, MTO Geocres 41K00-017 and 41K00-019) in the close vicinity of the existing bridge, shows an artesian conditions emanating from the basal granular deposits, a condition which frequently occurs due to an upward gradient the from confined pervious layer (the basal granular deposit at this site) between relatively impervious materials (i.e. silty clay to clay at top and bedrock at the bottom). However, an artesian condition was not encountered during our investigation but the water level was found at or very close to the o.g. levels. It is our opinion that an artesian condition might occur at the site depending on the water level in Lake Superior, located within about 120 m to the site. From the measured values, it is our opinion that groundwater level below the original grade (o.g.) at the time of investigation was at about El 182 to 183 m, while a perched water condition would likely occur due to the accumulation of the surface water in the fill materials and in the upper sand cap overlying the practically impervious clay deposit.

It should be pointed out that the water levels observed represent the conditions at the time of our investigation and that they would be subject to seasonal fluctuations as well as fluctuations due to weather events and the water level in the Harmony River which highly depends on the water level in Lake Superior (i.e. water level at the site would be largely regulated by the water level in Lake Superior), especially if there is a hydraulic connection through the water bearing, relatively pervious basal granular deposits underlying the site. We understand that the highest water level recorded in Lake Superior was 184.05 m, while the normal water level was 183.5 m (average of past five years).

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

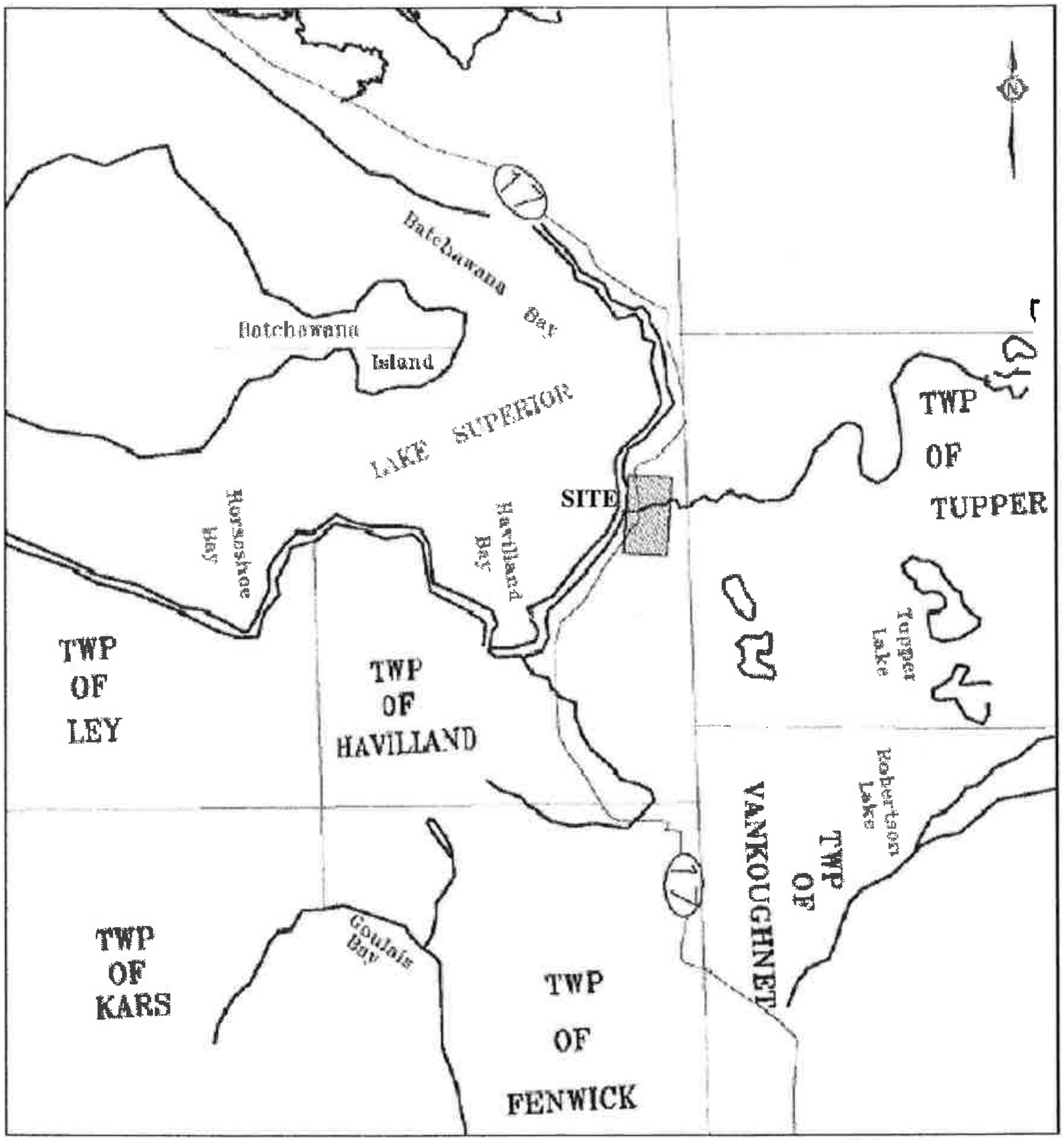
Ramon Miranda, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

Zuhtu Ozden, P.Eng.

Drawings

Harmony Beach Road Bridge
Site No. 38S-345
Highway 17-7045



Drawing 1 Site Plan

METRIC

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

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.

CONT No.

GWP: 5430-06-00

SHEET

HARMONY BEACH ROAD BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA (PROFILE)

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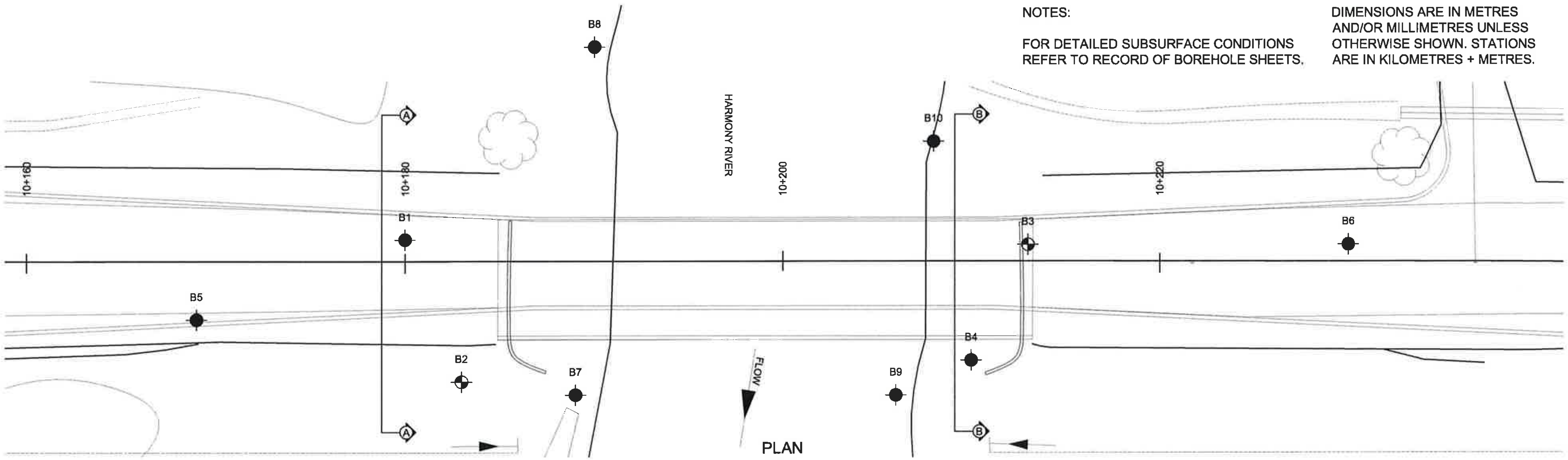
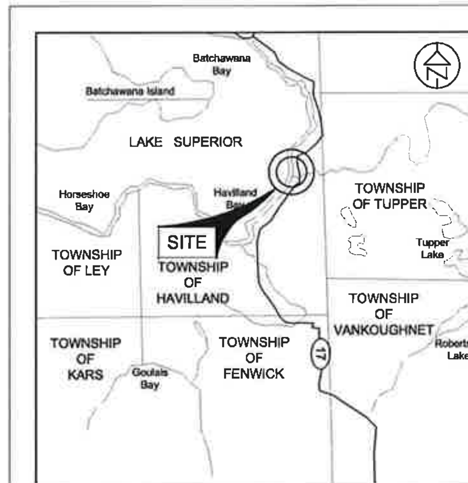
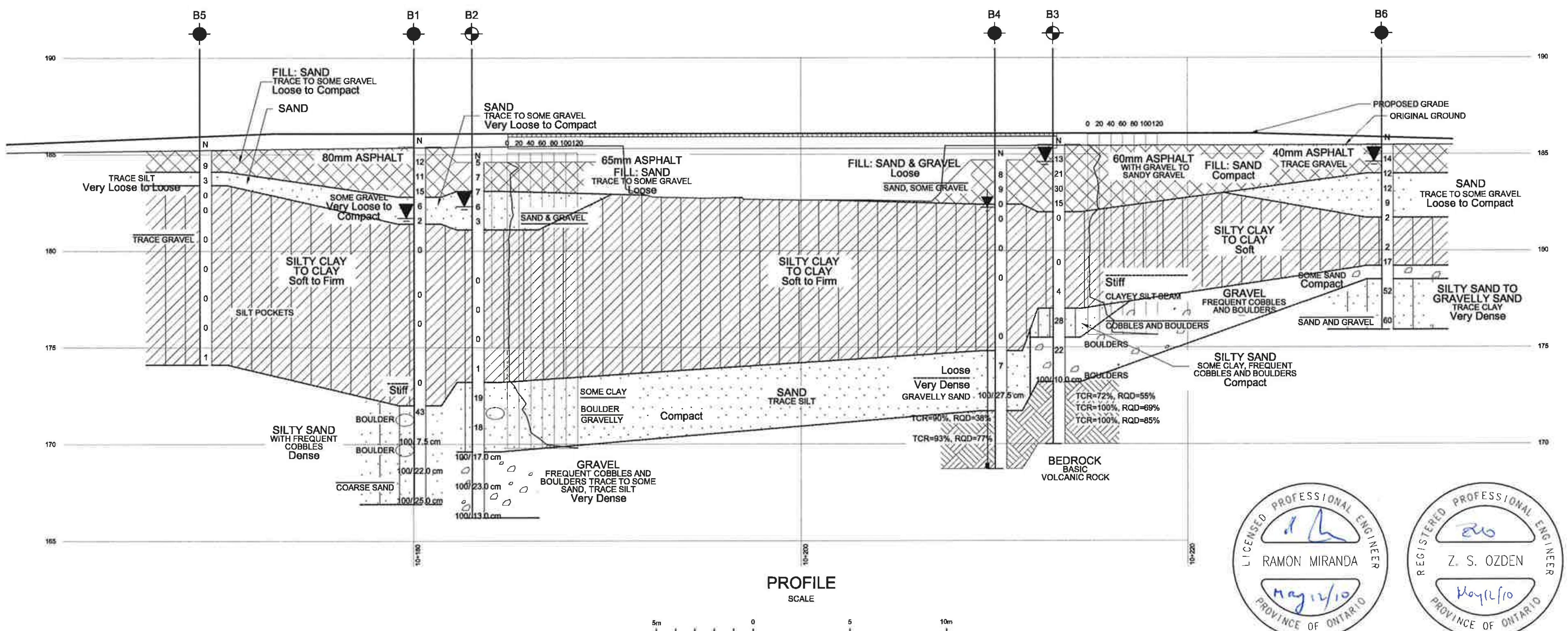


PLAN
SCALE



KEY PLAN
N.T.S.



PROFILE
SCALE

LEGEND

- Borehole
- Borehole & 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.	ELEVATION	NORTHING	EASTING
B1	185.3	5189580.7	276365.3
B2	184.6	5189578.8	276357.4
B3	185.4	5189548.1	276360.6
B4	184.7	5189551.9	276354.9
B5	185.1	5189592.2	276362.6
B6	185.5	5189531.2	276358.3
B7	183.3	5189572.9	276355.9
B8	183.1	5189569.5	276374.1
B9	182.9	5189556.1	276353.6
B10	183.1	5189552.3	276366.7

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

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 41K-83

TRANETOB01240AA			DIST
SUBM'D	CHECKED	DATE May 12, 2010	SITE 38S-345
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 2

CONT No.
GWP: 5430-06-00

HARMONY BEACH ROAD BRIDGE
SOIL STRATA (SECTIONS)

SHEET

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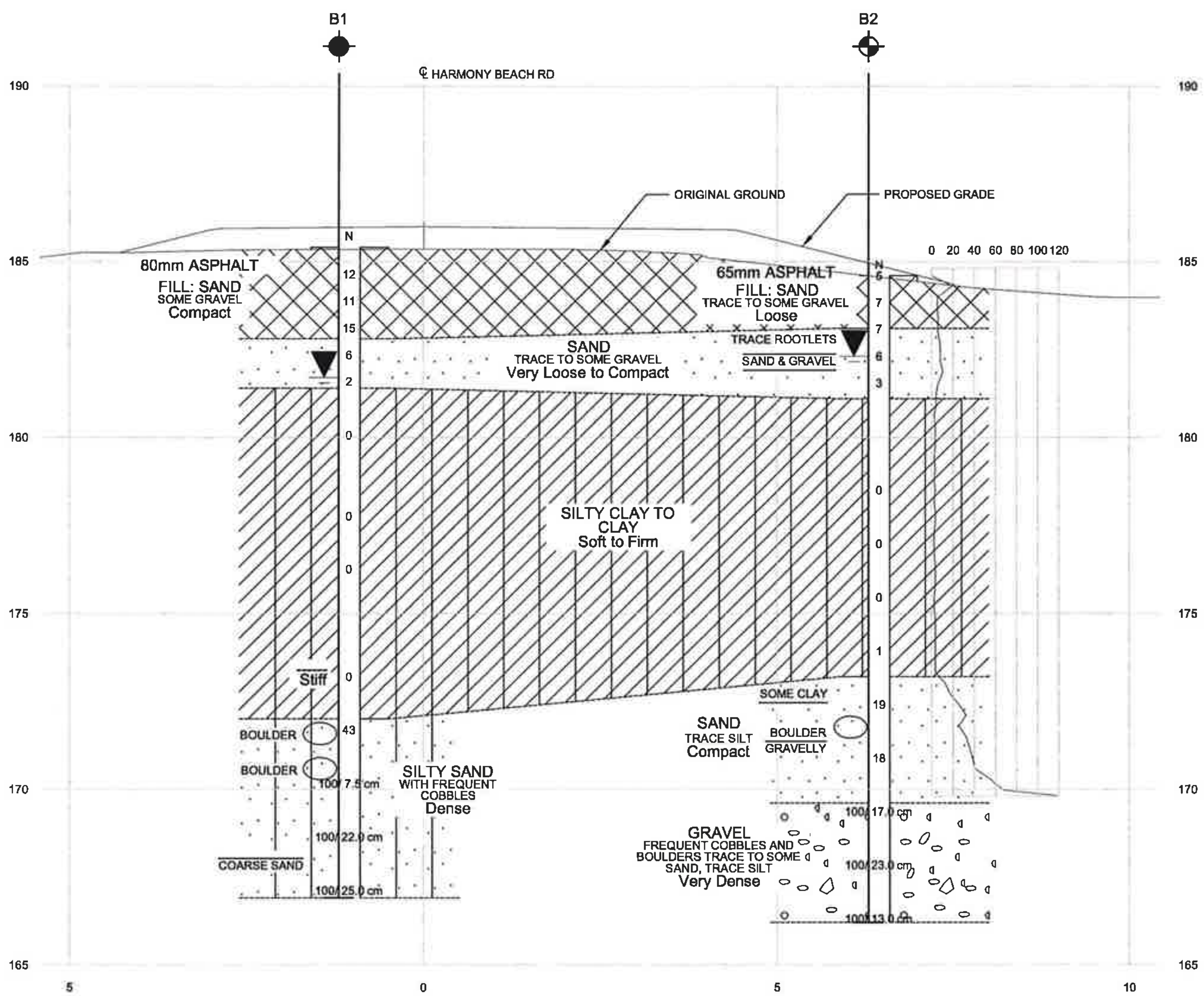
METRIC

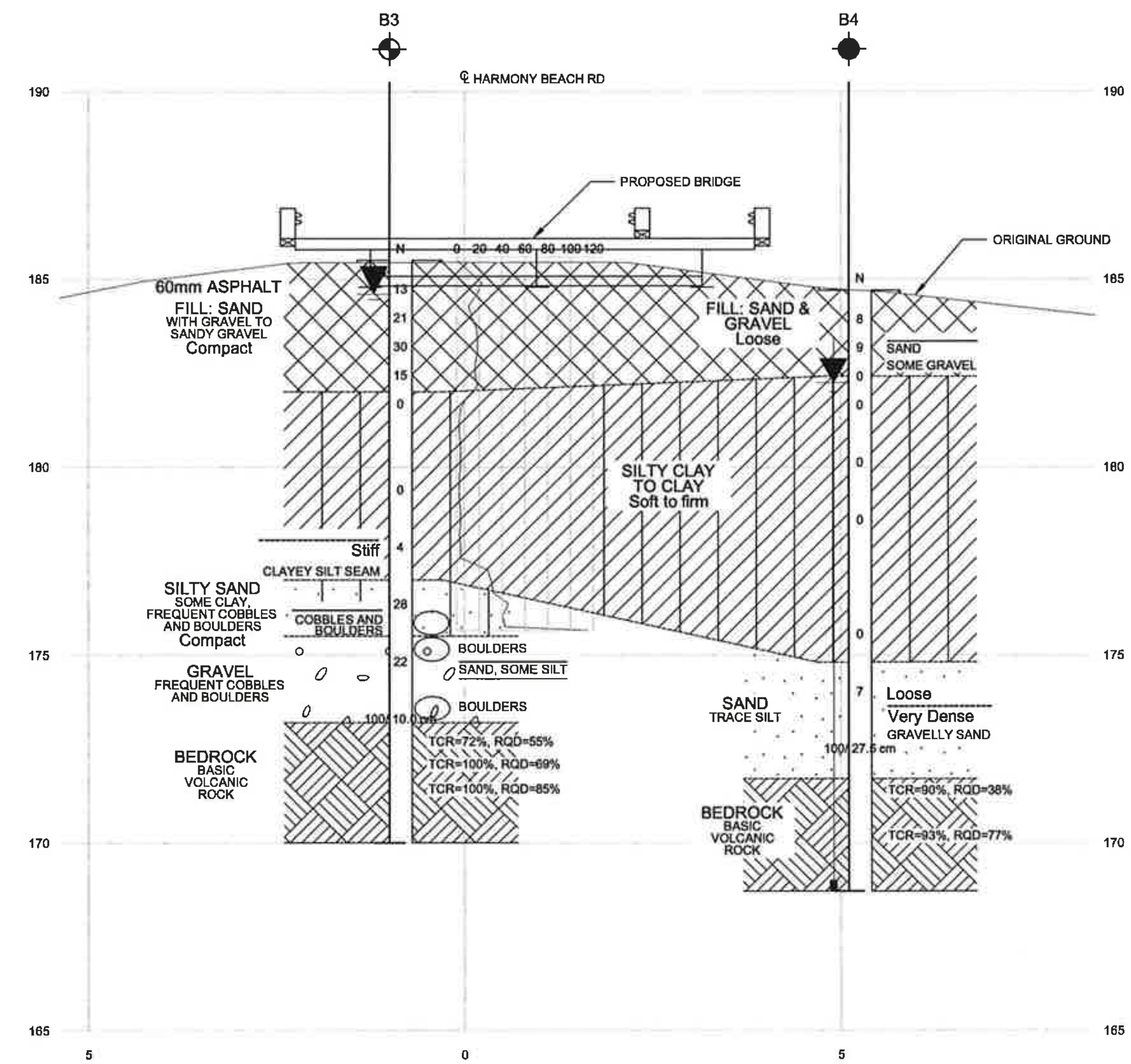
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.



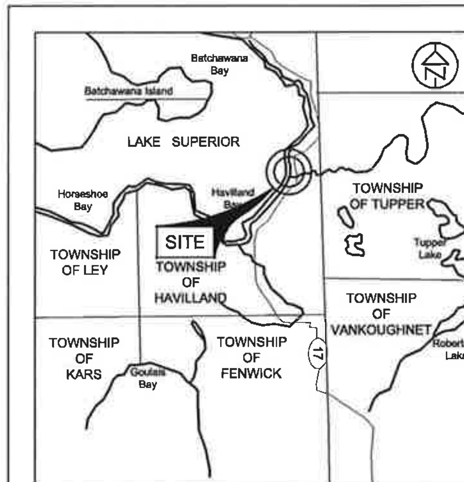
SECTION A-A



SECTION B-B

HORIZONTAL SCALE

2.5m 0 2.5 5m



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole & 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.	ELEVATION	STATION	OFFSET
B1	185.3	10+180	1.2m Lt C/L
B2	184.6	10+183	6.3m Rt C/L
B3	185.4	10+213	1.0m Lt C/L
B4	184.7	10+210	5.1m Rt C/L

-NOTE-
The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
May 12/10
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
May 12/10
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 41K-83

	TRANETOB01240AA		DIST
SUBM'D	CHECKED	DATE May 12, 2010	SITE 38S-345
DRAWN PHK	CHECKED RM	APPROVED ZO	DWG 3

Appendix A

Record of Borehole Sheets

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B1

METRIC

GWP 5430-06-00 | LOCATION Sta: 10+180 ; 1.25 m Lt. C/L | ORIGINATED BY RK

DIST | HWY HWY 17 | BOREHOLE TYPE Hollow Stem Augers, Wash boring | COMPILED BY RK

DATUM Geodetic | DATE 7/21/2009 | CHECKED BY RM

ELEV / DEPTH	DESCRIPTION	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.4	GROUND SURFACE	
0.0	80 mm **ASPHALT** 0.2 m **FILL:** sand with gravel 0.3 m **FILL:** sand, some gravel **FILL:** sand, some gravel brown, compact, moist	spoon wet
182.8 / 2.6	**SAND** some gravel brown, v. loose to compact, wet	5 89 (6) 7 87 (6)
181.4 / 4.0	**SILTY CLAY to CLAY** reddish brown, soft to firm, wet stiff	consolidation test 0 1 28 71 consolidation test
172.0 / 13.4	**SILTY SAND** with freq. cobbles, dense, wet boulder	auger refusal @ 14.2 m borehole further advanced using wash boring 20 40 26 14
170.4		

SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT (kN/m³)
1	AS		
2	SS	12	
3	SS	11	
4	SS	15	
5	SS	6	
6	SS	2	
7	TW	PM	15.5
8	SS	0	
9	SS	0	
10	SS	0	
11	TW	PM	15.3
12	SS	0	
13	SS	43	

ELEVATION SCALE: 185, 184, 183, 182, 181, 180, 179, 178, 177, 176, 175, 174, 173, 172, 171

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

Field vane sensitivity values: 4.0, 4.0, 4.0, 4.0, 4.0, 7.0

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

WATER CONTENT (%): 20 40 60

Continued Next Page

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

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

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B1

METRIC

GWP 5430-06-00 LOCATION Sta. 10+180 ; 1.25 m Lt. C/L ORIGINATED BY RK

DIST ____ HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Wash boring COMPILED BY RK

DATUM Geodetic DATE 7/21/2009 CHECKED BY RM

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLE NUMBER	SAMPLE 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
170.4 / 15.0	**SILTY SAND** with freq. cobbles, dense, wet; boulder		14	SS	100/ 7.5 cm		170				spoon bouncing on possible boulder, borehole advanced using coring
			15	SS	100/ 22.0 cm		169		o		21 58 (21)
	coarse sand						168				
166.9			16	SS	100/ 25.0 cm		167				
18.5	End of Borehole Borehole caved-in @ 9.5 m upon completion Groundwater level in open borehole @ 3.7 m (not stabilized)* upon completion										

Plot 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 (%) 20 40 60

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

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

RECORD OF BOREHOLE No B2

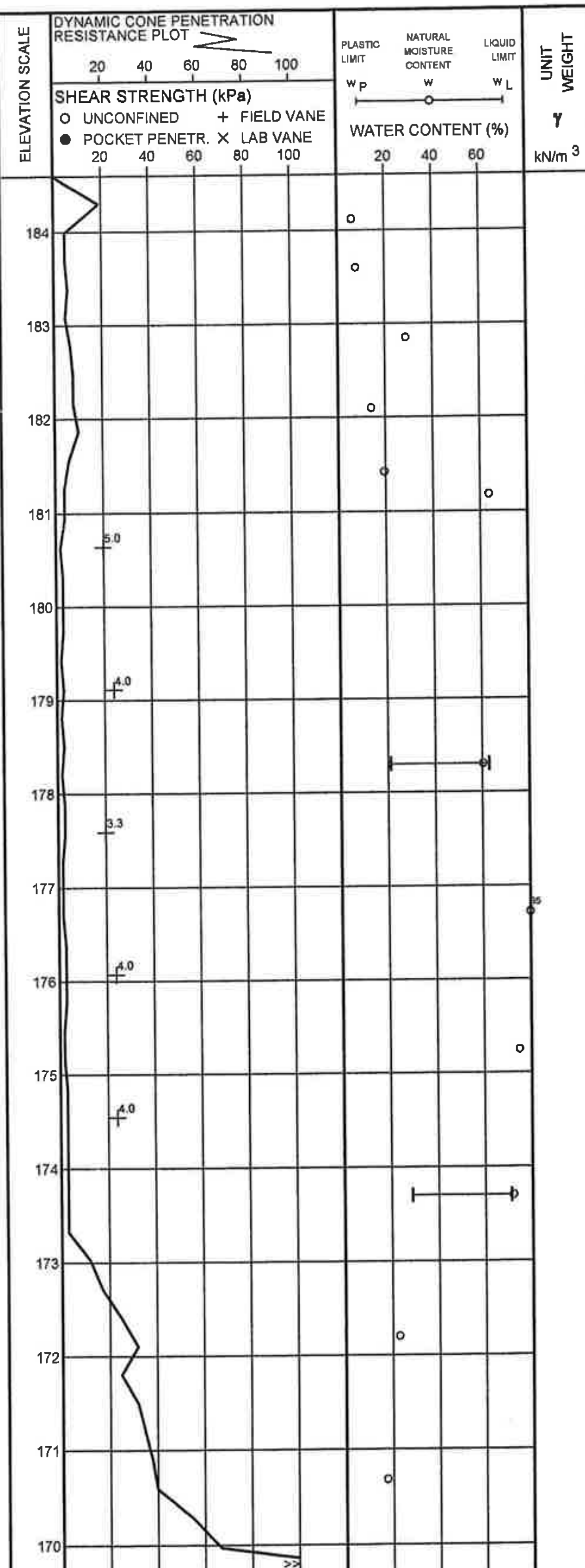
METRIC

GWP 5430-06-00 LOCATION Sta: 10+183; 6.3 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Wash boring COMPILED BY RK

DATUM Geodetic DATE 7/25/2009 CHECKED BY RM

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.6	GROUND SURFACE					
0.0	65 mm **ASPHALT** **FILL:** sand tr. to some gravel brown, loose, moist	1	SS	5		
		2	SS	7		
183.1 / 1.5	**SAND** tr. to some gravel tr. rootlets sand & gravel v. loose to loose, moist to wet	3	SS	7		
		4	SS	6	▼	spoon wet 46 51 (3)
		5	SS	3		
181.1 / 3.5	**SILTY CLAY to CLAY** reddish brown, soft, wet	6	TW	PM		
		7	SS	0		0 2 34 64
		8	SS	0		
		9	SS	0		
		10	SS	1		
173.2 / 11.4	**SAND** tr. silt, grey, compact, wet some clay boulder gravelly	11	SS	19		
		12	SS	18		auger refusal @ 13.1 m borehole further advanced using wash boring 2 87 (11)
169.6						



Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B2

METRIC

GWP 5430-06-00 LOCATION Sta: 10+183; 6.3 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Wash boring COMPILED BY RK

DATUM Geodetic DATE 7/25/2009 CHECKED BY RM

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
169.6 / 15.0	**GRAVEL** freq. cobbles and boulders tr. to some sand, tr. silt v. dense, wet		13	SS	100/ 17.0 cm		169				85 12 (3)
			14	SS	100/ 23.0 cm		168				
			15	SS	100/ 13.0 cm		167				
166.2 / 18.4	End of Borehole Borehole caved-in @ 6.7 m upon completion DCPT performed adjacent to Borehole to 14.8 m (El. 169.8 m) Groundwater level in open borehole @ 2.3 m (not stabilized)* upon completion										

Shear strength legend: O UNCONFINED, + FIELD VANE, ● POCKET PENETR., X LAB VANE

Plastic limit w_P, Natural moisture content w, Liquid limit w_L

+3, X3: Numbers refer to Sensitivity

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

TRANETOB01240AA: Harmony Beach Road Bridge

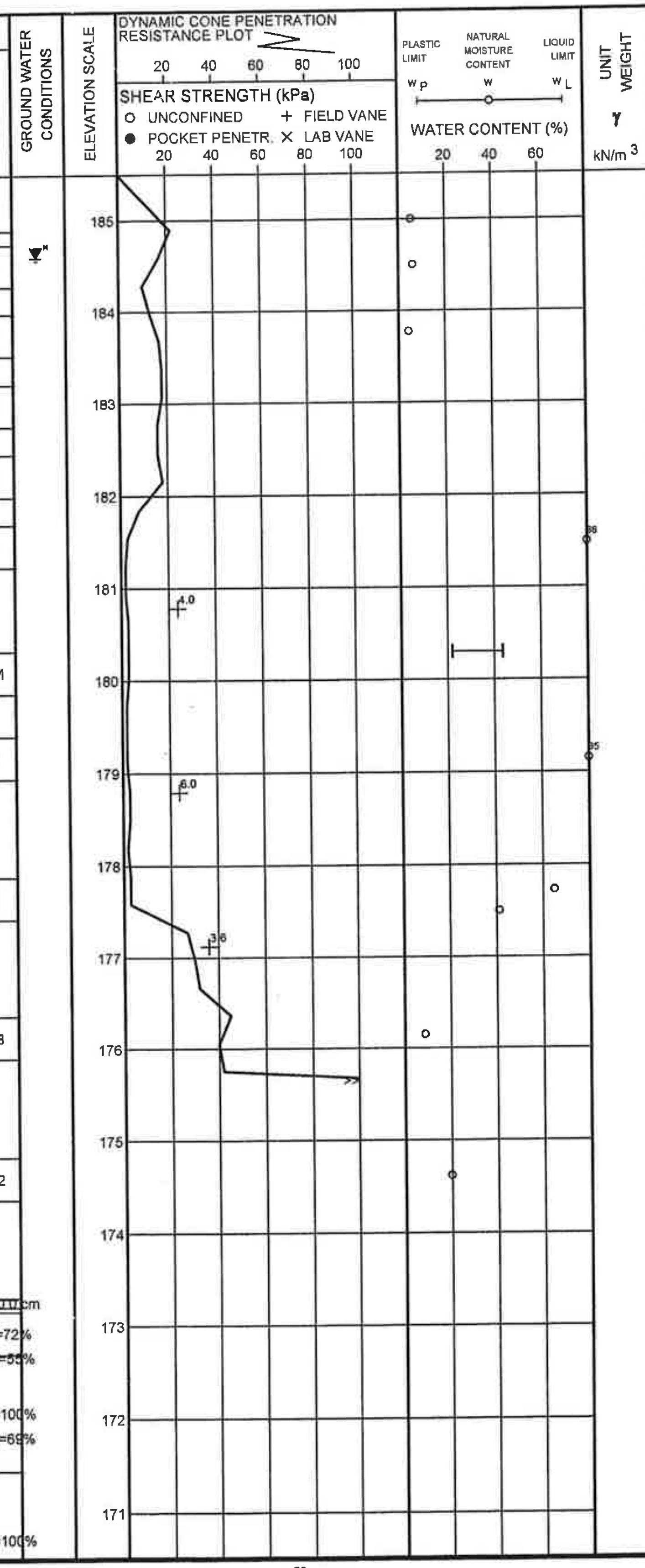
RECORD OF BOREHOLE No B3

METRIC

GWP 5430-06-00	LOCATION Sta: 10+213; 1.0 m Lt. C/L		ORIGINATED BY RK
DIST	HWY HWY 17	BOREHOLE TYPE Hollow Stem Augers, Wash boring, Rock coring	COMPILED BY RK
DATUM Geodetic		DATE 7/22/2009	CHECKED BY RM

SOIL PROFILE / SAMPLES

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
185.5	GROUND SURFACE			
0.0	60 mm **ASPHALT** 0.2 m **FILL:** sand & gravel 0.3 m **FILL:** sand, some gravel	1	AS	
		2	SS	13
184.1 / 1.4	**FILL:** sand with gravel to sandy gravel brown, compact moist wet	3	SS	21
		4	SS	30
		5	SS	15
182.0 / 3.5	**SILTY CLAY to CLAY** reddish brown, soft, wet	6	SS	0
		7	TW	PM
		8	SS	0
	stiff, clayey silt seam	9	SS	4
177.0 / 8.5	**SILTY SAND** some clay, freq. cobbles and boulders brown to reddish brown, compact, wet cobbles and boulders	10	SS	28
175.5 / 10.0	**GRAVEL** freq. cobbles and boulders, wet boulders	11	RC	
	sand, some silt	12	SS	22
	boulders	13	RC	
		14	SS	100/ 10 cm
173.2 / 12.3	**BEDROCK** reddish grey basic volcanic rock	15	RC	TCR=72% RQD=55%
		16	RC	TCR=100% RQD=69%
170.5		17	RC	TCR=100%



REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

- 61 33 (6)
- spoon wet
- 0 0 47 53
- 2 12 50 36
- 12 48 29 11
- auger refusal @ 9.8 m
- borehole further advanced using wash boring
- 0 86 (14)
- borehole advanced by coring
- UCS 161.5 MPa

Continued Next Page

+ 3 , X 3 : Numbers refer to Sensitivity

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

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B3

METRIC

GWP 5430-06-00	LOCATION Sta: 10+213; 1.0 m Lt. C/L		ORIGINATED BY RK
DIST	HWY HWY 17	BOREHOLE TYPE Hollow Stem Augers, Wash boring, Rock coring	COMPILED BY RK
DATUM Geodetic		DATE 7/22/2009	CHECKED BY RM

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 (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
170.5 15.0	**BEDROCK** reddish grey basic volcanic rock				RQD=85%						
170.0 15.5	End of Borehole DCPT performed adjacent to Borehole to 9.9 m (El. 175.6 m) Borehole caved-in @ 6.6 m upon completion Groundwater level in open borehole @ 0.9 m (not stabilized)* upon completion						170				

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

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B4

METRIC

GWP 5430-06-00 LOCATION Sta: 10+210; 5.15 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Rock Coring COMPILED BY RK

DATUM Geodetic DATE 7/27/2009 CHECKED BY RM

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
184.7	GROUND SURFACE					
0.0	**FILL:** sand & gravel brown, loose, moist	1	AS		184	
	sand, some gravel	2	SS	8		
		3	SS	9	183	
182.4 / 2.3	**SILTY CLAY to CLAY** reddish brown, soft to firm, wet	4	SS	0	182	spoon wet
		5	SS	0		0 0 33 67
		6	SS	0	181, 180, 179	
		7	SS	0	178, 177, 176	
		8	SS	0	175	0 1 27 72
174.8 / 9.9	**SAND** tr. silt	9	SS	7	174	augering slow
	reddish grey, loose				173	
	gravelly sand, brown, v. dense	10	SS	100/ 27.5 cm	172	22 61 (17)
171.7 / 13.0	**BEDROCK** reddish grey basic volcanic rock	11	RC	TCR=90% RQD=38%	171, 170	auger refusal @ 13.0 m; UCS=186.3 MPa
169.7						

Field vane shear strength values plotted with sensitivity labels: 4.0 (~180.8), 4.6 (~179.3), 4.0 (~177.8), 4.0 (~176.6). Groundwater level shown at ~182.3.

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 (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

Ministry of Transportation
Ontario

Foundation Design

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B4

METRIC

GWP 5430-06-00 LOCATION Sta: 10+210; 5.15 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers, Rock Coring COMPILED BY RK

DATUM Geodetic DATE 7/27/2009 CHECKED BY RM

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
169.7 15.0	**BEDROCK** reddish grey basic volcanic rock		12	RC	TCR=93% RQD=77%		169				
168.7 16.0	End of Borehole Piezometer installed to 12.2 m Piezometer readings July 27, 2009 4.1 m July 29, 2009 2.4 m										

Shear strength legend: O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE; plastic limit w_P, natural moisture content w, liquid limit w_L.

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

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

RECORD OF BOREHOLE No B5

METRIC

GWP 5430-06-00 LOCATION Sta: 10+169; 3.0 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WC

DATUM Geodetic DATE 7/26/2009 CHECKED BY RM

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.2	GROUND SURFACE				
0.0	0.3 m **FILL:** Sand & Gravel **FILL:** Sand tr. gravel, brown, loose, moist	1	AS		
184.1 / 1.1		2	SS	9	
	SAND tr. silt, brown, v.loose to loose, moist				29 68 (3)
183.4 / 1.8		3	SS	3	
	SILTY CLAY TO CLAY silt pockets, reddish brown soft, wet	4	SS	0	
		5	SS	0	
	tr. gravel	6	SS	0	
		7	SS	0	0 1 37 62
		8	SS	0	
		9	SS	0	
174.1		10	SS	1	
11.1	End of Borehole. Borehole caved-in @ 10.0 m. Borehole was dry (not stabilized) upon completion.				

Field vane shear strength sensitivities plotted: 4.0, 3.5, 4.0, 3.5

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (185 to 175); 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 (%) (20 40 60); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

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

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B6

METRIC

GWP 5430-06-00 LOCATION Sta: 10+230; 1.0 m Lt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WC

DATUM Geodetic DATE 7/23/2009 CHECKED BY RM

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
185.5	GROUND SURFACE					
0.0	40 mm **ASPHALT** 0.3 m **FILL:** Sand & Gravel **FILL:** Sand tr. gravel, brown, compact, moist	1	AS		185	
		2	SS	14		
184.0 / 1.5	**SAND** tr. to some gravel, brown to grey loose to compact, wet	3	SS	12	184	
		4	SS	12	183	spoon wet
		5	SS	9	182	
181.7 / 3.8	**SILTY CLAY TO CLAY** reddish brown, soft, wet	6	SS	2	181	0 2 46 52
		7	SS	2	180	
179.2 / 6.3	**GRAVEL** some sand, grey, compact, wet	8	SS	17	179	
178.5 / 7.0	**SILTY SAND to GRAVELLY SAND** tr. clay, reddish brown v. dense, wet	9	SS	52	178, 177	28 49 (23)
	sand and gravel	10	SS	60	176	
175.9 / 9.6	End of Borehole. Borehole caved in @ 6.7 m. Water level @ 2.4 m (not stabilized)* upon completion.					

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

5.0

PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L

WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

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

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

RECORD OF BOREHOLE No B7

METRIC

GWP 5430-06-00 LOCATION Sta: 10+189; 7.0 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY WC

DATUM Geodetic DATE 7/24/2009 CHECKED BY RM

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) ○ 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 (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.3	GROUND SURFACE										
0.0	freq. cobble		1	AS							
	SAND tr. silt, tr. gravel, grey, wet		2	AS		▼*	183				
			3	AS							
182.1 / 1.2	End of Borehole. Water level @ 0.4 m (not stabilized)* upon completion.										

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

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

RECORD OF BOREHOLE No B8

1 OF 1 **METRIC**

GWP 5430-06-00		LOCATION Sta: 10+190; 11.5 m Lt. C/L	ORIGINATED BY RK
DIST	HWY HWY 17	BOREHOLE TYPE Hand Auger	COMPILED BY WC
DATUM Geodetic		DATE 7/24/2009	CHECKED BY RM

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) ○ 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 (%) (20 40 60)	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.1	GROUND SURFACE										
0.0	**SAND** tr. gravel, some rootlets brown to grey, wet		1	AS		▼*	183				
			2	AS							
182.1 / 1.0	End of Borehole. Water level @ 0.3 m (not stabilized)* upon completion.										

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

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

TRANETOB01240AA: Harmony Beach Road Bridge

RECORD OF BOREHOLE No B9

1 OF 1 **METRIC**

GWP 5430-06-00 LOCATION Sta: 10+206; 7.0 m Rt. C/L ORIGINATED BY RK

DIST HWY HWY 17 BOREHOLE TYPE Hand Auger COMPILED BY WC

DATUM Geodetic DATE 7/24/2009 CHECKED BY RM

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 (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
182.9	GROUND SURFACE										
0.0	**GRAVELLY SAND** with cobble/rock pieces brown, wet		1	AS							
			2	AS		▼*					
181.9							182				
1.0	End of Borehole. Water level @ 0.5 m (not stabilized)* upon completion.										

+ ³, × ³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No B10

METRIC

GWP 5430-06-00	LOCATION Sta: 10+208; 6.5 m Lt. C/L		ORIGINATED BY RK
DIST	HWY HWY 17	BOREHOLE TYPE Hand Auger	COMPILED BY WC
DATUM Geodetic	DATE 7/24/2009		CHECKED BY RM

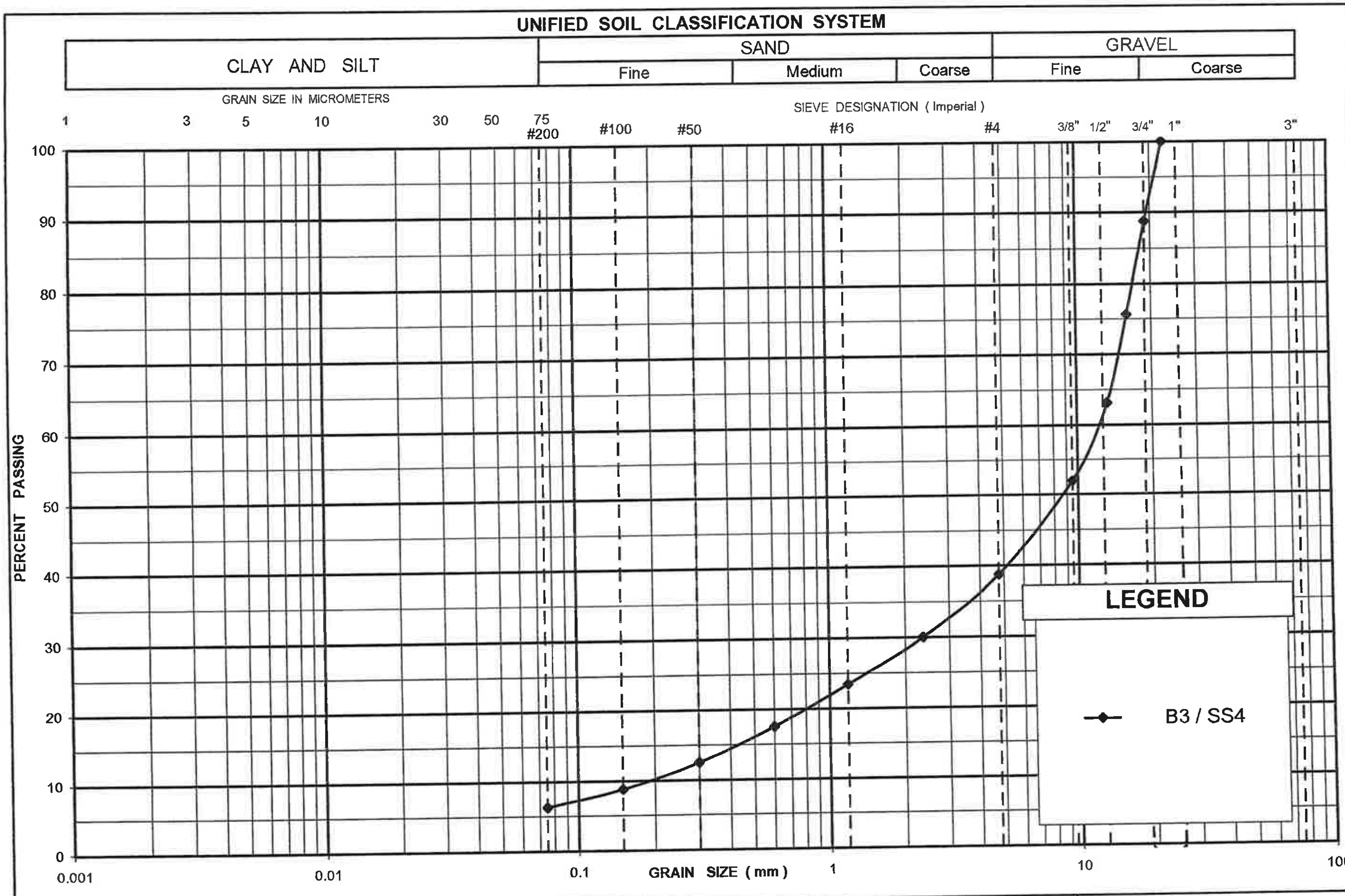
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH (kPa) O UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
183.1	GROUND SURFACE										
0.0	**SAND** some gravel and rock pieces/cobbles grey, wet		1	AS		▼*	183				
			2	AS							
181.9							182				
1.2	End of Borehole. Water level @ 0.4 m (not stabilized)* upon completion.										

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

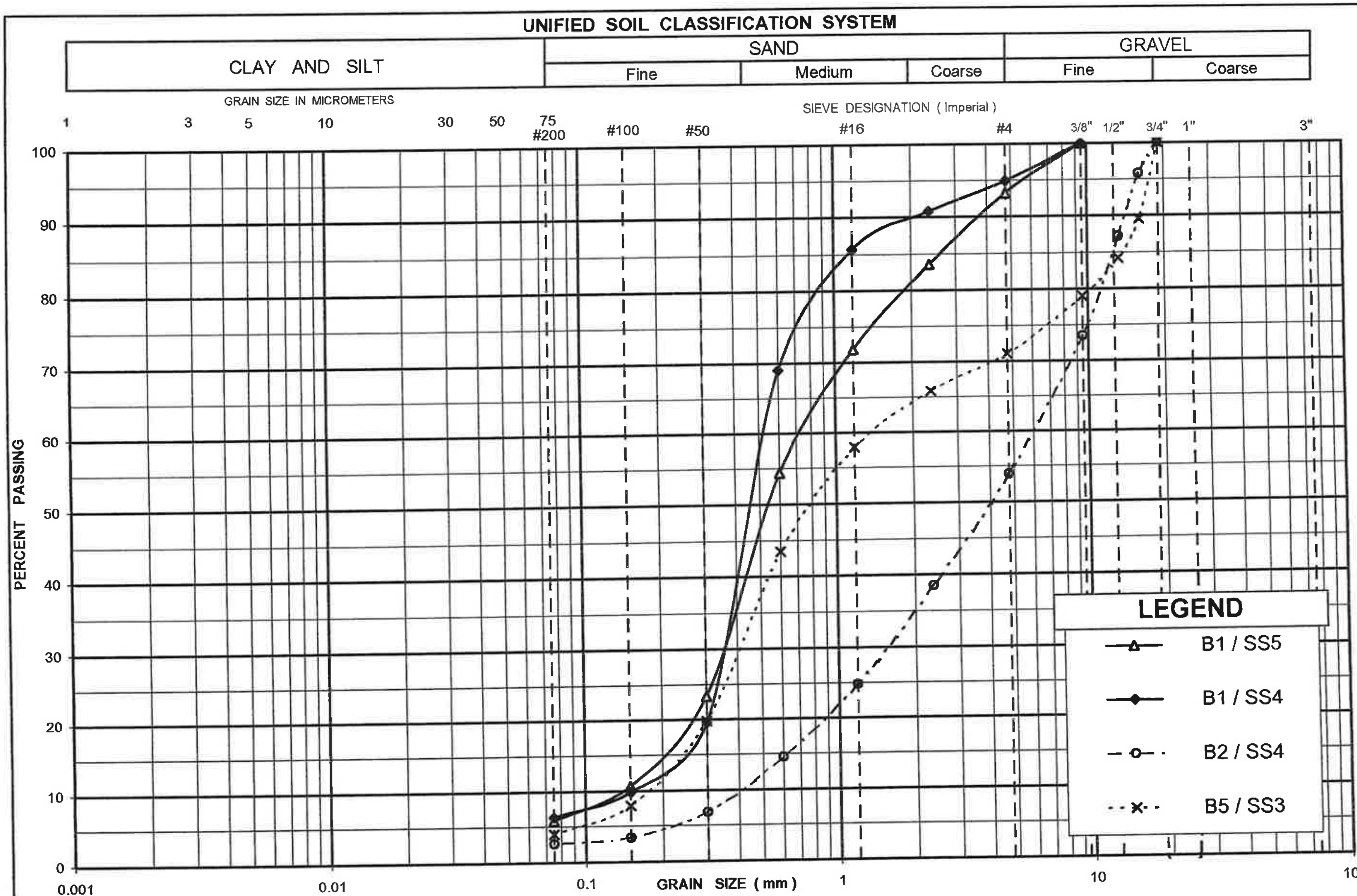
20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results



coffey geotechnics SPECIALISTS MANAGING THE EARTH	GRAIN SIZE DISTRIBUTION EMBANKMENT FILL: Sandy Gravel	Figure No. B-1 PROJECT # : TRANETOB01240AA DATE: October 09, 2009



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

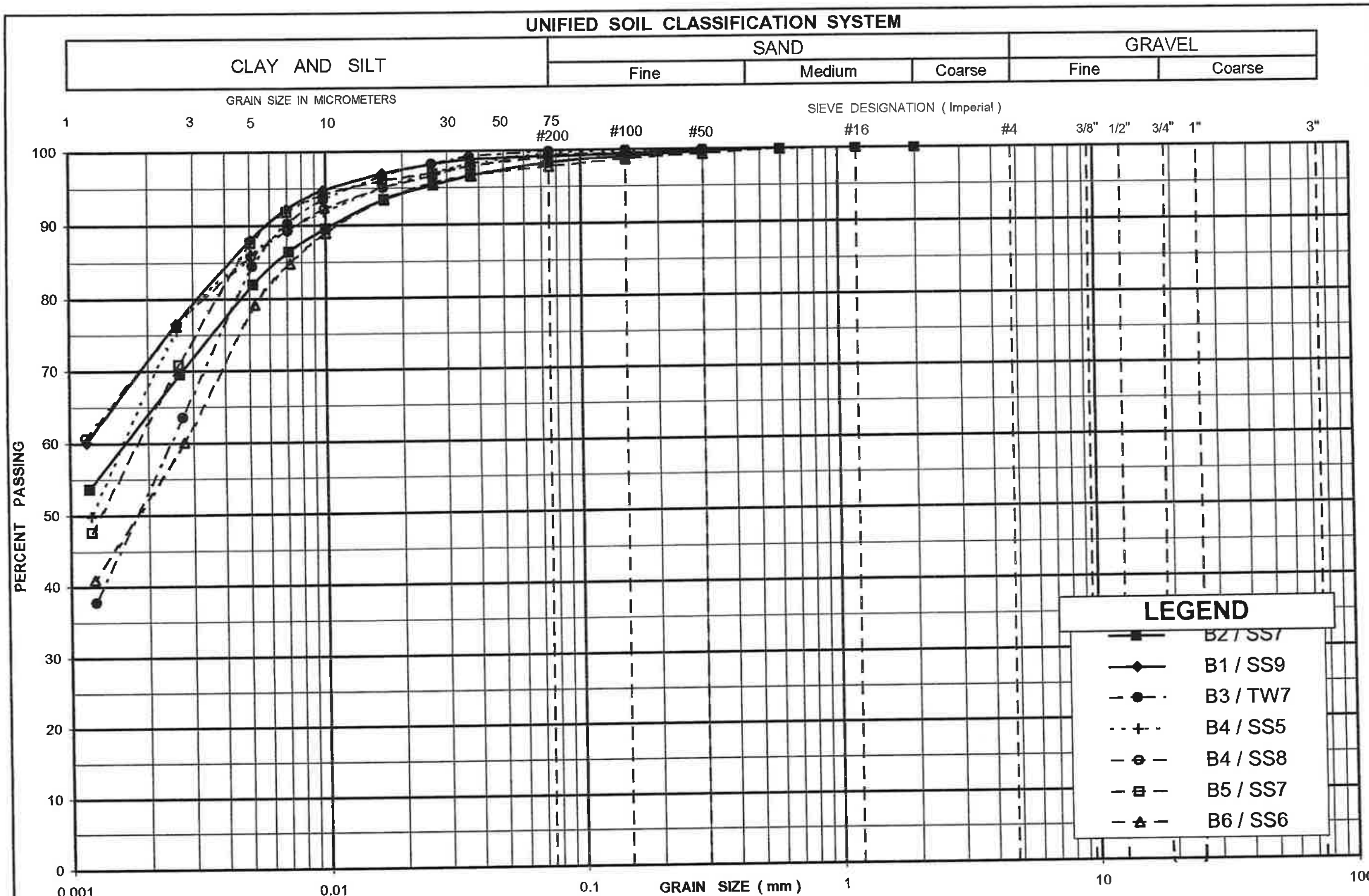
GRAIN SIZE DISTRIBUTION

SURFICIAL GRANULAR SOILS: Sand to Gravelly Sand

Figure No. B-2

PROJECT # : TRANETOB01240AA

DATE: October 09, 2009



coffey geotechnics
SPECIALISTS MANAGING THE EARTH

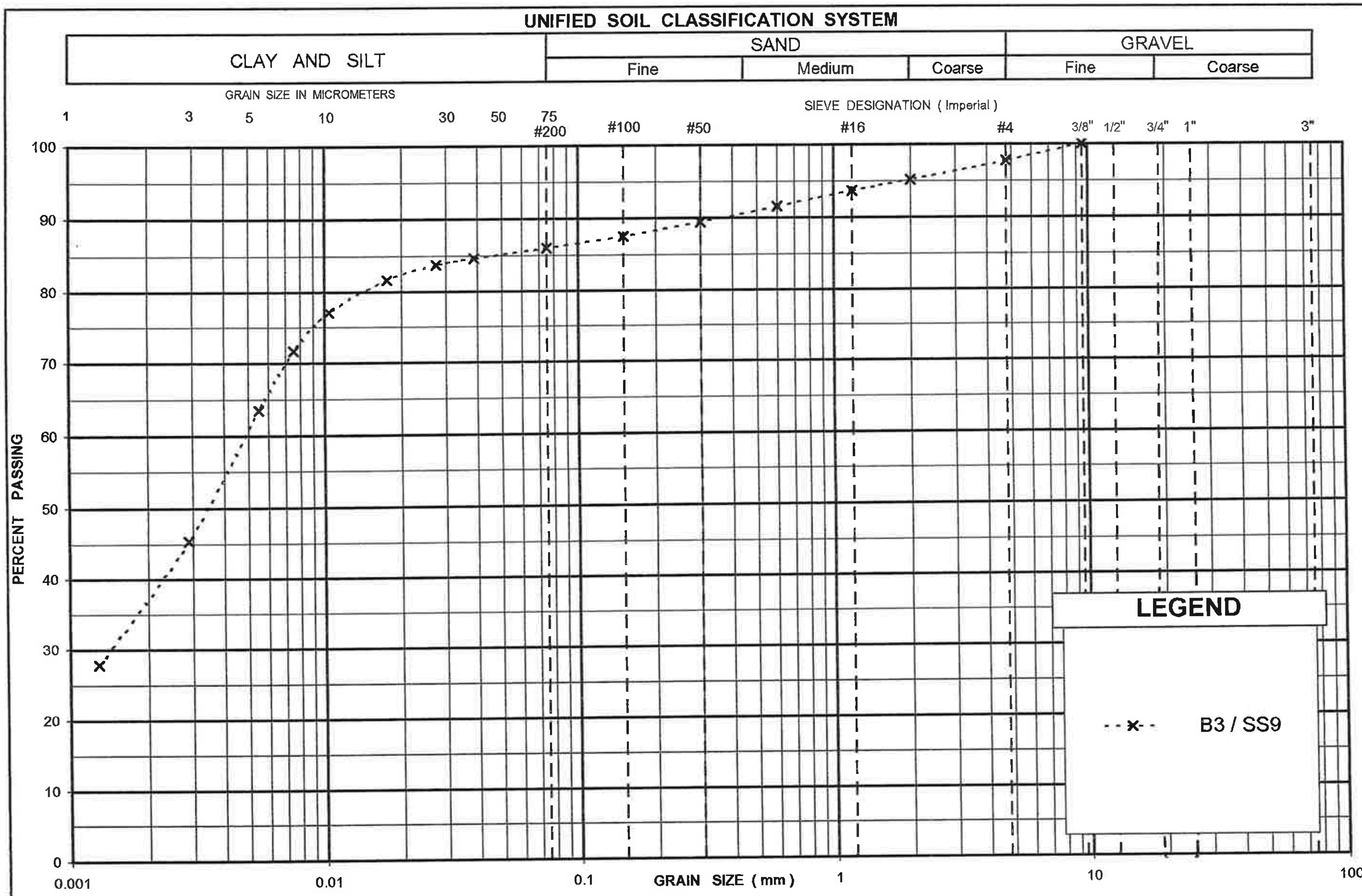
GRAIN SIZE DISTRIBUTION

SILTY CALY to CLAY

Figure No. B-3

PROJECT # : TRANETOB01240AA

DATE: October 09, 2009



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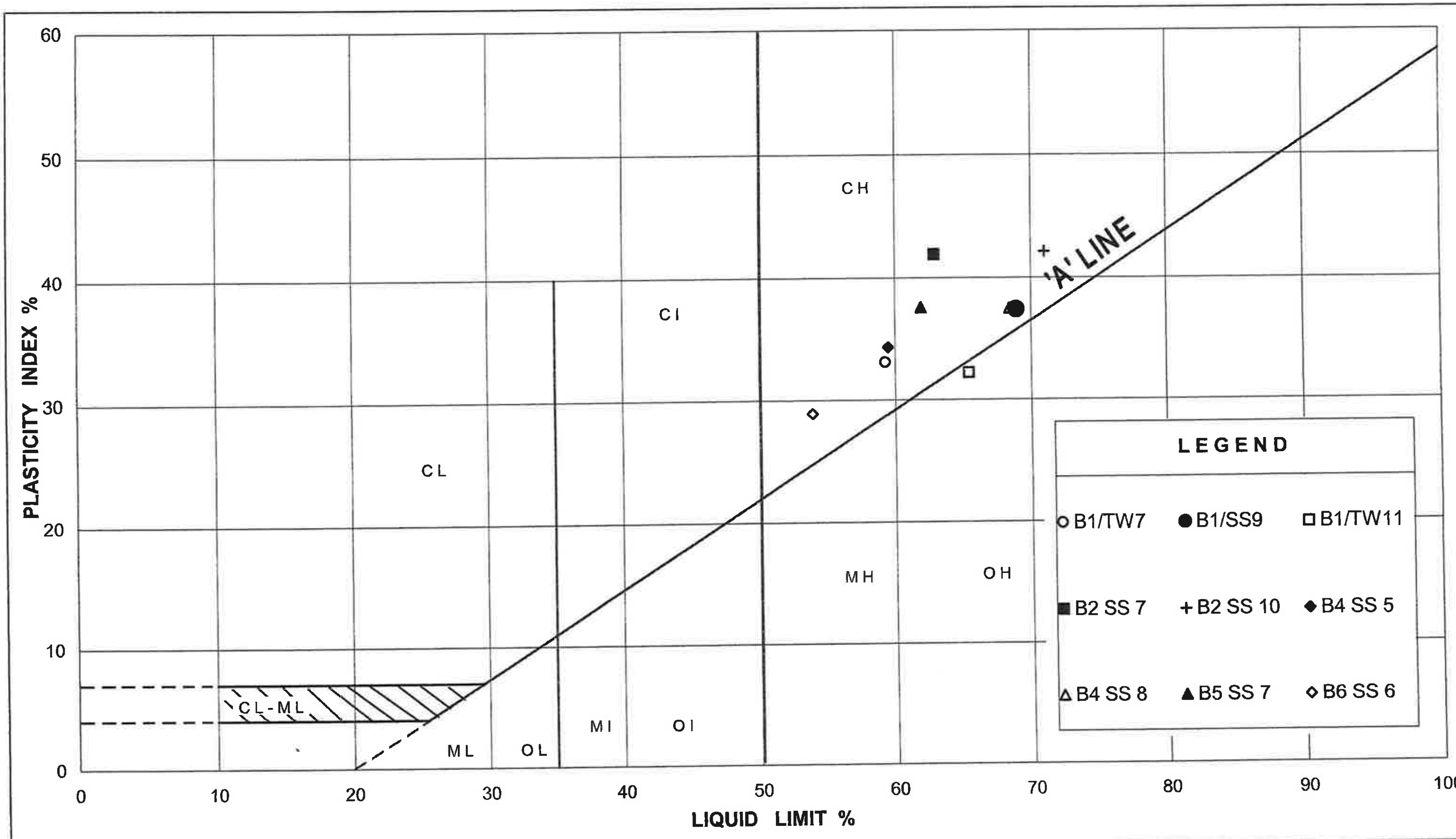
GRAIN SIZE DISTRIBUTION

SILTY CLAY

Figure No. B-4	
PROJECT # :	TRANETOB01240AA
DATE:	October 09, 2009



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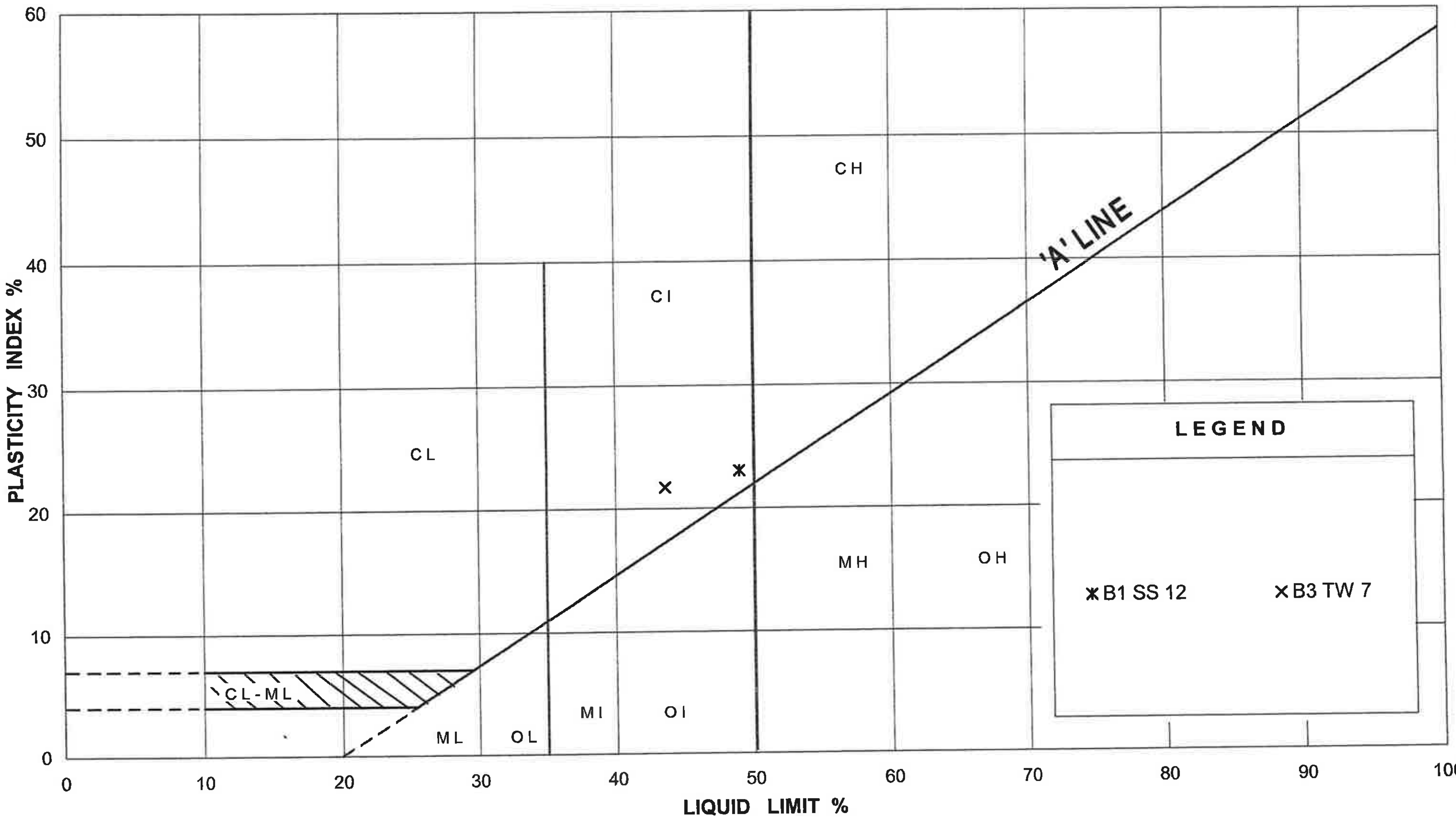
PLASTICITY CHART

SILTY CLAY TO CLAY

Figure No. B-5

Project No. TRANETOB01240AA

DATE Oct.16, 2009



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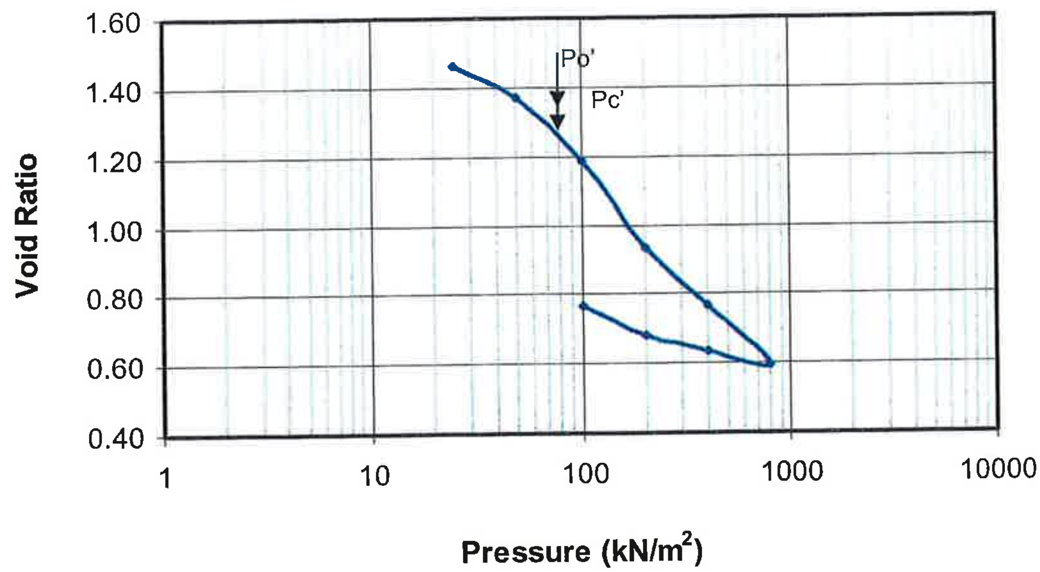
PLASTICITY CHART

SILTY CLAY

Figure No. B-6	
Project No. TRANETOB01240AA	
DATE	Oct.16, 2009

Figure B-7   Consolidation BH B1 TW7

Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

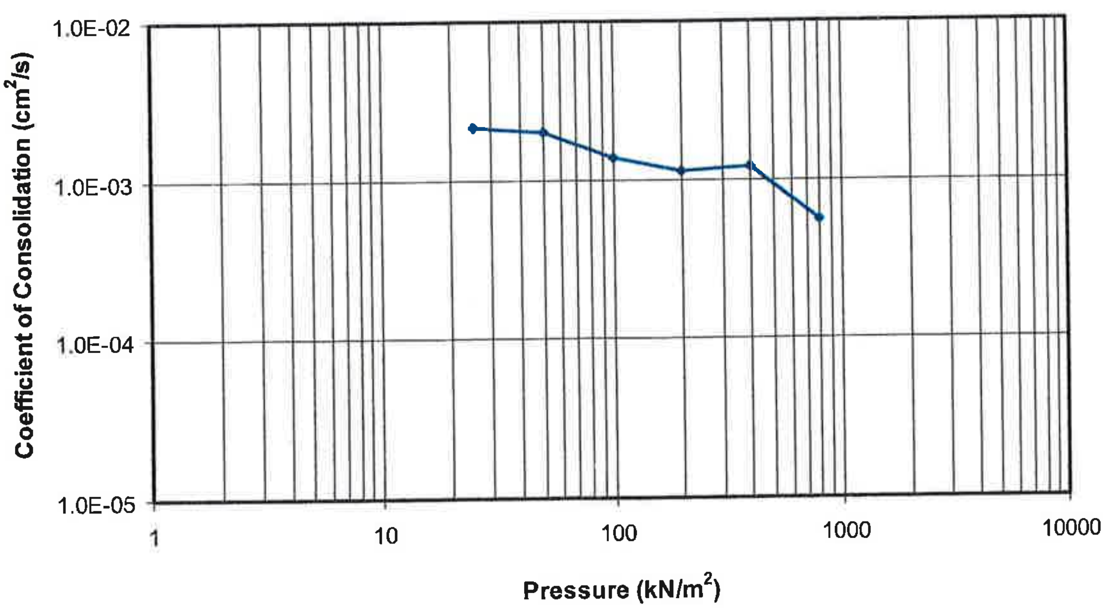
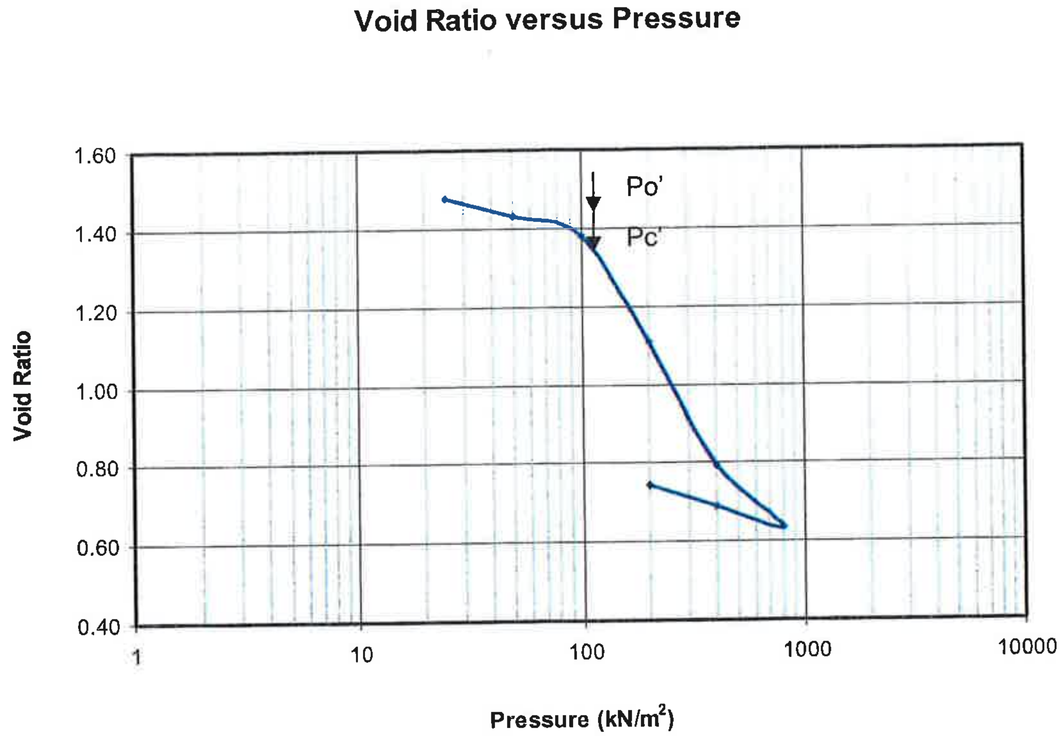
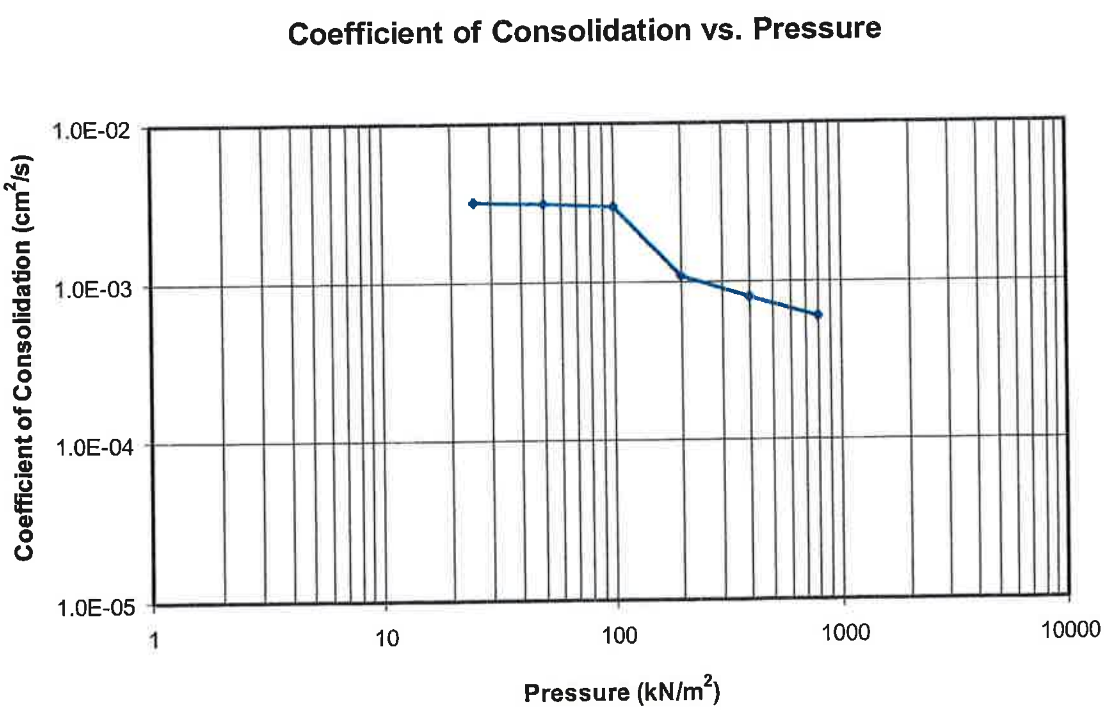
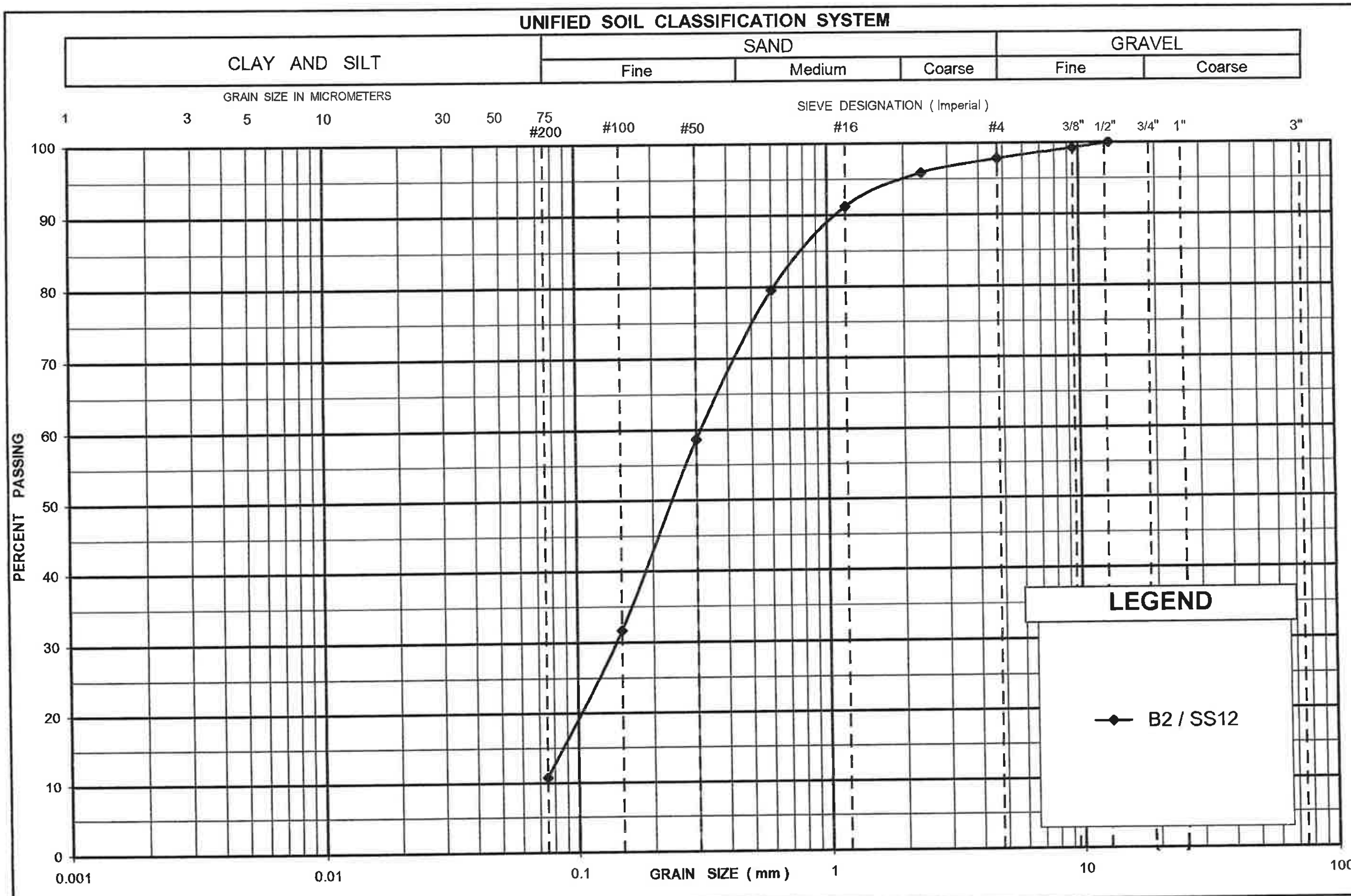


Figure B-8 Consolidation BH B1 TW11







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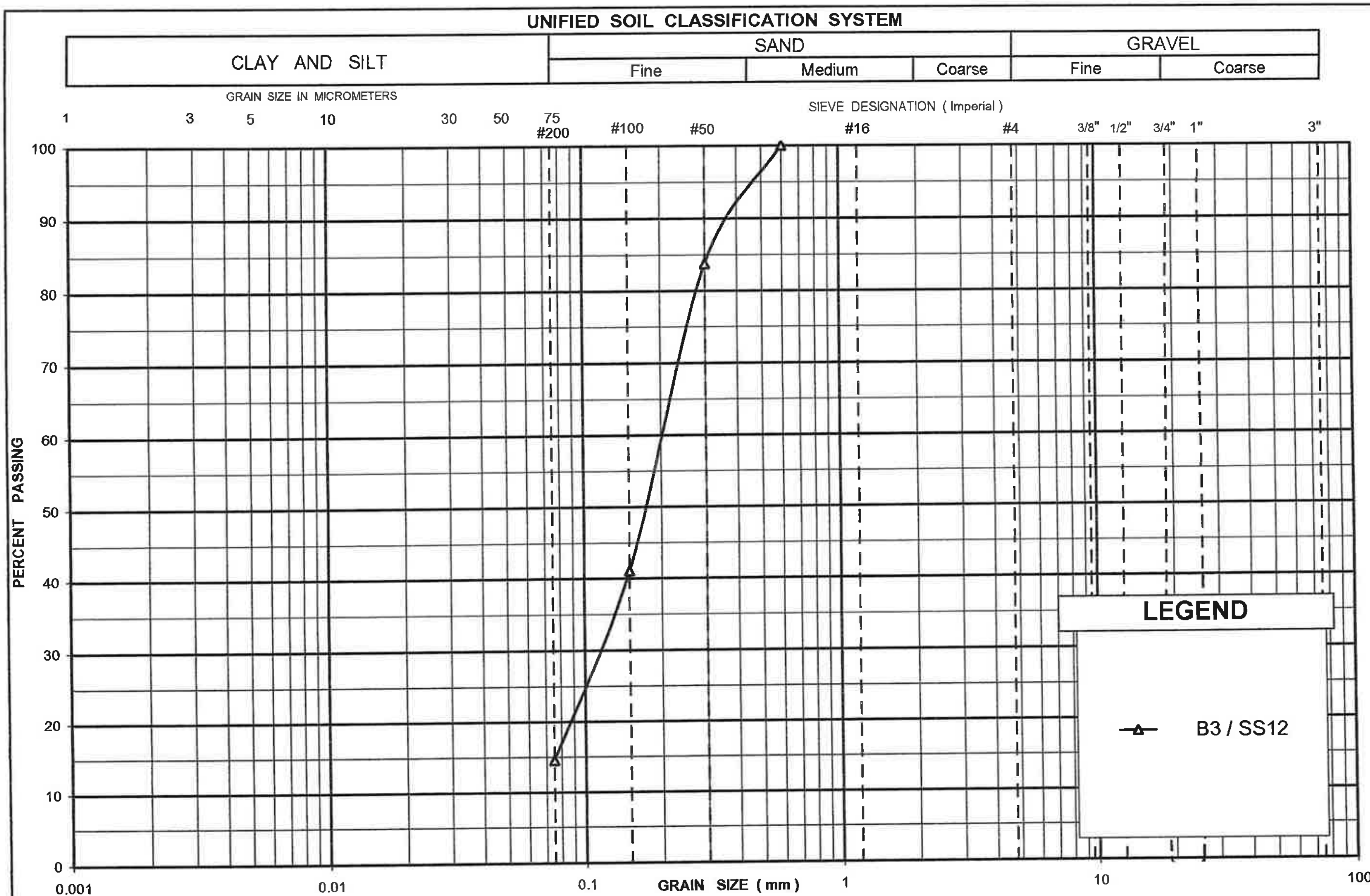
GRAIN SIZE DISTRIBUTION

BASAL GRANULAR SAND

Figure No. B-9	
PROJECT # :	TRANETOB01240AA
DATE:	October 09, 2009



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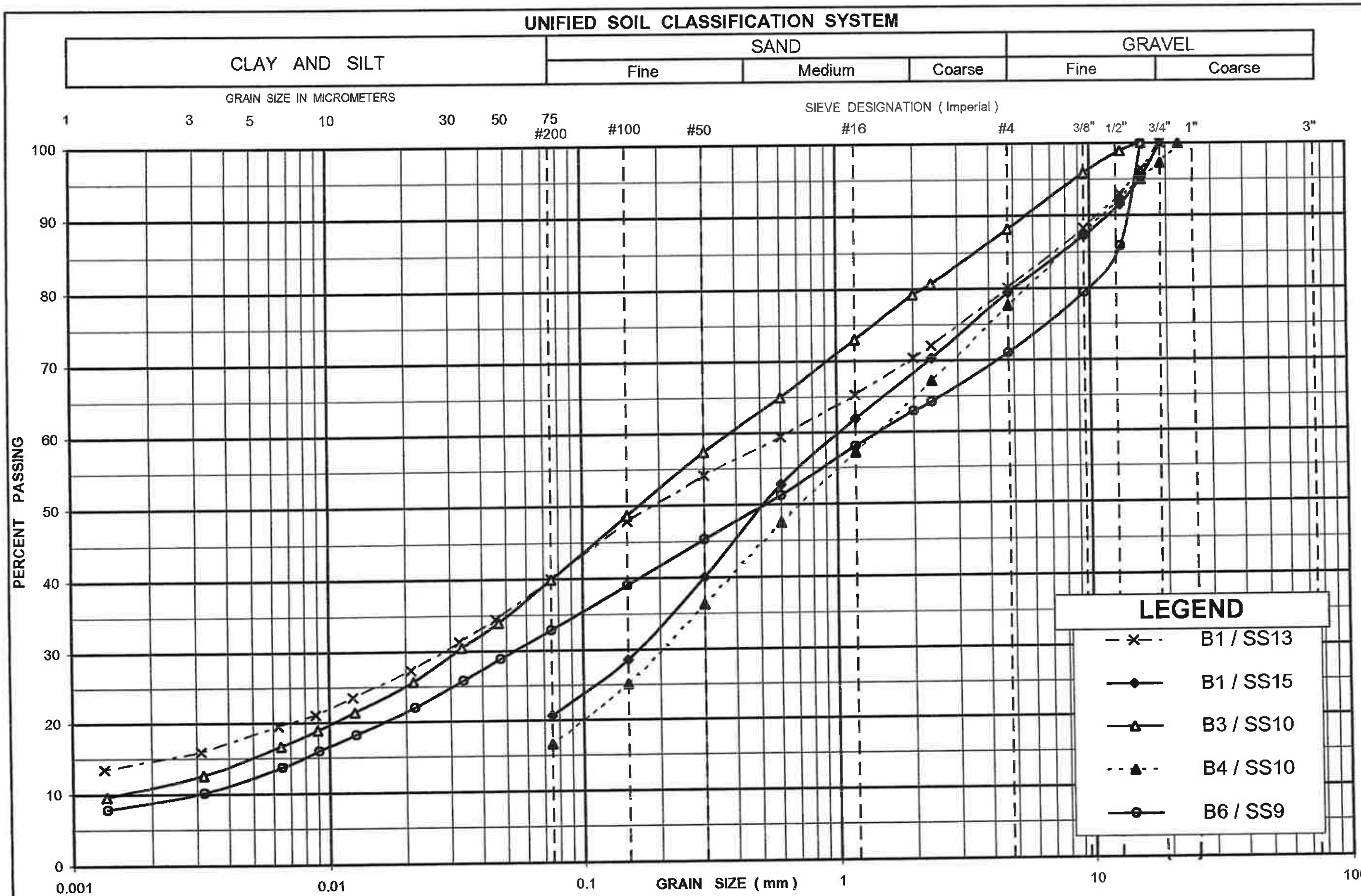
GRAIN SIZE DISTRIBUTION

BASAL GRANULAR SAND, Layer with gravelly soil

Figure No. B-10	
PROJECT # :	TRANETOB01240AA
DATE:	October 09, 2009



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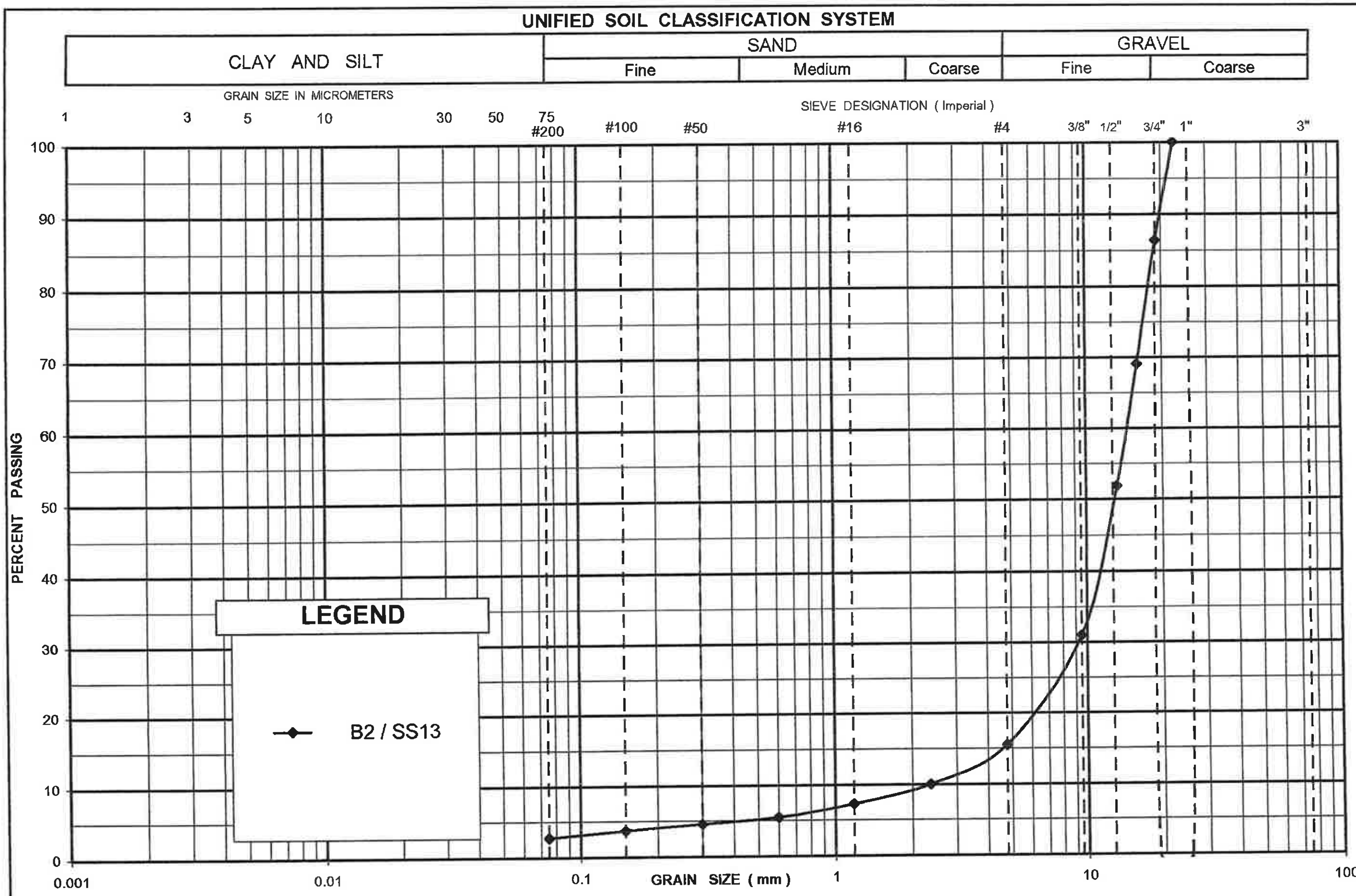
GRAIN SIZE DISTRIBUTION

BASAL GRANULAR SILTY AND GRAVELLY SAND

Figure No. B-11

PROJECT # : TRANETOB01240AA

DATE: October 09, 2009



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GRAIN SIZE DISTRIBUTION

BASAL GRANULAR SOIL: Gravelly Sand

Figure No. B-12	
PROJECT # :	TRANETOB01240AA
DATE:	October 09, 2009

Appendix C

Undrained Shear Strength Plots

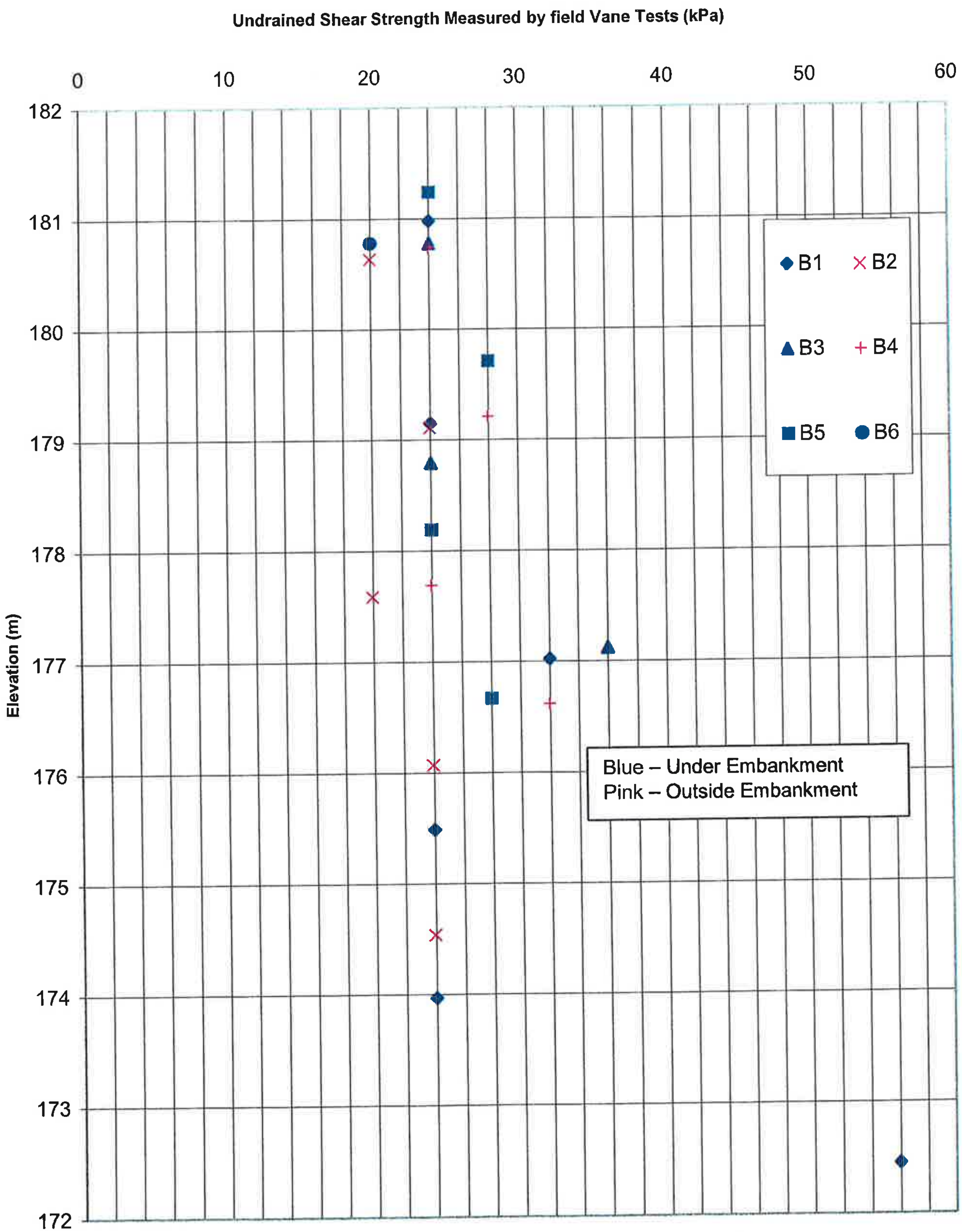
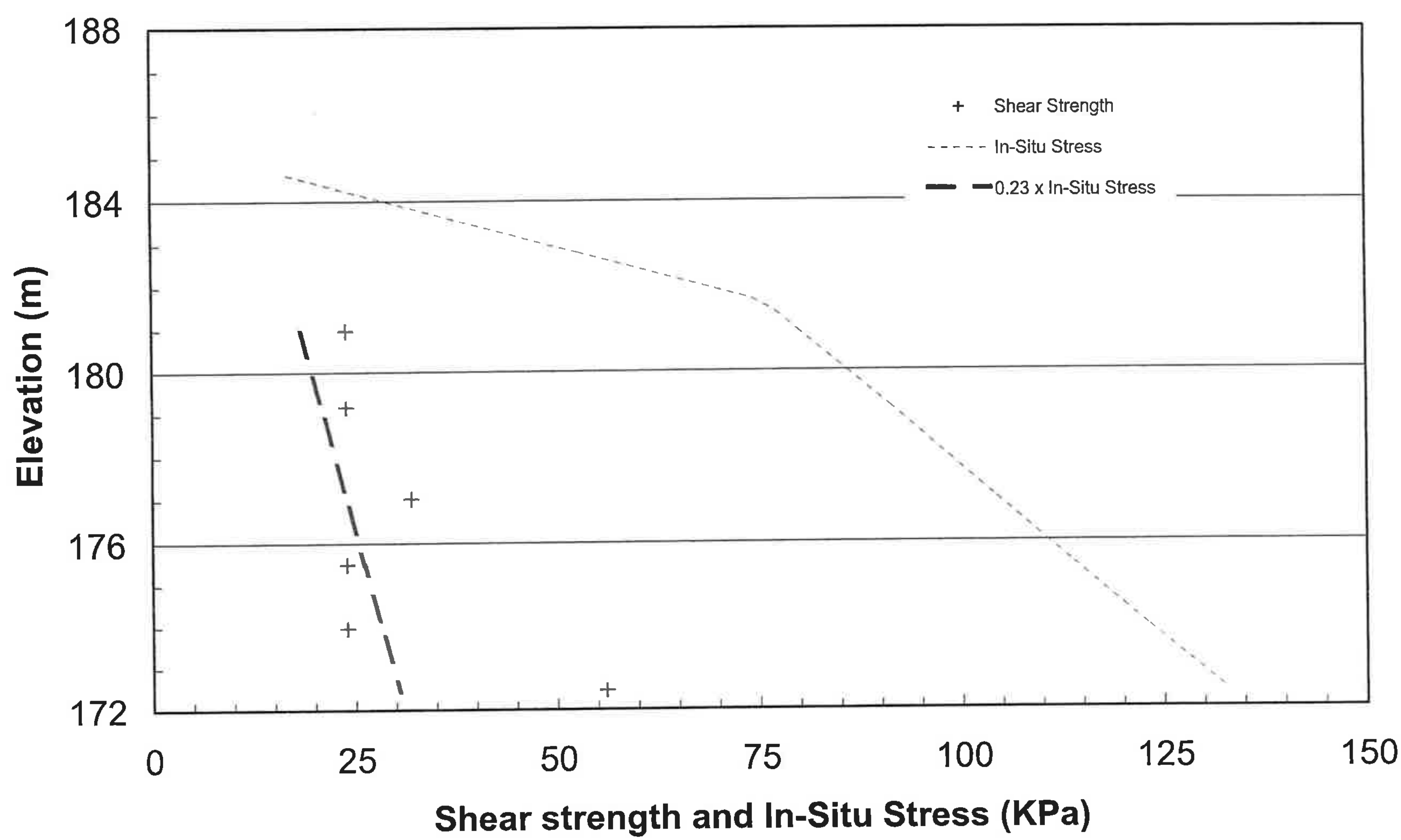


Figure C1. In-situ undrained Shear Strength versus Elevation

Figure C2. Shear Strength versus Elevation (BH B1)



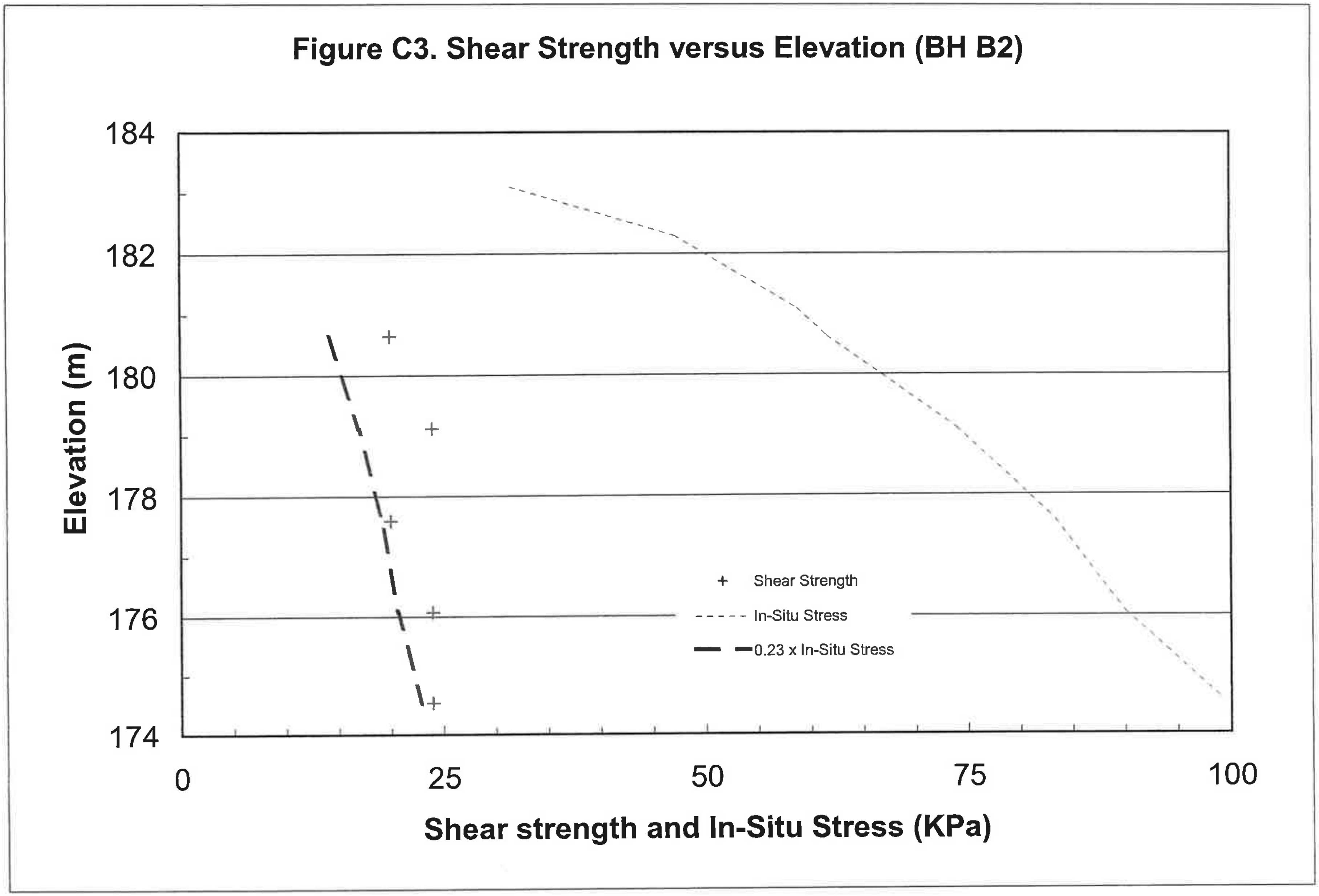


Figure C3. Shear Strength versus Elevation (BH B2)
184
182
180
178
176
174
Elevation (m)
0
25
50
75
100
Shear strength and In-Situ Stress (KPa)
Shear Strength
In-Situ Stress
0.23 x In-Situ Stress

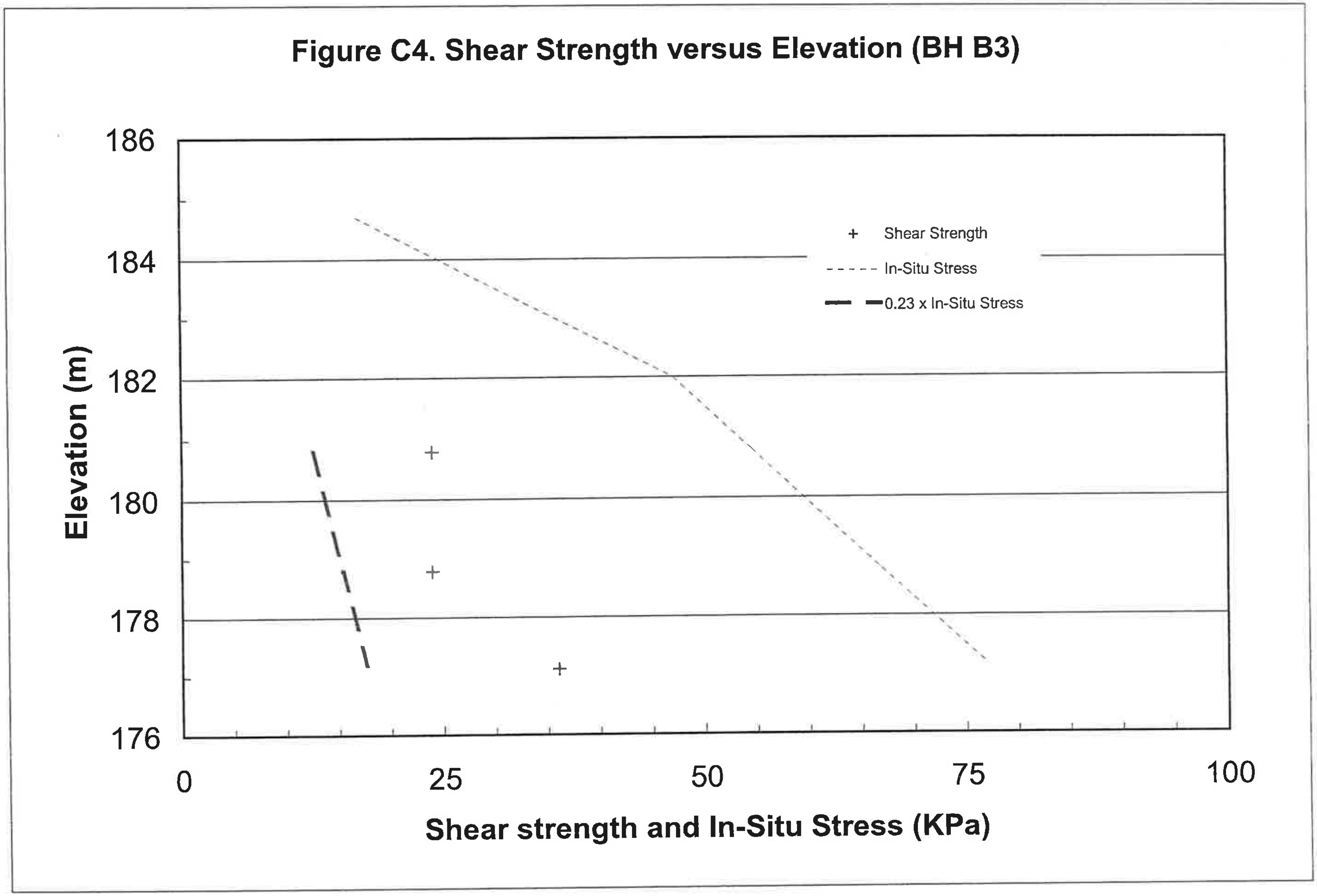


Figure C4. Shear Strength versus Elevation (BH B3)
Elevation (m)
186
184
182
180
178
176
0
25
50
75
100
Shear strength and In-Situ Stress (KPa)
Shear Strength
In-Situ Stress
0.23 x In-Situ Stress

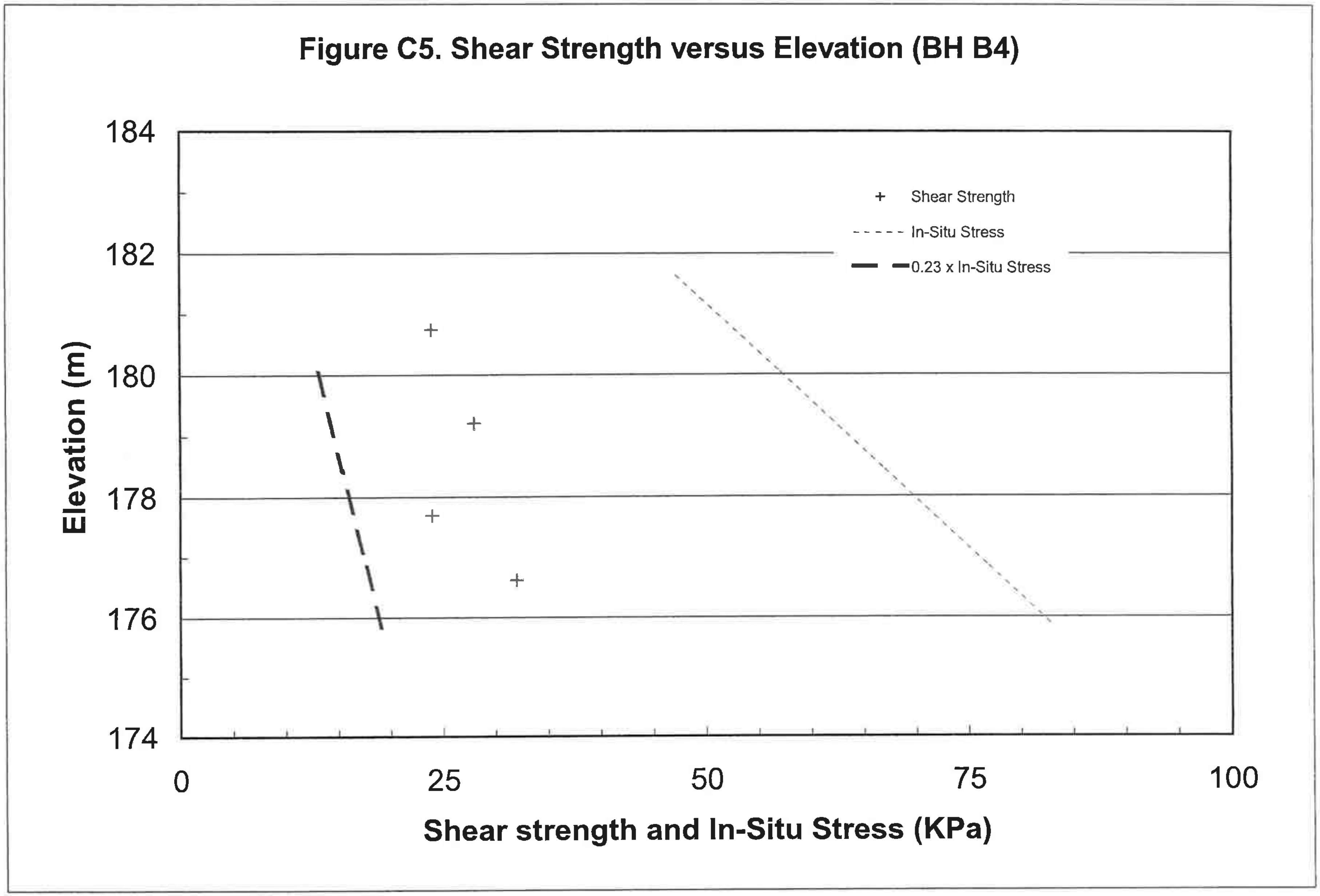
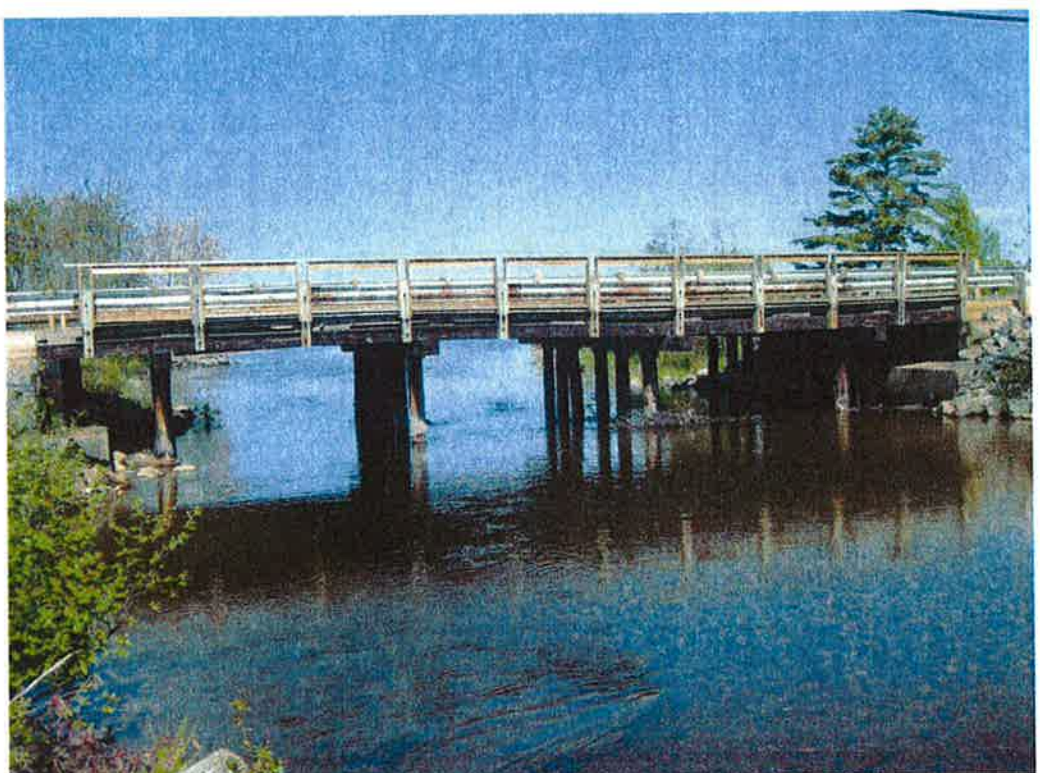


Figure C5. Shear Strength versus Elevation (BH B4)
Shear Strength
In-Situ Stress
0.23 x In-Situ Stress
Elevation (m)
184
182
180
178
176
174
0
25
50
75
100
Shear strength and In-Situ Stress (KPa)

Appendix D

Site Photographs

Photograph D1. Harmony Beach Road Bridge (looking south)



Photograph D2. Harmony Beach Road Bridge (looking west)

Photograph D3. Existing Harmony Beach Road Bridge Foundations

Photograph D4. Exposed bedrock near the junction of Harmony Beach Road and Highway 17

Appendix E

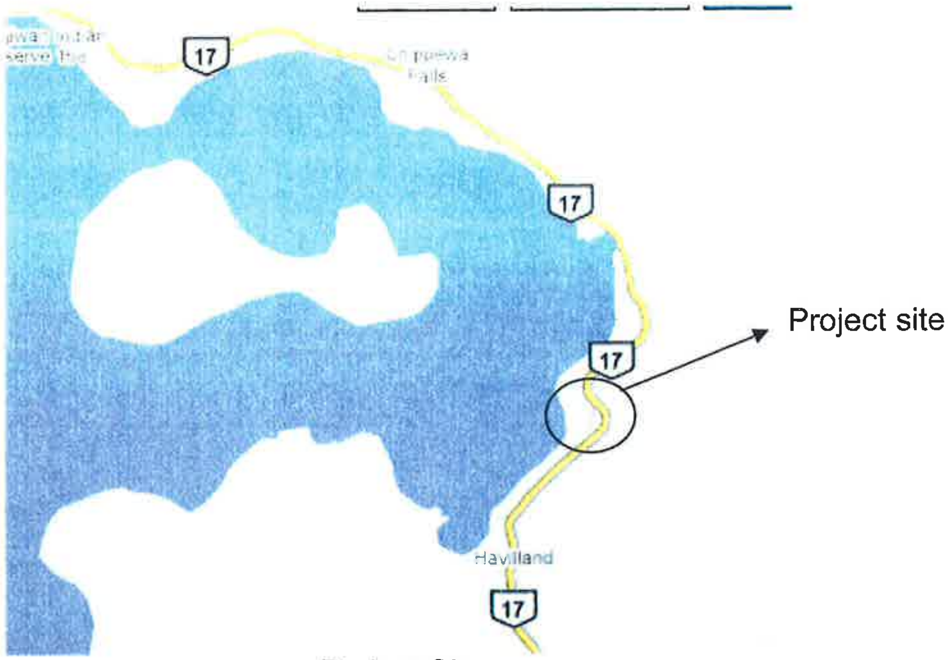
Rock Core Photographs and Geological Map

Photograph E1. Rock cores (RC11, BH B4)

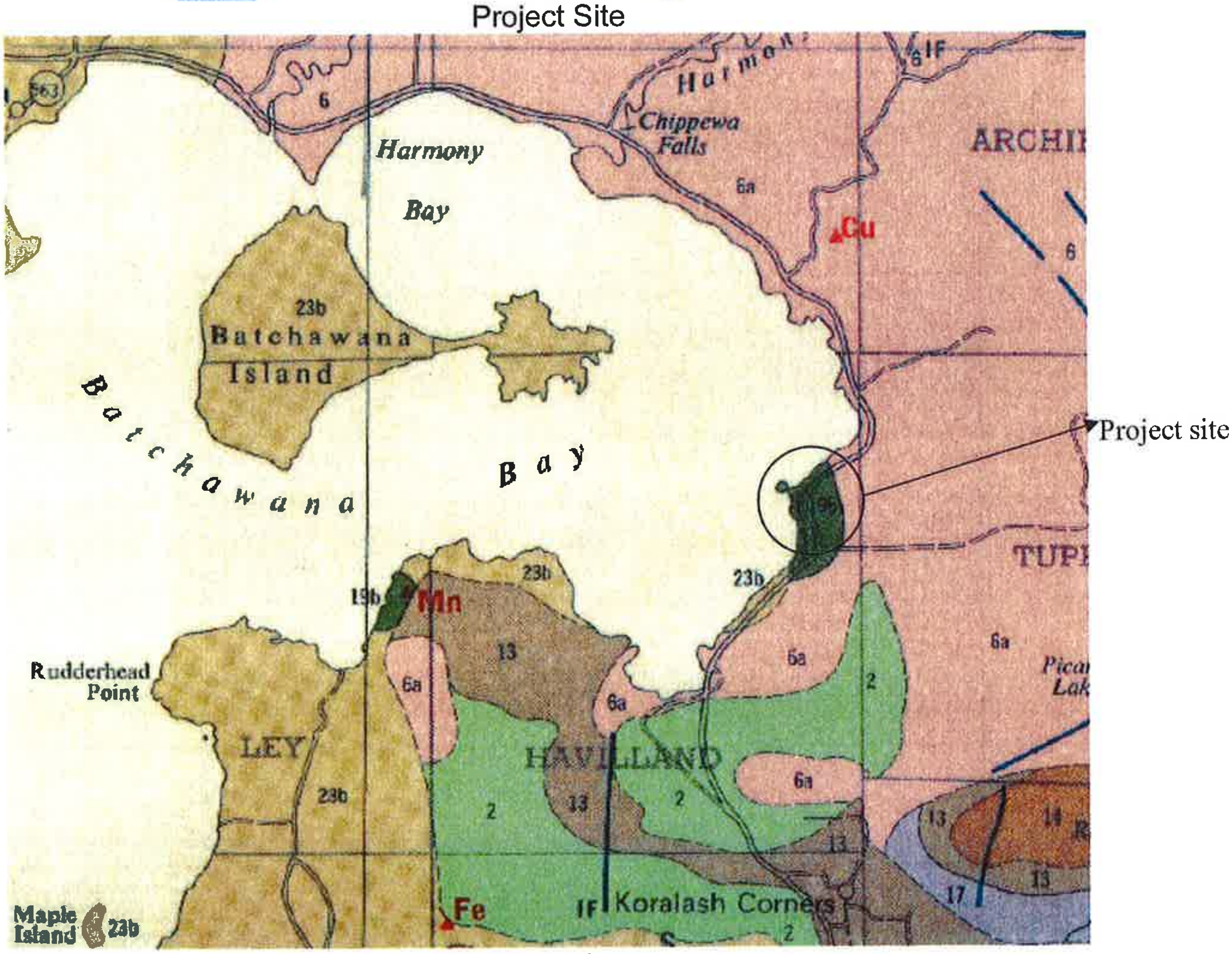
Photograph E2. Rock cores (RC12, BH B4)

Photograph E3. Rock cores (RC15 and 16, BH B3)

Photograph E4. Rock cores (RC17, BH B3)

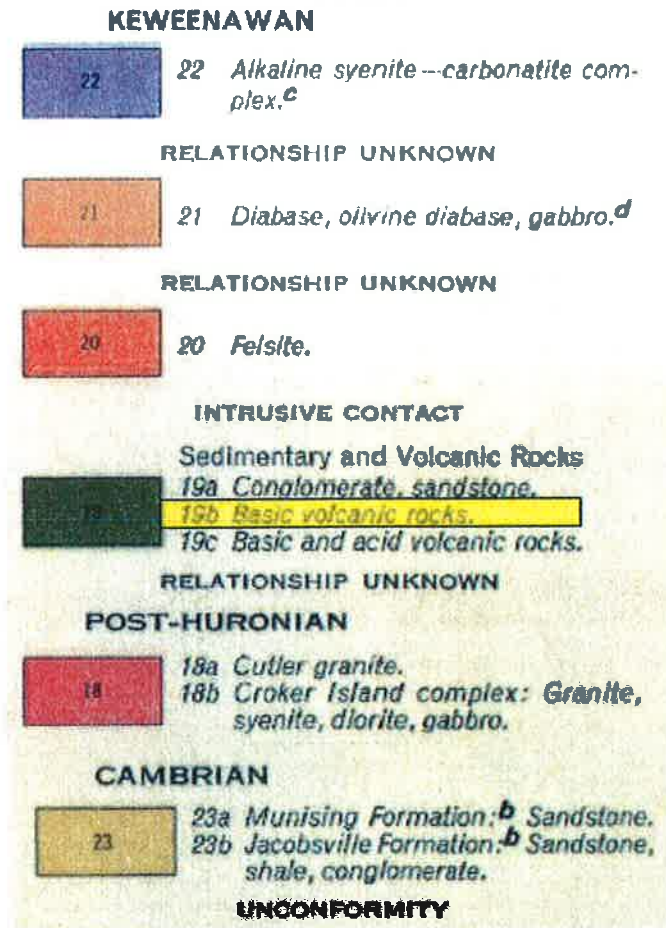


Project Site



Geological map

LEGEND



Basic volcanic rocks (low silicate) according to Ontario Geologic map m2108

Appendix F

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_l, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICALL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_P	%	PLASTICITY INDEX = $(W_L - W_L)$	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
γ_{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
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

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FOUNDATION DESIGN REPORT HARMONY BEACH ROAD BRIDGE REPLACEMENT, HIGHWAY 7090, TOWNSHIP OF HAVILLAND, DISTRICT OF ALGOMA, ONTARIO G.W.P. 5430-06-00, SITE 38S-345 GEOCRES 41K-83

D.M. Wills Associates Limited

Project: TRANETOB01240AA
May 12, 2010

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

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Appendices

FOUNDATION DESIGN REPORT
HARMONY BEACH ROAD BRIDGE REPLACEMENT, HIGHWAY 7045
TOWNSHIP OF HAVILLAND, DISTRICT OF ALGOMA, ONTARIO
G.W.P. 5430-06-00, SITE 38S-345

5 DISCUSSION AND RECOMMENDATIONS

5.1 Proposed Bridge Structure

The existing Harmony Road Bridge is located on Highway 7045 (Harmony Beach Road) approximately 1.3 km north east of Highway 17 junction with Highway 7045 in the Township of Havilland. The existing bridge will be replaced with a new bridge at the same location. This site is located approximately 45 km north of Sault Ste. Marie. The Harmony River flows westerly into the Bachawana Bay of Lake Superior and typical width of the river at the proposed bridge location is about 18 m. The water level in the River on June 1, 2009 was El. 183.0 m and the water level in the River depends primarily on the water level in Lake Superior. The recorded highest water level in Lake Superior was about El. 184.1 m. The bottom of the bridge deck will be at about El. 184.9 m based on the Drawing provided to us by D.M. Wills Associates Limited (Wills). The existing embankment height at the bridge abutment locations is about 2.5 m.

The existing bridge was originally built as a two lane timber bridge (about 7.6 m wide), consisting of five spans. The approximately 19 m long bridge structure is supported by four bents of timber piles and timber abutments founded on concrete footings and/or timber piles. In 2006, the bridge was rehabilitated and restricted to a single 4.1 m wide travelled lane centred on the structure with steel beam guide rail on both sides.

The new bridge structure is proposed to replace the existing bridge at the same location. The original plan was to utilize fully integrated abutment supported on H-piles. Subsequently consideration was given to a light-weight pre-fabricated structure. The originally proposed concrete bridge was an about 20 m long single span, two-lane (9.1 m wide) structure and the grade raise at the approach embankments would be about 0.5 - 0.85 m.

This investigation has shown that below some fill materials (pavement, pavement fill and embankment fill) and some surficial sand, the site is underlain, at about El. 183.4 – 181.1m, by an extensive silty clay to clay deposit. In the deep boreholes this deposit extends to about El. 179.2-172.0 m (i.e. about 2.5 to 9.3 m thick). Its consistency as measured by field vane and SPT field tests is described as soft to stiff but typically soft under the embankment; the clay deposit may be even weaker beyond the embankment. This cohesive deposit is in turn underlain by lower granular soils, with frequent cobbles and boulders. The bedrock was proven by NQ coring in Boreholes B3 and B4 at El. 173.2 and 171.7 m, respectively.

From the measured water levels in the open boreholes upon completion and the piezometer installed in Borehole B4, it is our opinion that groundwater level at the time of investigation was at about El 182 to 183 m. In addition, a perched water condition could also possibly be encountered at the site due to the accumulation of the surface water in the fill materials and in the underlying surficial sand, overlying the practically impervious silty clay to clay deposit.

* Highway 7045 has been changed to Highway 7090.

It should be pointed out that the water levels observed represent the conditions at the time of our investigation and that they would be subject to seasonal fluctuations as well as fluctuations due to weather events and the water level in the Harmony River which highly depends on the water level in Lake Superior.

5.2 Foundations

The very loose to compact surficial sand (in the north abutment location) and the underlying soft silty clay to clay are considered unsuitable to support normal shallow spread footing foundations, including the use of spread footings on engineered fill. As well, significant long term settlements can be expected due to the consolidation of the underlying weak silty clay to clay deposit induced by an about 0.5 to 0.85 m approach embankment grade raise and the widening of the existing embankment.

5.2.1 Integral Abutment Bridge

A concrete/steel bridge will, therefore, need to be supported on deep foundations and measures to reduce differential settlements between the bridge structure and the approach embankment will be required.

The use of drilled and cast-in-place concrete (caisson) foundations to support the structure is considered impractical due to water bearing granular deposits and the lack of a well-defined bearing stratum to support the caissons within the silty clay to clay deposit. This type of foundation is, therefore, not recommended based on reliability and cost. The cassions can be extended (socketed) into the bedrock but this will be very expensive. In addition, this type foundation support is unsuitable for integral abutment type design.

Expanded base (Franki-type) concrete piles and driven concrete piles are not considered to represent a practical, cost-effective and reliable solution.

The boreholes show that with the prevailing subsurface conditions, the use of a low displacement pile, such as a steel H-pile with a heavy section (e.g. HP 310 x 110 or HP 310 x 125), would be better suited than other pile types such as steel tube piles or steel H-piles with lighter sections or precast concrete type piles. In addition, steel H-piles are suitable for integral abutment type bridges.

Consideration was given to the use of steel H-piles with a lighter section (e.g. HP 310 x 74) or steel tube type piles utilizing friction and adhesion. Axial resistances provided by driven piles by this approach (i.e. primarily adhesion) are considered to be unsuitable (i.e. insufficient) for the bridge under consideration. For end bearing, lighter section H-piles are more vulnerable to damage due to the frequent cobbles and boulders which were encountered at feasible refusal elevations.

The most practical option for the bridge north abutment appears to drive the piles to refusal in the granular soils underlying the silty clay to clay deposit. Piles driven to the bedrock is the most practical option for the bridge south abutment, however some of the piles may be "hung-up" on boulders before reaching the bedrock and therefore the resistance will need to be selected taking into consideration this eventuality. The following table summarizes the recommended pile tip elevations and resistances for HP 310 x 110 steel H-piles.

Table 5.2.1.1 Recommended Axial Resistances and Anticipated Tip Elevations for HP 310X110 Steel H-Piles

Borehole No./Location	Existing Ground Elevation (m)	Anticipated Pile Tip Elevation (m)	Assumed Pile Cut-Off Elevation (m)	Corresponding Approximate Pile Length Below Pile Cap (m)	Recommended Geotechnical Resistances for HP 310x110 H-Piles		Refusal Medium
					ULS (kN/pile)	SLS (kN/pile)	
B1 (North Abutment)	185.4	168.0	182.8	14.8	1700	1100	Silty sand, Freq. cobbles
B2 (North Abutment)	184.6	168.0	182.8	14.8	1700	1100	Gravel, Freq. cobbles and boulder
B3 (South Abutment)	185.5	175.5 – 173.2	182.8	9.6 – 12.3	1700	1100	Boulder/Bedrock
B4 (South Abutment)	184.7	171.7	182.8	11.1	1700	1100	Bedrock

For piles driven to bedrock, MTO standard resistance of 2000 – 2400 kN/pile is not recommended here since this is only expected at the south abutment location and it is likely that some of the piles even at this abutment location will encounter refusal before reaching the surface of the bedrock. Due to the anticipated grade raise, the piles can be expected to be subject to downdrag. This is due to the fact that as the clay settles, it will drag the piles down, thus inducing an additional load on the piles due to a phenomenon known as negative skin-friction/adhesion. As per 6.8.4 of Canadian Highway Bridge Design Code (CHBDC, CSA, S6-06), downdrag on the piles is considered as a load. For this project, the unfactored downdrag load can be taken as 300 kN/pile. Load factor typically used for this purpose is 1.25 for ULS. The downdrag acting on the piles can be reduced by the application of bituminous or other viscous coatings to the pile surface before the installation. But this is costly and is not recommended as it is not cost effective for this project. A prolonged surcharge/preload process can also be considered but this too is considered impractical. In any event, high resistances are not necessary for the relatively short-span bridge in consideration for this project.

According to a drawing provided to us by Wills, the elevation for the pile tops will be approximately 182.8 m and therefore length of the piles based on the borehole data can be expected to range from about 9.6 m to 14.8 m. However, the actual pile lengths may vary considerably, as the tip elevations given above are for general guidance purposes only. We recommend that consideration be given to this aspect when ordering the piles. The possibility of piles encountering potential cobbles and boulders in the lower granular soil deposit should be anticipated. We recommend that an NSSP be provided in the Contract Documents to warn the Contractor of the possible presence of cobbles and boulders in the overburden and the possible variations in the actual pile lengths.

Pile Driving at South Abutment Location

At the location of bridge south abutment, the piles may be driven to bedrock. It should be noted that the Hiley Formula is not applicable for piles driven to refusal on bedrock. The pile driving termination or set criteria will depend on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, for piles driven to refusal on bedrock, it is generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then gradually increase the energy over a series of blows to seat the pile.

Alternatively, a refusal criterion of 5 blows for 6 mm for three consecutive sets can be maintained for practical refusal on bedrock, based on our pile driving experience in Ontario. As well, 16 blows for 20 mm or 20 blows for 25 mm penetration can also be used. These values are based on typical hammer energy of 60 kilojoules/blow, with an energy transfer (efficiency) of 40%.

If and where the piles encounter refusal before proper penetration is achieved, 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 a suitable bearing zone is reached, the Foundation Design Engineer and the Bridge Design Engineer should be consulted to assess the axial resistance and the minimum pile length requirements. It is also possible that the piles may be driven some distance below the estimated pile tip elevations.

Pile Driving at North Abutment Location

At the north abutment location, piles are expected drive into the lower granular soil deposit (i.e. encounter refusal within the overburden) and thus the driving of the piles in the field should be monitored by a recognized pile driving formula such as the Hiley Formula. The estimated ultimate resistance of the piles by the Hiley Formula can be calculated by dividing the recommended axial resistance at U.L.S. by a resistance factor of 0.4 as per current MTO practice. In this instance the recommended ultimate resistance is 1700 ÷ 0.4 = 4250kN. As the actual driving of the piles in the field will be governed by the Hiley Formula, the pile tip elevations given in the Table 5.1.1.1 are for general guidance purposes only and the actual pile lengths may be different than the lengths quoted. We recommend that an NSSP be prepared to inform the Contractor of this possibility.

In accordance with the above criterion, the piles may be driven to about 4 m above the design elevation and driving then monitored by employing the Hiley Dynamic Formula in accordance with MTO Standard Drawing SS103-11.

Overall Comments on Pile Foundations

All pile driving should be carried out in accordance with SP903S01. Re-striking should be done as per SP903S01. After each pile is installed, an elevation should be taken of the pile top or on a suitable mark on the side of the pile. This elevation should be checked periodically to confirm that the pile has not heaved as a result of the driving of adjacent piles. Piles that are heaved must be re-driven to the required resistance as required by the engineer. 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 SP903 S01, to check that relaxation has not occurred. If it has, then all the piles should be re-tapped. Re-tapping of piles driven to bedrock is not required.

While pile heave/relaxation is not anticipated, if it is observed, it may be necessary to stagger the driving of the piles. The use of light-weight (e.g. HP 310 x 79) piles is not recommended as lighter piles are more vulnerable to damage. Consideration should be given to provide an NSSP to alert the contractor of the possible presence of cobbles and boulders in overburden and possible heavy driving requirements through the dense to very dense strata.

In view of the fact that the frequent cobbles and boulders were encountered in the deep boreholes, it is desirable that the piles be reinforced as per OPSD 3000.100 or Titus standard H-bearing points (or APF hard bite or approved equivalent) can be utilized to reinforce the pile tip to prevent damage to the pile during the anticipated heavy driving conditions and to ensure adequate seating of the piles (on the bedrock at south abutment location). Care must be taken to avoid overdriving and damaging the pile tip.

For frost protection, all pile caps should have a permanent earth cover of at least 2.2 m or be provided with an equivalent thickness of extruded rigid exterior-grade polystyrene insulation.

Eccentric loading on piles and the required pile spacing should be considered as per the latest CHBDC (CSA, S6-06). Reference may be made to Section C6-8.7.1 of the CHBDC (CSA, S6-06), 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 5.2.1.2

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 ks = coefficient of horizontal subgrade reaction

c_u = undrained shear strength

d = width of pile

Table 5.2.1.2

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)
B1	182.8-181.4 181.4-172.0 172.0-166.9	Sand Silty clay to Clay Silty sand	19.0 16.0 20.5	30 - 32	1.3 - 11.0	- 25 -	182.5*
B2	182.8-181.1 181.1-173.2 173.2-169.6 169.6-166.2	Sand Silty clay to Clay Sand Gravel	19.0 16.0 19.5 20.5	30 - 31 34	1.3 - 6.0 11.0	- 25 - -	182.5*
B3	182.8-182.0 182.0-177.0 177.0-175.5 175.5-173.2	Fill: Sand with gravel Silty clay to Clay Silty sand Gravel	20.0 16.0 20.0 18.0	32 - 31 33	1.3 - 8.0 10.0	- 25 - -	183.0*
B4	182.8-182.4 182.4-174.8 174.8-173.0 173.0-171.7	Fill: Sand & gravel Silty clay to clay Sand Sand	19.5 16.0 19.0 20.5	31 - 30 32	1.3 - 1.3 10.0	- 27 - -	182.3**

* estimated.

** Groundwater measurement in piezometer (stabilized).

For preliminary estimating purposes, the recommended horizontal resistances for HP310 x 110 steel H-piles are as follows:

Factored Horizontal Resistance at U.L.S. = 110 kN/pile

Horizontal Resistance at S.L.S. = 30 kN/pile

At the abutments, if integral type abutments are not to be constructed, then the use of battered piles can be considered to obtain additional lateral resistance. In this case, we recommend that the batter be limited to 4:1, for practical purposes.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles would be driven.

Pile lengths could be different than the quoted figures and therefore, this aspect will need to be considered for estimating purposes and when ordering the piles. It would be prudent to mention this in the contract documents.

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone.

MTO structural office requirements (Report SO-96-01) indicate that 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 supply and installation of the CSP's, including the gradation of the sand. The special provision is given in Appendix L; the required gradation of the uniform sand is presented in the following Table.

Table 5.2.1.3 Sand Gradation required for the Flex Zone

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

5.2.1.1 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 and OPSD 3101.200.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B' Type I or Type II, with minus 0.075 mm sieve size material not exceeding 5%) and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with C.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

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

Angle of Internal Friction, $\phi = 35°$ (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, $\phi = 32°$ (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 retaining structure occurs but not in a sufficient magnitude to fully mobilize an active condition (as such an intermediate condition between K_o and K_a occurs).

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 (e.g. supported on bedrock), 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 can be consulted.

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

5.2.2 Foundations For Prefabricated Bridge

After the analysis and preparation of a design report incorporating a 20 m long single span integral abutment concrete bridge, MTO expressed a preference for a prefabricated steel girder bridge, for which further analysis was carried out and our report was revised to include this option.

MTO Northeastern Region has experience with this type of bridge where for foundation support a small (typically 0.3 m wide) precast concrete footing is placed on a pad of 0.3 m thick granular fill (see Figure K-1 in Appendix K). We understand that the main advantages of this type of structure are its cost, being considerably more economical than a built-in-place conventional bridge, along with the capability to withstand settlements and frost movements without detrimental effects, if periodically adjusted, where necessary. The new bridge at this site would be 5.5 m wide which is narrower than the existing 7.6 m wide structure and the originally proposed concrete bridge (9.1 m wide).

We understand that the prefabricated bridge is proposed to be 80 feet (24.4 m) long, but the length can be increased by up to 13 feet to 93 feet (28.4 m) without causing an increase in the depth for the steel girders. We recommend this approach (i.e. 28.4 m long structure), as a longer bridge

- reduces disturbance immediately adjacent to the existing bridge abutments (i.e. the existing support can be left in place if desired, which is environmentally less intrusive and may be beneficial for scour protection)
- reduces the amount of grade raise required near the edges of the embankment which would reduce future differential settlements
- provides better foundation conditions (i.e. moves the foundations for this bridge away from the forward slopes thus increasing the geotechnical resistance of the footing).
- improves slope stability.

- reduces the risk of dewatering.

We recommend that the bridge be supported on a 0.6 m high, 0.8 m wide and 6.4 m long footings as shown in Figure K-2 in Appendix K, prepared by Wills. The width of the footings (i.e. 0.8 m) was chosen using preliminary calculations based on proposed geotechnical resistances of ULS = 230 kPa and SLS = 150 kPa. . This should however be checked by the Structural Engineer, using the resistance values given in this report. We understand that the underside of the 0.6 m high and 0.8 m wide concrete footing will be at El. 184.2 m. The following procedure is recommended to prepare the base for the footing.

- Excavate to one footing width below the underside of the proposed footing. As the underside of the proposed footing will be at El. 184.2 m, the excavation would be carried out to El. 183.4 m. This excavation elevation was chosen in order to keep the stresses in the underlying weak clay to acceptable levels while keeping the excavation level to above the recorded groundwater/river water levels, at the time of our investigation.

- The bottom of the excavation would be 0.8 m wide (i.e. proposed footing width). A wider excavation would be more effective in distributing the footing pressures, but would result in a very wide excavation and disturbance near the surface.

- The temporary side slopes would be carefully constructed as steep as practically possible providing they are safe, but in no case steeper than 1H:1V, as shown in Figure K-2 (see detail A) in Appendix K. If necessary, shoring will need to be applied to ensure a safe excavation.

- We recommend that the excavation be carried out when the water level in the River is low and certainly no higher than El. 183.0 m.

- After the excavation is carried to the required elevation, the exposed subgrade at the bottom of the excavation would be inspected to ensure that the base is free of organic and otherwise unsuitable materials. If encountered, these should be removed and replaced with suitable soils. If conditions permit, after its inspection, evaluation and approval, the bottom of the excavation should be lightly compacted from the surface. This must however be conducted in a safe manner whereby

  - The sides of excavation are safe

  - The bottom of the excavation is not unduly disturbed due to water pumping upwards, from beneath the base of the excavation, due to vibrations induced by compaction and foot traffic.

- The excavation would then be backfilled with Granular 'B' Type II material (with less than 5% fines), placed in two lifts; the bottom 0.4 m thick lift would be compacted to not less than 95 % of the material's Standard Proctor Maximum Dry Density (SPMDD) and would include a properly filtered subdrain at the bottom which is free to drain, while the top 0.4 m lift would be compacted to not less than 96% of its SPMDD.

With these procedures, the following geotechnical resistances can be assigned for a 0.8 m wide footing, embedded at least 0.4 m into the granular fill.

Factored Geotechnical Resistance at ULS = 230 kPa

Geotechnical Resistance at SLS= 150 kPa*

As mentioned before, the width of the proposed footing should be checked using these values. Based on this configuration, the following settlements are anticipated

Settlement of footing due to structural loading:

- 20 mm settlement in the fill and the underlying surficial sand (short term settlements)
- 50 mm consolidation settlement in the underlying silty clay to clay deposit (i.e. long term settlements)
- Total settlements due to the foundation loading = 70 mm*

The long term (consolidation) settlement due to grade raise

- 0.6 m grade raise = 100 mm
- 0.85 m grade raise = 155 mm

bringing the total long term settlements to 150 mm and 205 mm, respectively.

We understand that settlement of these magnitudes is acceptable for this type of structure, since the structure can be lifted periodically as the settlements take place. In addition, some up and down movements can be expected to occur due to frost heave, since the depth of the frost penetration in the general area is 2.2 m. It should, however, be pointed out that with this configuration (i.e. 0.8 m thick fully drained, non-frost susceptible soil), movements due to frost would be less than if only 0.3 m thick granular soil would be provided beneath the abutment footings, as was originally proposed (see Figure K-1 in Appendix K).

It the anticipated settlements are unacceptable, then the proposed structure can be supported on driven piles. A discussion of driven piles was provided in section 5.2.1 of this report. We will provide further project specific information on this aspect if driven piles are to be considered for the proposed prefabricated bridge.

5.3 Approach Embankments

It is expected that the grade will be raised by up to 0.85 m above the existing embankment top, along with embankment widening (i.e. in the widened sections the grade raise will exceed 0.85 m), as shown in the cross-sections (provided to us by Wills) in Appendix G.

Based on the borehole data, no foundation failures are anticipated with normal 2H:1V side slopes, assuming that all organic or otherwise unsuitable materials within the proposed embankment footprint will be removed as per MTO standards prior to placing the embankment fills.

After stripping, the exposed subgrade in the proposed embankment widening area should be inspected and approved. The approved subgrade should be compacted from the surface using a suitable compactor.

The material used for the construction of the embankment fills should consist of approved, acceptable earth fill. Oversize materials (having a nominal diameter in excess of 75 mm) should not be used in embankment fills through which piles would be driven. Fill used for construction of the embankments should be in accordance with OPSS 212 and fill placement should meet or exceed the requirements of OPSS 501 and OPSS 206. In general, the fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density.

Proper erosion control measures should be implemented by seed and cover (OPSS 572) and sodding (OPSS 571).

The settlement of embankment fills in the widening portion, under their own weight, prepared as described above, should not exceed 12 mm for embankment heights of up to about 2 m. This settlement is additional to foundation soil settlements, which will be discussed in Section 5.3.2. The time-rate of settlement will depend on the materials used for the construction. For example, granular soils will settle more rapidly than finer soils.

The settlement of the existing embankment fills after the proposed grade raise will depend on the compactness or consistency of the embankment fill materials at the site. But typically this should not exceed 15 mm, under the proposed grade raises, as shown in Appendix G. These settlements are expected to take place rather rapidly (say with four weeks), as based on the boreholes drilled, the existing embankments appear to have been constructed from basically granular soils.

5.3.1 Approach Embankment Stability

Slope stability analysis was carried out using the information provided to us by Wills, as given in Appendix H. The stability of the proposed embankments was analysed by the limit equilibrium approach. The analysis was carried out using the commercial two-dimensional slope stability computer program Slope/W and the Mogerstern-Price method of analysis for both short term (undrained) and long term (drained) analysis calculations.

The soil profiles used for slope stability were based on the boreholes drilled on each side of the River (i.e. Boreholes 1 and 2 on the north side and Boreholes 3 and 4 on the south side). The soil parameters adopted in the analysis are summarized in Table 5.3.1.1.

Table 5.3.1.1

Soil Type	Unit Weight (kN/m^3)	Shear Strength Parameters			
		Undrained		Drained	
		Shear Strength (kPa)	Angle of internal friction (deg)	Cohesion (kPa)	Effective angle of internal friction (deg)
New Embankment Fill	21	-	32	-	32
Existing Embankment Fill	19.5	-	30	-	30

Soil Type	Unit Weight (kN/m^3)	Shear Strength Parameters			
		Undrained		Drained	
		Shear Strength (kPa)	Angle of internal friction (deg)	Cohesion (kPa)	Effective angle of internal friction (deg)
Surficial Sand	19.0	-	30	-	30
Silty Clay to Clay	16.0	25	-	2	26
Lower Granular Soil	21	-	32-34	-	32-34

The analyses were carried out using the cross sections shown in Appendices G2 and G3, which were provided to us by Wills.

In summary the calculated minimum safety factors range between 1.31 and 1.63 and therefore, it is our opinion that the proposed grade raise and embankment widening constructed to 2H:1V side slopes for the bridge approaches should not cause stability issues, due to foundation failures.

5.3.2 Approach Embankment Settlements

We understand that the project incorporates a grade raise of up to 0.85 m on the existing approach embankment with widening based on the drawings presented in Appendix G, as provided to us by Wills. Appendices G2 and G3 present the proposed grade raises for a concrete bridge and a prefabricated bridge, respectively. For both cases, a maximum 0.85 m grade raise is proposed on south approach embankment while an about 0.6 m grade raise is proposed on north approach embankment. As mentioned before the grade raise along the widened area is as high as 2 m for the original proposal (i.e. concrete bridge). For the prefabricated, which is longer, bridge, the maximum grade raise in the widened area is only up to 1.0 m (i.e. considerably less than the originally proposed shorter bridge). These grade raises are expected to cause settlements. The anticipated settlements due to compression of the new embankment under its own weight and the settlement of the existing embankments were previously discussed (see Section 5.2). In addition to these settlements, the settlement of the foundation soils will also occur, including the consolidation settlement of the underlying silty clay to clay deposit. Obviously, in this respect, the longer prefabricated bridge presents a significant advantage as the consolidation settlement of the silty clay to clay deposit underlying the site due to up to 1.0 m grade raise can be expected to be considerably less in comparison with a 2.0 m grade raise, in the widened zones. In addition, the widened zone itself can be somewhat narrower under the prefabricated bridge in comparison with the widened concrete bridge, which also reduces the anticipated stresses and thus the settlements.

The consolidation characteristics of the silty clay to clay deposit were investigated by means of two one-dimensional consolidation (oedometer) tests. The results of these tests are presented in Appendix B. The tests were performed from samples recovered from Borehole B1 (TW 7 and TW11), located at the proposed north abutment location. The test results indicate normally consolidated soil that has a pre-consolidation pressure (p'_c) which is very close to the existing overburden in-situ pressure (P'_o).

At the north abutment location, the estimated primary settlement of the 7.9 to 9.4 m thick silty clay to clay under up to 0.6 m grade raise, based on the consolidation test data, is about 100 mm (t_{90} = 6 years) while at the south abutment location, the primary settlement of the 5.0 to 7.6 m thick silty clay to clay under up to 0.85 m grade raise is estimated to be about 155 mm (t_{90} = 4 years).

5.3.2.1 Concrete Bridge Option

In the case of a concrete bridge, based on the cross sections given in Appendix G2, beyond the existing embankment, up to about 360 mm total settlement (t_{90} = 6years) is expected to occur due to the embankment widening (e.g. where the grade raise is 2.0 m – see proposed cross section at Station 10+190). This will cause differential settlements between the center and the widened portions of the road. The settlement of the surficial granular soil at the north abutment location can be expected to take place rapidly, while the consolidation of the underlying silty clay to clay deposit at both abutment locations can be expected to proceed at a much slower pace. A secondary settlement amounting to about 4 % of the total settlements quoted above can be expected to occur due to secondary consolidation, after the completion of the primary settlement periods.

To estimate the rate of settlement of the 5.0 to 9.4 m thick clay deposit, C_v values obtained from the laboratory consolidation tests were utilized. These range from $3x10^{-3}$ to $6x10^{-4}$ cm^2/sec. However, experience shows that C_v values obtained from laboratory tests are typically 5 to 10 times lower than values in the field. Using a value of C_v=$1X10^{-3}cm^2$/sec, the time required to obtain a 90% consolidation is about 2 years for 5.0 m thickness (at Borehole B3 location) to 6 years (for 9.4 m thickness at Borehole B1).

Four options were considered to provide alternative construction approaches to alleviate these settlements:

- Use of surcharging to promote early embankment settlement (with full road closure)
- Use of pre-loading on the proposed widening area (without road closure) prior to paving with progressive maintenance after paving.
- Utilizing light weight fill
- Lengthen the bridge to reduce grade raise

These options are described below.

Surcharge Option with Full Road Closure

Since settlements of above mentioned magnitude would result in a requirement for progressive maintenance immediately adjacent to the bridge (i.e. settlements will manifest themselves as differential settlements), as well as causing downdrag on the piles, surcharging may be required to speed up the rate of consolidation of the clay. We understand that for this project surcharging can be applied to the existing embankment as well as the proposed embankment widening areas, but the maximum surcharging period would only be six months (with full road closure). High surcharge heights would jeopardize slope stability, as well as being impractical. Monitoring may be required to measure settlement response to check design assumptions and forecast settlement performance during operation.

A stability analyses was performed and typical results are presented in Appendix H.

Figures H-2 and H-4 in Appendix H show several surcharge cross sections using a surcharge height of 1.0 m. Due to the property limitations, side slopes of the embankments during surcharging will be slightly steeper than 2H:1V slope. The calculated factor of safety is slightly less than 1.3. For this reason we recommend that the surcharge height be kept a maximum of 0.8 m. The anticipated post construction settlements after surcharging for a time period of six months using a surcharge height of 0.8 m over and above the proposed new road grades are presented in Appendix I.

Pre-loading Option without Road Closure

Pre-loading is a feasible option to reduce the anticipated settlements at this project location but it is less effective in speeding up the consolidation of the clay deposit in comparison with surcharging. Progressive maintenance after construction would be required for this option, as the remaining settlements after preloading are greater than surcharging option, as shown in Appendix I.

Light Weight Fill Option

If the anticipated embankment settlement is not considered acceptable, another approach would be use light weight fill. The light weight fill can be used with or without surcharging. The use of expanded polystyrene or slag can be considered. In this event, the use of expanded polystyrene blocks (EPS) would be the recommended option.

In principle, the EPS thickness would reflect the fill thickness (i.e. where the proposed fill thickness is larger, the EPS would be thicker). As such, since the height of the fill is greatest immediately adjacent to the bridge abutments and towards the edge of the embankment, the thickness of the EPS would be greater near the bridge abutments and near the edges, gradually decreasing further south and north. The presently proposed vertical and horizontal alignments are given in Appendix G. The following design criteria with the EPS option are recommended.

- The recommended thickness of the pavement fill over the EPS is 1.3 m with a concrete cover over the EPS and 1.4 m without a concrete cover. At present, MTO design requirements include a 125 mm thick concrete cover over the EPS, as shown in Appendix J, but this is under review since there have been reported cases of cracking of the concrete especially where post construction settlements occur. The design and construction of the EPS should be in accordance with MTO Special Provision entitled "Expanded Polystyrene Embankment."
- We understand that possible highest water level in Lake Superior is 184.05 m. The bottom of EPS should be therefore not be extended below El. 181.8 m, to prevent an uplift condition. This elevation is for preliminary design purposes. It may vary, depending on the details and it should therefore be recalculated by us, if this approach is to be considered.
- Depending on the design, an earth cover of 0.7 to 1.0 m should be provided over the EPS on the side slopes to prevent a possible uplift, as well as to avoid damage due to ultra-violet light exposure.
- The soil underlying the EPS should be well compacted and the top 0.15 m of the soil should consist of sand with no gravel to prevent damage to the EPS.

The following procedures will likely apply.

The site to receive the EPS would be surcharged by 0.8 m over the full foot-print of the proposed embankment. Since, for example, immediately adjacent to the proposed north abutment location, the proposed grade for the finished roadway is at 185.8 m, the surcharge fill would be placed to El. 186.6 m (i.e. 185.8+0.8 m surcharge), while at Station 10+170, the proposed elevation of the embankment is 185.6 m and therefore the top of surcharge would be at 186.4 m. The surcharge would be placed at least six months prior to construction. After surcharging (i.e. at least six months), the fills would be removed and EPS would be placed and road would be built. The granular fill and asphalt overlying the EPS, as was mentioned before, would be 1.3 m underlain by a 125 mm thick concrete covering the EPS.

Depending on the thickness of EPS used, the settlements with this approach would be within an acceptable range for a secondary highway, especially with surcharging.

We will be pleased to discuss further details of the approach if a pre-fabricated bridge construction is not planned.

Lengthening the Bridge to Reduce Grade Raise

Lengthening the concrete bridge to reduce grade raise is a feasible but expensive option. As was discussed earlier in the report, however, this would be a recommended option for a pre-fabricated bridge.

5.3.2.2 Prefabricated Bridge Option Incorporating A 28.4 m Long Span

As was discussed before, we understand that the length of the prefabricated bridge can be increase to 28.4 m without a substantial cost increase. Among other benefits this approach reduces the anticipated differential settlements. In this case the maximum grade raise due to widening will be about 1.0 m and the anticipated maximum settlement would be 120 mm. As the anticipated settlement under the central portion is about 100 mm, this will create some minor distortions. This can be rectified by providing a 0.5 m high surcharge for a period of about six months which we understand is feasible. If surcharging is impractical then a preload period of six months can be considered. The anticipated residual settlements after surcharging/preloading are 60 / 75 mm with 0.6 m grade raise (at north abutment location) and 90 / 110 mm with 0.85 m grade raise (at south abutment location), respectively. These settlements are probably acceptable for secondary highway but may require some future maintenance. In any event, we recommend that to minimize the effects of possible deformations in the pavement due to uneven loading effects, the paving of the road be delayed, if possible, for a period of one year, but not less than three months.

5.4 Construction Comments

It is anticipated that the bulk of the construction, including stripping operations for embankment construction (widening) will take place in surficial granular soils above or close to the groundwater level and therefore, no major problems are foreseen during earthworks due to groundwater, especially for the longer prefabricated bridge. After stripping, the exposed subgrade should be inspected, approved and properly compacted (i.e. proof rolled) from the surface, using a suitable compactor. Where necessary (e.g. close to the River in the case of a 20 m long concrete bridge), the groundwater table would be lowered to about 0.6 m in below the subgrade level, before any proofrolling and the application of significant compaction effort. Depending on the water level in the River, it may be possible to achieve this by gravity drainage and pumping from strategically placed filtered sumps.

Surcharge construction, including the cross sections for the embankments during the surcharging, was discussed in earlier sections. The erosion of the embankments must be prevented during the surcharging period.

Assuming properly compacted, acceptable inorganic earth fill materials are utilized, 2H:1V side slopes can be used for the construction of the approach embankment. 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) where embankment widening is proposed.

5.5 Scour Protection

We recommend that channel and bridge scour protection and erosion control be designed by an experienced Hydraulic Engineer. The following should be considered for erosion/scour control measures.

- Flow rate
- Water depth
- Type of transported sediments
- Detailed cross section survey
- Stream pattern and alignment
- Channel gradient
- Effects of the constriction of river flow due to the construction of the bridge piers
- Effect of flooding

The following are some suggestions which would be subject to review and revision during design by an experienced Hydraulic Engineer. The scour/erosion protection can possibly consist of 0.5 m thick R-50 size rock, as per OPSS 1004. A granular filter or a suitable geotextile will be required for separation and filtering purposes. Granular filter can consist of a 150 mm thick layer of concrete Fine Aggregates (Type FA1) underlain by another 150 mm thick layer of Concrete Coarse Aggregates (Group I/20-5). Alternatively, a robust geotextile such as Terafix R-400 (or equivalent) can be placed in lieu of the natural filter materials. All materials will need to be machine placed in a manner to avoid segregation. The scour/erosion system should be placed at least 0.3 m above the 1:100 year storm elevation. It is furthermore recommended some form of scour/erosion protection be extended at least 5 m into the river bed. One advantage of a 28.4 m long prefabricated bridge is that where feasible the existing abutments and other existing scour and erosion protection measures can be left in place, provided they appear to be working for the existing bridge. We will be pleased to further discuss these aspects, if you wish us to do so.

The prevention of erosion and scour is particularly important for the prefabricated bridge which is expected to be supported on spread footing foundations.

5.6 Frost Protection

Design frost protection depth for the general area is about 2.2 m. Therefore, a permanent soil cover of about 2.2 m or its thermal equivalent of artificial insulation is required for frost protection of foundations, including pile caps. In case of rockfill, only one-half of the rockfill thickness should be assumed to be effective in providing frost protection.

6 CLOSURE

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

For and on behalf of Coffey Geotechnics Inc.

Gwangha Roh, Ph.D.

LICENSED PROFESSIONAL ENGINEER
RAMON MIRANDA
PROVINCE OF ONTARIO

Ramon Miranda, P.Eng.

REGISTERED PROFESSIONAL ENGINEER
Z. S. OZDEN
PROVINCE OF ONTARIO

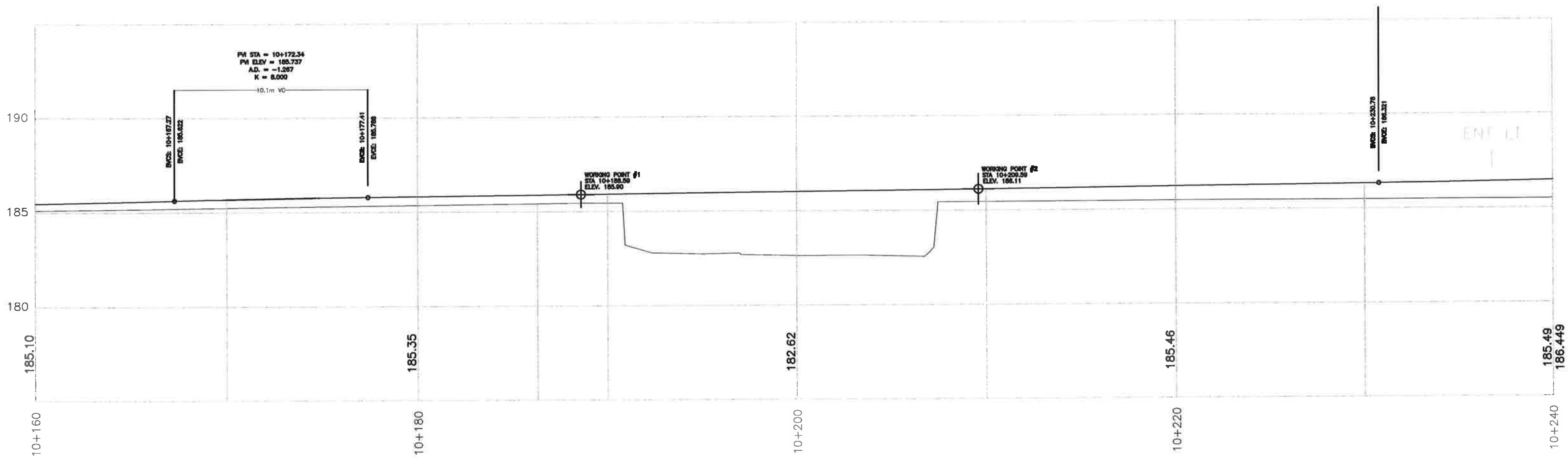
Zuhtu Ozden, P.Eng.

Appendix G

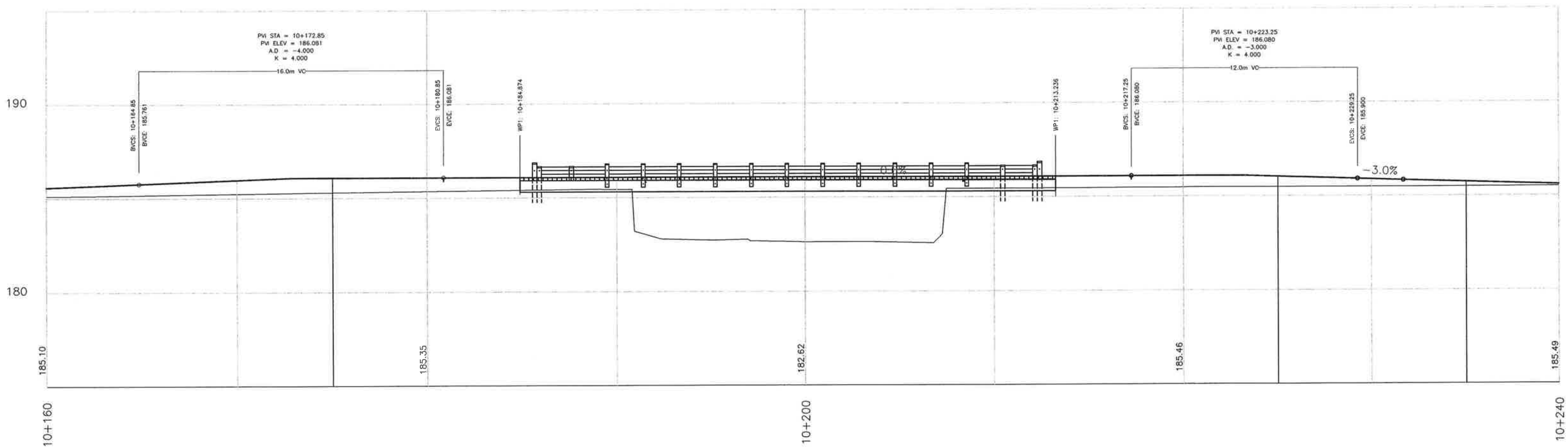
Profile and Cross-Sections of the Harmony Beach Road

Appendix G1

Profile Drawings



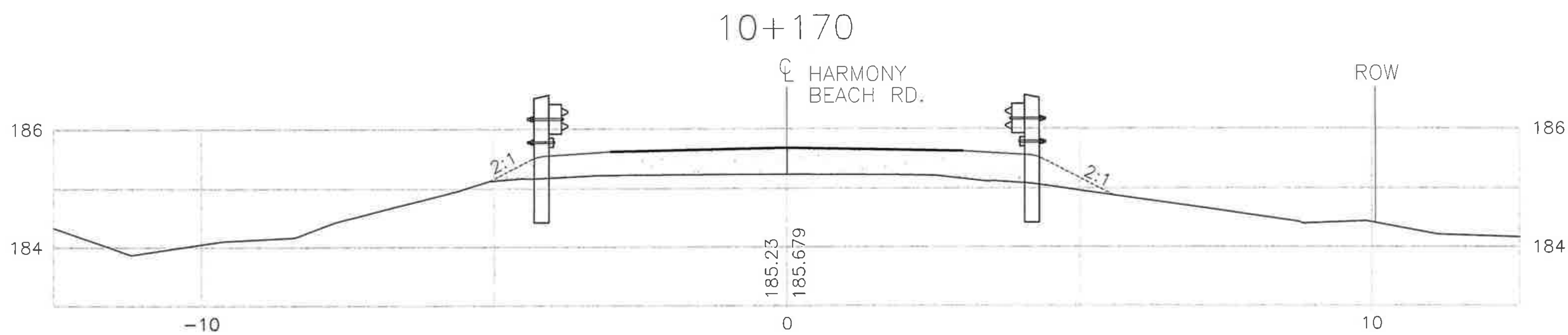
PROFILE
CONCRETE BRIDGE



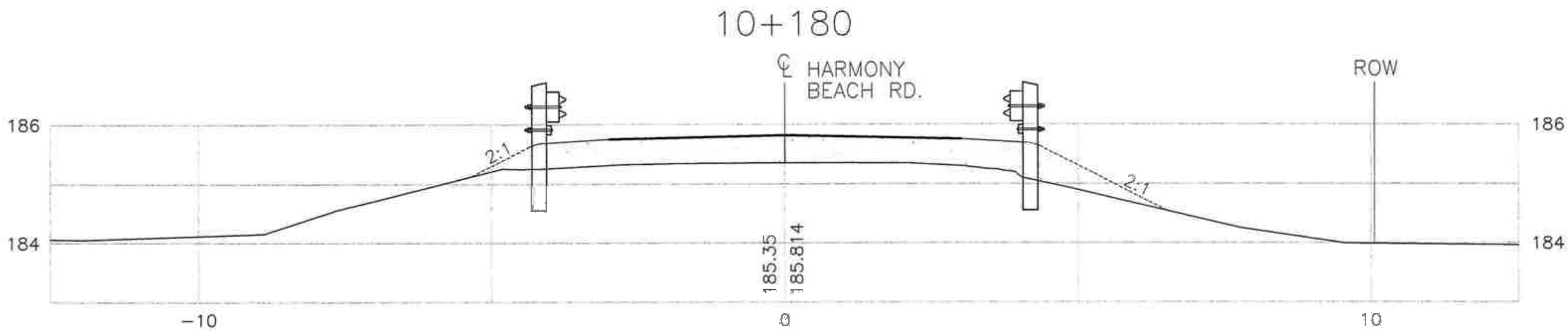
PROFILE
PREFABRICATED BRIDGE

Appendix G2

Section Drawings (Concrete Bridge)



SECTION
CONCRETE BRIDGE



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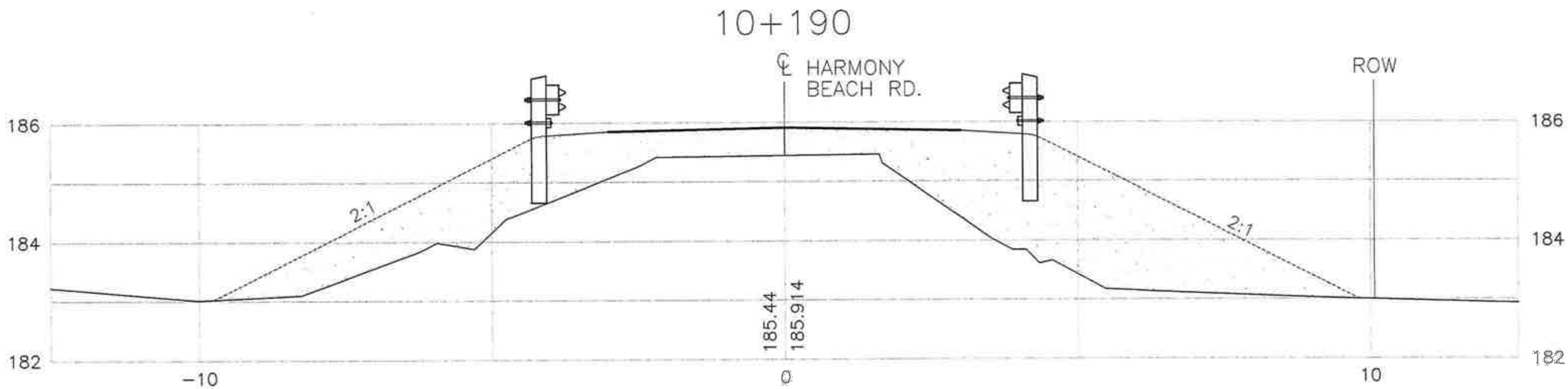

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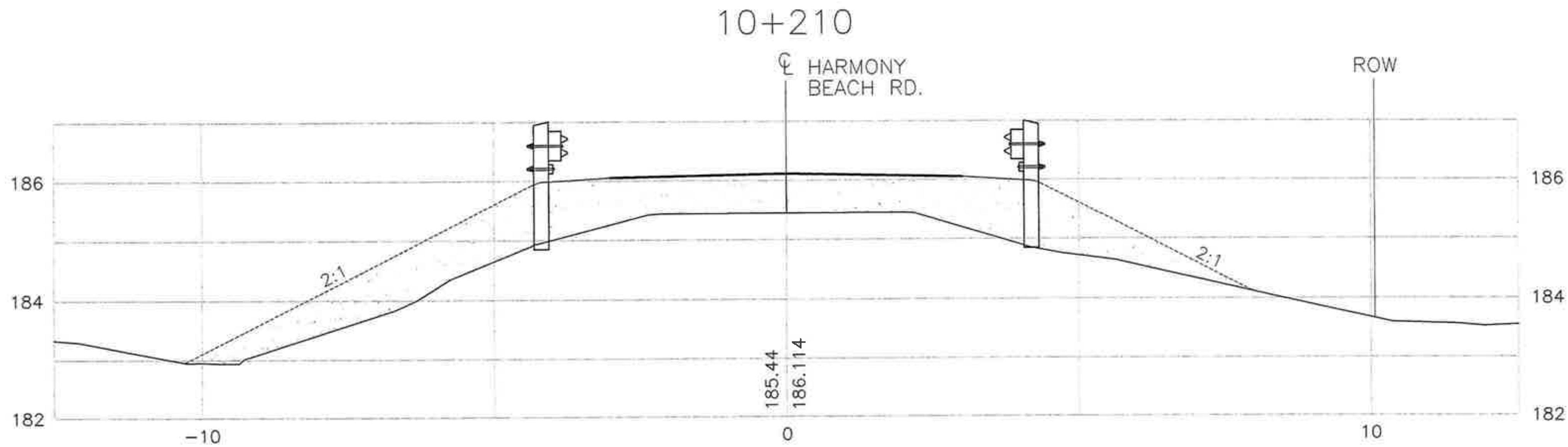

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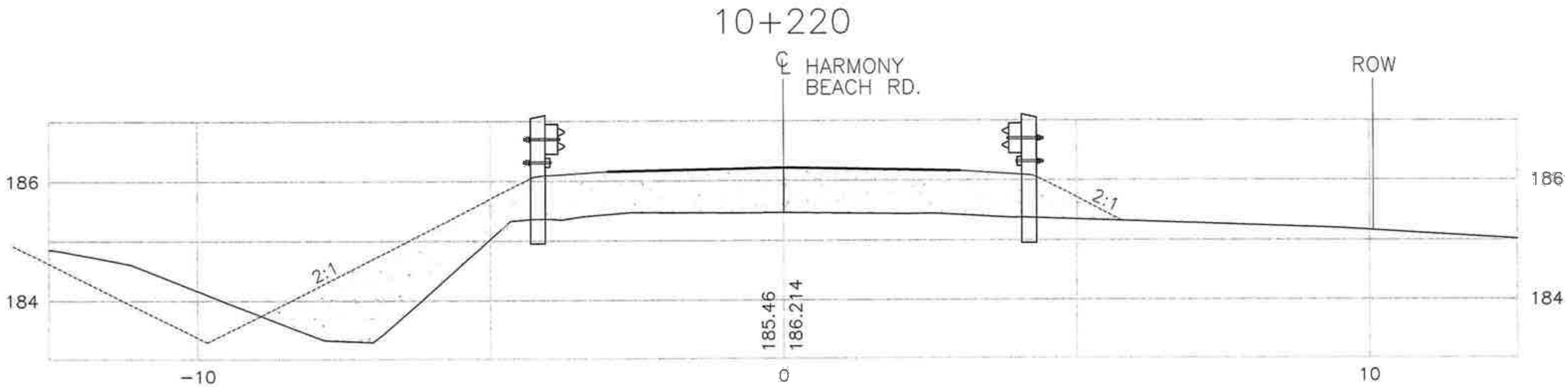
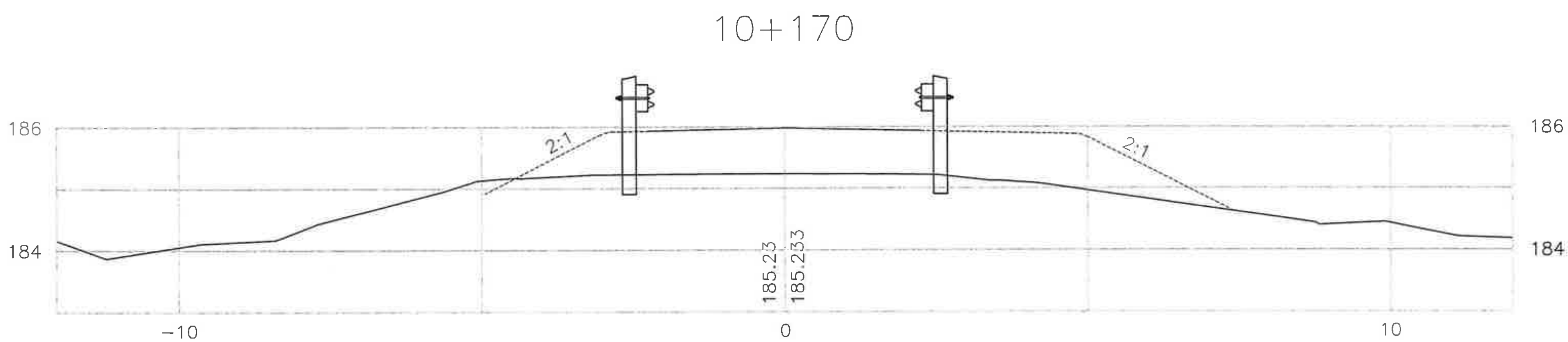


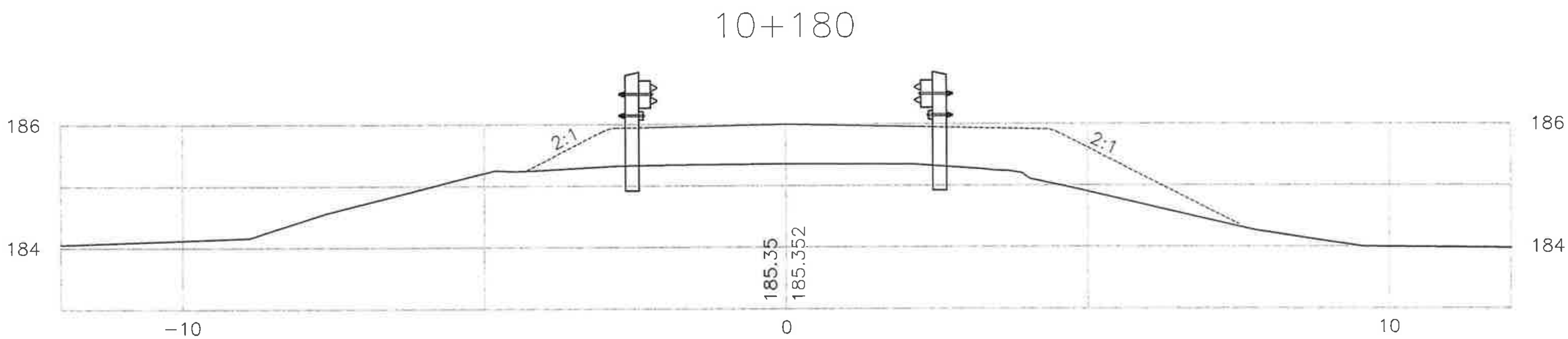
SECTION
CONCRETE BRIDGE

Appendix G3

Section Drawings (Prefabricated Bridge)



SECTION
PREFABRICATED BRIDGE



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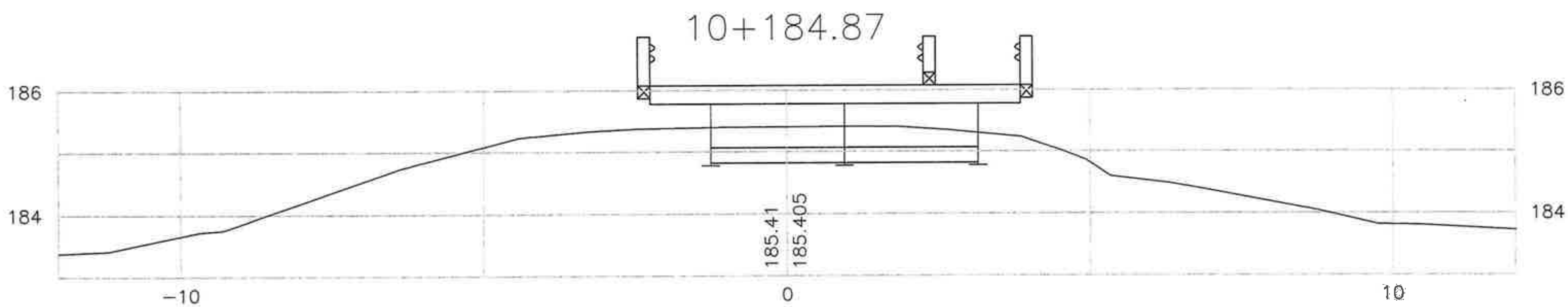
PREFABRICATED BRIDGE



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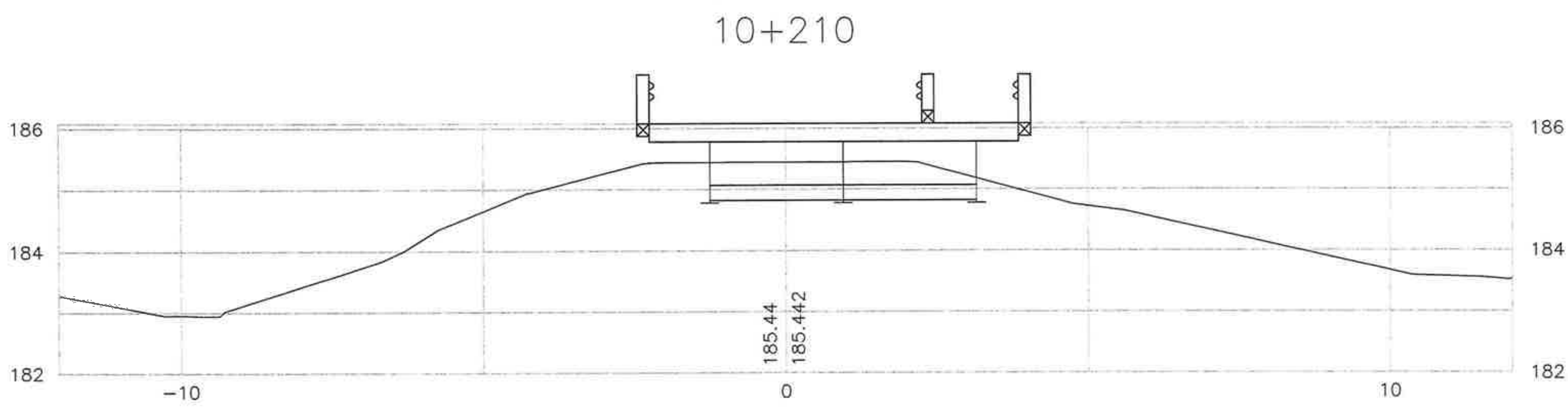
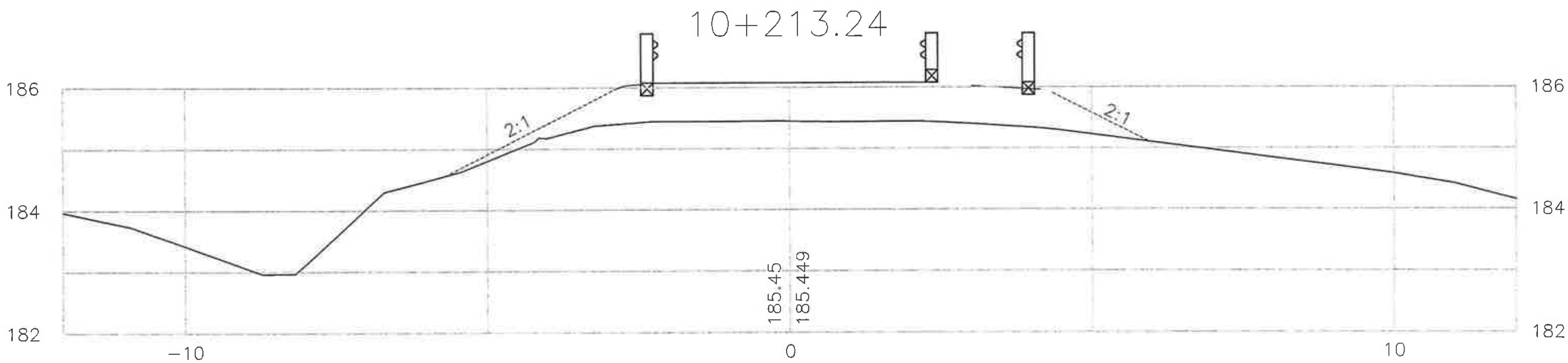
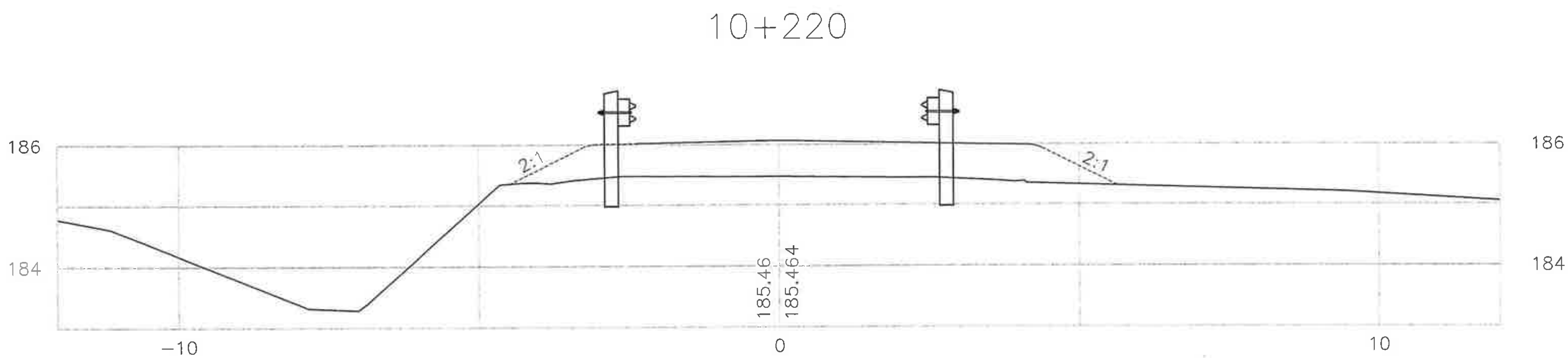
PREFABRICATED BRIDGE



SECTION
PREFABRICATED BRIDGE



SECTION
PREFABRICATED BRIDGE



SECTION
PREFABRICATED BRIDGE

Appendix H

Typical Embankment Stability Analyses

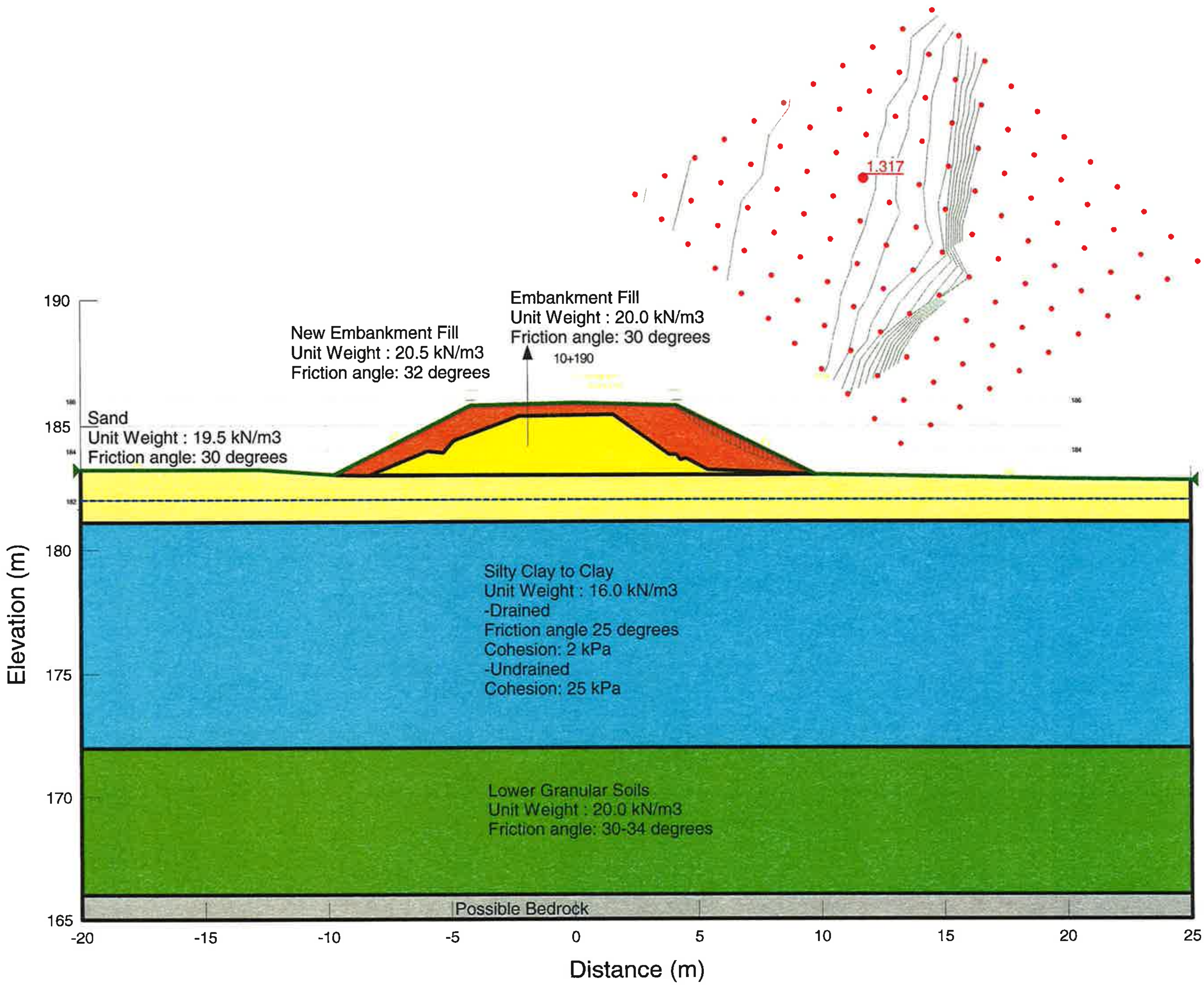


Fig H-1 Stability analysis @ STA. 10+190 (north abutment location) for concrete bridge option

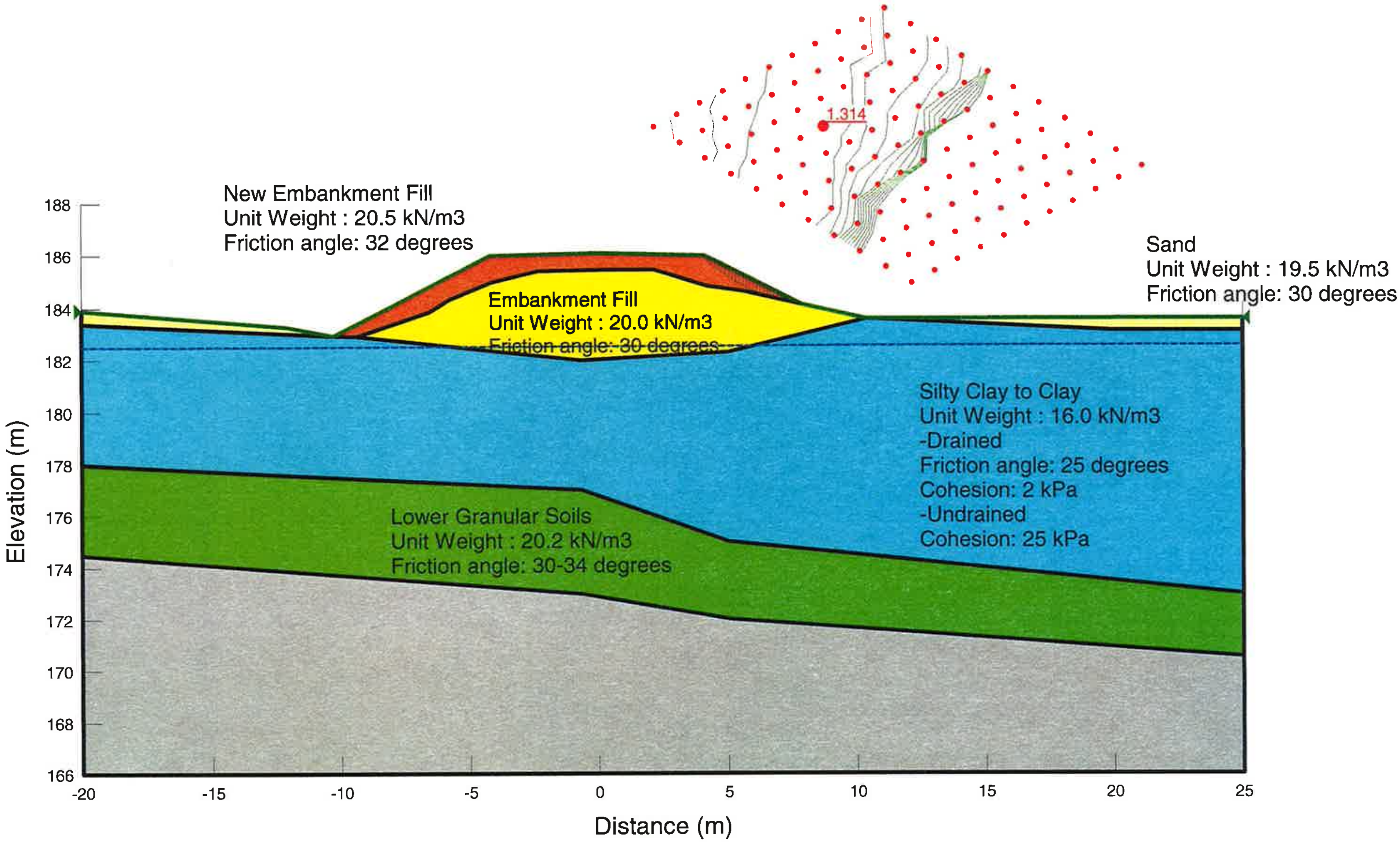


Fig. H-2 Stability analysis @ STA 10+210 (south abutment location) for concrete bridge option

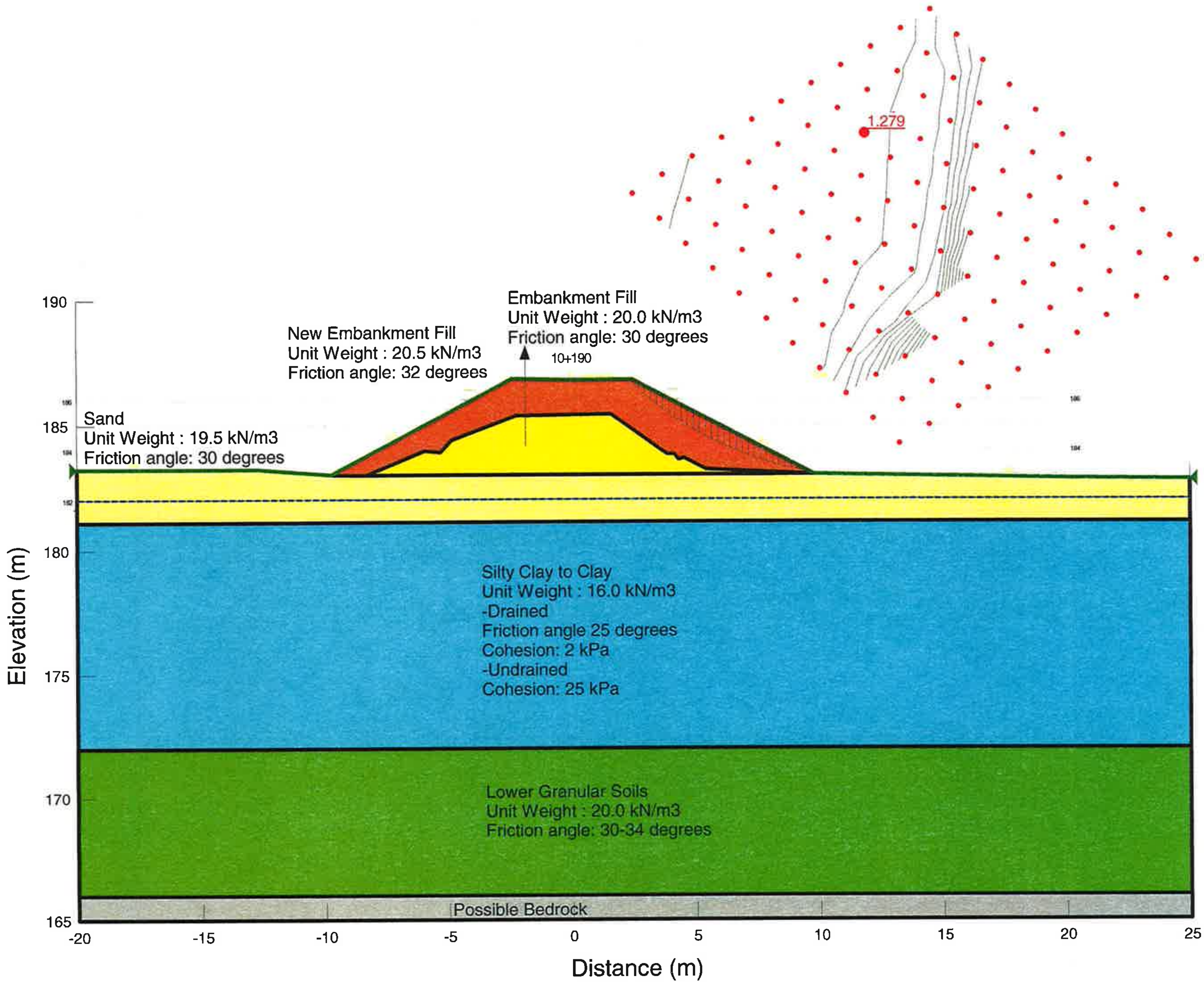


Fig H-3 Stability analysis @ STA. 10+190 with 1.0 m surcharge (north abutment location) for concrete bridge option

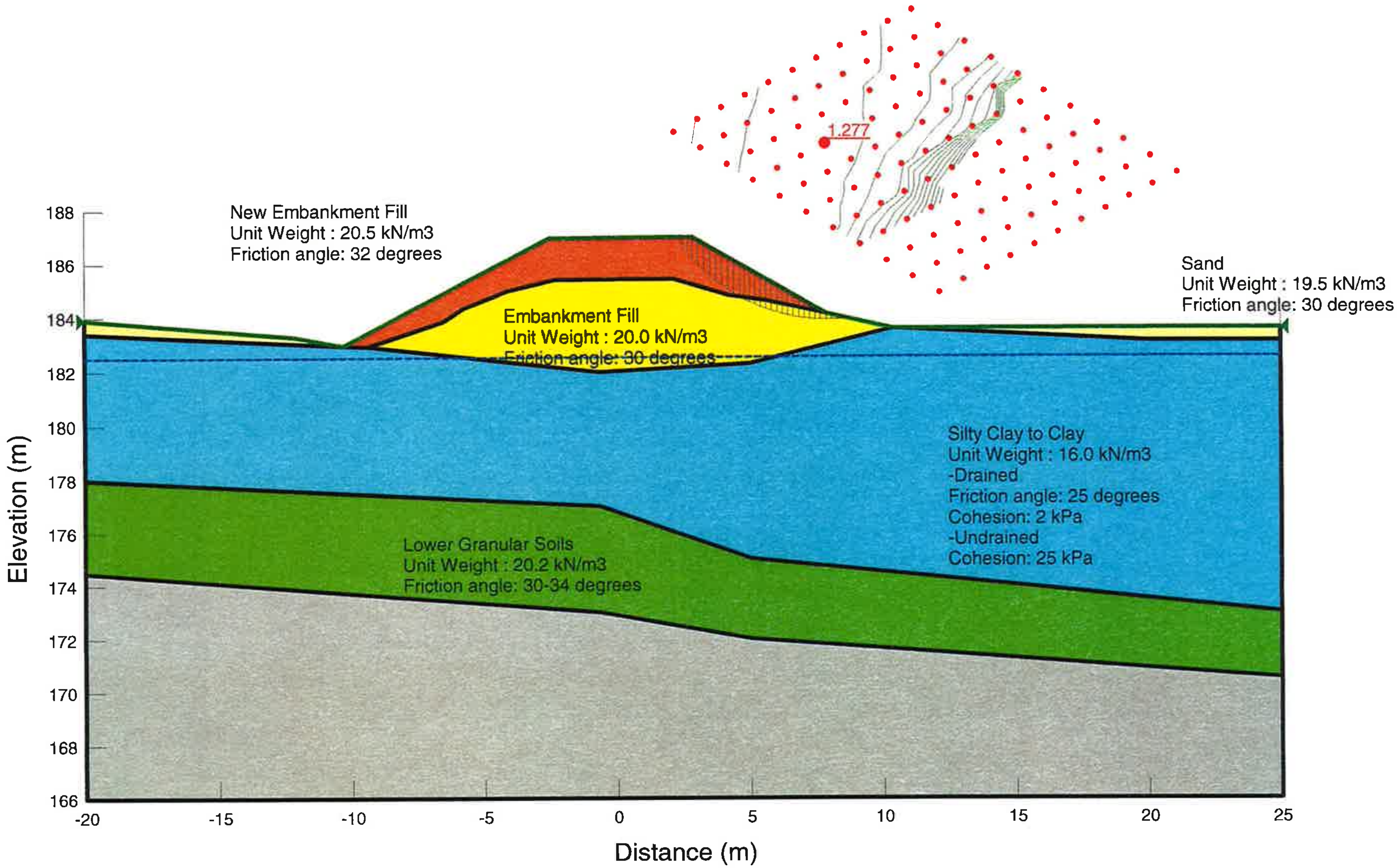


Fig. H-4 Stability analysis @ STA 10+210 with 1.0 m surcharge (south abutment location) for concrete bridge option

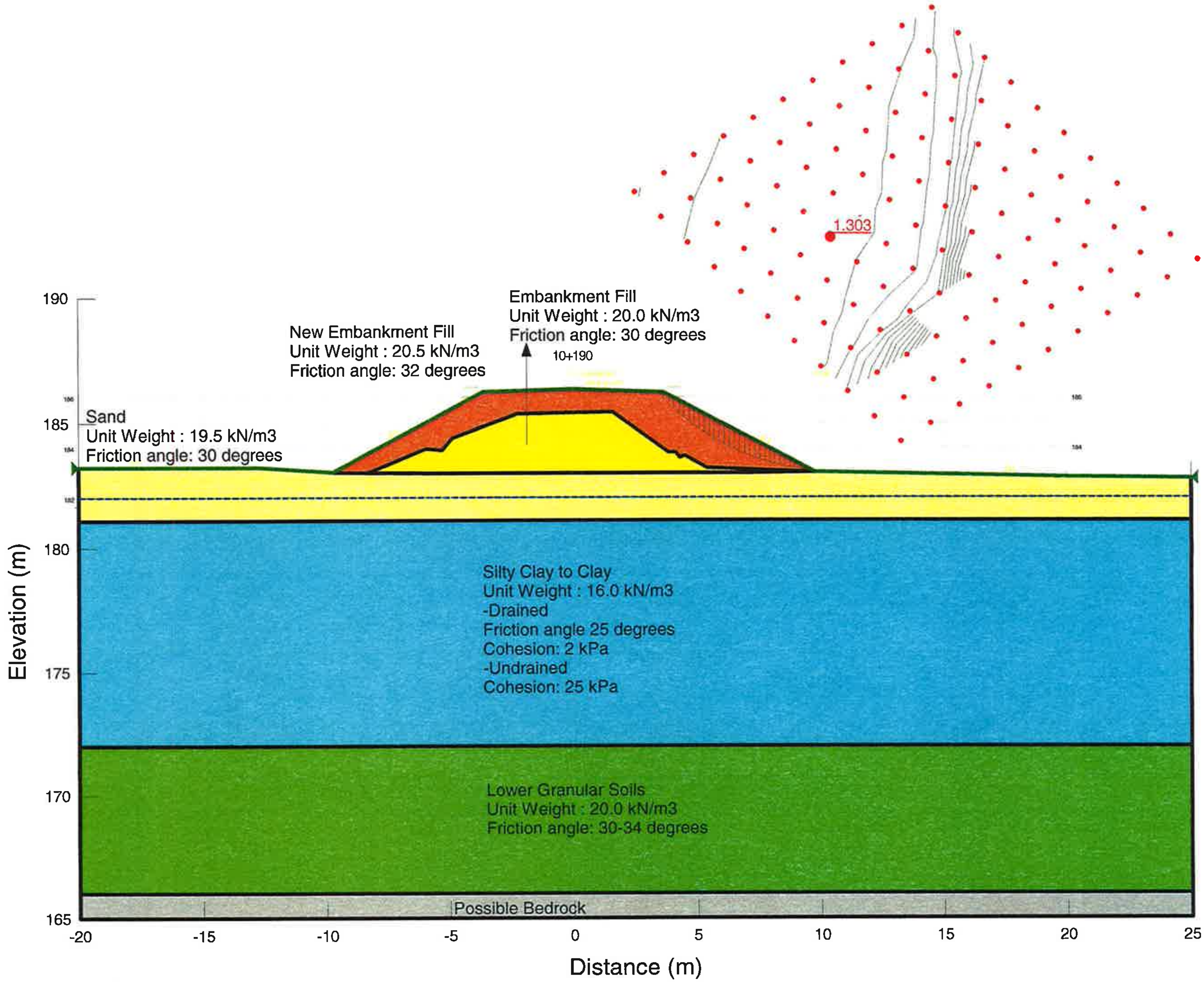


Fig H-5 Stability analysis @ STA. 10+190 with 0.8 m surcharge (north abutment location) for concrete bridge option

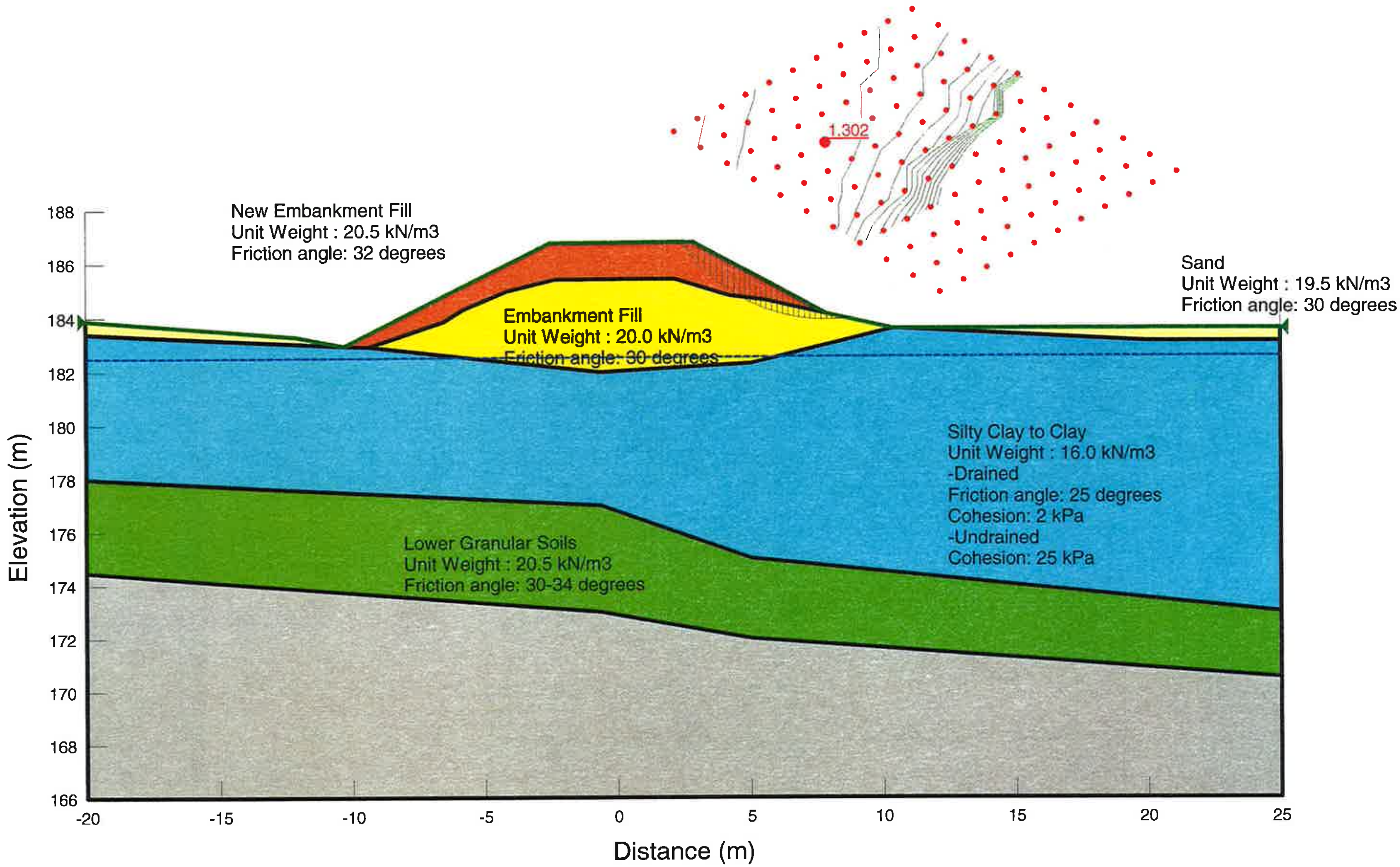


Fig. H-6 Stability analysis @ STA 10+210 with 0.8 m surcharge (south abutment location) for concrete bridge option

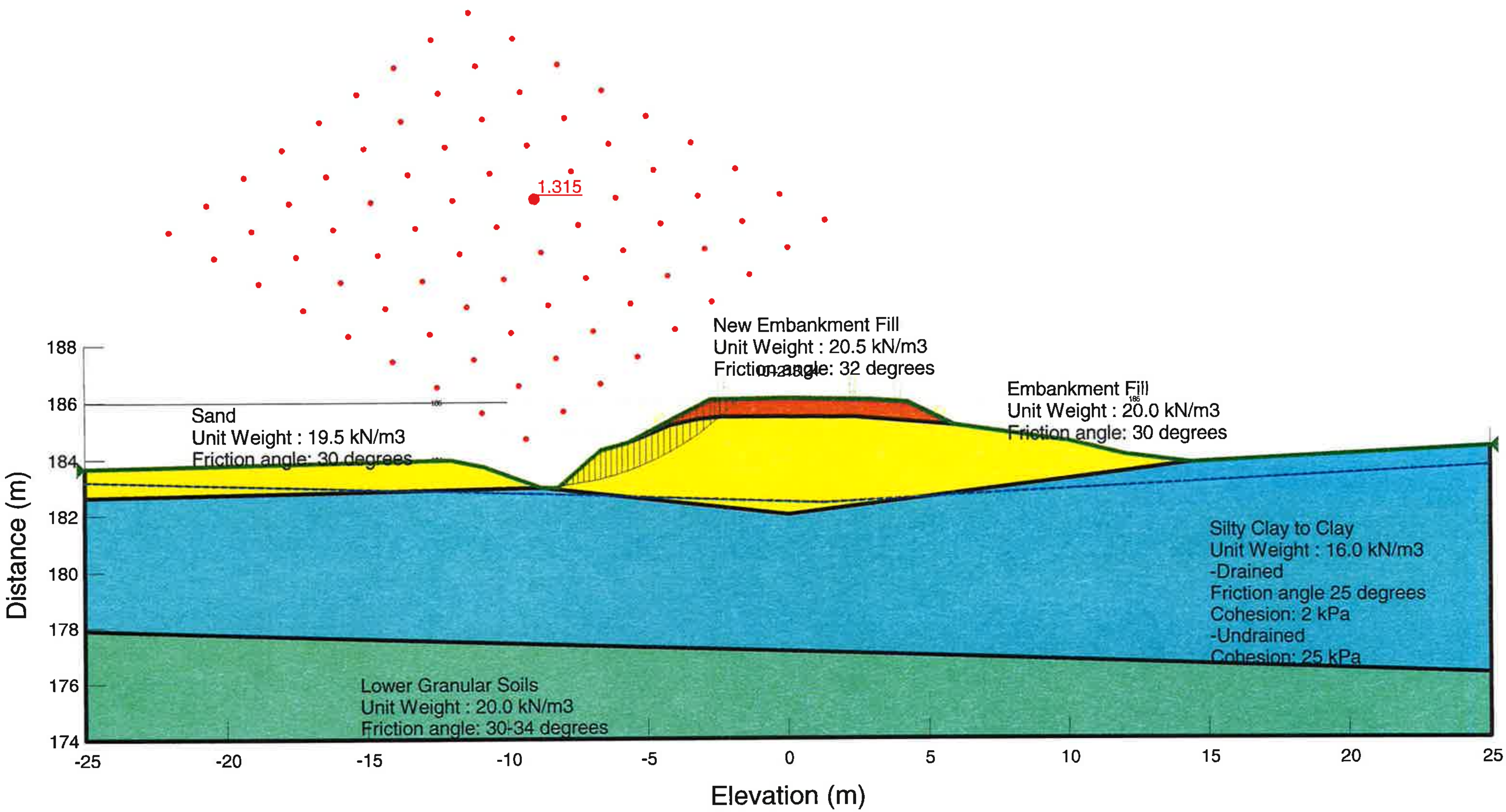


Fig H-7 Stability analysis @ STA. 10+213 (south abutment location) for prefabricated bridge option

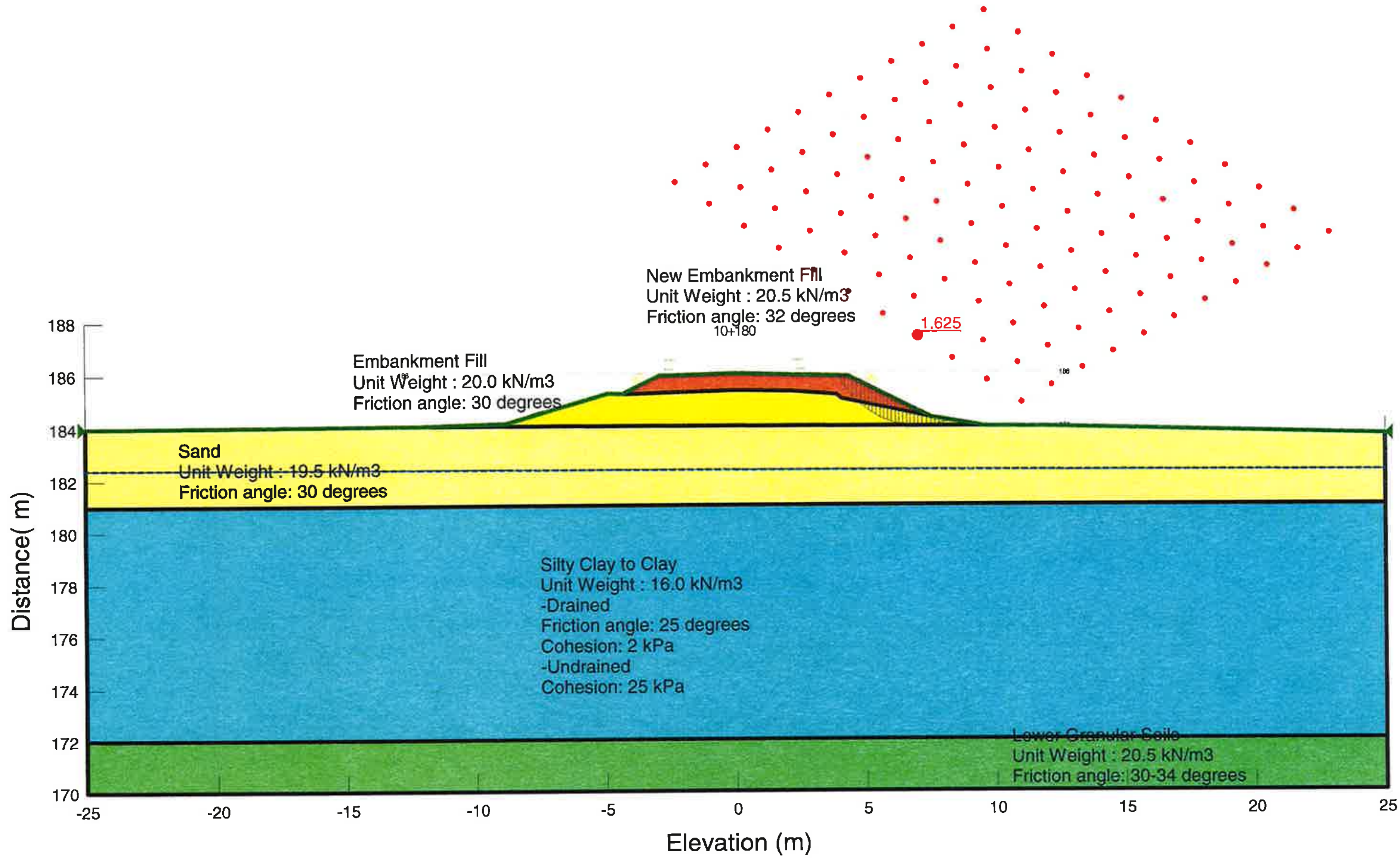


Fig. H-8 Stability analysis @ STA 10+180 (north abutment location) for prefabricated bridge option

Appendix I

Typical Settlement Analyses

Station 10+190 (North Abutment Location for Concrete Bridge Option)

Figure I-1 Embankment Centreline Settlement VS Time (0.6 m grade raise, 0.5 m surcharge and 1.0 m surcharge)

Figure I-2 Embankment Settlement VS Time (at Centreline and Widening Portion)

Figure I-3 Embankment Centreline Settlement VS Time (0.6 m grade raise and 0.8 m surcharge)

Figure I-1 Embankment Centreline Settlement vs Time, Concrete Bridge (STA 10+190)

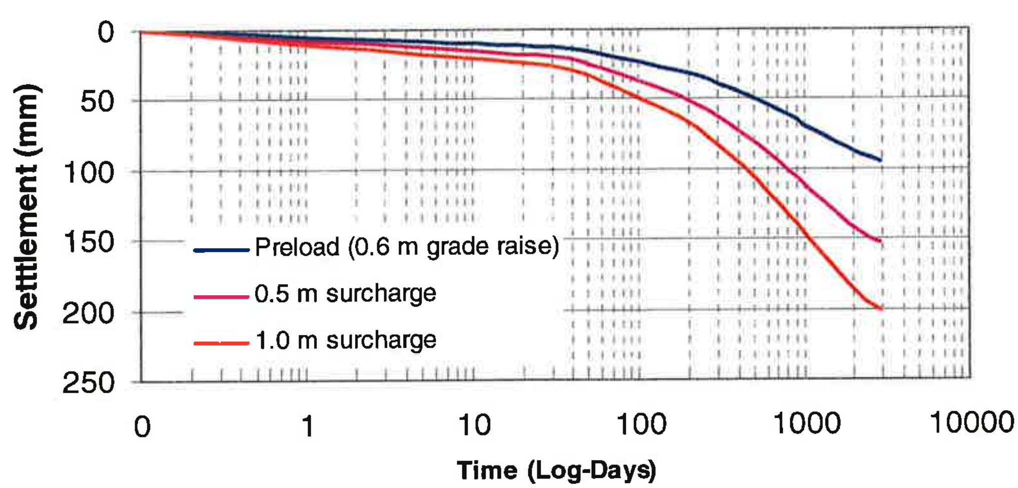
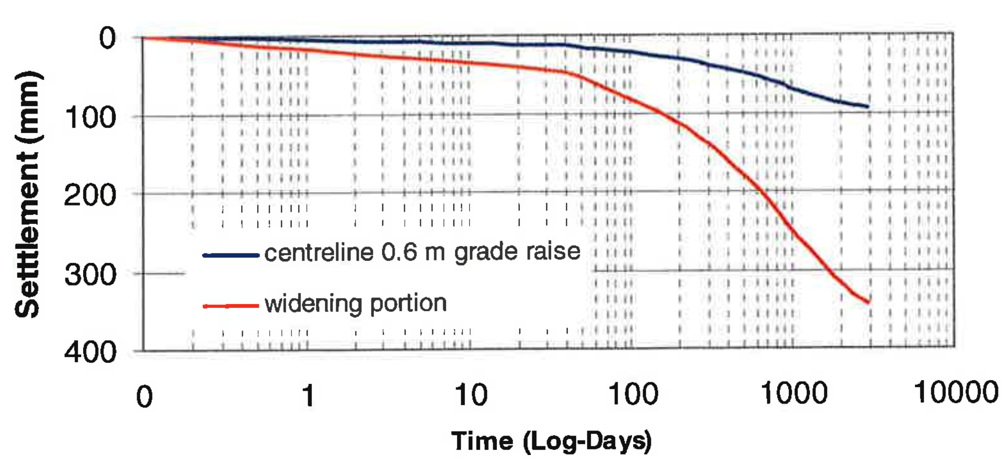


Figure I-2 Embankment Settlement vs Time, Concrete Bridge (STA 10+190)



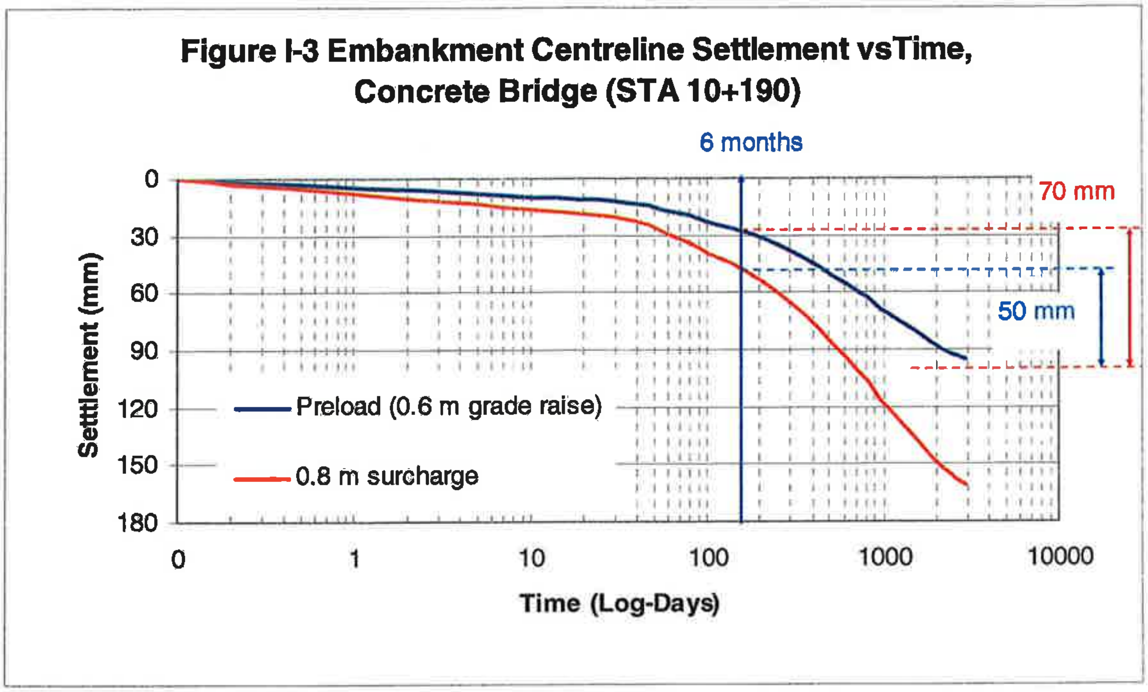


Figure I-3 Embankment Centreline Settlement vsTime, Concrete Bridge (STA 10+190)
6 months
70 mm
50 mm
Settlement (mm)
0
30
60
90
120
150
180
Preload (0.6 m grade raise)
0.8 m surcharge
0
1
10
100
1000
10000
Time (Log-Days)

Station 10+210 (South Abutment Location for Concrete Bridge Option)

Figure I-4 Embankment Centreline Settlement VS Time (0.85 m grade raise, 0.5 m surcharge and 1.0 m surcharge)

Figure I-5 Embankment Settlement VS Time (at Centreline and Widening Portion)

Figure I-6 Embankment Centreline Settlement VS Time (0.85 m grade raise and 0.8 m surcharge)

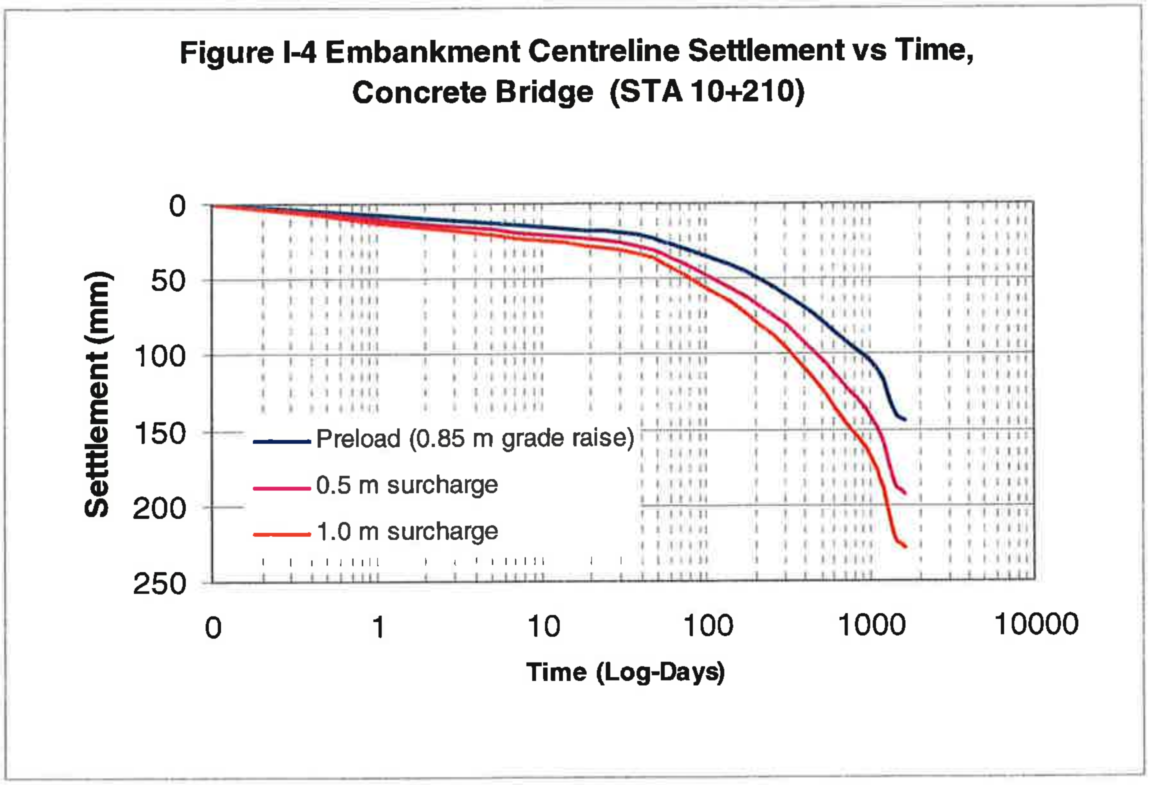


Figure I-5 Embankment Settlement vs Time, Concrete Bridge (STA 10+210)

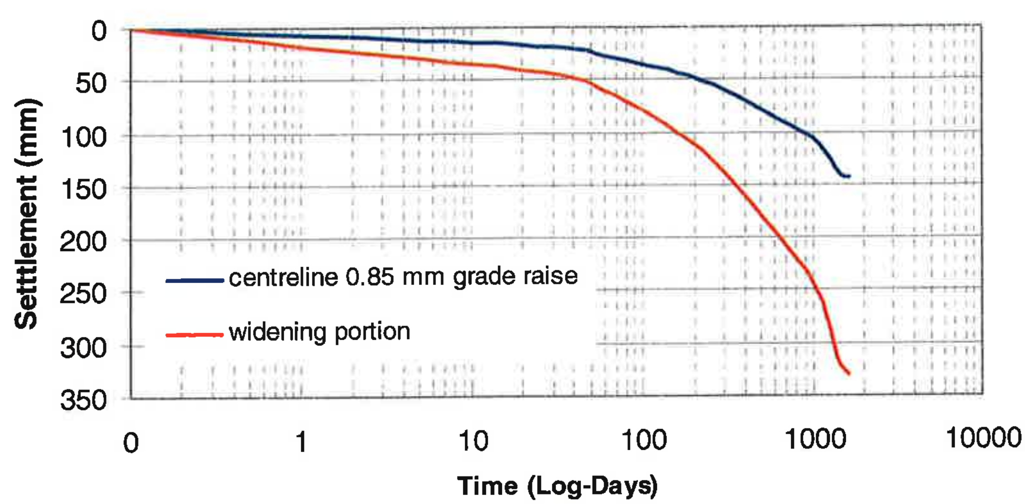
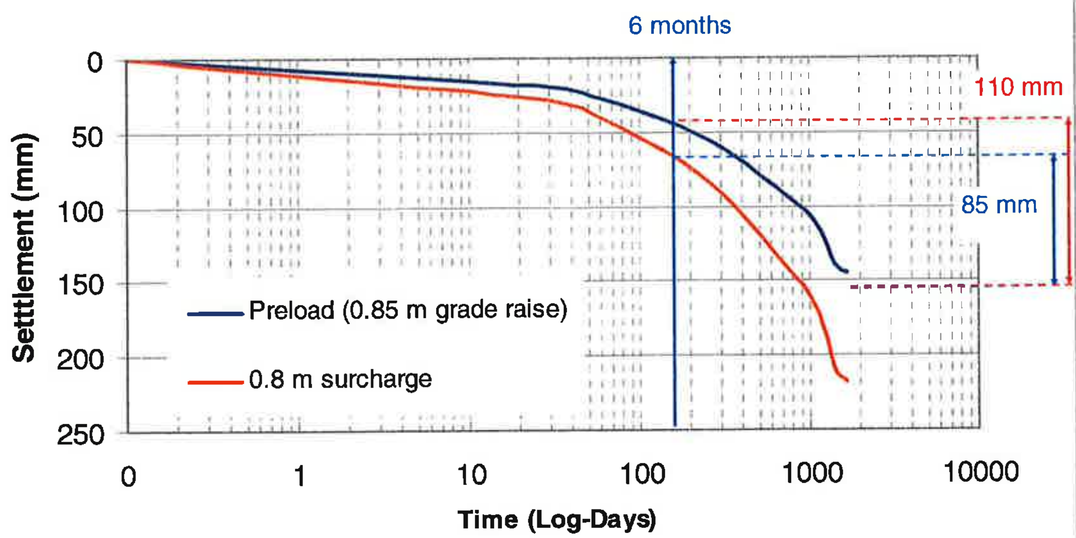


Figure I-6 Embankment Centreline Settlement vs Time, Concrete Bridge (STA 10+210)

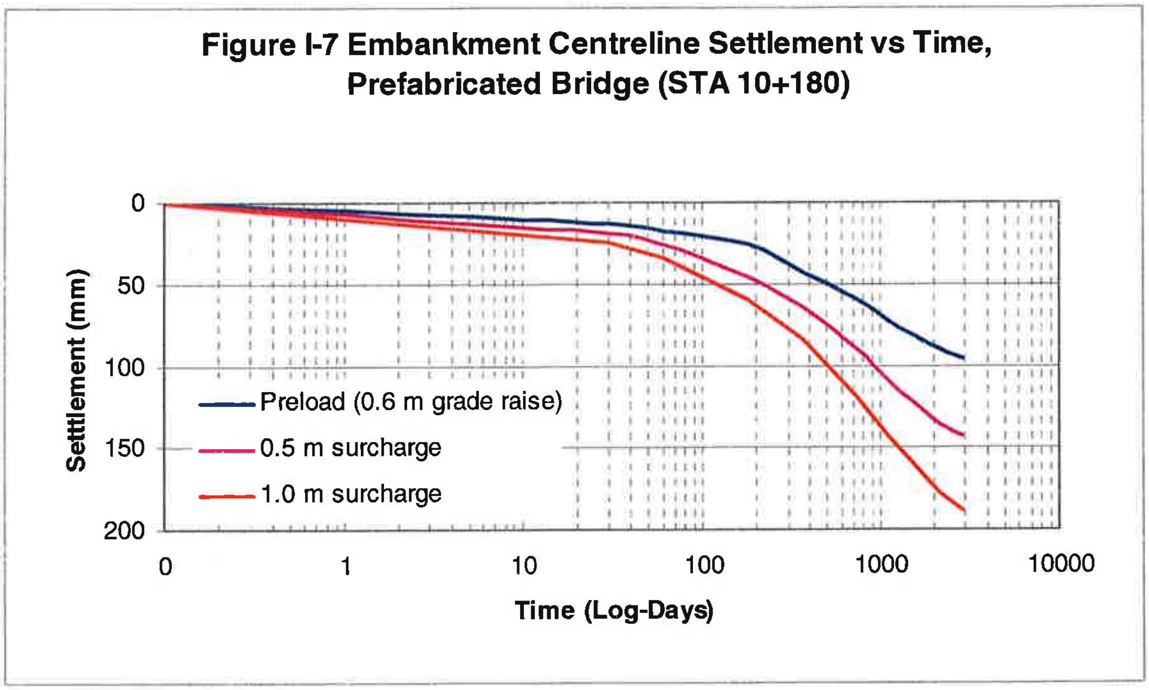


Station 10+180 (North Abutment Location for Prefabricated Bridge Option)

Figure I-7 Embankment Centreline Settlement VS Time (0.6 m grade raise, 0.5 m surcharge and 1.0 m surcharge)

Figure I-8 Embankment Settlement VS Time (at Centreline and Widening Portion)

Figure I-9 Embankment Centreline Settlement VS Time (0.6 m grade raise and 0.5 m surcharge)



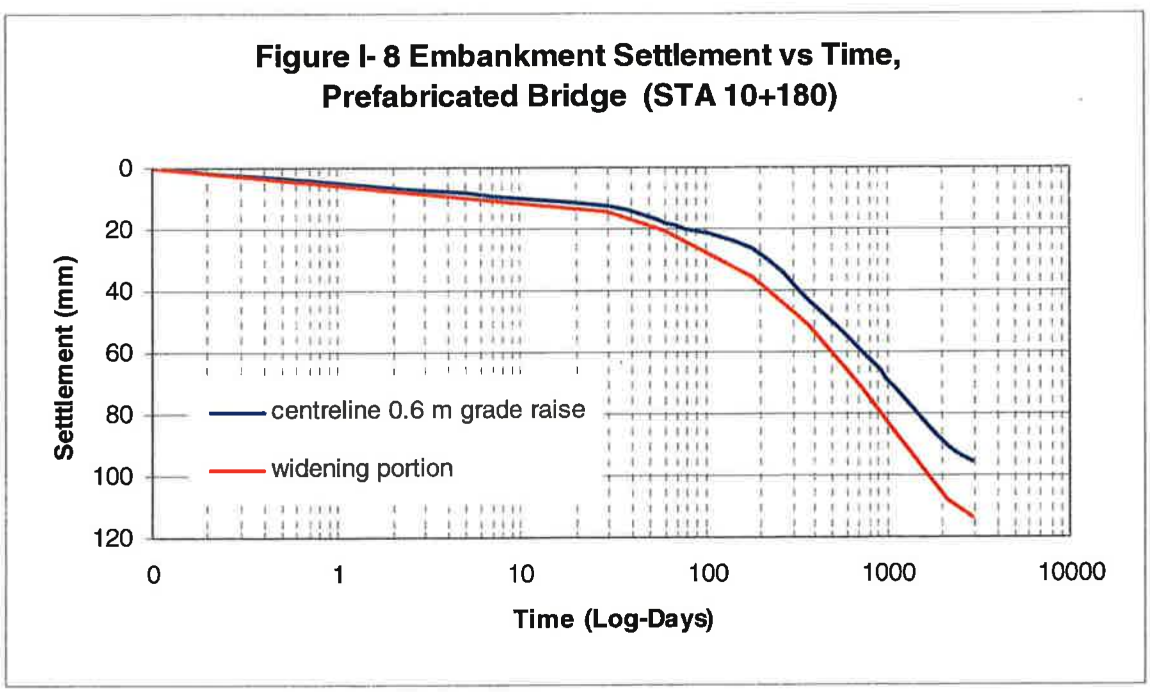
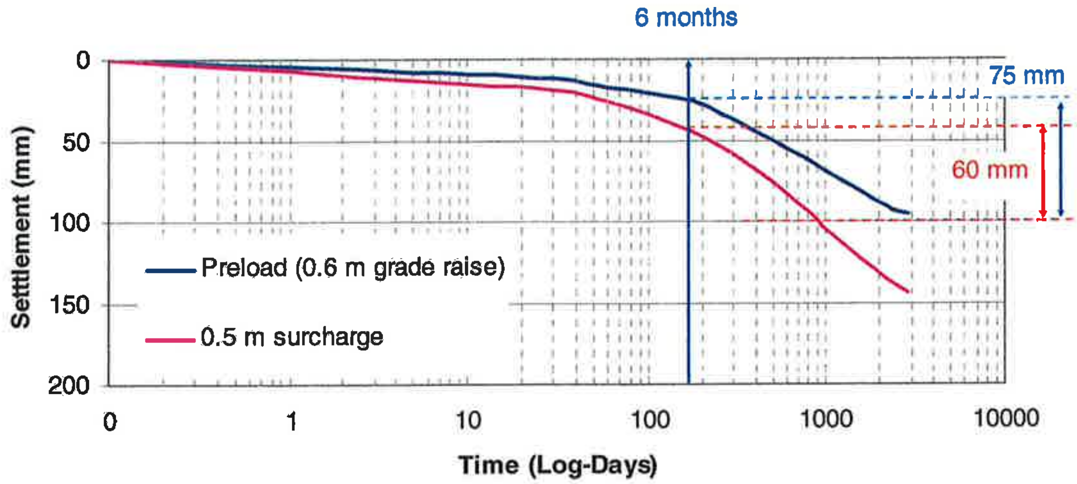


Figure I-9 Embankment Centreline Settlement vs Time, Prefabricated Bridge (STA 10+180)



Station 10+213 (South Abutment Location for Prefabricated Bridge Option)

Figure I-10 Embankment Centreline Settlement VS Time (0.85 m grade raise, 0.5 m surcharge and 1.0 m surcharge)

Figure I-11 Embankment Centreline Settlement VS Time (0.85 m grade raise and 0.5 m surcharge)

Figure I- 10 Embankment Centreline Settlement vs Time, Prefabricated Bridge (STA 10+213)

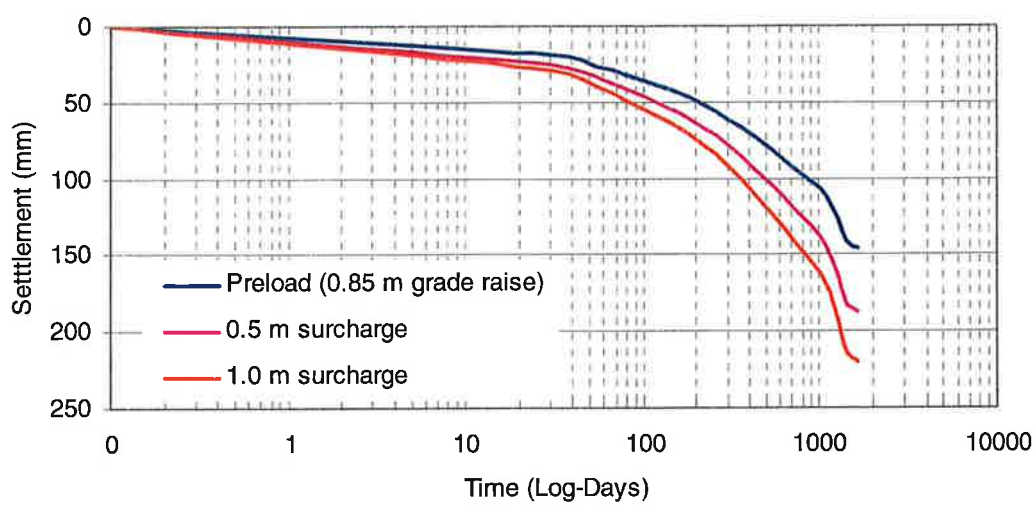
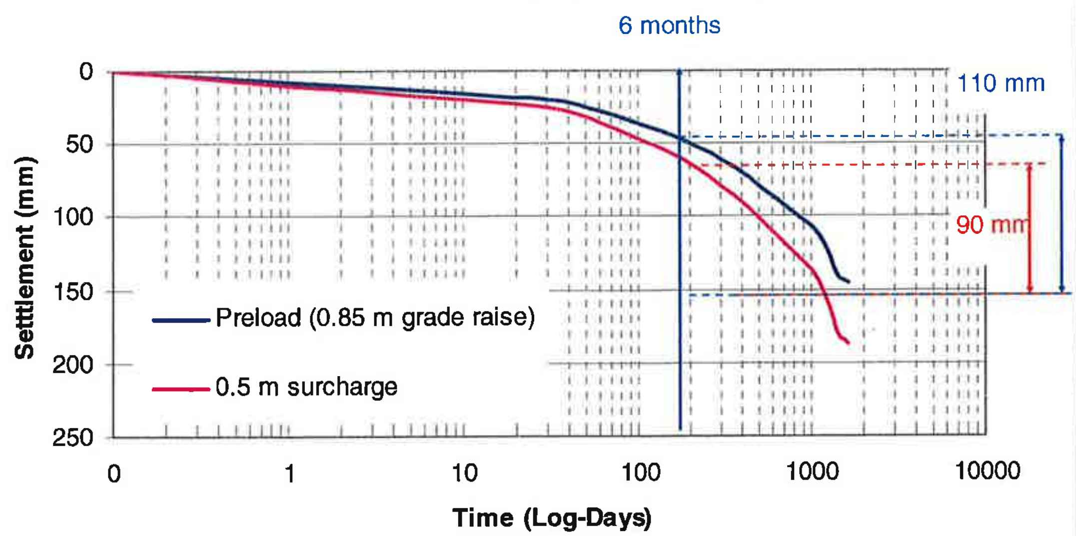


Figure I- 11 Embankment Centreline Settlement vs Time, Prefabricated Bridge (STA 10+213)



Appendix J

MTO Procedures for EPS Design

Special Provision

REQUIREMENTS FOR EXPANDED POLYSTYRENE EMBANKMENT FILL

1.0 SCOPE

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill and associated works as shown on the contract drawings.

2.0 REFERENCES

This special provision refers to the following standards, specifications or publications.

2.1 National Standards of Canada

CAN/CGSB - 51.20 M87

2.2 ASTM

ASTM D1621	Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM C203	Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation
ASTM C177	Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus
ASTM D2842	Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Test Method for Measuring the Minimum Oxygen Content
ASTM D2126	Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

2.3 OPSS - Ontario Provincial Standard Specification

OPSS 212	Borrow
OPSS 501	Compaction
OPSS 517	Dewatering
OPSS 1010	Aggregates – Granular A, B, M, and Selected Subgrade Material
OPSS 1605	Expanded Extruded Polystyrene Pavement Insulation
OPSS 1860	Geotextiles

3.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene: Moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

Rigid Extruded Expanded Polystyrene: Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot: The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: Quality Verification Engineer means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s)of conformance.

5.0 QUALIFICATION

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6.0 SUBMISSION AND DESIGN REQUIREMENTS

6.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 Delivery, Storage, Handling, and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturers requirement.

6.3 Construction

The contractor shall submit full details of the following.

a) The method of foundation excavation and preparation.

b) Construction of levelling pad.

c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.

d) The method and limits of placement of polyethylene sheeting.

e) The method of placement of 125 mm reinforced concrete base pad (or equivalent).

f) The method of placement of subbase material.

g) The method of placement of side slope cover.

6.4 Quality Verification Engineer

(1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

(2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.

7.0 MATERIALS

7.1 Granular Levelling Pad

The levelling pad shall consist of a Granular "A" material with gradation and physical requirements as specified in OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.

2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.

3. Certification of compliance of physical and mechanical properties.

4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.

5. The physical and mechanical properties of the rigid expanded polystyrene including:

    1. Geometry
    2. Nominal Density
    3. Compressive Strength
    4. Flexural Strength
    5. Thermal Resistance
    6. Dimensional Stability
    7. Flammability
    8. Water Absorption

6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.

7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.

8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Production Lots

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	 1200 x 600 x 300 with tolerances ± 1% 10 mm in 3 m ± 0.5% -3, +5	
Compressive Strength	kPa (min)	110	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	$m^2.°C/W$ (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be ± 1%. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner-to-corner dimensions (squareness) shall be 0.5%. The thickness shall be within –3 to +5 mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 110 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be 0.7 $m^2.^{\circ}C/W$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25mm} = \frac{R_{measured}}{\text{thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less then 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 DELIVERY, STORAGE AND HANDLING

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 CONSTRUCTION

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

(1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.

(2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.

(3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.

(4) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.

(5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.

(6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.

(7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.

(8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

(9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

(10) The top surface and side surfaces of the expanded polystyrene shall be covered with 0.6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the

embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

(11) The contractor shall install the concrete base pad as detailed elsewhere in the contract.

(12) The side slope of the rigid expanded polystyrene embankment shall be covered with Lightweight fill and waste material as detailed elsewhere in this contract.

10.0 EQUIPMENT

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11.0 QUALITY ASSURANCE

General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. A recognized testing laboratory accredited by the Standards Council of Canada shall conduct the testing.

Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12.0 MEASUREMENT FOR PAYMENT

Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13.0 PAYMENT

Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

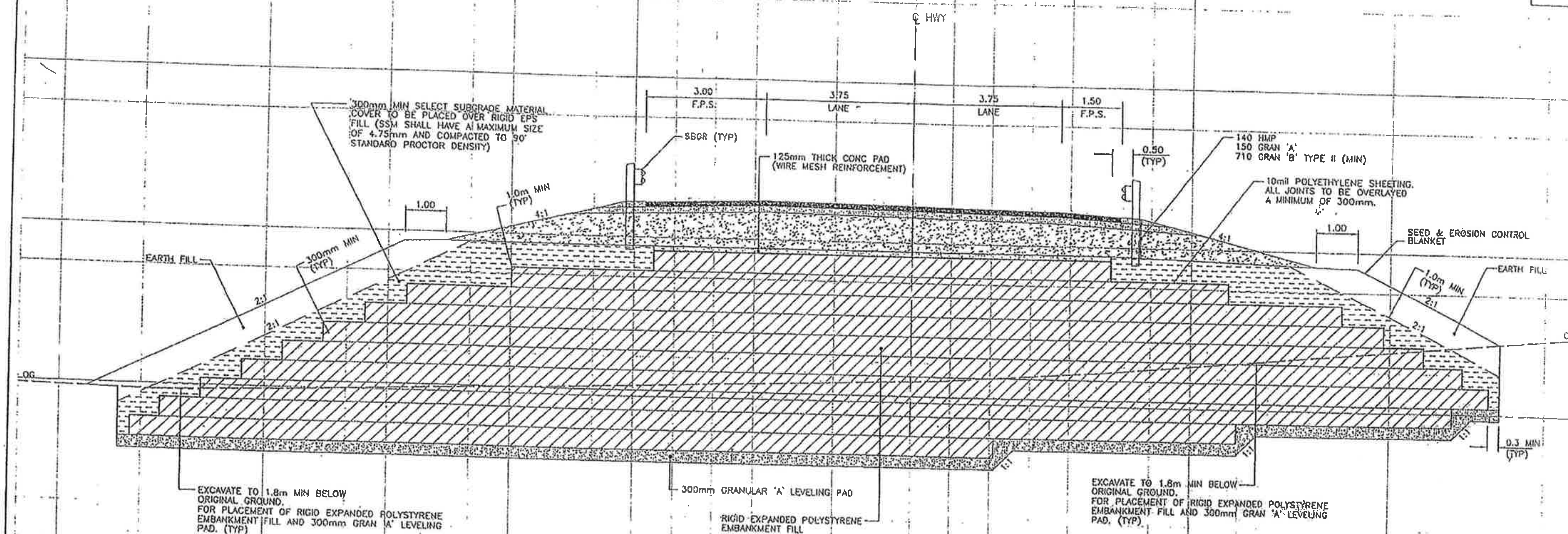
WARRANT: Always with this tender item.

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT WP	
TYPICAL SECTIONS	SHEET 1

TSH engineers architects planners



RIGID EXPANDED POLYSTYRENE EMBANKMENT TYPICAL SECTION

N.T.S.

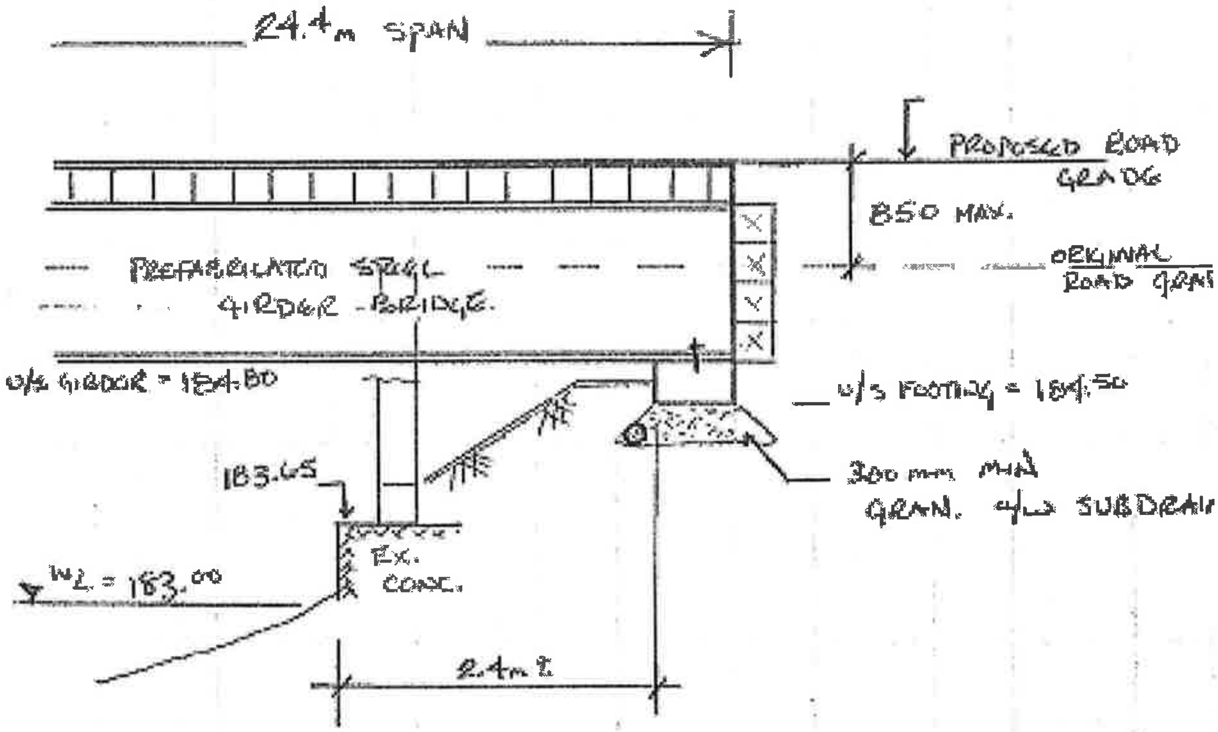
Appendix K

Bridge Design Drawings

WILLS
Consulting Engineers

D.M. Wills Associates Limited
452 Charlotte Street,
Peterborough, ON. K9J 2W3
P: 705.742.2297 • F: 705.741.3568

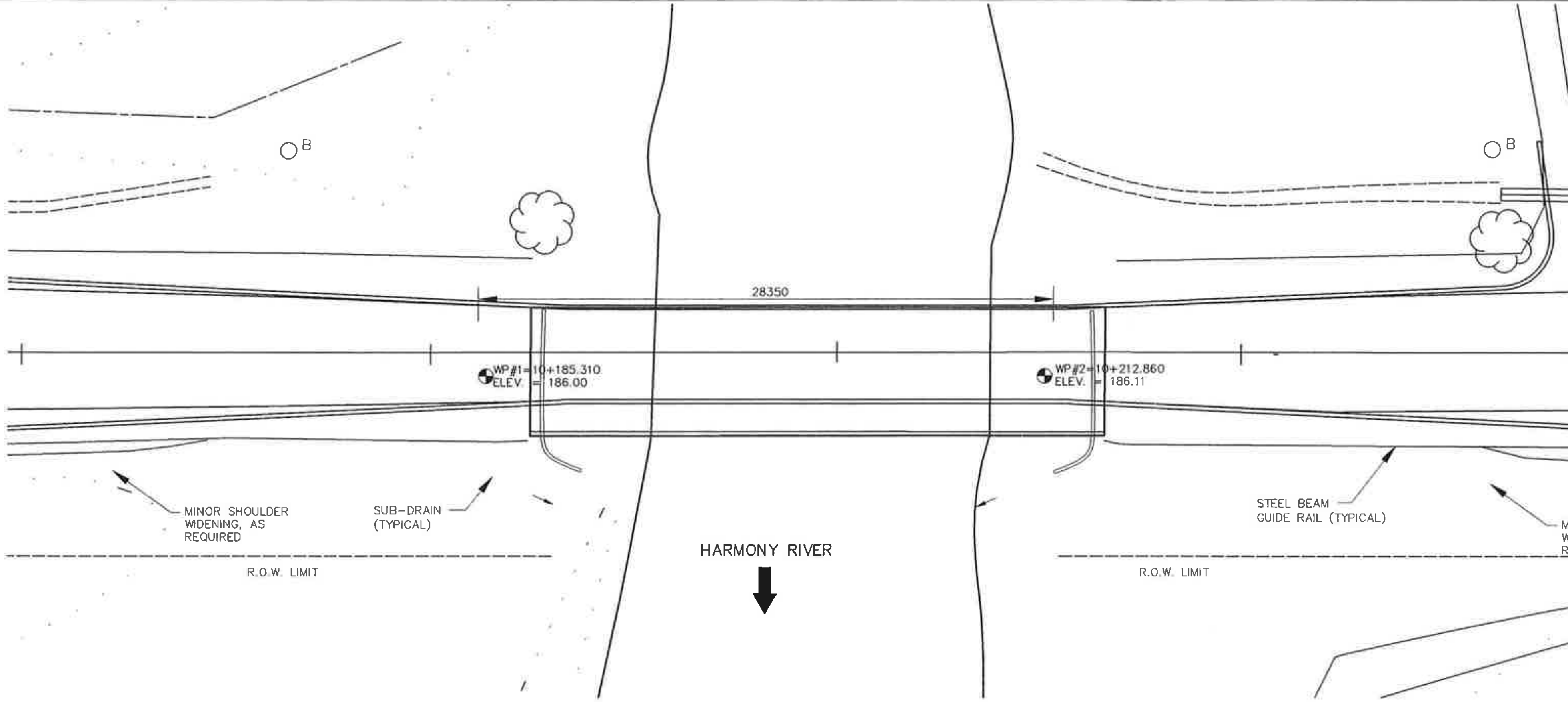
Project No. 09-4283 Sheet 1 of 1
Project Title HARMONY BEACH
Author D. BONSALL
Subject
Date 16 DEC 2009



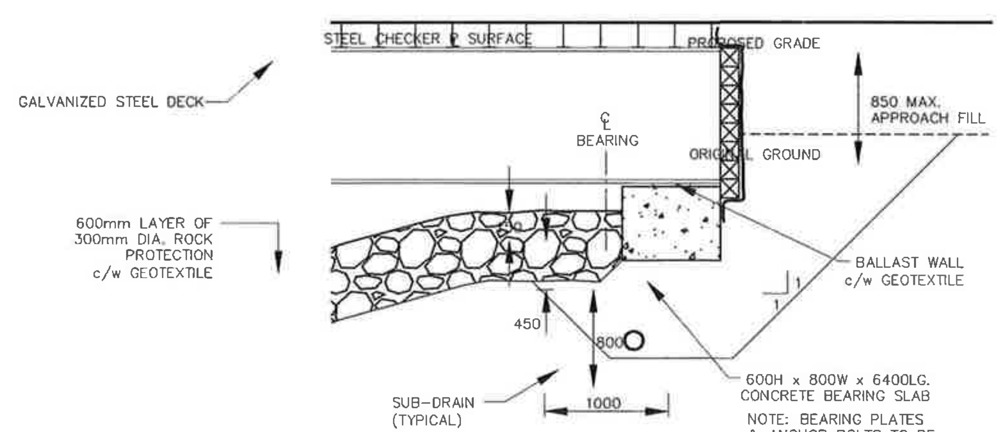
NOTE:
EX. TIMBER PILES & FOOTING TO REMAIN FOR SCOUR PROTECTION

SCALE 1:50 ±

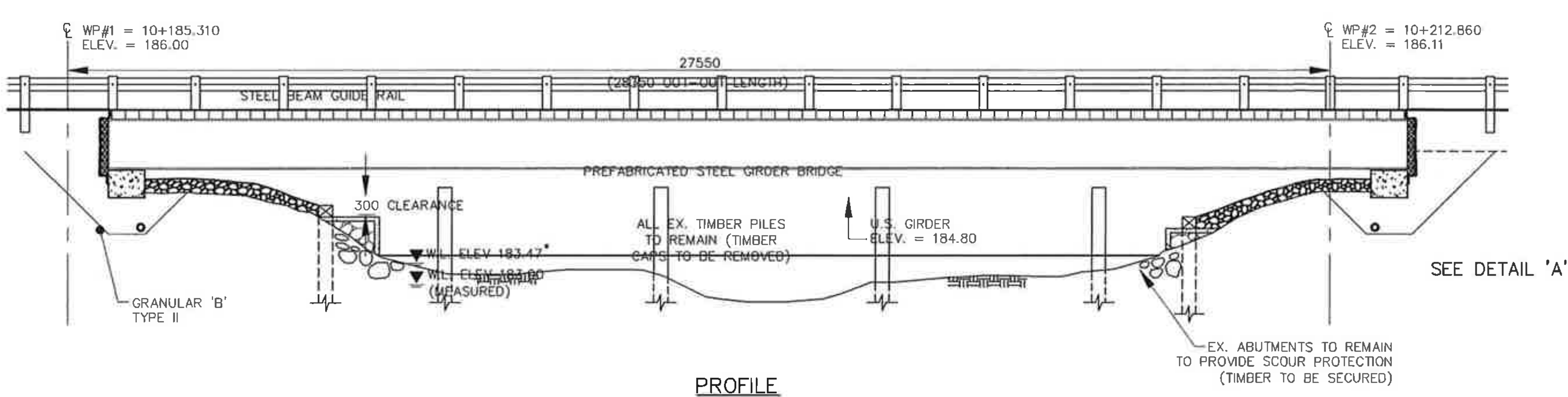
Figure K-1 Prefabricated Bridge Design



PLAN
1:150

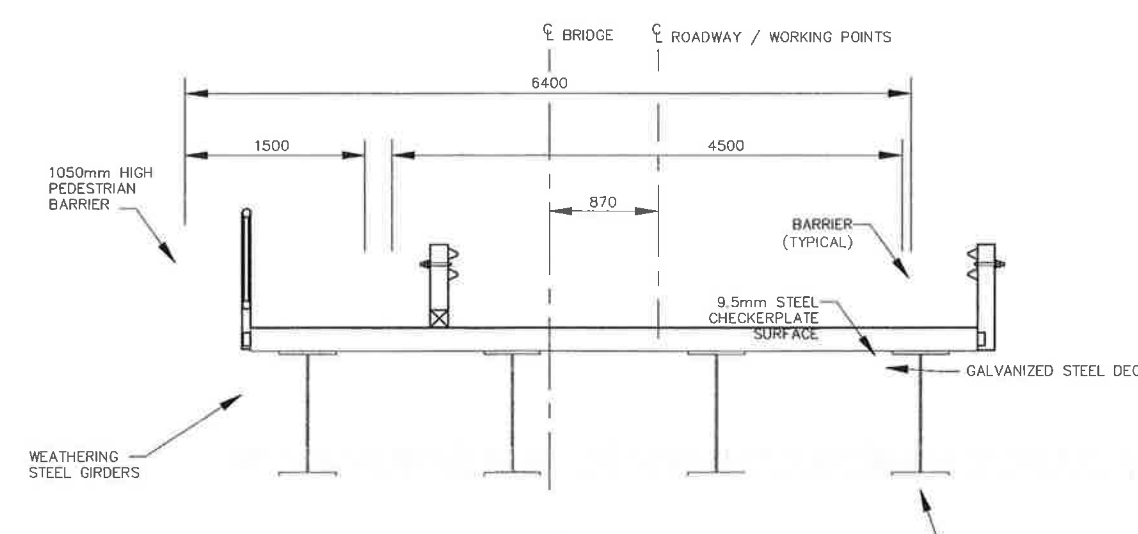


DETAIL 'A'
N.T.S.



PROFILE
1:75

* HISTORICAL AVERAGE LAKE SUPERIOR WATER LEVEL



SECTION 1
N.T.S.

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

LICENSED PROFESSIONAL ENGINEER — D. S. BONSALL — PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER — G. ZANZOUL — PROVINCE OF ONTARIO

DISTRICT NO
CONT No
WP No 5430-06-00

HARMONY RIVER BRIDGE
HIGHWAY 7045

GENERAL ARRANGEMENT

FIGURE
K-2

WILLS

GENERAL NOTES:

SPECIFICATIONS AND STANDARDS

- CONTRACT SPECIFICATIONS
- ONTARIO PROVINCIAL STANDARD SPECIFICATIONS (OPSS)
- CANADIAN HIGHWAY BRIDGE DESIGN CODE (CAN/CSA-S6-06) AS MODIFIED BY ONTARIO DIRECTIVE FOR LOW VOLUME ROADS

CONSTRUCTION NOTES

- BRIDGE DESIGN LIVE LOAD: CL-625-ONT
- THE CONTRACTOR SHALL DETERMINE THE EXACT LOCATION OF ALL UTILITIES THROUGHOUT THE SITE. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ADEQUATE PROTECTION OF ALL UTILITIES AND PREVENT DAMAGE DURING CONSTRUCTION.
- THE CONTRACTOR SHALL CARRY OUT THE WORK TO PREVENT DEBRIS FROM ENTERING THE WATERCOURSE.
- CHECK AND VERIFY ALL DIMENSIONS AND SITE CONDITIONS BEFORE PROCEEDING WITH WORK AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BREFORE COMMENCING.

REFERENCE DRAWINGS

OPSD 912.140 – GUIDE RAIL SYSTEM, STEEL BEAM, WOODEN POST ASSEMBLY, INSTALLATION – SINGLE RAIL

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN TS	CHK DB	CODE CHBDC-2006	LOAD CL-625-ONT	DATE 03/2010	
DRAWN TS	CHK DB	SITE 44-265	STRUCT	SCHEME	DWG 1

Appendix L

List of Standard Specifications

List of Standard Specifications

OPSD

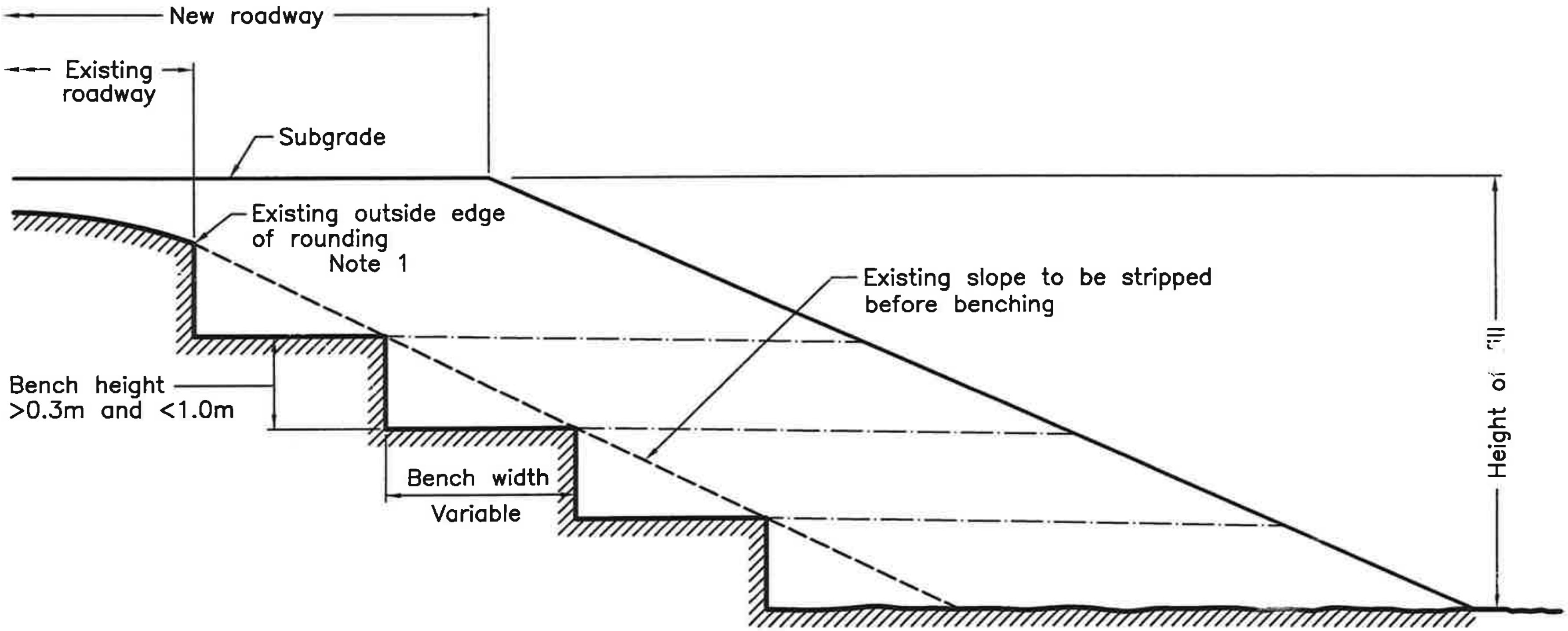
- 208.010 BENCHING OF EARTH SLOPES (included)
- 3000.100 FOUNDATION PILES STEEL H-PILE DRIVING SHOE (included)
- 3101.150 WALLS, ABUTMENT, BACKFILL MINIMUM GRANULAR REQUIREMENT (included)
- 3101.200 WALLS, ABUTMENT, BACKFILL ROCK (included)

OPSS

- 206 CONSTRUCTION SPECIFICATION FOR GRADING
- 212 CONSTRUCTION SPECIFICATION FOR BORROW
- 501 CONSTRUCTION SPECIFICATION FOR COMPACTING
- 571 CONSTRUCTION SPECIFICATION FOR SODDING
- 572 CONSTRUCTION SPECIFICATION FOR SEED AND COVER
- 1004 MATERIAL SPECIFICATION FOR AGGREGATES - BASE, SUBBASE, SELECT SUBGRADE, AND BACKFILL MATERIAL

SP

- SP903S01 PILING 2007
- CSP FOR INTEGRAL ABUTMENT (included)



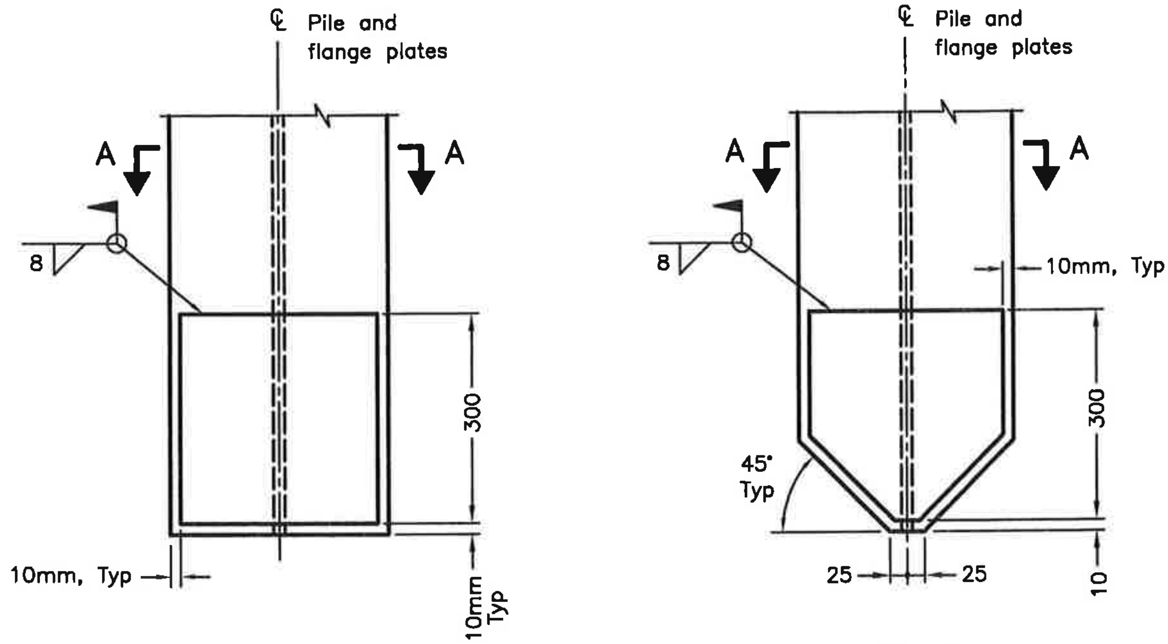
NOTES:

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.

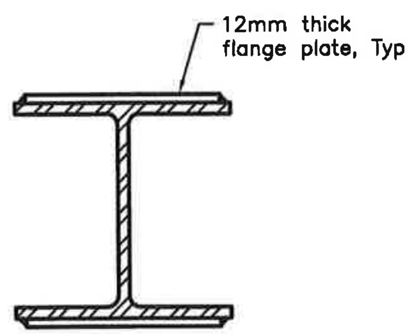
A Benching is not required on existing slopes flatter than 3H:1V.

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**	



ELEVATION



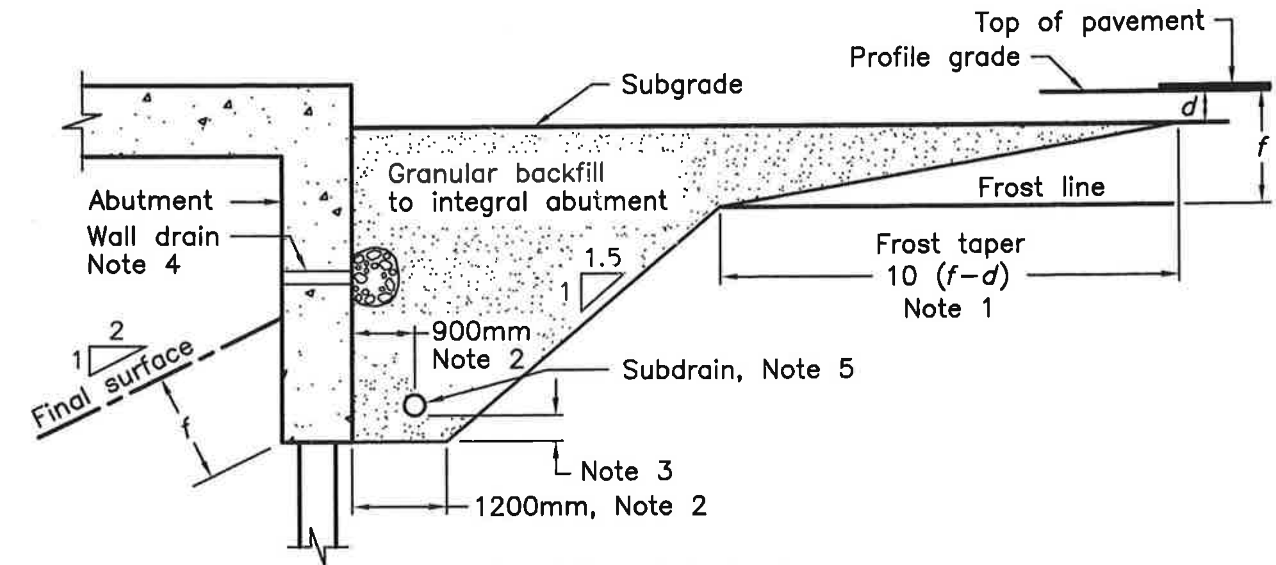
PILE DRIVING SHOE

SECTION A–A

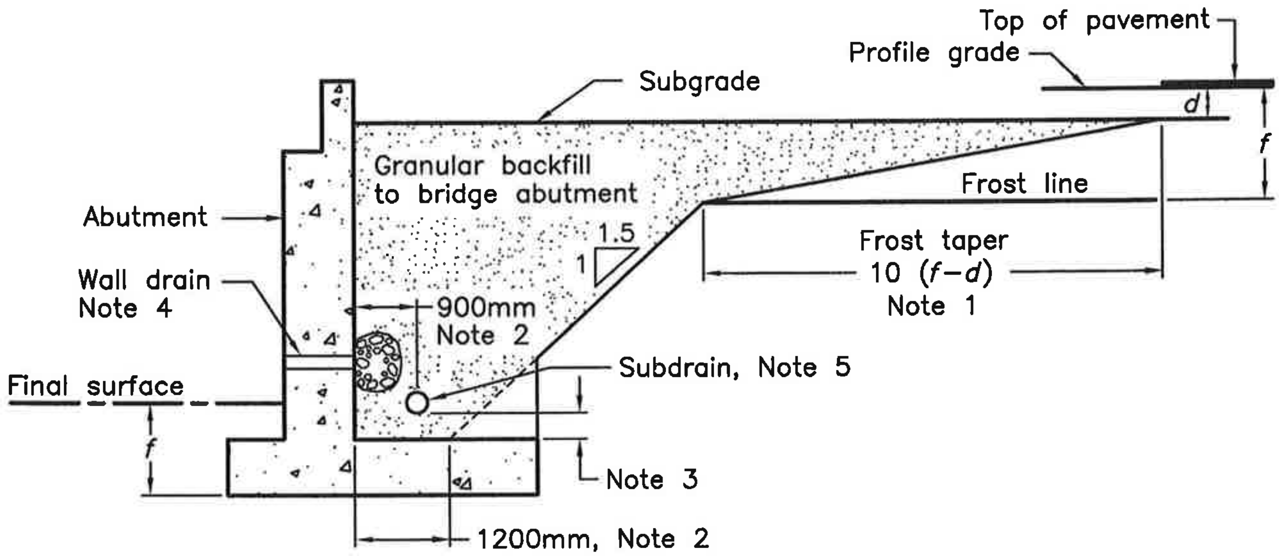
NOTES:

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.

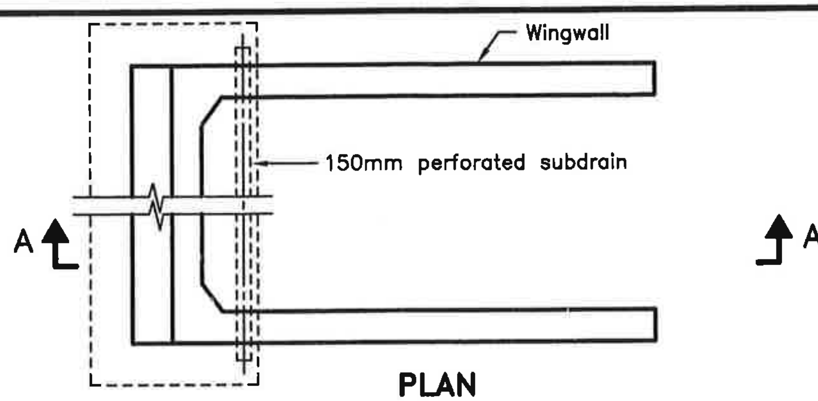
A Lateral limits of granular backfill to bridge abutment to be inside face to inside face of retaining wall or wingwall. Frost taper shall extend the full width of the fill unless interrupted by the retaining wall or wingwall.

B Sections shown are parallel to centreline of roadway.

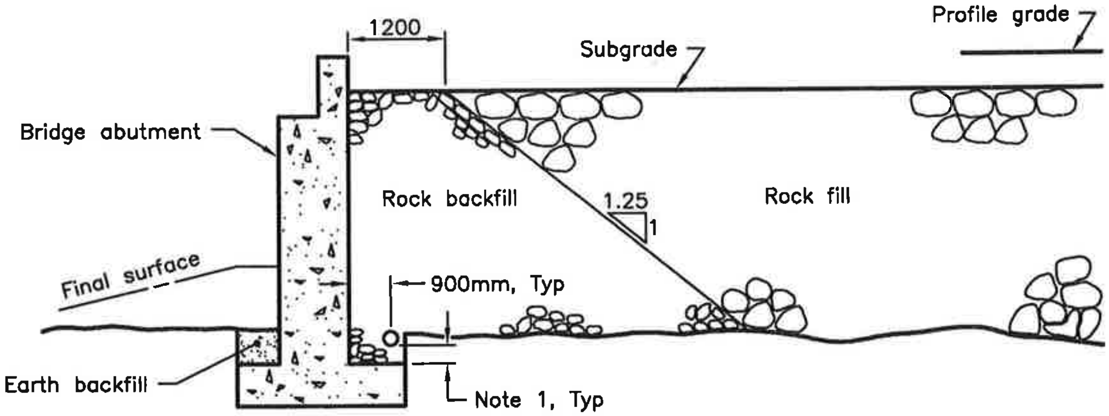
C Subdrain to be installed with a 2% gradient behind wall.

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	

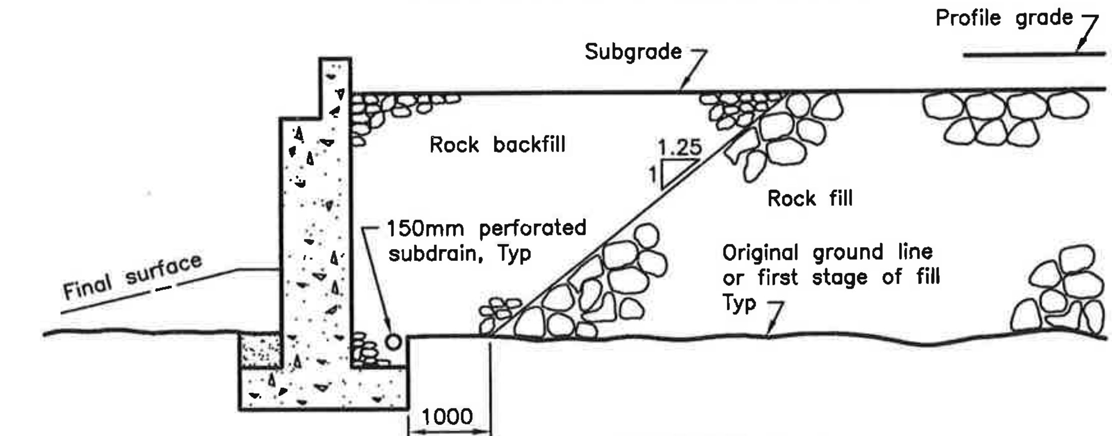


PLAN



SECTION A–A

STRUCTURE CONSTRUCTED BEFORE ROCK FILL



SECTION A–A

STRUCTURE CONSTRUCTED AFTER ROCK FILL

NOTES:

1 Height to be consistent with positive drainage of subdrain as specified.
A Dimensions perpendicular to back face of abutment.
B Grading and compaction of rock backfill and rock fill shall be as specified.
C Lateral limits of backfill to be inside face to inside face of wingwall.
D Section A–A parallel to centreline of roadway.
E All dimensions are in millimetres unless otherwise shown.

ONTARIO PROVINCIAL STANDARD DRAWING	Nov 2005	Rev 0
WALLS **ABUTMENT, BACKFILL** **ROCK**		
	OPSD – 3101.200	

CSP FOR INTEGRAL ABUTMENT - Item No.

Special Provision

Scope

This specification covers the requirements for the installation of the CSP's, including sand fill and polystyrene sheets, at the integral abutments.

References

This specification refers to the following standards, specifications or publications:

Ontario Provincial Standard Specifications, Construction:

OPSS 906 Structural Steel
OPSS 909 Prestressed Concrete - Precast Members

Ontario Provincial Standard Specifications, General:

OPSS 180 Management and Disposal of Excess Materials

Ontario Provincial Standard Specifications, Material:

OPSS 1605 Expanded Extruded Polystyrene
OPSS 1801 Corrugated Steel Pipe Products

Canadian Standards Association Standards:

CSA G164-M Galvanizing of Irregularly-Shaped Articles

Ministry of Transportation Publications

MTO Manual of Designated Sources of Materials

Definitions

For the purposes of this specification, the following definitions apply:

Abutment Stem: means the cast-in-place concrete component of the abutment placed over the top of the piles and forming the bearing seat for the girders.

CSP: means helical corrugated steel pipe.

Design Engineer: means the Engineer who produces the design and/or working drawings, and who has a minimum of five (5) years in the design and/or construction of bridges.

Submission and Design Requirements

Submissions

All submissions shall bear the seal and signature of the Design Engineer.

At least two weeks prior to commencement of installation of the abutment, the Contractor shall submit to the Contract Administrator, for information purposes only, three (3) sets of the working drawings.

The Contractor shall have a copy of the submitted working drawings on site at all times.

Working Drawing Requirements

Working drawings shall include at least the following:

1. Layout and Elevations of the CSP's;
2. Source of the sand fill, and description of placing method and equipment;
3. Location and details of all temporary bracing, including permanent and temporary spacers, for the piles, CSP's and abutment stems;
4. Detailed construction sequence for the work, including installation and removal of the temporary bracing.

Design Requirements

The Contractor shall be responsible for the complete detailed design of the construction sequence for the work, including the installation and removal of all temporary bracing. The general sequence of construction shall be as shown on the Contract drawings.

The Contractor shall be responsible for the complete detailed design of all temporary bracing, including temporary and permanent spacers, required to maintain the piles, CSP's, abutment stems and girders in their specified positions through all stages of construction until concrete in deck has reached a compressive strength of 25 MPa. All temporary bracing, except spacers identified as permanent on the Contract drawings, shall be removed.

Temporary bracing for prestressed, precast girders shall meet the requirements of OPSS 909. Temporary bracing for structural steel girders shall meet the requirements of OPSS 906.

Material

Corrugated Steel Pipe

CSP shall be in accordance with OPSS 1801, and shall be from a supplier listed under DSM # 4.60.80. The CSP shall be of the diameter and wall thickness specified on the Contract drawings, and shall be galvanized in accordance with CSA G164-M.

Permanent Spacers and Associated Hardware

Permanent spacers and associated hardware left in place shall not consist of wood and corrodible material.

Sand Fill

The sand fill for backfilling the inner CSP shall meet the gradation requirements of Table 1 below:

Table 1 - Sand Fill Gradation Requirements

MTO Sieve Designation		Percentage Passing by Mass
2 mm	# 10	100 %
600 μm	# 30	80 % to 100 %
425 μm	# 40	40 % to 80 %
250 μm	# 60	5 % to 25 %
150 μm	# 100	0 % to 6 %

Expanded Extruded Polystyrene

Expanded extruded polystyrene shall be in accordance with OPSS 1605, and shall be from a supplier listed under DSM # 3.30.30.

<u>Construction</u>

General

The sequence of construction for installing the concrete pads, CSP's, sand fill and abutment stems, including the installation and removal of the temporary bracing, shall be in accordance with the working drawings.

The Contractor shall not proceed with the abutment backfill above the level of the bottom of the CSP's without written permission from the Contract Administrator.

Corrugated Steel Pipe

CSP's shall be supplied in the lengths and with the end treatments, either square or skew, as specified on the Contract drawings; field cutting and splicing of CSP's will not be permitted. Cut ends shall be neat and free of burrs. The planes defined by the end treatments of each CSP shall be parallel to each other.

Handling and storage of CSP's shall be in accordance with the manufacturer's recommendations. Damaged CSP's shall be rejected. Localized areas of damaged galvanizing on otherwise acceptable CSP's shall be repaired by two coats of zinc-rich paint.

The Contractor shall set the inner and outer CSP over each pile in the abutment into the concrete pad, following the batter of the pile, while the concrete in the concrete pad is still plastic. The CSP's shall extend at least 150 mm into the concrete pad.

The Contractor shall ensure the full perimeter of the tops of all CSP's at each abutment are at the elevation shown on the working drawings.

After the CSP's have been set, the Contractor shall take all measures necessary to prevent the ingress of water, backfill and debris into the CSP's.

Sand Fill

The sand fill shall be placed dry of optimum and free-flowing, completely filling the volume between the inner CSP and pile. No additional compaction effort other than the action of placing the sand fill itself shall be applied to the sand fill.

The placing of the sand fill shall be carried out in a manner such as to not damage and displace the CSP's.

After the sand fill has been placed to the top of each inner CSP, the Contractor shall take all measures necessary to prevent the ingress of water and other liquids into the sand fill until after the concrete in the abutment stem has been placed and cured.

Expanded Extruded Polystyrene

The expanded extruded polystyrene sheets shall completely cover the area under the abutment stem as shown on the Contract drawings. The sheets shall be placed in one piece for the width of the abutment stem, with butt joints perpendicular to the centre-line of abutment bearings. The minimum length of sheet shall be 500 mm.

Joints between sheets within 500 mm of a pile centre-line will not be permitted. At each pile location, a minimum 1000 mm long sheet shall be centred on the pile and a 500 mm diameter hole neatly cut in the sheet so as to fit over the pile in one piece, fully spanning the annular space between the double CSP's.

The Contractor shall adjust the backfill to ensure full and uniform contact of the sheets with the backfill and the full perimeter of the tops of the CSP's. The vertical step at joints between sheets shall not exceed 5 mm.

The Contractor shall protect the sheets from damage during installation of the reinforcing for the abutment stem, and shall secure the sheets from "floating" during placing of the concrete in the abutment stem. Only hardware approved by the Owner shall be used to secure the sheets. All hardware used to secure the sheets shall be installed so as not to project above the top surface of the sheets into the abutment stem.

Temporary Bracing

Temporary bracing shall be installed and removed in accordance with the working drawings.

The temporary bracing shall not distort, nor pierce the walls of, the CSP's. Welding to the CSP's will not be permitted.

Concrete anchors shall be removed and the holes filled with non-shrink grout.

Tolerances

The CSP's at each pile shall be constructed to the following tolerances:

Criteria	Tolerance
Maximum deviation of inner and outer CSP from pile centroid.	± 25 mm
Maximum deviation from specified spacing between inner and outer CSP's.	± 25 mm
Maximum deviation of any point on the top perimeter of the CSP's from the specified Elevation.	± 10 mm

Quality Assurance

Prior to placing the CSP's, the Contractor shall establish reference points at each abutment and determine the location of the centroid of each pile in the abutment with respect to these reference points. The Contractor shall maintain the reference points until written permission to proceed with the backfill above the level of the bottom of the CSP's has been given by the Contract Administrator.

Measurement for Payment

There will be no measurement for this item.

Basis of Payment

Payment at the contract price for the above items shall be full compensation for all labour, equipment and material required to do the work.

Appendix M

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.