

**FOUNDATION INVESTIGATION AND
DESIGN REPORTS, SHEWFELT BRIDGE
REPLACEMENT, GOULAIS BAY ROAD,
3 KM WEST OF HIGHWAY 17, DISTRICT OF
ALGOMA, ONTARIO, G.W.P. 5290-04-00,
SITE 38S-031
GEOCRES NO. 41K-82**

LEA Consulting Limited

Project: SPT1156
December 09, 2009

December 09, 2009

LEA Consulting Limited
625 Cochrane Drive, Suite 900
Markham, Ontario
L3R 9R9

Attention: Peter Ojala, P.Eng

Dear Sirs:

**RE: Foundation Investigation and Design Reports, Shewfelt Bridge Replacement Goulais Bay
Road, 3 Km West of Highway 17, District of Algoma, Ontario, G.W.P. 5290-04-00, Site 38S-031
GEOCRES No. 41K-82**

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

For and on behalf of Coffey Geotechnics Inc.



Ramon Miranda, P.Eng
Manager, Transportation Division

**FOUNDATION INVESTIGATION REPORT
SHEWFELT BRIDGE REPLACEMENT
GOULAIS BAY ROAD, 3 KM WEST OF
HIGHWAY 17,
DISTRICT OF ALGOMA, ONTARIO
G.W.P. 5290-04-00, SITE 38S-031
GEOCRES NO. 41K-82**

LEA Consulting Limited

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**FOUNDATION INVESTIGATION REPORT
SHEWFELT BRIDGE PLACEMENT, GOULAIS BAY ROAD
3 KM WEST OF HIGHWAY 17, DISTRICT OF ALGOMA
ONTARIO, G.W.P. 5290-04-00, SITE 38S-031**

1 INTRODUCTION

Coffey Geotechnics Inc. (Coffey) was retained by LEA Consulting Limited (LEA) to carry out a foundation investigation at the site of the proposed replacement of the Shewfelt Bridge over the Goulais River on Goulais Bay Road between Highway 552 and Pine Shores Road, in the Township of Fenwick, approximately 3 km west of Highway 17. The site is located within the District of Algoma and has MTO Site Number 38S-031.

The existing Shewfelt Bridge is a four-span bridge with a total length of 84.7 m, which contains a two-span single lane Bailey bridge (63.1 m) with a timber deck, and two steel girder end spans with a concrete deck, each 10.8 m in length. It is understood that the performance of the existing bridge is affected by the problems of bridge foundation settlements and rotation, slope stability, active erosion and riverbank slumping (upstream of the existing bridge).

In 2006, Shaheen & Peaker Limited (now known as Coffey Geotechnics Inc.) was retained to carry out an advance foundation investigation. During this investigation, a borehole was put down on each side of the river at the proposed new bridge location. Borehole 1 was drilled at the proposed east abutment location and Borehole 2 was advanced at the west abutment location. The findings of that investigation were presented in our report entitled "Advance Foundation Investigation, Shewfelt Bridge Replacement, Goulais Bay Road, 3 km west of Highway 17, District of Algoma, Ontario, G.W.P. 5290-04-00, Site 38S-031," dated May 24, 2006, Project No. SPT1156A.

Since then the location of the proposed bridge was finalized and Coffey was retained by LEA to conduct another investigation for the detail design of the bridge and for the approach roads.

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 Goulais River is located in a deep and wide valley (the Goulais River Valley) north of Sault Ste. Marie. In the general vicinity of the project site, the area is referred to as the Goulais River Beach Ridges, which is described as ancient beach ridges of an alluvial plain. The river meanders on its way toward Lake Superior and numerous oxbow lagoons are evident.

The Goulais River in the vicinity of the project site has steep banks, with bank failures having occurred in many areas. It is evident that the Goulais River is continuing to undercut its banks at turns in the river,

resulting in slope failures and re-alignment of the river channel. It is noted that a section of the existing Goulais Bay Road located to the north of the existing bridge near the west bank of the river is at close proximity to such a bend in the present river channel. Appendix D provides photographs illustrating the site setting.

Based on available information, the Goulais River Valley was probably cut by a major pre-glacial river. At the time of the retreat of the last glaciations, a river flowed in the Goulais Valley carrying glacial materials into Glacial Lake Algonquin, resulting in deep glacial deposits. As well, it appears that deep clays were deposited and followed by sands and silts deposited by the present river itself.

3 INVESTIGATION PROCEDURES

The fieldwork for the proposed Shewfelt Bridge was performed during the period of September 22, 2008 through October 11, 2008. As agreed with MTO, the fieldwork consisted of drilling and sampling two boreholes (Boreholes 101 and 102) for the bridge structure, six boreholes for the approach fills (Boreholes 103 through 107) and two boreholes for a cut section of the proposed road, some 120 to 150 m east of the proposed bridge location (Boreholes 108 and 109), as well as performing field and Dynamic Cone Penetration tests (DCPT). In addition, a 9.1 m deep borehole was drilled adjacent to Borehole 102 to install two shallow piezometers. As mentioned before, two boreholes (Boreholes 1 and 2) were previously drilled at the site in 2006, for the advance investigation. The plan location of the previous and present boreholes is shown in Drawing Nos. 1, 3 and 4. 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
BH1	East side of Goulais River	48.3	19.8 m to 24.3 m 25.9 m to 30.5 m 48.3 m to 54.9 m	1 deep piezometer
BH2	West side of Goulais River	41.6	40.2 m to 41.6 m	-
BH101	In the River	47.9*	38.1* m to 40.8 m* 41.3* m to 42.9 m*	-
BH102	West side of Goulais River	56.5	43.4 m to 53.8 m	1 deep piezometer
BH102A	Adjacent to BH102	9.1	-	2 shallow piezometers
BH103	West side of Goulais River	16.5	-	-
BH104	West side of Goulais River	10.4	-	1 piezometer
BH105	West side of Goulais River	8.8	-	-
BH106	West side of Goulais River	4.4	-	-
BH107	East side of Goulais River	6.7	-	-
BH108	East side of Goulais River	6.7	-	1 piezometer

Borehole No.	Location	Drilling Depth Below Existing Ground Surface (m)	Dynamic Cone Penetration Tests	Piezometer
BH109	East side of Goulais River	8.8	-	1 piezometer

*below the water level in the River

Walker Drilling of Utopia, Ontario carried out the drilling, testing and sampling work, under the direction and supervision of a Professional Engineer from Coffey. Boreholes which were put down from the land were advanced using a truck and a track-mounted drilling rigs, both outfitted with tools and equipment for soil sampling and testing. Drilling was effected using hollow-item augers, however, in Borehole 102 wash boring methods were also utilized below a depth of 9 m. As well coring was effected to advance the borehole through cobbles and boulders.

Borehole 101 was located in the river and this borehole was drilled from a raft. The borehole was advanced by wash boring methods, using NW casing. Coring was also utilized to advance the borehole.

In the deep boreholes (BH1, BH2, BH 101 and BH102) drilling mud was utilized to counter-balance the hydrostatic uplift due groundwater.

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 or 70 mm 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 Field Vane was used to conduct the tests but when the soil became stiffer this was changed to small Field Vane.

As mentioned in Boreholes 101 and 102 (and also in Boreholes 1 and 2), 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 to the piezometer that was installed in Borehole 1 during the previous investigation, six additional piezometers were installed during this investigation, as detailed in Table 3.1, to enable

groundwater level monitoring in the boreholes over a prolonged period of time without interference from surface water in Boreholes 102, 102A, 104, 108 and 109. The remaining boreholes were grouted upon their completion using a cement/bentonite mixture as per MTO procedures.

The borehole locations were established by our field supervisor in relation to centreline stakes that were previously established in the field by surveyors (retained by LEA) prior to our field crew's arrival at the site. Based on the provided bench mark information, the Geodetic ground surface elevations at the borehole locations were surveyed by our fieldwork supervisor. The benchmark used was HCP 1110 which we understand has Geodetic elevation of 189.48 m. This benchmark, which is a railway spike in a root of a 0.6 m diameter spruce tree is located to the south of the road alignment near the eastern bank of the Goulais River.

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 oedometer (consolidation) and Atterberg Limits tests, was performed on selected representative soil samples. 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 nine boreholes (see Table 3.1 in Section 3) during the current investigation and two boreholes during the 2006 investigation (Advance Foundation Investigation). The plan locations of the boreholes are shown on Drawing Nos. 1, 3 and 4. Details of sub-surface conditions encountered at each borehole location for the current and the prior (2006) investigation, including the results of in-situ testing, groundwater observations and laboratory test results, are presented on the Record of Borehole Sheets in Appendix A. A stratigraphic profile at bridge section, cut section and fill section is shown on Drawing Nos. 2, 3 and 4. Detailed laboratory test results are enclosed in Appendix B. Rock core photographs are shown in Appendix E.

In general, the sub-surface stratigraphy comprises surficial topsoil and/or minor fill materials overlying typically very loose to loose cohesionless sand, sandy silt and silty sand to sandy silt deposits, which are in turn underlain by a 33 to 38 m thick deposit of soft to very stiff silty clay. The upper and lower 2 to 4 meters of the deposit was found to contain frequent silt and clayey silt interbeds. In the deep boreholes, the silty clay is further underlain by a clayey silt deposit, followed by silty sand, gravels and cobbles at the bottom of the boreholes. Probable bedrock was encountered in Borehole 101 at a depth of about 46 m/ El. 137 m. Borehole 102 was extended some 6 m below this elevation without encountering bedrock.

4.1 Topsoil

A 0.2 to 0.5 m thick sandy topsoil layer was contacted in Boreholes 1, 2, 103, 104, 105, 106, 108 and 109.

4.2 Surficial Fill

Borehole 107 contacted a 0.15 m thick sand and gravel layer followed by silty fine sand fill extending to a depth of 0.8 m below the ground surface or to El. 193.4 m.

Based on a recorded N-value of 24 blows/0.3 m this basically granular surficial fill is considered compact.

4.3 Silty Fine Sand

A surficial silty fine sand deposit was contacted in Boreholes 1, 2, 102 and 106. This granular (non-cohesive) soil extended to a depth of 0.7 to 0.8 m in Boreholes 1, 2 and 102 and to 1.4 m (El. 186.0 m) in Borehole 106.

Standard Penetration tests performed in this deposit yielded N-values which range from 4 blows/0.3 m (Boreholes 2, 102 and 106) to 8 blows/0.3 m (Borehole 1) which indicate a very loose to loose condition.

4.4 Sand

On the east side of the river (where the grade is approximately 6 m higher than the west side) the boreholes show the presence of a surficial sand deposit. In Boreholes 1, 107, 108 and 109, which were put down on the east side of the river, the surficial sand was contacted immediately underlying the topsoil, fill or surficial silty fine sand at depths of 0.3 to 0.8 m below the ground surface or below Elevations 192.7 to 193.4 m. The thickness of this non-cohesive granular deposit was found to range from 1.9 to 3.9 m and it extended to depths of 2.2 to 4.6 m below the ground surface or to El. 188.8 to 191.4 m.

The grain-size distribution of three samples from the deposit from Boreholes 107 and 109 is given in Figure B-1, in Appendix B. This indicates the following grain-size distribution.

Gravel:	1-9%
Sand:	89-95%
Silt & Clay:	2-4%

From the grain-size distribution, the material is considered to be more pervious than the underlying sandy silt to silty sand deposits, with an estimated coefficient of permeability (k) of the order of 1×10^{-1} to 4×10^{-2} cm/sec.

N-values recorded in this deposit generally range from 5 to 27 blows/0.3 m (typically 8 to 16 blows/0.3 m) which indicate a loose to compact condition.

A similar sand layer was also contacted in Borehole 101 immediately below the river bed, as well as in all the boreholes drilled on the west bank (i.e. Boreholes 2, 102, 103, 104, 105 and 106) at depths of 2.5 to 4.1 m below the ground surface or below El. 183.6 m to 184.9 m. The deposit was also contacted on the east bank in Boreholes 108 and 109 at a depth of 4.4 m below the ground surface or below El. 189.2 and 189.3 m, respectively. The thickness of this granular soil was found to range from 1.6 to 4.5 m and it was found to extend to Elevations ranging from 187.7 m to 179.4 m.

The grain-size distribution of seven samples from this granular deposit is given on Figure B-2 in an envelope form. As shown, the following grain-size distribution is indicated:

Gravel:	0-17%
Sand:	80-95%
Silt & Clay:	2-5%

Standard Penetration tests performed in this sand deposit gave N-values which range from 0 to 22 blows/0.3 m indicating a very loose to compact but typically loose to compact relative density.

4.5 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand was encountered in all the boreholes drilled at the site, except for Borehole 101. In this borehole, the deposit appears to have been eroded by the meandering river.

In the east bank area where the o.g. levels are higher, this deposit was found underlying the sand deposit at depths ranging from 2.2 m (Borehole 108) to 4.6 m (Borehole 1) or below elevations ranging from 191.4 to 188.8 m. In Borehole 1 the thickness of the deposit was found to be 4.4 m where it extended to El. 184.3 m whereas in Boreholes 107, 108 and 109, the thickness of the deposit was found to range from 1.5 to 2.3 m and it extended to El. 189.3 to below El. 187.5 m. In Boreholes 108 and 109 a second layer of the deposit was found below El. 187.6 and 187.7 m, respectively.

On the west bank area, the deposit was found immediately below the topsoil or the surficial silty fine sand at depths of 0.3 to 1.4 m. It appears that the surficial sand deposit was probably eroded by the meandering river. The thickness of the sandy silt to silty sand deposit in the boreholes drilled on the west side of the river was found to range from about 3.4 to 3.7 m near the river (in Boreholes 2, 102, 103 and 104) to 2.1 to 1.6 m in boreholes 105 and 106, away from the watercourse. The deposit was found to extend to El. 182.6 to 184.9 m.

This is basically sandy silt deposit with some silty fine sand zones. The grain-size distribution of five samples from the deposit is presented in an envelope form in Figure B-3 in Appendix B. The results show the following gradation:

Gravel:	0%
Sand:	33-59%
Silt:	33-58%
Clay:	4-14%

This deposit is considered to be less pervious than the sand deposit which overlies it on the east side and underlies it on the west side of the present watercourse location, but more pervious than the massive silty clay deposit which underlies the entire site.

N-values recorded in this unit ranged from 2 to 24 blows/0.3 m but typically 3 to 9 blows/0.3 m. These results indicate a generally very loose to loose relative density with occasional compact zones (e.g. Borehole 107).

4.6 Silty Clay

Underlying the non-cohesive deposits described in the previous sections, all the deep boreholes contacted a massive cohesive deposit at depths ranging from 3.9 m (El. 179.4 m) in Borehole 101 (below the river bottom) to 9.0 m (El. 184.3). 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)
1	9.0/184.3	47.2/146.1
101	3.9/179.4	37.0/146.3
102	7.0/180.7	42.3/145.4
2	7.6/179.5	40.5/146.6

The deposit consists of a reddish silty clay to clay with occasional clayey silt and grey silt seams. The upper and lower portions of the deposit (typically the upper and lower 2 to 4 m) was found to contain frequent clayey silt and silt seams and thus the upper and lower zones typically resemble a layered clayey silt material.

The grain-size distribution of four samples from the deposit (from Boreholes 101 and 102) is given in Figure B-4. Five samples from Boreholes 1 and 2 were tested during our 2006 investigation and these are included as Figure B-5. The results of the tests on the nine samples show the following grain-size distribution:

Gravel:	0-2%
Sand:	0-2%
Silt:	22-64%
Clay:	32-78%

Figure B-6 from 2006 investigation shows the grain-size distribution of a sample from the upper clayey silt zones of Borehole 1. This indicates 74% silt and 26% clay size particles.

When analysing these grain-size results it should be kept in mind that the samples tested is a mixture of several or more individual interbeds.

The Atterberg limits tests performed during the present (17 samples) and the previous investigation (6 samples) are given in Figures B-7, B-8, B-9 and B-10 in Appendix B. These tests yielded the following index values:

Liquid Limit:	25-79% (Average 55%)
Plastic Limit:	16-31% (Average 24%)
Plasticity Index:	9-55 (Average 31)

These results indicate clayey soils of low to high but typically medium to high plasticity. The lower plasticity index values are from the upper or lower more silty zones, typically above and below Elevations 175 and 153 m, respectively. There is also some variation within each zone, where annual deposition shows a range from more plastic (i.e. fatter) to relatively less plastic (i.e. leaner) clay content. As shown on the individual Record of Borehole Sheets, the measured natural moisture contents are generally near or in

excess of the measured liquid limits which indicate the likelihood of a normally consolidated soil deposit, or only a slight pre-consolidation.

Standard Penetration tests conducted in the silty clay deposit gave N-values which typically range from 0 to 3 blows/0.3 m which indicate a very soft consistency but N-values as high as 22 blows/0.3 m were also recorded indicating firm to very stiff zones. The higher N-values were typically recorded in the clayey silt zones which contain stiffer than silt/clayey silt interbeds (generally in the upper and lower zones of the deposit).

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 12 to in excess of 100 kPa, indicating a very soft to very stiff consistency.

In Figures C1 and C2 (Appendix C) the variation of the measured in-situ vane strength values (i.e. in-situ undrained shear strengths) at each deep borehole versus elevation is presented. Also plotted on each figures are 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, perhaps due to removal of previously existing overburden. A total of four oedometer (one dimensional consolidation) tests was performed in the laboratory on 50 to 70 mm diameter Shelby tube (TW) samples (three from Borehole 102 and one from Borehole 103). The results are presented in Figures B-11 through B-14, in Appendix B. These show a possible pre-consolidation pressure in excess of existing overburden pressure $P'_c - P'_o$ in order of 80 to 150 kPa. It should be pointed out that the presence of silty seams was noted in samples TW 11 (Borehole 102) and TW 12 (Borehole 103) and this is expected to have affected consolidation test results because the silty soil typically be less compressible than the adjacent clay.

The measured bulk unit weight of the TW samples range from 15.3 to 18.3 kN/m³.

4.7 Silty Sand with Gravel (Possible Till)

In Boreholes 101 and 102 the silty clay to clayey silt deposit is underlain at depths/elevations of 37.0/146.3 m and 42.3/145.4 m by a relatively coarse grained soil consisting of silty sand with gravel and occasional cobbles and boulders. From an examination of the soil samples recovered from the deposit, it is likely to be of glacial till origin (i.e. a silty sand till). The deposit was found to extend to about depths/elevations of 40.0 m/ El. 143.3 m and 50.0 m/ El. 137.7 m in Boreholes 101 and 102, respectively.

The grain-size distribution of two samples from the deposit from Borehole 102 gave the following results:

Gravel:	34-40%
Sand:	38-43%
Silt:	16-17%
Clay:	6%

as shown in Figure B-15 in Appendix B.

Two Standard Penetration tests performed in this deposit yielded N-values of 36 and 85 blows/0.3 m and Dynamic Cone Penetration tests (DCPT) gave blow counts which range from 14 to 45 blows/0.3 m. Based on these values, the relative density of the deposit is considered to be typically compact to dense with a very dense zone in the upper portion of Borehole 101.

4.8 Silty Fine Sand

Underlying the silty sand till, Borehole 101 contacted at a depth of 40 m (El. 143.3 m) a relatively fine-grained soil consisting of silty fine sand with some gravel and occasional cobbles and boulders. This deposit was found to extend to 42.0 m below the river bed or to El. 140.1 m.

Based on DCPT test results which range from 25 to 60 blows/0.3 (typically 30 to 40 blows/0.3 m), the relative density of the deposit is described as compact to dense.

4.9 Gravel and Cobbles

Underlying the silty clay, Boreholes 1 and 2 encountered at 47.2 m (El. 146.1 m) and 40.5 m (El. 146.6 m), respectively, a deposit which consists of gravel and cobbles. The presence of some sand in-fill as well as occasional boulders was also noted. These two boreholes were terminated in this coarse grained granular deposit at 48.3 m and 41.6 m (El. 145.0 m and 145.5 m), respectively.

Measured SPT N-values in this deposit ranged from 44 blows per 0.3 m (Borehole 1) to 50 blows per 0.08 m penetration (Borehole 2), indicating dense to very dense relative density. The measured natural moisture content of one soil sample was 11%.

Dynamic Cone Penetration Tests (DCPT) were carried out at the bottom of Boreholes 1 and 2. In Borehole 1, DCPT was carried out from 48.5 m to 54.9 m (El. 144.8 m to El. 138.4 m) below ground surface. As can be seen in the DCPT plots in the Record of Borehole Sheets, the DCPT blow counts had high variation with depth (a "zigzag" curve pattern) with test results varying from 19 to over 200 blows per 0.3 m penetration, and this may be due to the presence of cobbles and/or boulders which obstructed the penetration of the cone/rod assembly. This may also be the result in the bending of the rods and the non-vertical penetration of the cone/rod assembly. The DCPT encountered refusal at 54.9 m (El. 138.4 m) below ground surface. From the results, the relative density of the soil below El. 145.0 m can be surmised to be compact to very dense.

In Borehole 2, Dynamic Cone Penetration Test was carried out from 40.2 m to 41.6 m (El. 146.9 m to El. 145.5 m) and encountered refusal at 41.6 m (El. 145.5 m) below ground surface.

The deposit also contacted in Boreholes 101 and 102 at depths/elevations 43.2 m/140.1 m and 50.0 m/137.7 m, respectively. The boreholes were extended in this deposit by a vertical distance of 2.8 m and 6.5 m, respectively. Due to the presence of boulders and cobbles, frequent coring was resorted to advance the boreholes. Borehole 102 was terminated in this deposit while in Borehole 101, coring results indicate the possible presence of bedrock, underlying this deposit at a depth of 44.8 m below the river bottom, or below El. 137.3 m.

In these two boreholes, reliable N-values could not be obtained due to the presence of boulders and advancing by means of coring. But based on observations made while drilling and DCPT results, the relative density is probably compact to dense.

4.10 Probable Bedrock

In Borehole 101, which was put down by washboring methods from a raft (at the proposed pier location) in the river, a reddish brown colored metamorphised sandstone was contacted at a depth of 44.8 m below the bottom of the river or at El. 137.3 m. This was cored for a vertical distance of 1.9 m to 46.7 m below the bottom of the river or El. 135.4 m where the borehole was abandoned and plugged due to a severe artesian condition. This possibly represents the bedrock, although Borehole 102 was extended below this elevation to El. 131.2 m without encountering bedrock.

The percentage of recovery was 100% while the RQD value recorded from El. 137.3 to 137.0 m was 48% and below this elevation to El. 135.4 m, it was 72%.

4.11 Groundwater Conditions

Groundwater conditions were observed in the open boreholes while drilling and upon completion of each borehole. In the deep boreholes, where washboring methods 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 4.11.1 Summary of Groundwater Level Measurements

Borehole No./Elevation	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers	Remarks
1 (193.3 m)	47.8/145.5	5.6/187.7	March/06	Yes	Lower hydrostatic level
2 (187.1 m)		3.4/183.7	March/06	No	On completion Upper hydrostatic level
101 (182.1 m)	44.0/138.1 46.7/135.4	+3.3/185.4 +5.5/187.6	Nov/08	No	Artesian condition measured inside casing while drilling in lower hydrostatic level
102 (187.7 m)	56.5/131.2	+1.4 to +2.3/ 189.1 to 190.0	Sept-Oct/08	Yes	Artesian condition Lower hydrostatic level
102A (187.7 m)	6.1/181.6 9.1/178.6	4.2/183.5 4.2/183.5	Sep-Oct/08 Sept-Oct08	Yes Yes	Upper Hydrostatic level Upper Hydrostatic level
103 (187.6 m)		3.6/184.0	Sept/08	No	On completion Upper hydrostatic level
104 (187.4)	9.1/178.3	3.8/183.6	Sept/08	Yes	Upper hydrostatic level
105		3.8/183.6	Sept/08	No	On completion

Borehole No./Elevation	Depth/Elevation of the Tip of Piezometer (m)	Water Level Measurement Depth/Elevation (m)	Date	Piezometers	Remarks
(187.4 m)					Upper hydrostatic level
106 (187.4 m)		3.5/183.9	Sept/08	No	On completion Upper hydrostatic level
107 (194.2 m)		4.2/190.0	Sept/08	No	On completion Upper hydrostatic level
108 (193.6 m)	6.0/187.6	4.4/189.2	Sept-Oct 08	Yes	Upper hydrostatic level
109 (193.7 m)	7.6/186.1	4.2/189.5	Oct/08	Yes	Upper hydrostatic level

From the measured values, it appears that there are two distinct water/piezometric levels at the site, namely an upper level and a lower level (below the silty clay deposit).

On the east bank, the upper water level was measured at about El. 189.2-190.0 m while the lower piezometric level deeper in the profile was measured at El. 187.7 m. On the west bank, the upper water level was measured at El. 183.5-184.0 m, while the lower piezometric level deeper in the profile was measured at 189.1-190.0 m (i.e. similar to the water level measured in the piezometer installed in the east bank area). This represents an artesian condition in relation to the ground levels on the west side of the river. In the borehole drilled in the middle (i.e. within the river), an artesian condition was noted which was measured to reach 5.5 m above the bottom of the river or El. 187.6 m. It is likely that this level could have reached El. 189-190 m. It is also of interest to note that when the artesian condition was being measured, the water level in the deep piezometer installed on the west bank (i.e. Borehole 102) was noted to start reacting (i.e. dropping).

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

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.





Zuhtu Ozden, P.Eng.



Drawings

METRIC

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

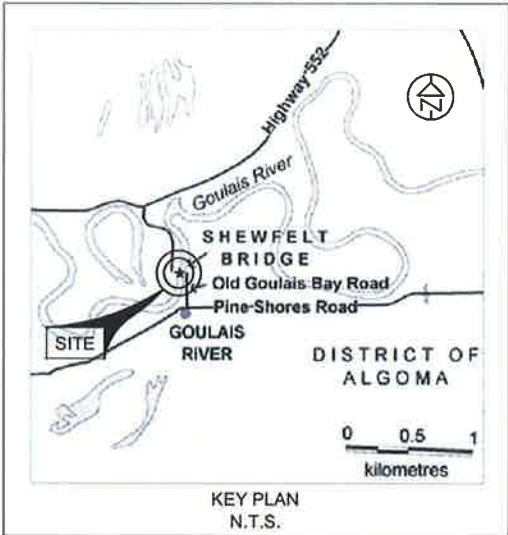
CONT No.
GWP: 5290-04-00

OLD GOULAIS BAY ROAD
SHEWFELT BRIDGE
BOREHOLE LOCATION PLAN



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



LEGEND

- Borehole
- Borehole & Cone

No.	ELEVATION	NORTHING	EASTING
BH 101	183.3m	5175927.5	275561.3
BH 102	187.7m	5175938.2	275525.9
BH 103	187.6m	5175948.2	275498.7
BH 107	194.2m	5175904.3	275627.4
BH 1	193.3m	5175908.7	275614.1
BH 2	187.1m	5175945.1	275512.4

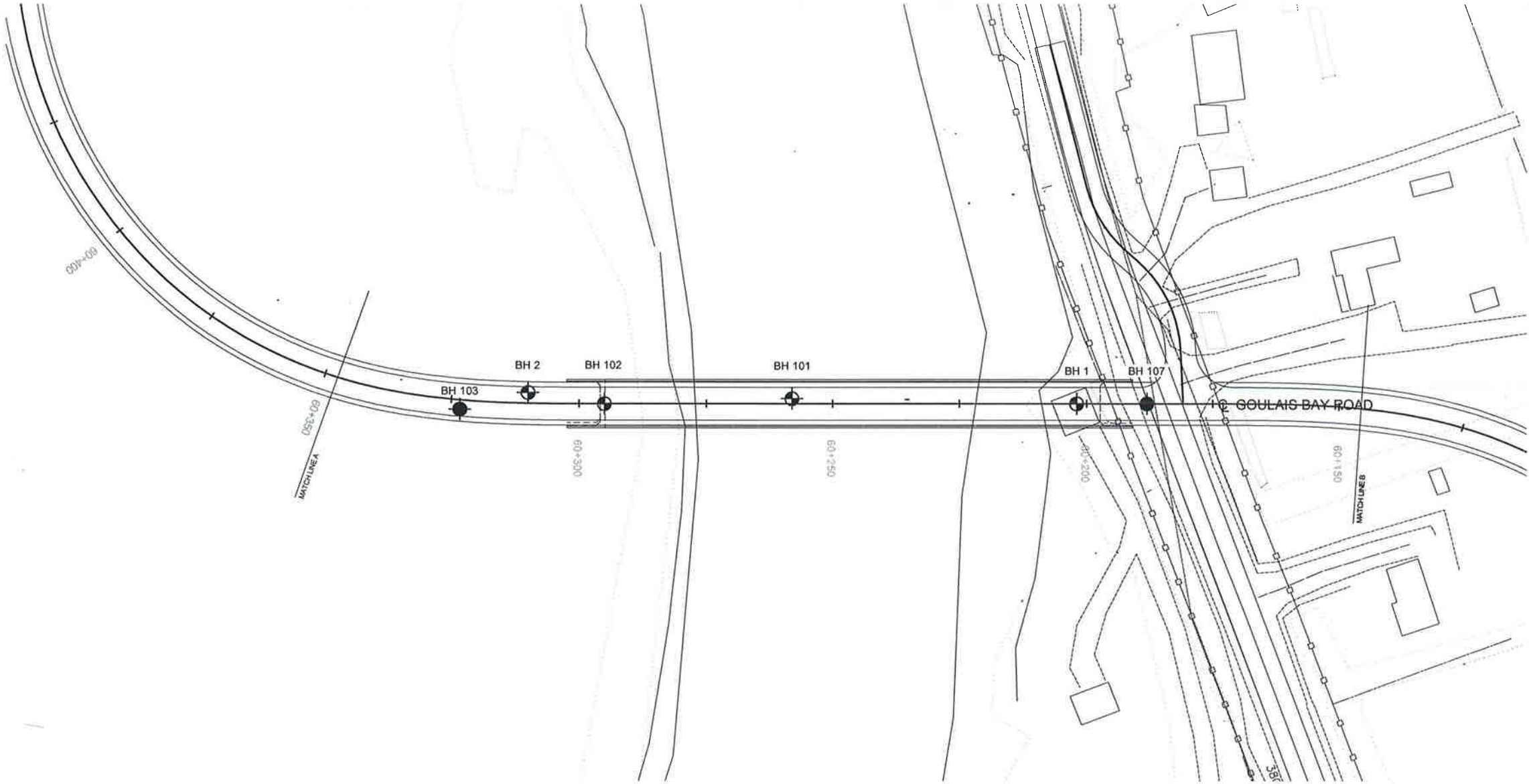
-NOTE-

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-82				DIST	
TRANETO801156AA				SITE	
SUBMD	CHECKED	DATE	Dec 8, 2009	38S-031	
DRAWN	PHK	CHECKED	RM	APPROVED	ZO
				DWG	1



PLAN
SCALE



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.
GWP: 5290-04-00

OLD GOULAIS BAY ROAD
SHEWFELT BRIDGE
SOIL STRATA

SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



LEGEND

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

No.	ELEVATION	NORTHING	EASTING
BH 101	183.3m	5175927.5	275561.3
BH 102	187.7m	5175938.2	275525.9
BH 103	187.6m	5175948.2	275498.7
BH 107	194.2m	5175904.3	275627.4
BH 1	193.3m	5175908.7	275614.1
BH 2	187.1m	5175945.1	275512.4

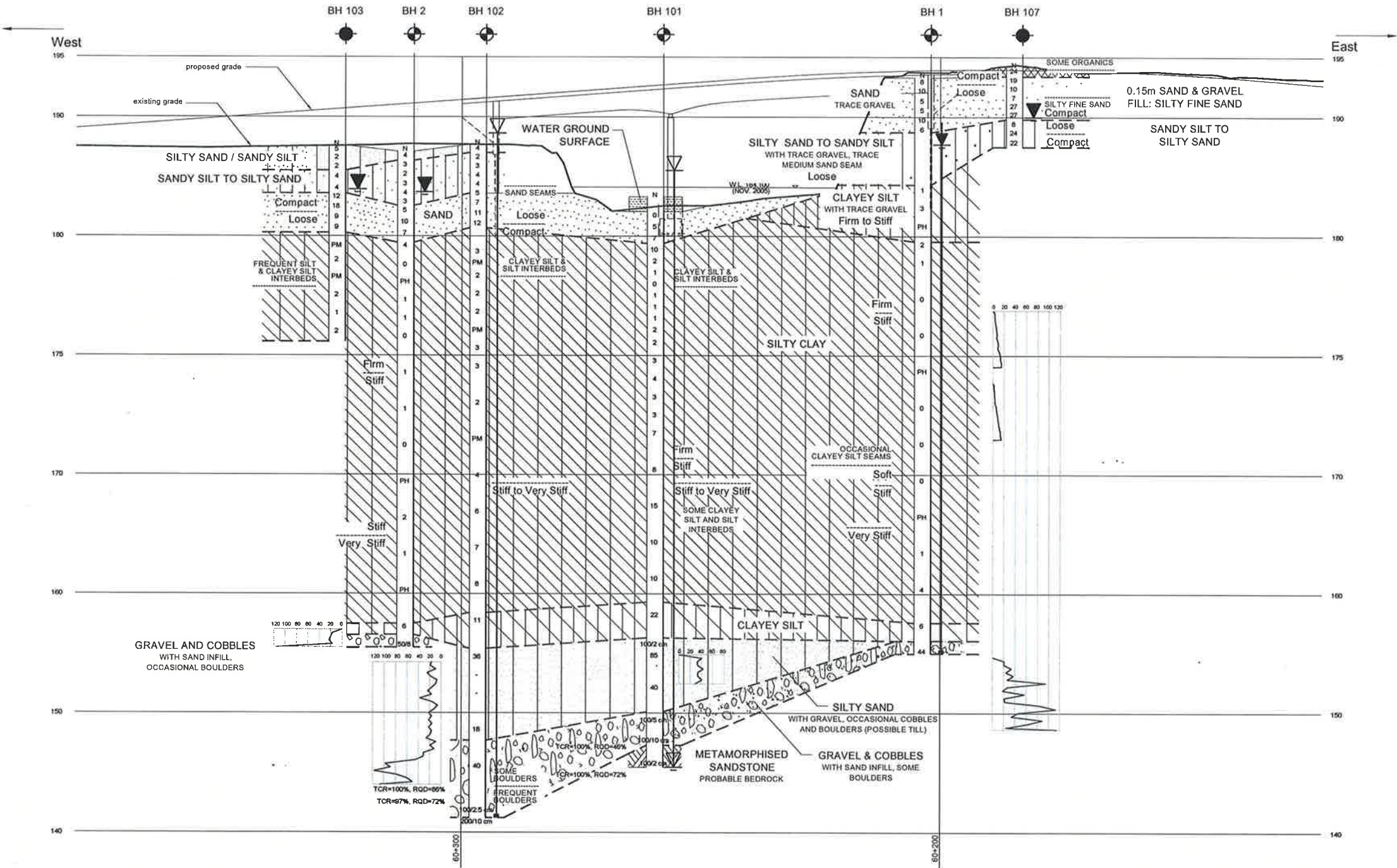
-NOTE-

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-82			
TRANETOB01156AA			
SUBMD	CHECKED	DATE	SITE
DRAWN	PHK	APPROVED	DWG



PROFILE
HORIZONTAL SCALE



METRIC

NOTES:

FOR DETAILED SUBSURFACE CONDITIONS
REFER TO RECORD OF BOREHOLE SHEETS.

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

CONT No.
GWP: 5290-04-00

OLD GOULAIS BAY ROAD
SHEWFELT BRIDGE
BOREHOLE LOCATION PLAN
AND SOIL STRATA



SHEET

coffey geotechnics
SPECIALISTS MANAGING THE EARTH



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole & Cone
- 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
BH 104	187.4m	5175959.3	275478.6
BH 105	187.4m	5175975.0	275461.9
BH 106	187.4m	5176017.2	275442.4

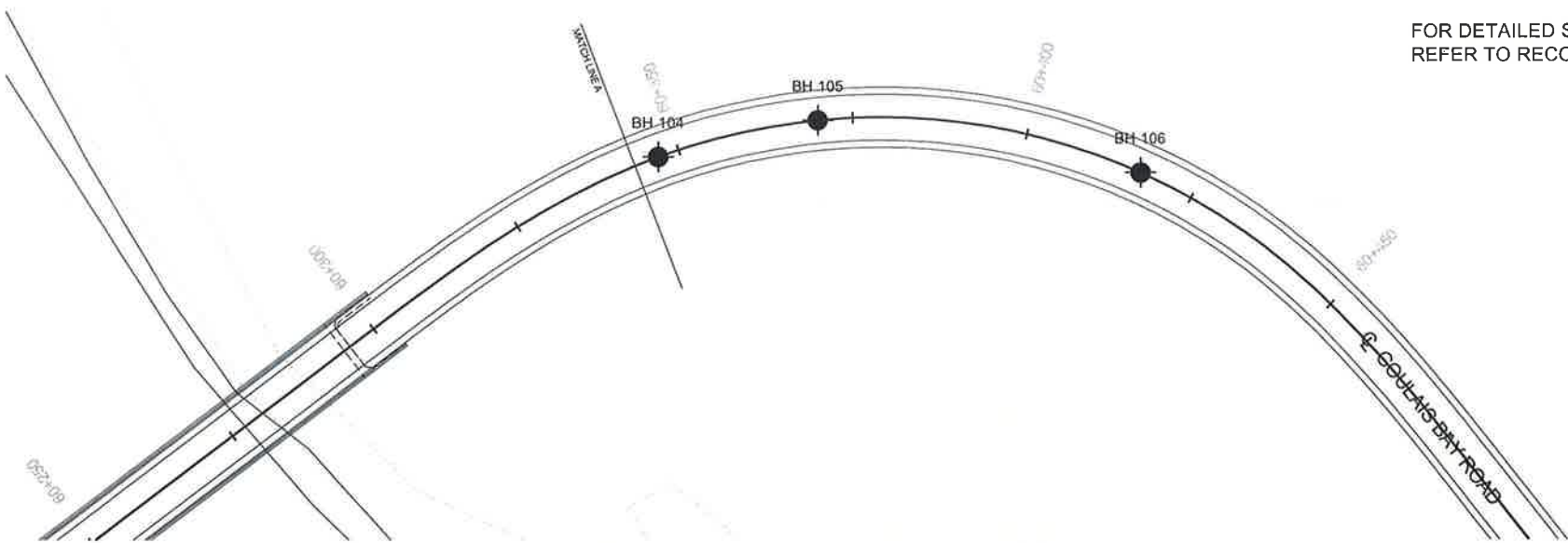
-NOTE-

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

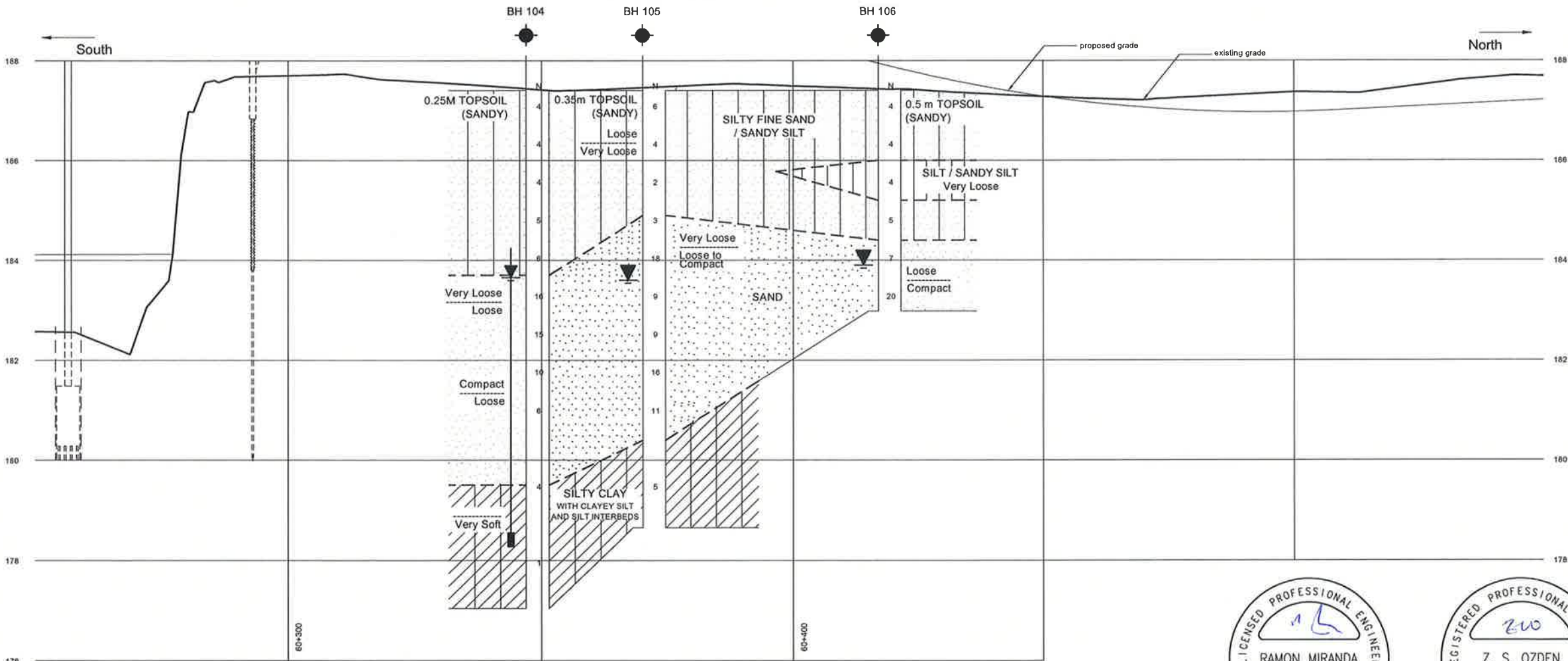
NOTE: This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 41K-82			
TRANETO01156AA			
SUBMD	CHECKED	DATE	SITE
DRAWN	PHK	CHECKED	RM
APPROVED	ZO	DATE	SITE
DWG	3		

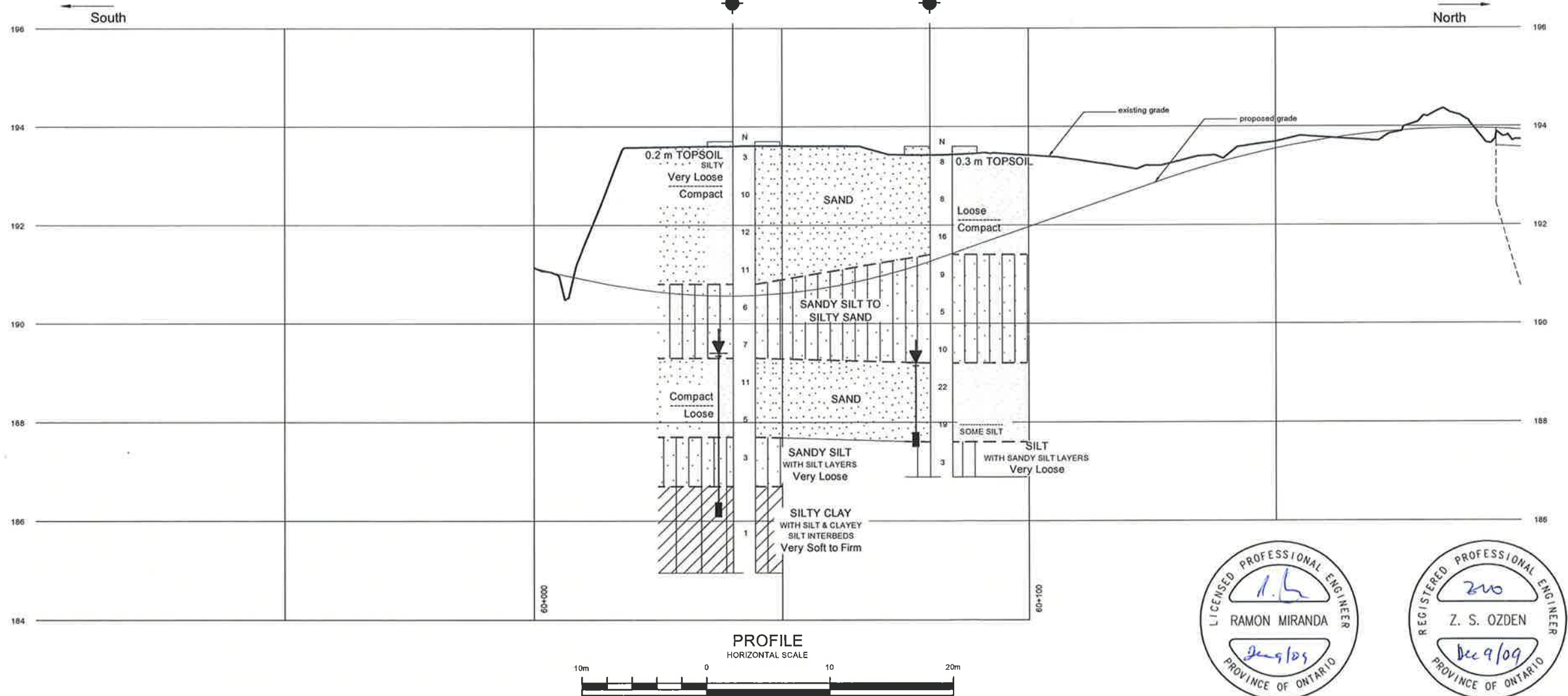
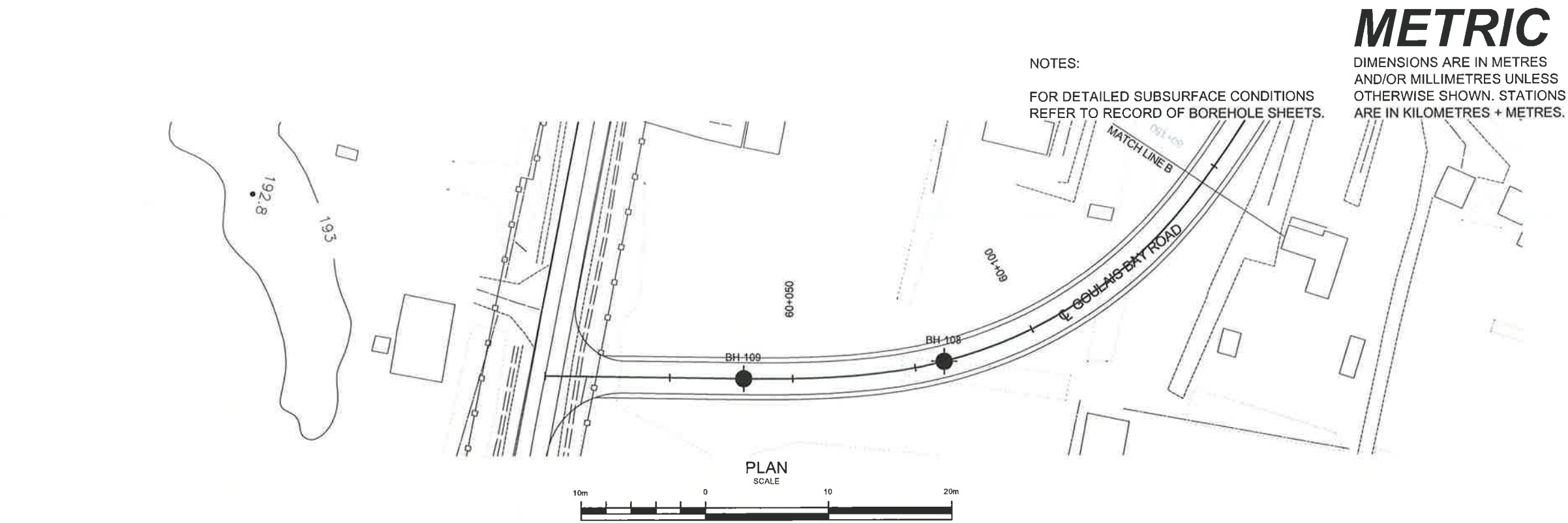


PLAN
SCALE



PROFILE
HORIZONTAL SCALE





CONT No.	SHEET
GWP: 5290-04-00	
OLD GOULAIS BAY ROAD SHEWFELT BRIDGE BOREHOLE LOCATION PLAN AND SOIL STRATA	



KEY PLAN
N.T.S.

LEGEND

- Borehole
- Borehole & Cone
- 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
BH 108	193.6m	5175847.1	275713.9
BH 109	193.7m	5175808.2	275726.4

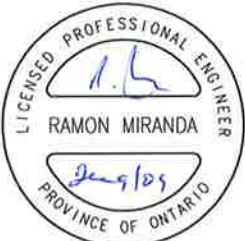
-NOTE-

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-82			
TRANET001156AA			
SUBMD	CHECKED	DATE	DIST
DRAWN	PHK	Dec 8, 2009	38S-031
CHECKED	RM	APPROVED	ZO
DWG	4		



Appendix A

Record of Borehole Sheets

RECORD OF BOREHOLE No BH1

1 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.J.

DIST Algoma HWY 17

BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 2/28/2006

CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				
								○ UNCONFINED ● POCKET PENETR.	+ FIELD VANE x LAB VANE			
							20 40 60 80 100					
193.3 0.0	Ground Surface											
192.7 0.7	250mm TOPSOIL SILTY FINE SAND with trace rootlets & topsoil dark brown, loose, moist (frozen)		1	SS	8							
			2	SS	10							
	SAND with trace gravel, trace topsoil pockets brown, loose, moist		3	SS	5							
			4	SS	5							
			5	SS	10							
			6	SS	6							
188.8 4.6			7	SS	7							
	SILTY SAND to SANDY SILT with trace gravel brown loose, wet		8	SS	6							
	trace medium sand seam		9	SS	7							
184.3 8.0			10	SS	1							
	CLAYEY SILT with trace gravel grey, firm to stiff		11	SS	3							
			12	TW	PH							
179.6 13.7			13	SS	2							
	SILTY CLAY reddish grey, firm											

Continued Next Page

+³, x³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

50mm diameter
Shelby tube
sample
(0) 79 21

RECORD OF BOREHOLE No BH1

2 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewell Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.I.

DIST Algoma HWY 17

BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 2/20/2006

CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N-VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE	WATER CONTENT (%) 20 40 60					
	occ. clayey silt seams		14	SS	1		176							
							177							
							176							
	SILTY CLAY reddish grey, firm to stiff	firm stiff	15	SS	0		175							
							174							
							173							
							172							N-value not reliable
							171							
							170							
							169							
			17	TW	PH		168						16.6	70mm diameter Shelby tube sample (0) 34 66
							167							
							166							N-value not reliable
			18	SS	0		165							
							164							

Continued Next Page

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

SPT1156A

RECORD OF BOREHOLE No BH1

3 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewell Bridge, Goulais River -Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.I.

DIST Algoma HWY 17

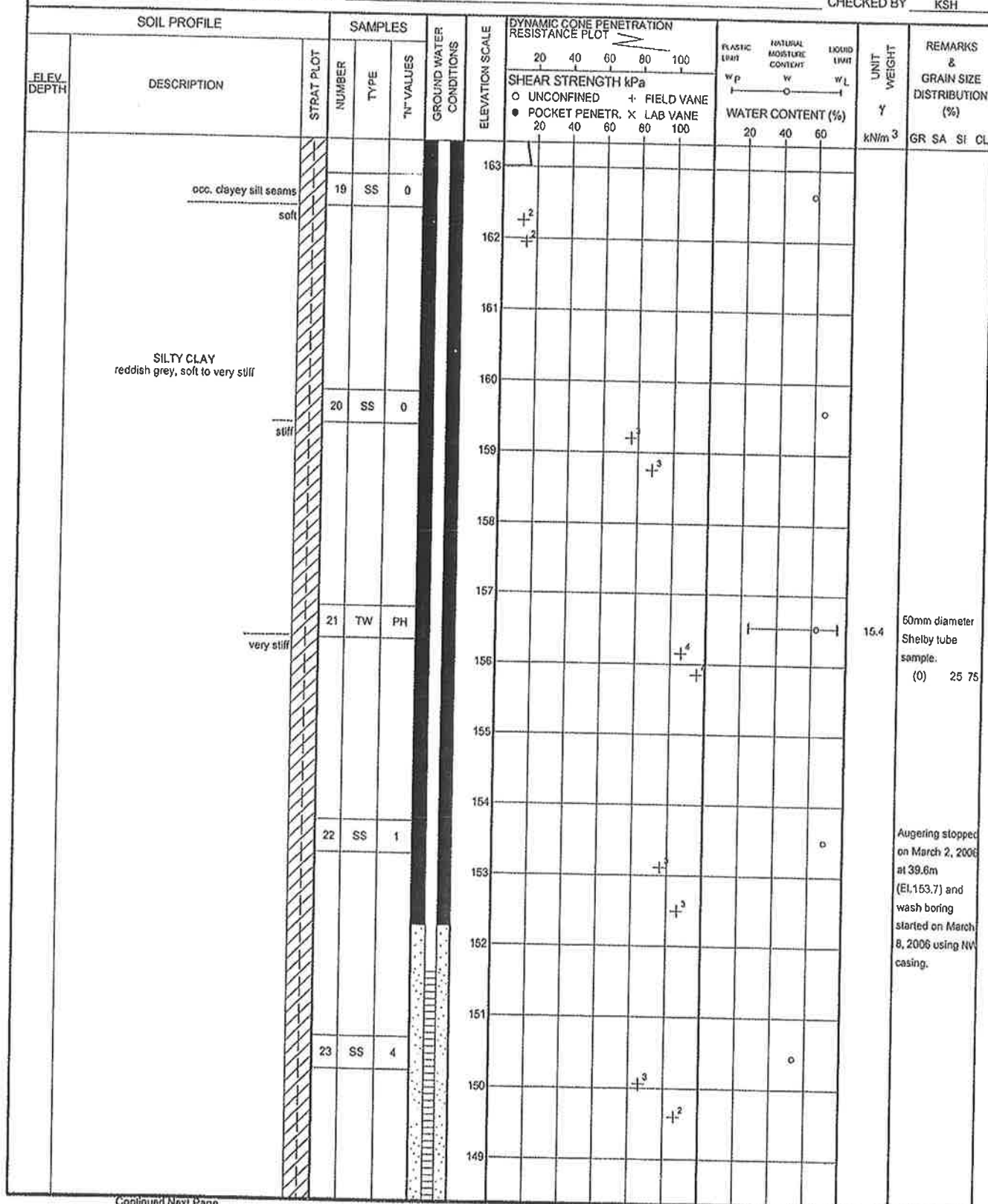
BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.2.

DATUM Geodetic

DATE 2/28/2006

CHECKED BY KSH



Continued Next Page

+³, x³: Numbers refer to Sensitivity 20 15-0.5 10 (%) STRAIN AT FAILURE

SPT1156A

RECORD OF BOREHOLE No BH1

4 OF 4

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 914.4; E 275 624.7

ORIGINATED BY G.I.

DIST Algoma HWY 17

BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 2/28/2006

CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE	WATER CONTENT (%) w _p — w — w _L					
147.6 45.7	CLAYEY SILT grey, very stiff		24	SS	6									
146.1 47.2	GRAVEL & COBBLES with some sand, occasional boulders reddish brown, wet		25	SS	44									
145.0 48.3	End of Borehole. Piezometer Installed to 47.6m (El. 145.5m). Water level on: Mar. 9, 2006=3.3m (El. 190.0m) Mar. 10, 2006=4.5m (El. 188.8m) Mar. 11, 2006=5.8m (El. 187.5m) Mar. 12, 2006=5.6m (El. 187.7m) below ground surface.													
138.4 54.9	End of DCPT. Dynamic Cone Penetration Test (DCPT) performed from: 19.8m (El. 173.5m) to 24.3m (El. 169.0m), 25.9m (El. 167.4m) to 30.5m (El. 162.8m), and 48.5m (El. 144.8m) to 54.9m (El. 139.4m).													

+³, X³: Numbers refer to Sensitivity 20 15-10-5 10 (%) STRAIN AT FAILURE

SPT1156A

RECORD OF BOREHOLE No BH2

1 OF 3

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River - Coords: N 5 175 950.6; E 275 523.1

ORIGINATED BY G.I.

DIST Algoma HWY 17

BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 3/10/2006

CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa								WATER CONTENT (%)
							○ UNCONFINED	+ FIELD VANE	● POCKET PENETR. × LAB VANE						
187.1 0.0	Ground Surface						20	40	60	80	100	20	40	60	
186.3 0.8	250mm TOPSOIL SILTY FINE SAND with trace topsoil & rootlets brown, very loose, moist		1	SS	4										
	SANDY SILT brown, very loose, moist		2	SS	3										
			3	SS	2										
			4	SS	3										
			5	SS	4										
			6	SS	3										
182.6 4.6	SAND with trace gravel brown, loose, wet		7	SS	5										
			8	SS	10										
			9	SS	7										
179.5 7.6	CLAYEY SILT grey, stiff		10	SS	4										
178.0 9.2			11	SS	0										
	SILTY CLAY reddish grey, firm		12	TW	PH										
			13	SS	1										
			14	SS	1										

50 mm diameter
Shelby tube
sample.
2 2 64 32

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity
20
15 10 5 0
(%) STRAIN AT FAILURE

50 mm diameter
Shelby tube
sample.
2 2 64 32

SPT1156A

RECORD OF BOREHOLE No BH2

2 OF 3

METRIC

GWP 5290-04-00

LOCATION Shewfelt Bridge, Goulais River -Coords: N 5 175 950.6; E 275 623.1

ORIGINATED BY G.I.

DIST Algoma HWY 17

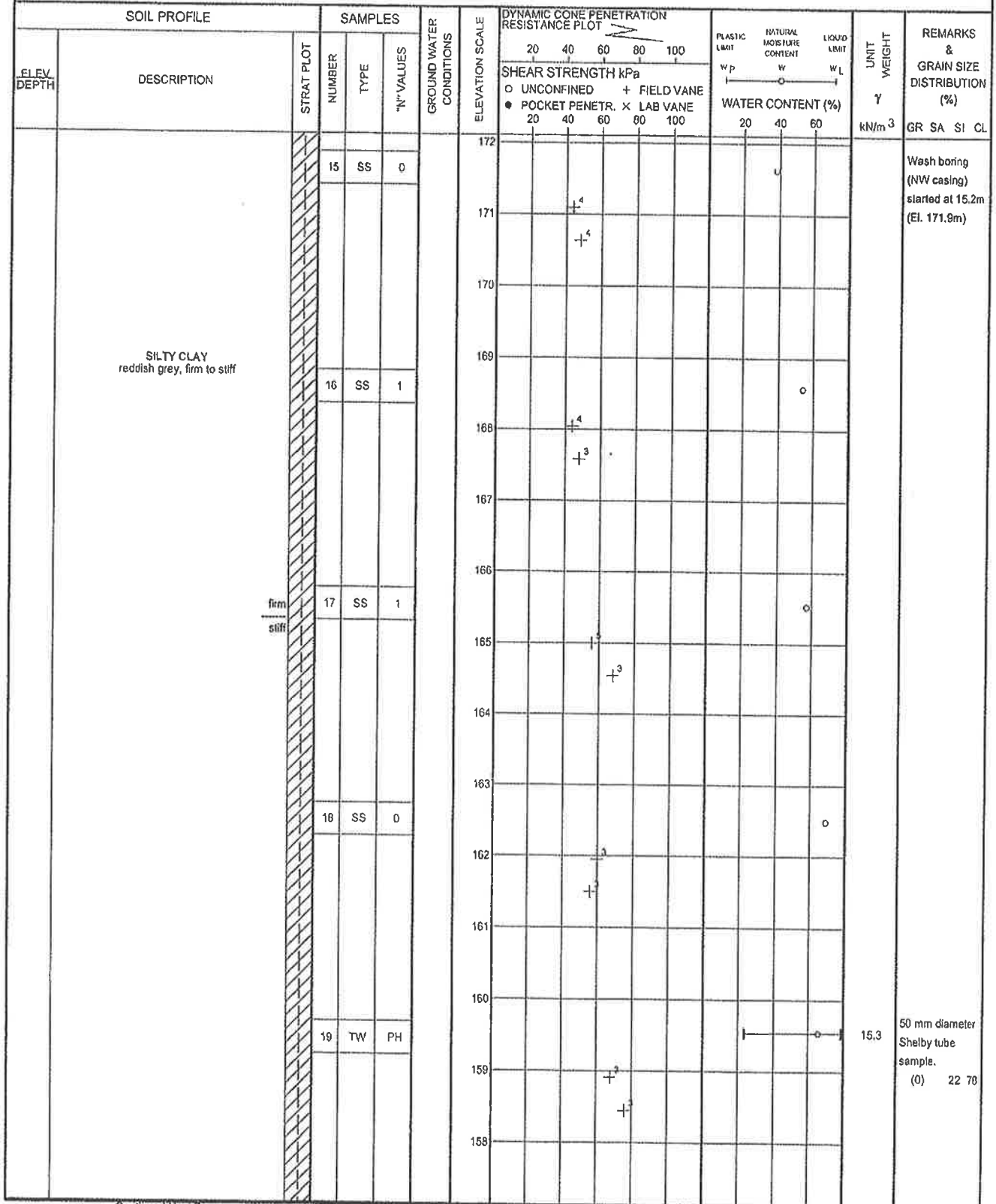
BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT

COMPILED BY J.Z.

DATUM Geodetic

DATE 3/10/2006

CHECKED BY KSH



Continued Next Page

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

SPT1156A

RECORD OF BOREHOLE No BH2

3 OF 3

METRIC

GWP 5290-04-00 LOCATION Shewfelt Bridge, Goulais River --Coords: N 5 175 950.6; E 275 523.1 ORIGINATED BY G.I.
DIST Algoma HWY 17 BOREHOLE TYPE Hollow Stem Augers+Wash Boring+DCPT COMPILED BY J.Z.
DATUM Geodetic DATE 3/10/2006 CHECKED BY KSH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE	WATER CONTENT (%)					
							20 40 60 80 100	20 40 60						
			20	SS	2									
			21	SS	1									
			22	TW	PH									
147.5 39.6	CLAYEY SILT gray, very stiff		23	SS	6									
146.6 40.5	GRAVEL and COBBLES with sand infill, occasional boulders reddish brown, wet													
145.5 41.6	End of borehole.		24	SS	50/8									
	Dynamic Cone Penetration Test carried out from 40.2m (El. 146.9m) to 41.6m (El. 145.6m). * Water level in open borehole: 3.4m (not stabilized).													

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

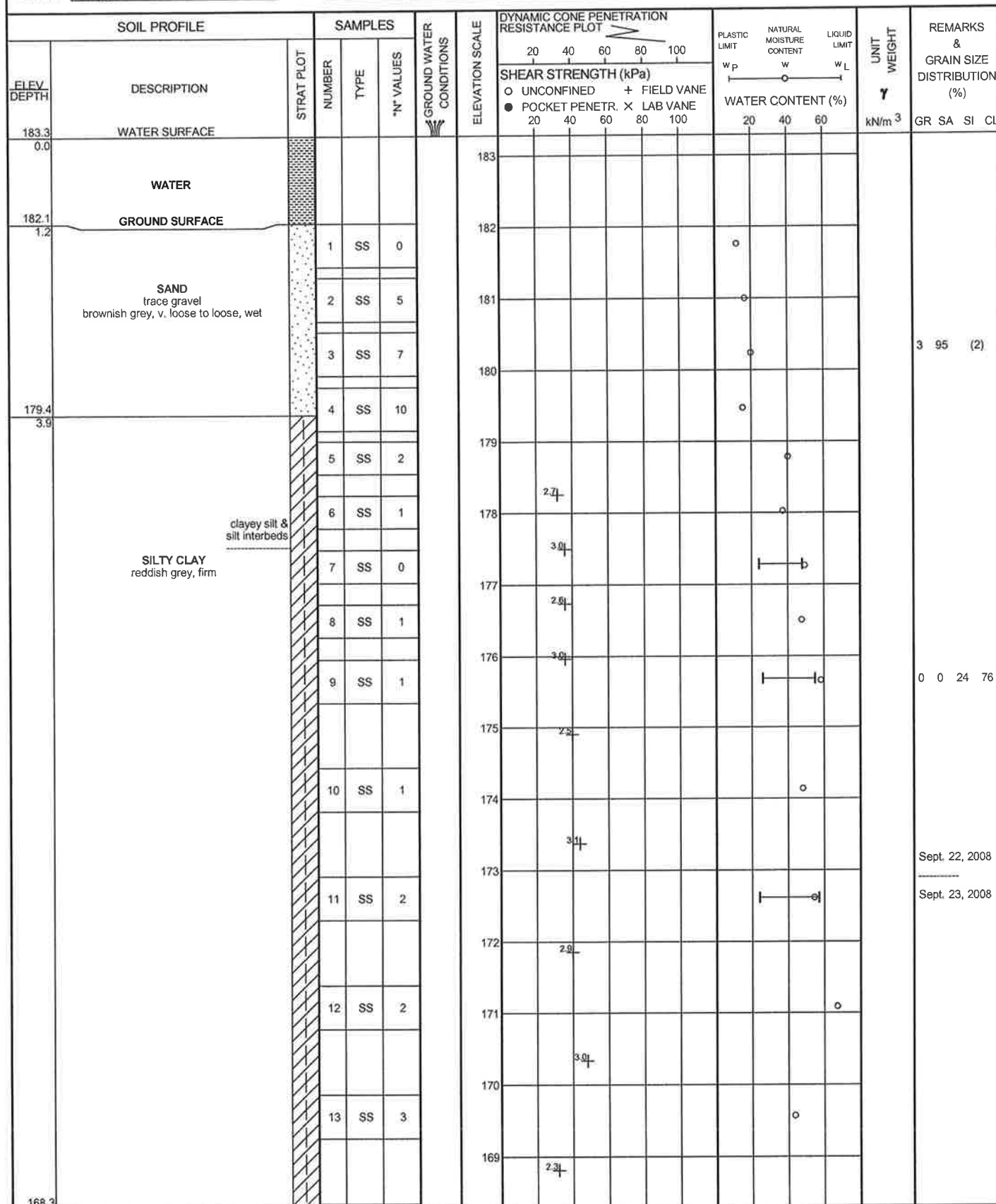
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 101

1 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+258, N: 5175927.45 E: 275561.25 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY SS
DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO



Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

2 OF 4

METRIC

TEST RESULTS	COMMENTS		DYNAMIC CONE PENETRATION		
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Continued Next Page

(%) STRAIN AT FAILURE




SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 101

3 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+258, N: 5175927.45 E: 275561.25 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY SS
DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)										
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							WATER CONTENT (%)									
153.3 30.0	SILTY CLAY reddish grey stiff to v. stiff		21	SS	10	153	20	40	60	80	100	20	40	60	GR SA SI CL									
149.3 34.0			22	SS	22											152	20	40	60	80	100	20	40	60
146.3 37.0	SILTY SAND with gravel, occ. cobbles and boulders (possible till) reddish grey, compact to v. dense, wet		23	SS	100/2 cm	151	20	40	60	80	100	20	40	60	GR SA SI CL									
143.3 40.0			24	RC												150	20	40	60	80	100	20	40	60
140.1 43.2			25	SS	85																			
138.3	GRAVEL AND COBBLES with sand infill, some boulders reddish brown, wet		26	SS	40	148	20	40	60	80	100	20	40	60	GR SA SI CL									
			27	SS	100/5 cm											147	20	40	60	80	100	20	40	60
		28	RC		146	20	40	60	80	100	20	40	60											

Continued Next Page

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

Dynamic Cone Penetration Test (DCPT) performed from 38.4 to 40.8 m when Casing is @ 37.8 m.

DCPT performed from 41.5 m to 43.9 m

Oct. 7, 2008

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 101

4 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+258, N: 5175927.45 E: 275561.25 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE NW Casing and Wash Boring COMPILED BY SS
DATUM Geodetic DATE 22/09/2008 11/10/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE			WATER CONTENT (%) w _p w w _L				
138.3 45.0	GRAVEL AND COBBLES with sand infill, some boulders reddish brown, wet					138								GR SA SI CL Artesian condition @ El. 138.1 m 3.3 m above ground (to El. 185.4 m)	
137.3 46.0			29	SS	100/10 cm										
137.0 46.3	METAMORPHISED SANDSTONE (possible bedrock) reddish brown		30	RCT	CR=100%		137								
	METAMORPHISED SANDSTONE (probable bedrock) reddish brown	31	SS	RQD=48% 100/2 cm											
			32	RCT	CR=100% RQD=72%	136									
135.4 47.9	End of Borehole.													Artesian condition @ bottom of borehole 5.5 m above ground (to El. 187.6 m)	

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

1 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+295, N:5175938.23 275525.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & Rock Coring COMPILED BY SS
DATUM Geodetic DATE 03/09/2008 10/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
187.7	GROUND SURFACE													
0.0	SILTY FINE SAND organic stained to 0.4 m brown, v. loose, moist		1	SS	4									
187.0			2	SS	2									
0.7	SANDY SILT TO SILTY SAND brown, v. loose, moist		3	SS	3									0 33 54 13
			4	SS	4									
			5	SS	4									
183.6		sand seams	6	SS	5									
4.1	SAND trace gravel wet		7	SS	7									sampler wet @ 4.5 m
		brown loose	8	SS	11									17 80 (3)
		compact grey	9	SS	12									
180.7			10	SS	3									
7.0			11	TW	PM									H/S augering
	SILTY CLAY reddish grey, firm		12	SS	2								18.3	NW casing wash boring consolidation test
		clayey silt & silt interbeds	13	SS	2									0 1 32 67
			14	SS	2									
172.7														

Continued Next Page

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

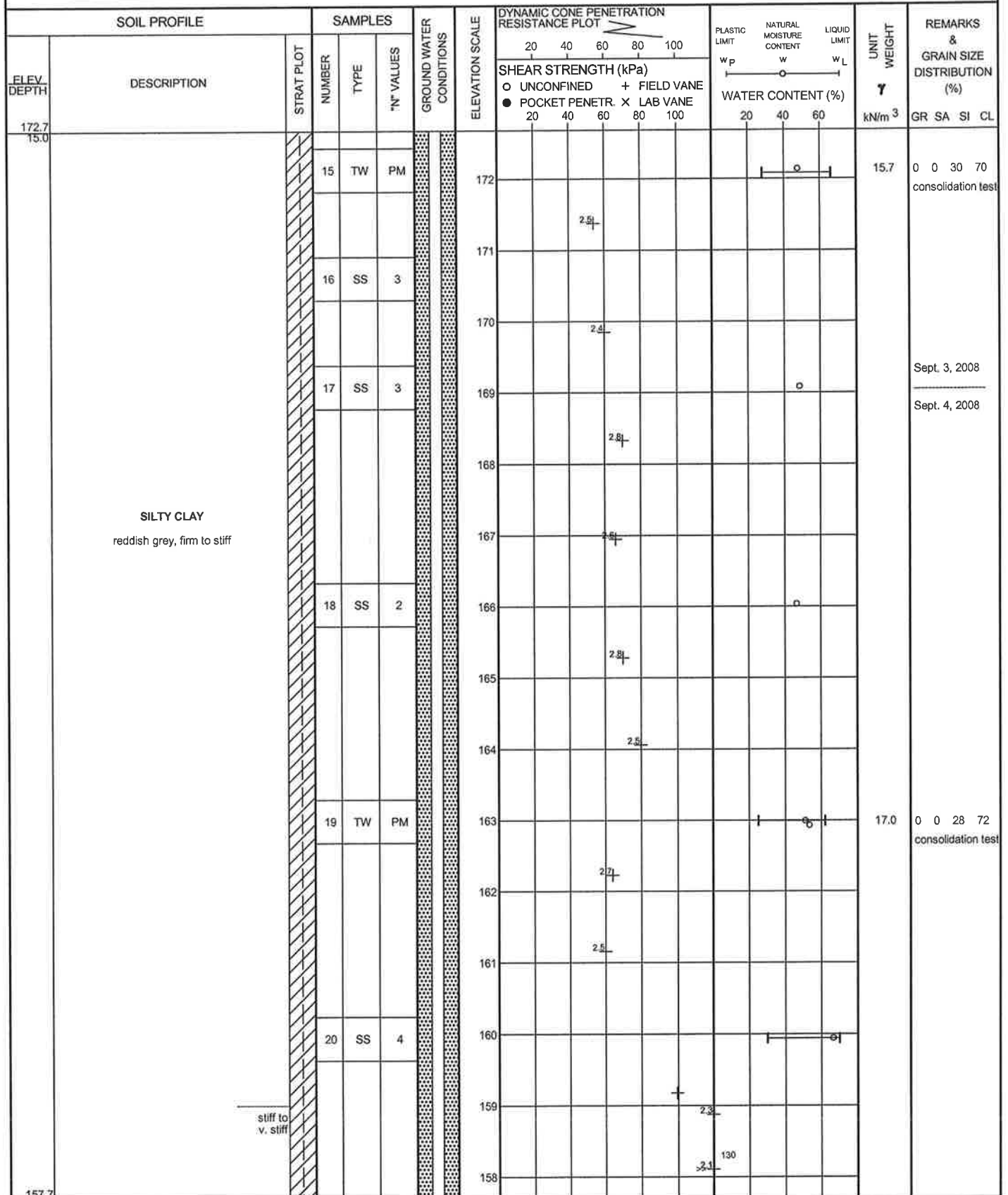
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

2 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+295, N:5175938.23 275525.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & Rock Coring COMPILED BY SS
DATUM Geodetic DATE 03/09/2008 10/09/2008 CHECKED BY ZO



Continued Next Page

+ 3, X 3; Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE



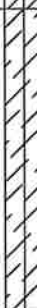

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

3 OF 4

METRIC

GWP	5290-04-00	LOCATION	Sta. 60+295, N:5175938.23 275525.92	ORIGINATED BY	GI
DIST	HWY 17	BOREHOLE TYPE	Hollow Stem Auger, Wash Boring & Rock Coring	COMPILED BY	SS
DATUM	Geodetic	DATE	03/09/2008 10/09/2008	CHECKED BY	ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60						80	100	SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE		
157.7 30.0	SILTY CLAY reddish grey stiff to v. stiff		21	SS	6		157													
			22	SS	7		154													
							153													
							152													
			23	SS	8		151													
							150													
							149													
148.4 39.3	CLAYEY SILT with silt seams grey, stiff to v. stiff wet, dilatant		24	SS	11		148													
							147													
							146													
145.4 42.3	SILTY SAND with gravel occ. cobbles and boulders (possible till) reddish grey, compact to dense, wet		25	SS	36		145													
					26	SS	-		144											
142.7							143													

1.5m sand
back-up in casing
Sept. 4, 2008
34 44 16 6

Sept. 5, 2008
Dynamic Cone
Penetration Test
(DCPT) performed
from 43.3m to
54.1m.
40 38 16 6

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

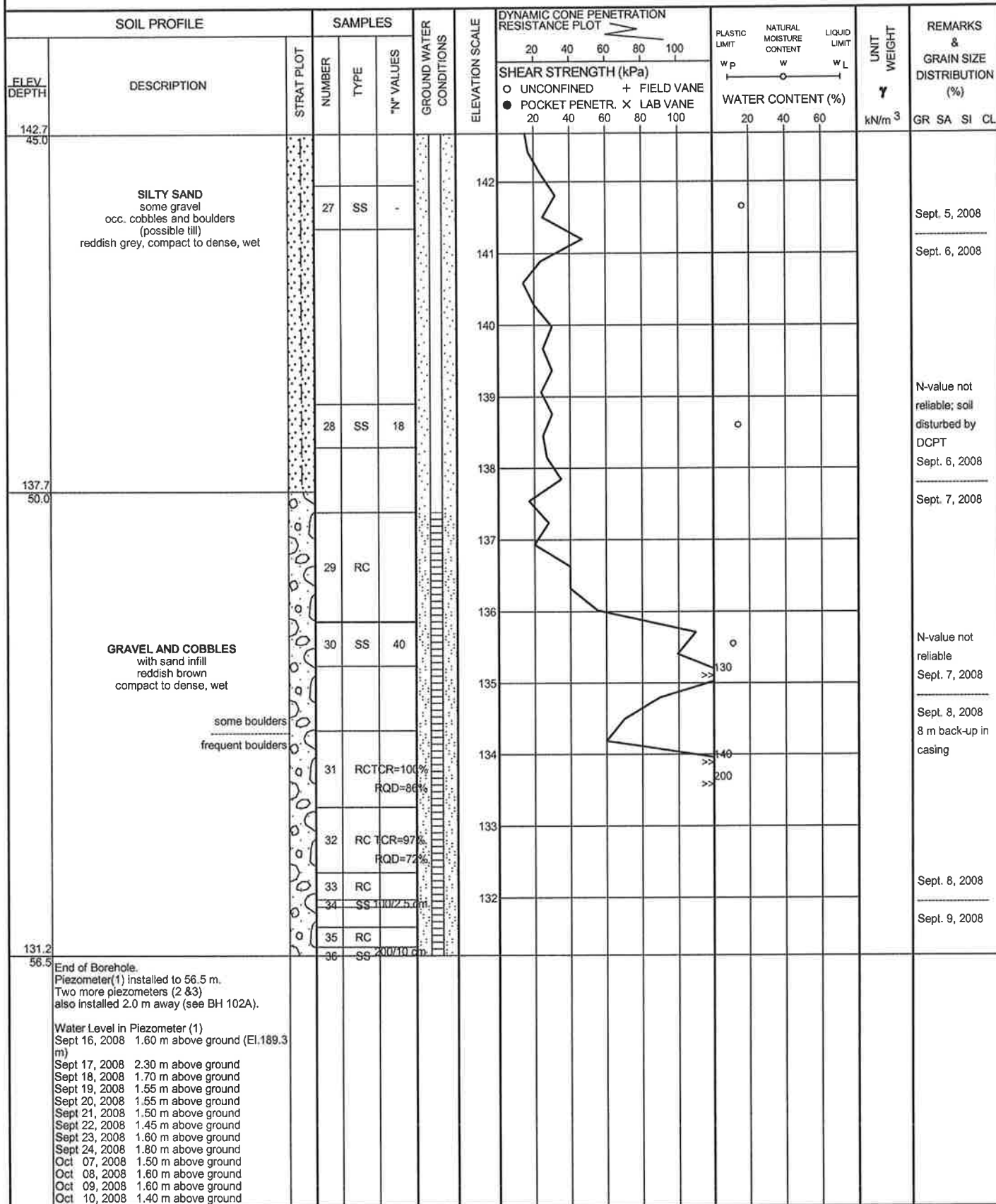
SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102

4 OF 4

METRIC

GWP 5290-04-00 LOCATION Sta. 60+295, N:5175938.23 275525.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger, Wash Boring & Rock Coring COMPILED BY SS
DATUM Geodetic DATE 03/09/2008 10/09/2008 CHECKED BY ZO



+³, X³: Numbers refer to Sensitivity

20
15 10
10 (%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102A

1 OF 2

METRIC

GWP 5290-04-00

LOCATION

Sta. 60+295 : 2m away from BH 102

ORIGINATED BY GI

DIST _____ HWY 17

BOREHOLE TYPE

Hollow Stem Auger

COMPILED BY SS

DATUM Geodetic

DATE _____

9/3/2008

9/10/2008

CHECKED BY ZO

[illegible]

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 102A

2 OF 2

METRIC

GWP 5290-04-00 LOCATION Sta. 60+285 : 2m away from BH 102 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 9/3/2008 9/10/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
172.7	Oct 08, 2008 4.20 m Oct 09, 2008 4.20 m Oct 10, 2008 4.20 m																

+³ ×³ : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 103

1 OF 2

METRIC

GWP 5290-04-00 LOCATION Sta. 60+324, N:5175948.19 E:275498.72 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 17/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)			
187.6 0.0	GROUND SURFACE							20 40 60 80 100			
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE			
								20 40 60 80 100			
								w _p w w _L			
								WATER CONTENT (%)			
185.5 2.1	0.3 m TOPSOIL (Sandy) SILTY SAND/SANDY SILT some topsoil inclusions (previously disturbed) brown, v.loose, moist		1	SS	5		187				
			2	SS	2		186				
			3	SS	2		185				
183.6 4.0	SANDY SILT/SILTY SAND brown, v.loose, moist upto 3.3 m, wet below		4	SS	4		184				0 37 51 12
			5	SS	4		183				
	SAND wet		6	SS	12		182				
		brown compact	7	SS	18		181				10 87 (3)
		grey loose	8	SS	9		180				
			9	SS	9		179				
180.3 7.3			10	TW	PM		178				
			11	SS	2		177				
	SILTY CLAY reddish grey, soft to firm		12	TW	PM		176				
		frequent silt & clayey silt interbeds	13	SS	2		175				
			14	SS	1		174				
172.6							173				consolidation test

Continued Next Page

+³ . X³ : Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

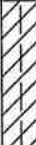

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 103

2 OF 2

METRIC

GWP 5290-04-00 LOCATION Sta. 60+324, N:5175948.19 E:275498.72 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 17/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	*N* VALUES			SHEAR STRENGTH (kPa)						
172.6 15.0	SILTY CLAY reddish grey, firm to stiff		15	SS	2	172	20 40 60 80 100		20 40 60					
171.2 16.5							20 40 60 80 100		20 40 60					
End of Borehole. Water level @ 3.6 m upon completion.														

+³, X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 104

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+347, N:5175959.35 E:275478.64 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 17/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
187.4 0.0	GROUND SURFACE							20 40 60 80 100		20 40 60					
	0.25 m TOPSOIL (Sandy)		1	SS	4		187								
	SILTY FINE SAND/SANDY SILT brown, moist		2	SS	4										
			3	SS	4		186								
			4	SS	5		185								
		v.loose	5	SS	6		184								
	wet below 3.3 m	loose	6	SS	16		183								
183.7 3.7			7	SS	15		182								
	SAND wet		8	SS	10		181								
		brown compact	9	SS	6		180								
		loose grey	10	SS	4		179								
		v.loose	11	SS	1		178								
179.5 7.9															
	SILTY CLAY with silt and frequent clayey silt interbeds reddish grey, soft to firm														
177.0 10.4															
	End of Borehole: Piezometer installed to 9.1 m Water Level in Piezometer Sept 18, 2008 6.2 m Sept 19, 2008 5.5 m Sept 20, 2008 4.9 m Sept 21, 2008 4.3 m Sept 22, 2008 4.1 m Sept 23, 2008 3.8 m Sept 24, 2008 3.8 m Oct 07, 2008 3.8 m Oct 08, 2008 3.8 m Oct 09, 2008 3.75 m Oct 10, 2008 3.75 m														

+ 3, X 3: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 105

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+370, N:5175974.99 E:275461.86 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 18/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. x LAB VANE							W _p	W	W _L
							20	40	60	80	100	20	40	60			
187.4 0.0	GROUND SURFACE																
	0.35 m TOPSOIL(Sandy)	loose	1	SS	6												
		v. loose	2	SS	4												
	SILTY FINE SAND/SANDY SILT brown, damp to moist		3	SS	2											0 49 37 14	
184.9 2.5			4	SS	3												
		v. loose	5	SS	18												
	SAND	loose to compact	6	SS	9												
	moist to 3.5 m, wet below		7	SS	9											7 88 (5)	
		brown	8	SS	16												
		grey	9	SS	11												
180.4 7.0			10	SS	5												
	SILTY CLAY with clayey silt and silt interbeds reddish grey, firm																
178.6 8.8	End of Borehole. Water level @ 3.8 m upon completion (not stabilized)*																

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 106

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+417, N:5176017.18 E:275442.41 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY SS
DATUM Geodetic DATE 18/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE												
187.4	GROUND SURFACE						20	40	60	80	100									
0.0	0.5 m TOPSOIL (Sandy)		1	SS	4		187													
	SILTY FINE SAND brown, v.loose, damp		2	SS	4		186													
186.0																				
1.4	SILT/SANDY SILT brown, v.loose, moist		3	SS	4		185													
185.2			4	SS	5		184													
2.2	SILTY FINE SAND brown, loose, moist																			
184.4			5	SS	7		184									10 87 (3)				
3.0	SAND moist to 3.6 m wet below		6	SS	20		183													
183.0																				
4.4	End of Borehole. Water level @ 3.5 m and hole caved @ 3.5 m upon completion (not stabilized)*																			

+³.X³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 107

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+188, N:5175904.27 E:275627.39 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Solid Stem Auger COMPILED BY SS
DATUM Geodetic DATE 20/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								
194.2	GROUND SURFACE							20	40	60	80	100				
0.0	0.15 m SAND & GRAVEL some organics FILL: SILTY FINE SAND brown, compact moist		1	SS	24		194									
193.4																
0.8			2	SS	19		193									
	SAND brown, damp to moist															
	compact		3	SS	10		192									9 89 (2)
	loose															
			4	SS	7		191									
	silty fine sand compact															
			5	SS	27		190									
			6	SS	27		189									1 95 (4)
189.8																
4.4	SANDY SILT TO SILTY SAND wet		7	SS	8		188									
	loose															
	compact brown		8	SS	24											
	grey															
			9	SS	22											
187.5																
6.7	End of Borehole. Water level @ 4.2 m upon completion (not stabilized)*															

+³ . X³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 108

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+080, N:5175847.1 E:275713.92 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 21/09/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								
193.6	GROUND SURFACE							20	40	60	80	100				
0.0	0.3 m TOPSOIL		1	SS	8		193									
	SAND		2	SS	8											
	brown, dry to moist															
	loose															
	compact		3	SS	16		192									
191.4																
2.2	SANDY SILT TO SILTY SAND		4	SS	9		191									
	loose, wet															
	brown		5	SS	5											
	grey						190									
			6	SS	10											
189.2																
4.4	SAND		7	SS	22		189									
	grey, compact															
	wet															
	some silt		8	SS	19		188									
187.6																
6.0	SILT with sandy silt layers															
	grey, v.loose															
	wet, dilatant		9	SS	3											
186.9																
6.7	End of Borehole. Water level @ 4.5 m on completion. Piezometer installed to 6.0 m Water Level in Piezometer Sept 22, 2008 4.4 m Sept 23, 2008 4.4 m Sept 24, 2008 4.4 m Oct 07, 2008 4.4 m Oct 08, 2008 4.4 m Oct 09, 2008 4.4 m Oct 10, 2008 4.4 m															

+ 3, X 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT 1156: Shewfelt Bridge

RECORD OF BOREHOLE No BH 109

1 OF 1

METRIC

GWP 5290-04-00 LOCATION Sta. 60+040, N:5175808.21 E:275726.41 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 21/09/2008 CHECKED BY ZO

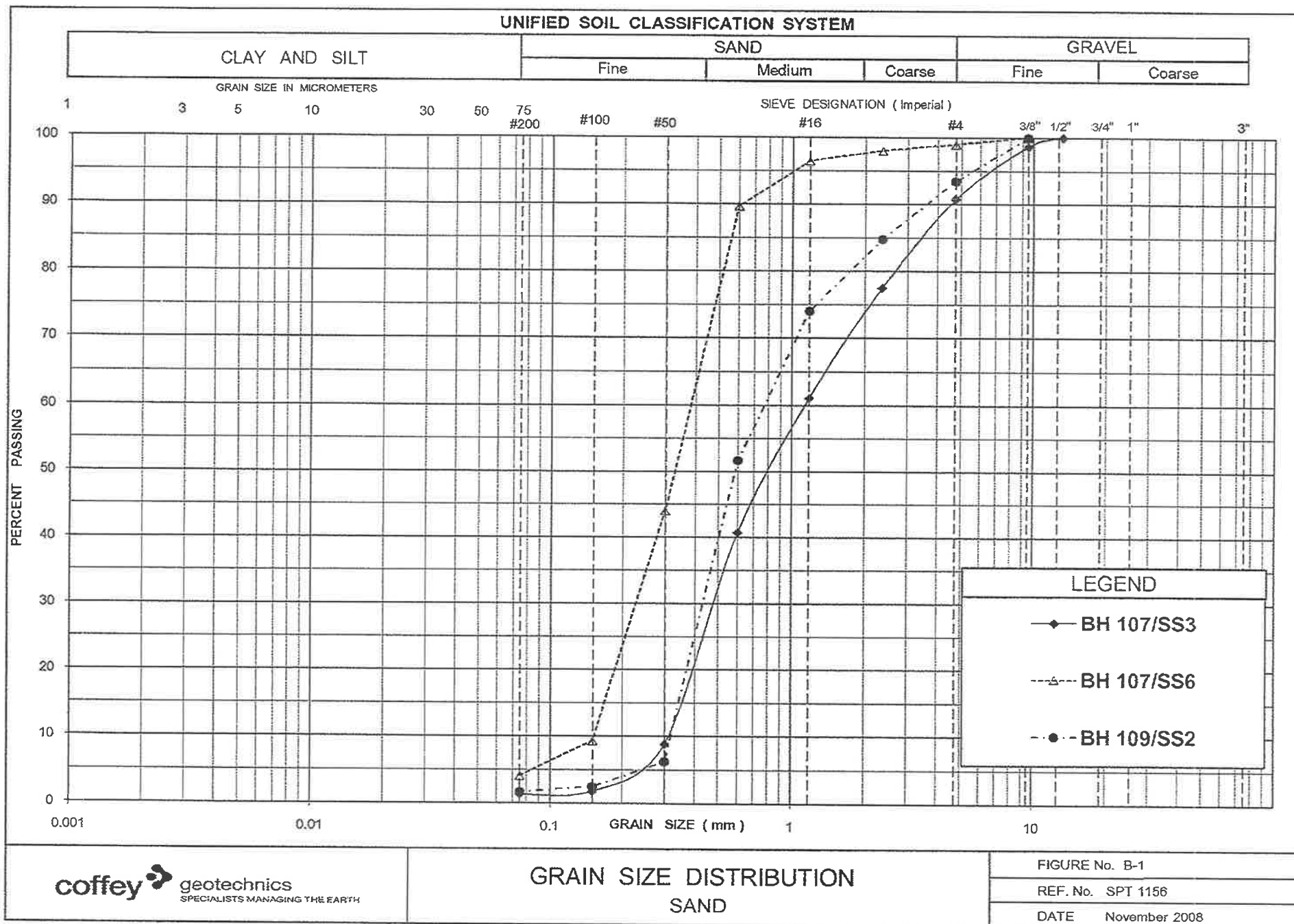
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)							
								20 40 60 80 100	○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE						
193.7	GROUND SURFACE														
0.0	0.2 m TOPSOIL 0.15 m SANDY SILT org. stained	v. loose silty	1	SS	3		193								
		compact	2	SS	10										7 91 (2)
	SAND brown, compact, damp to moist		3	SS	12		192								
			4	SS	11		191								
190.8			5	SS	6		190								
2.9	SANDY SILT with silt layers grey, loose, wet, dilatant		6	SS	7										
189.3			7	SS	11		189								0 95 (5)
4.4	SAND brown, wet	compact loose	8	SS	5		188								
187.7			9	SS	3										
6.0	SANDY SILT with silt layers grey, v. loose, wet, dilatant						187								
186.7															
7.0	SILTY CLAY with silt & clayey silt interbeds reddish grey (clay), grey (silt) very soft to firm		10	SS	1		186								
184.9							185								
8.8	End of Borehole. Piezometer installed to 7.6 m Water Level in Piezometer Sept 22, 2008 6.2 m Sept 23, 2008 5.4 m Sept 24, 2008 4.8 m Oct 07, 2008 4.3 m Oct 08, 2008 4.3 m Oct 09, 2008 4.3 m Oct 10, 2008 4.3 m														

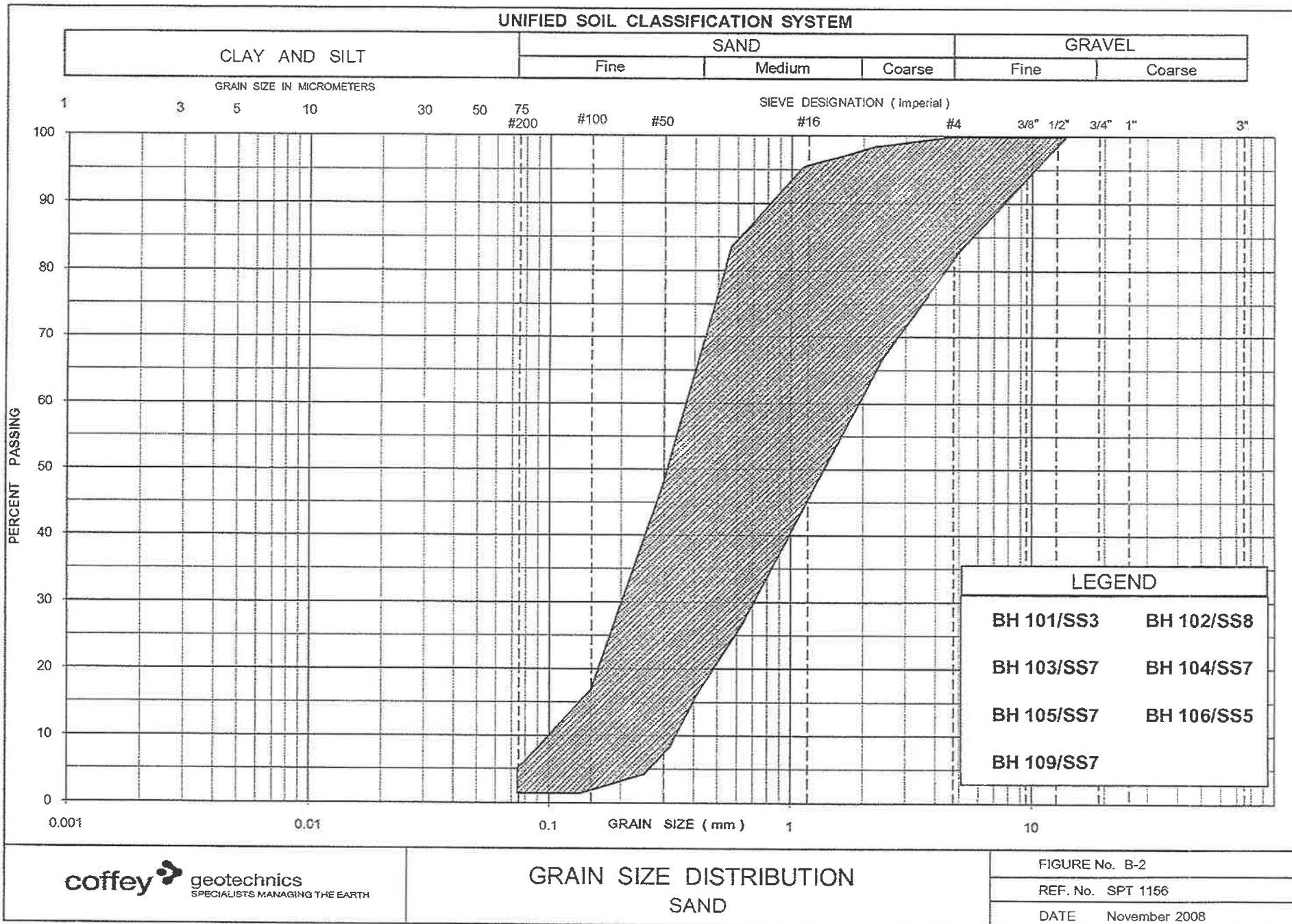
+³, ×³: Numbers refer to
Sensitivity

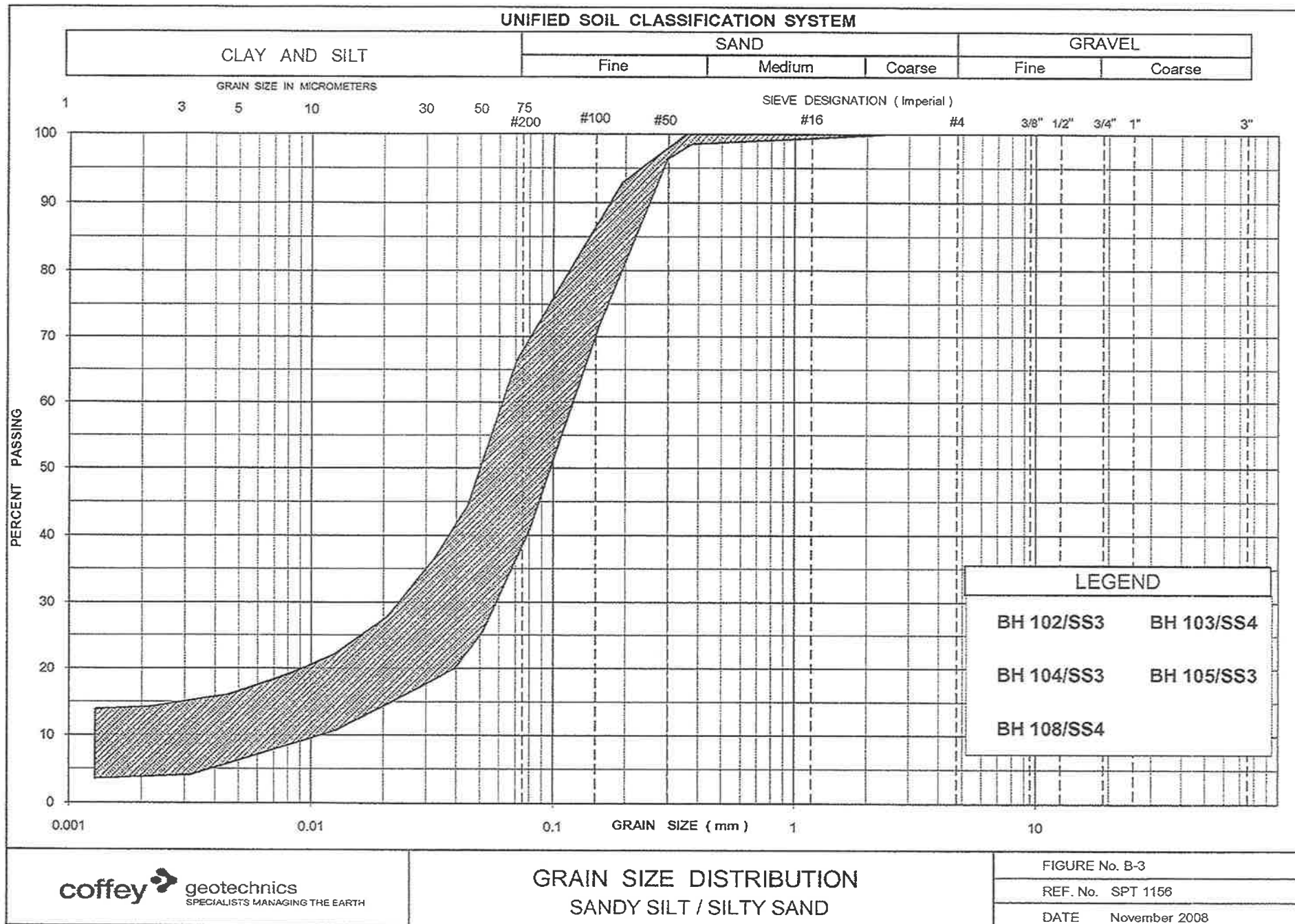
20
15 10 5
(%) STRAIN AT FAILURE

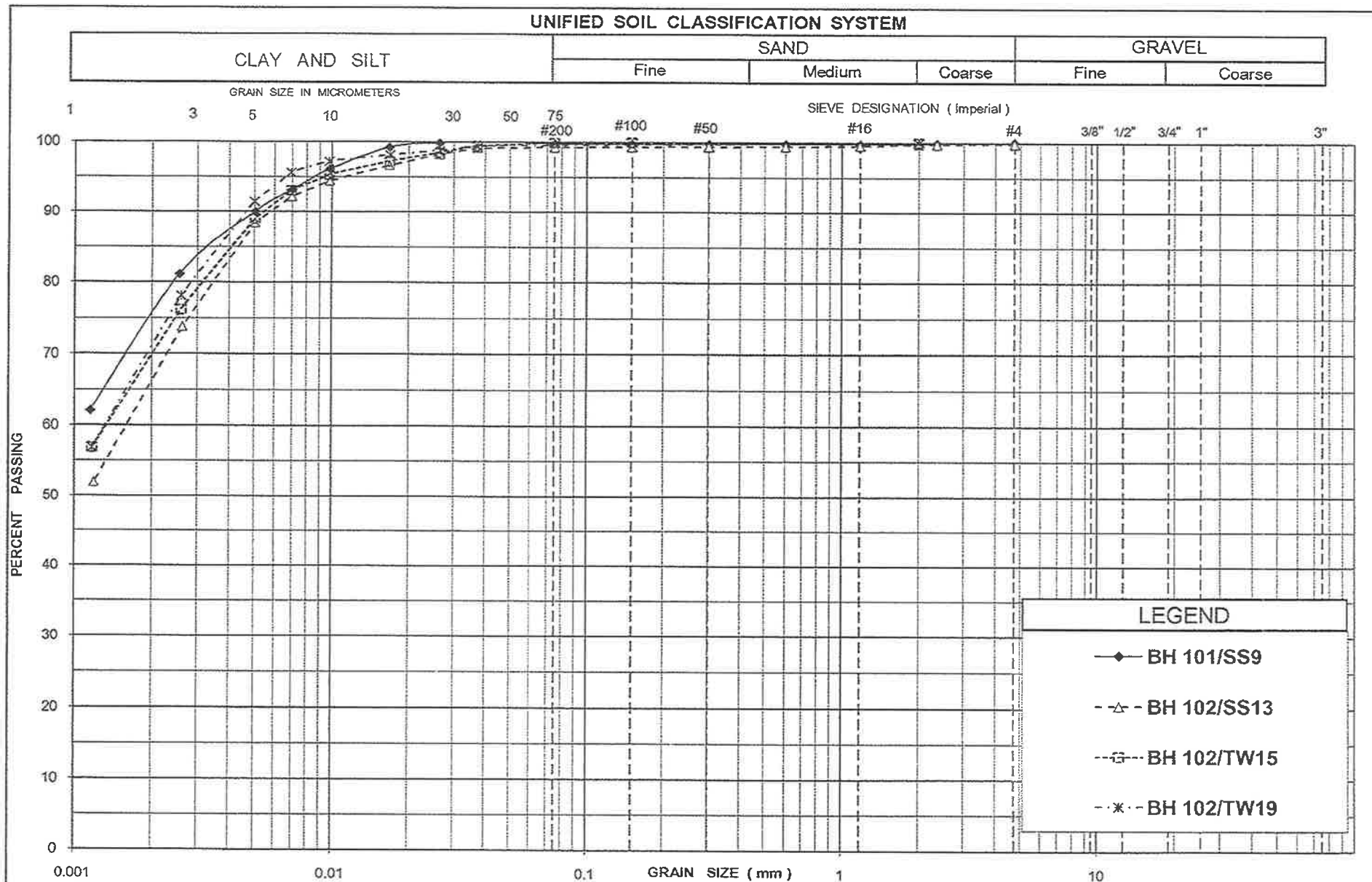
Appendix B

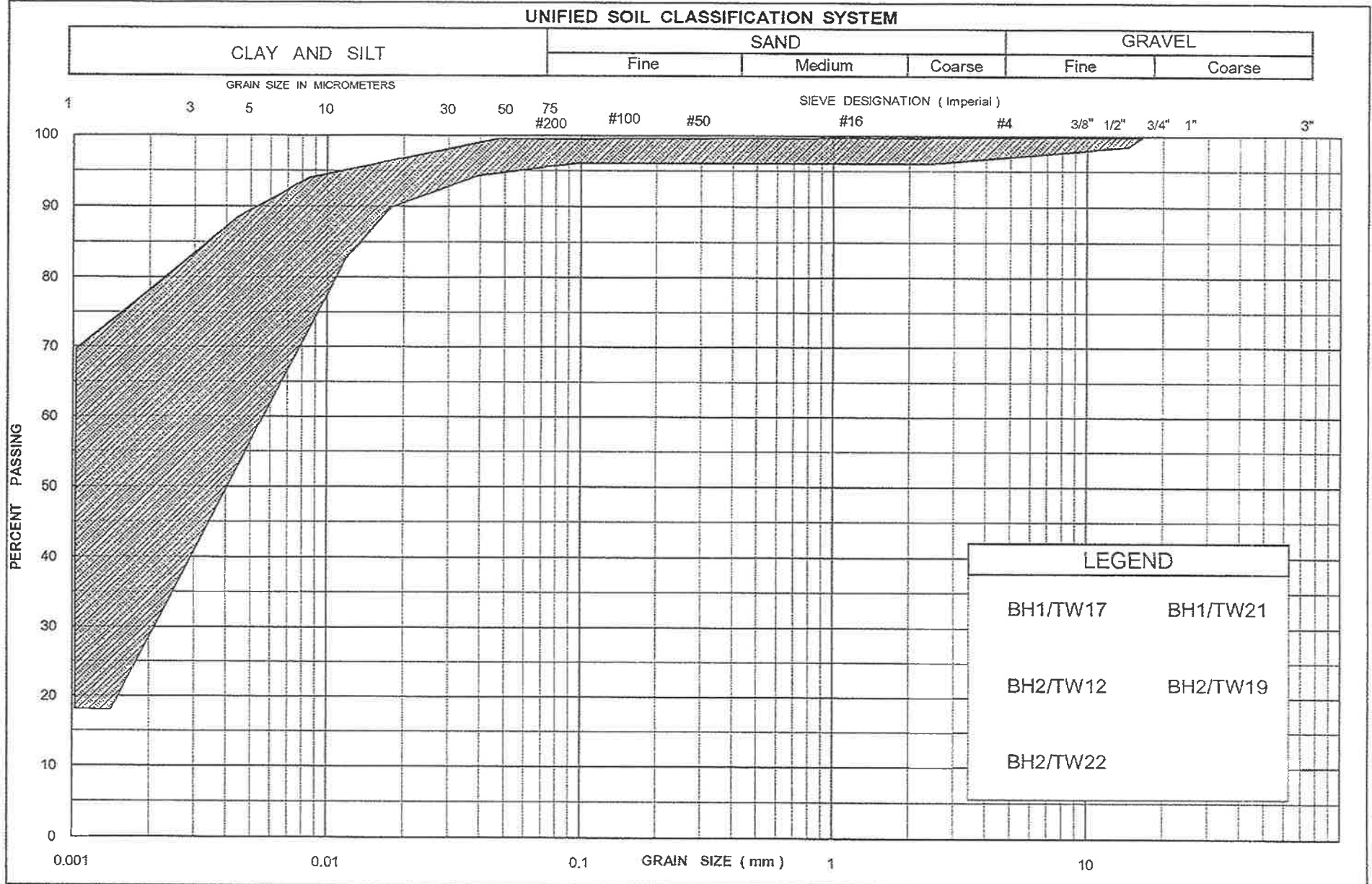
Laboratory Test Results

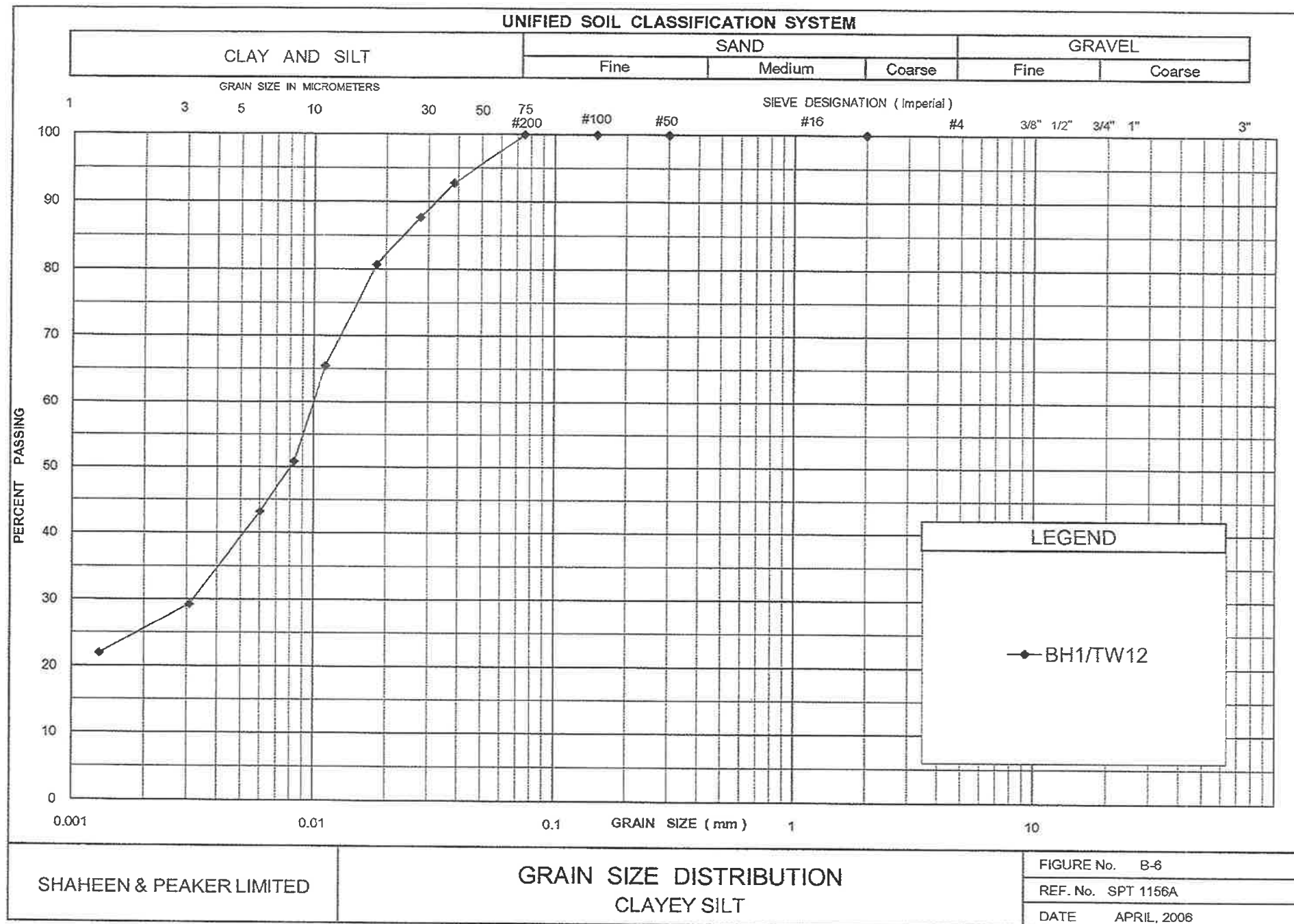


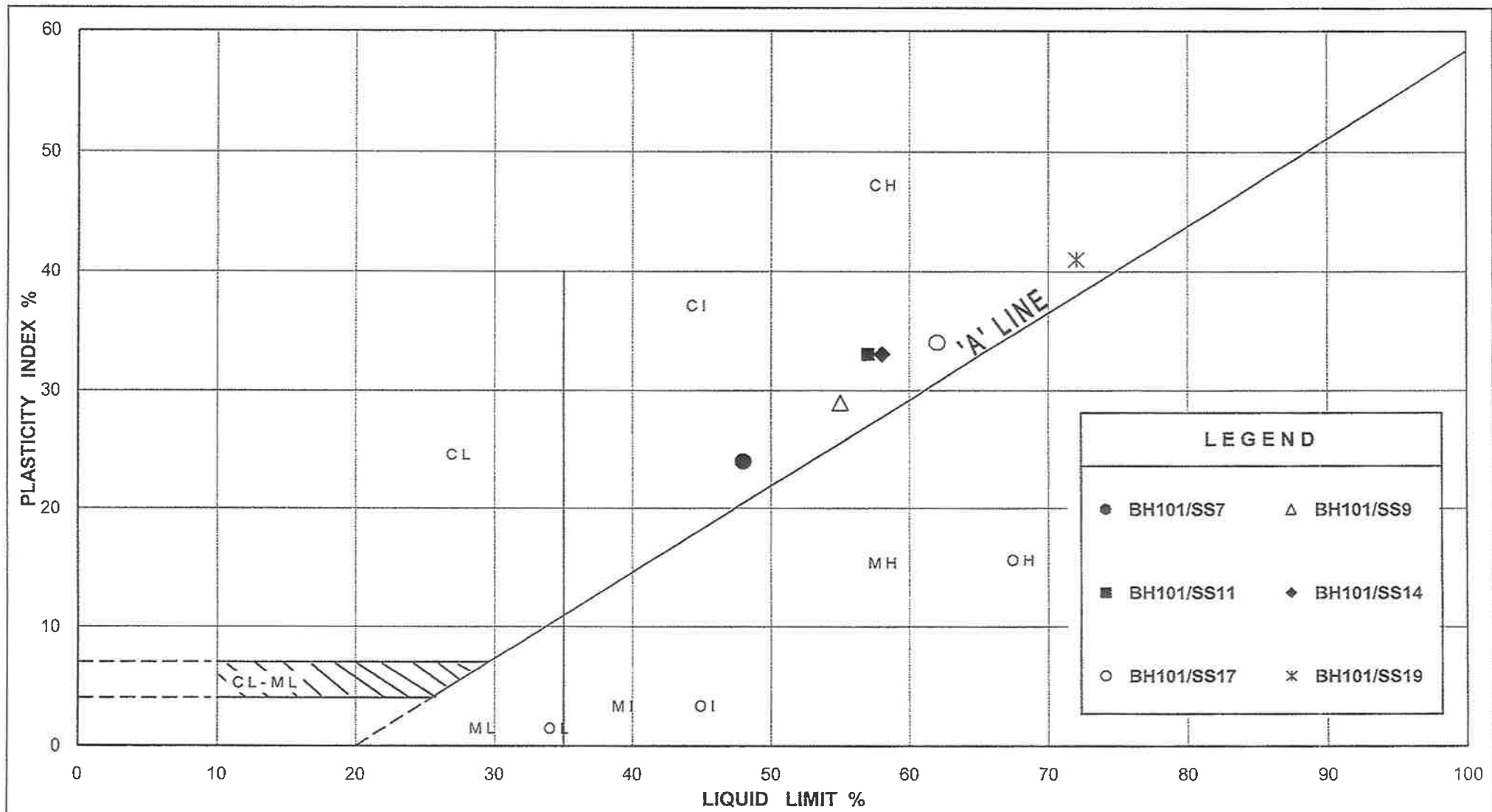


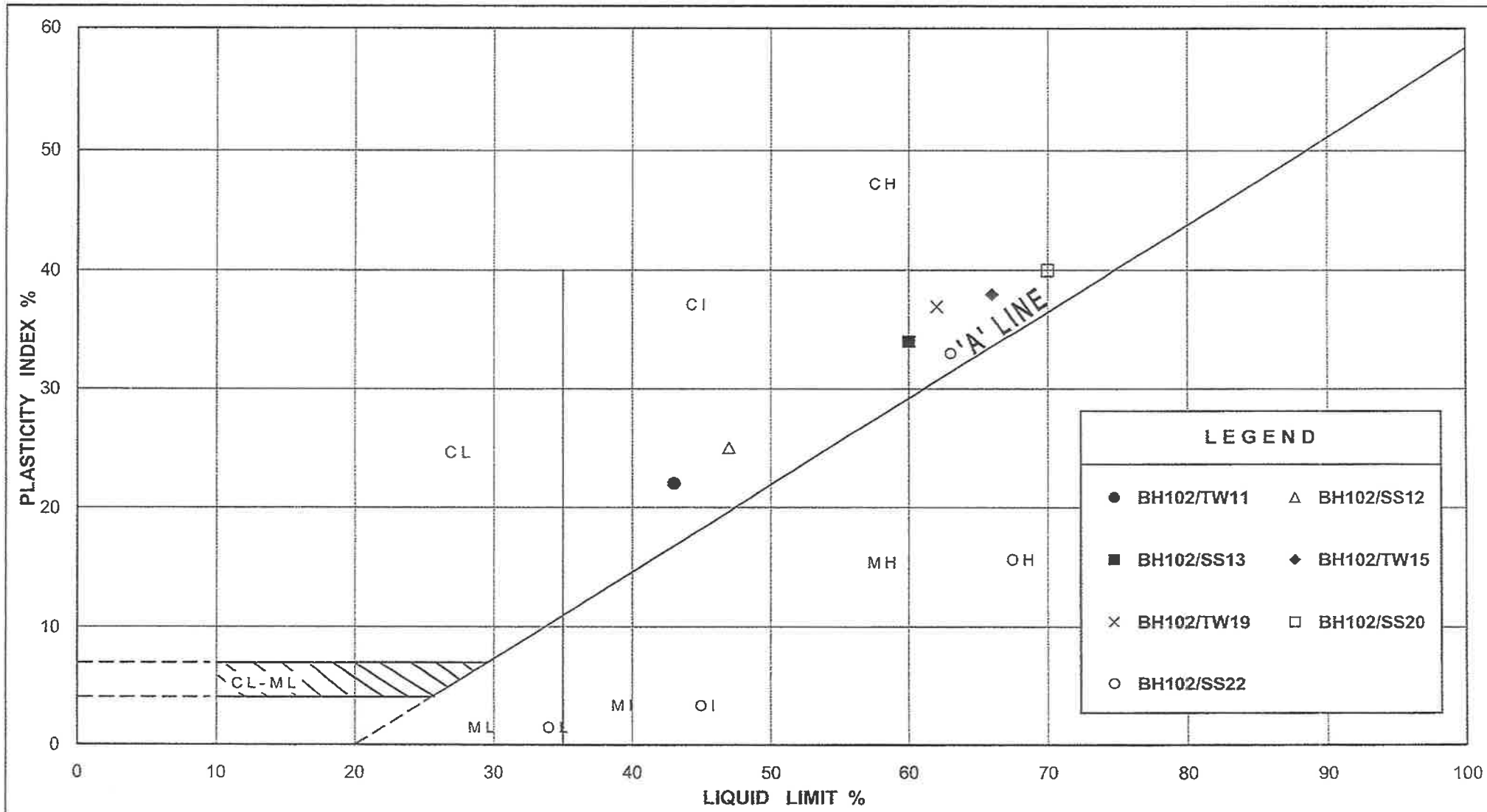


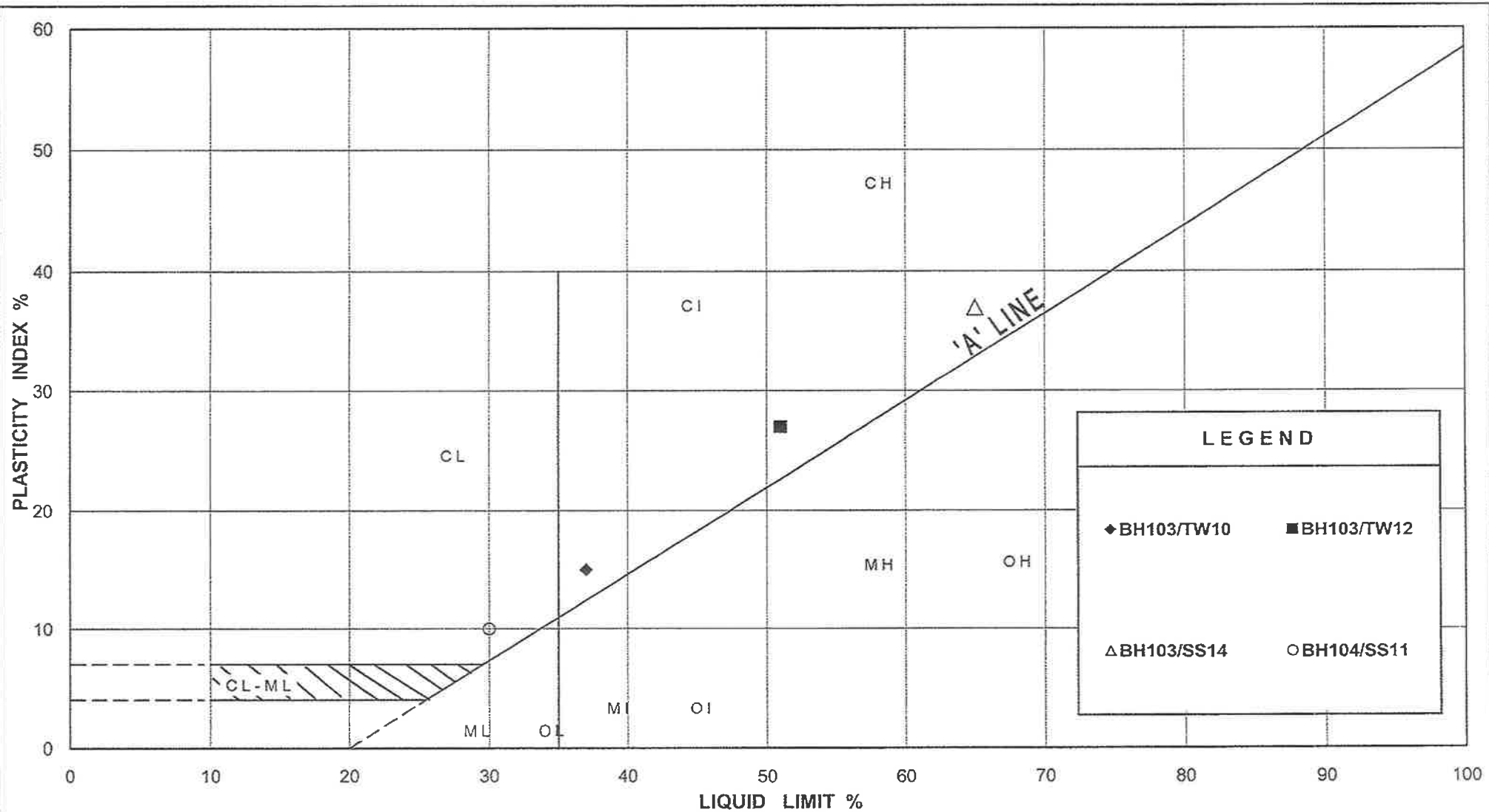


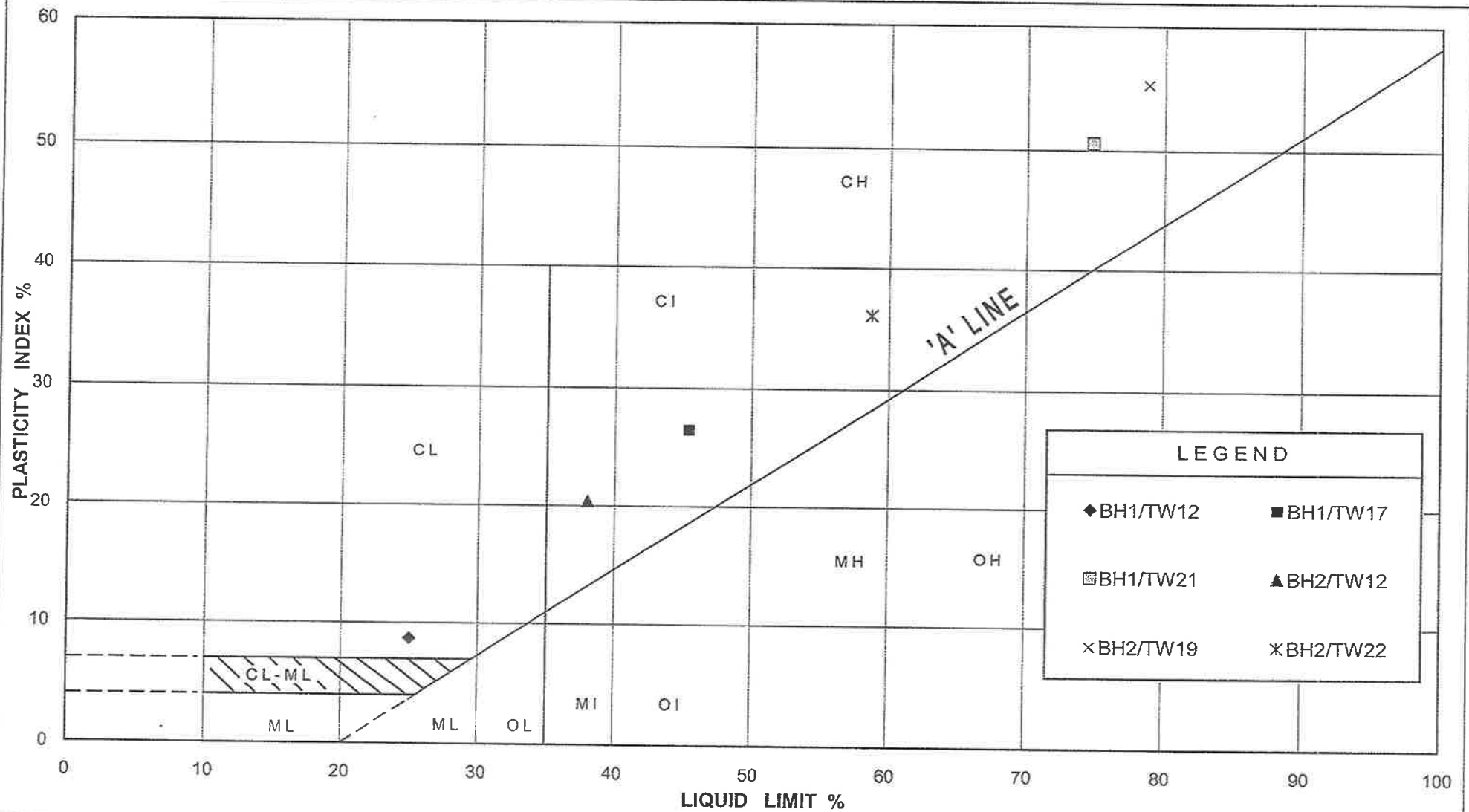












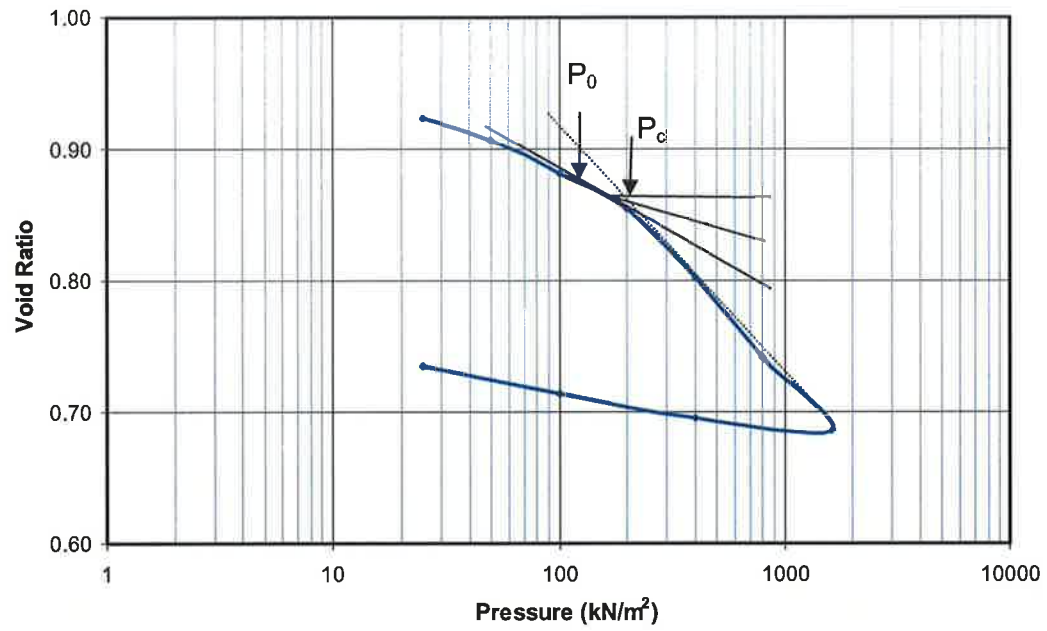
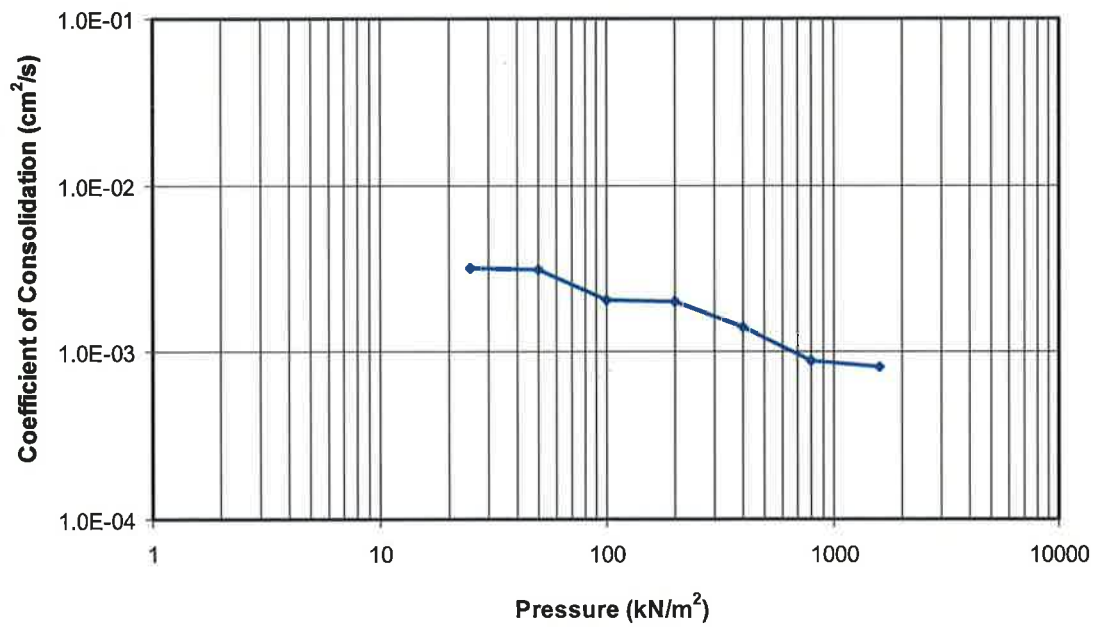
SHAHEEN & PEAKER LIMITED

PLASTICITY CHART
SILTY CLAY

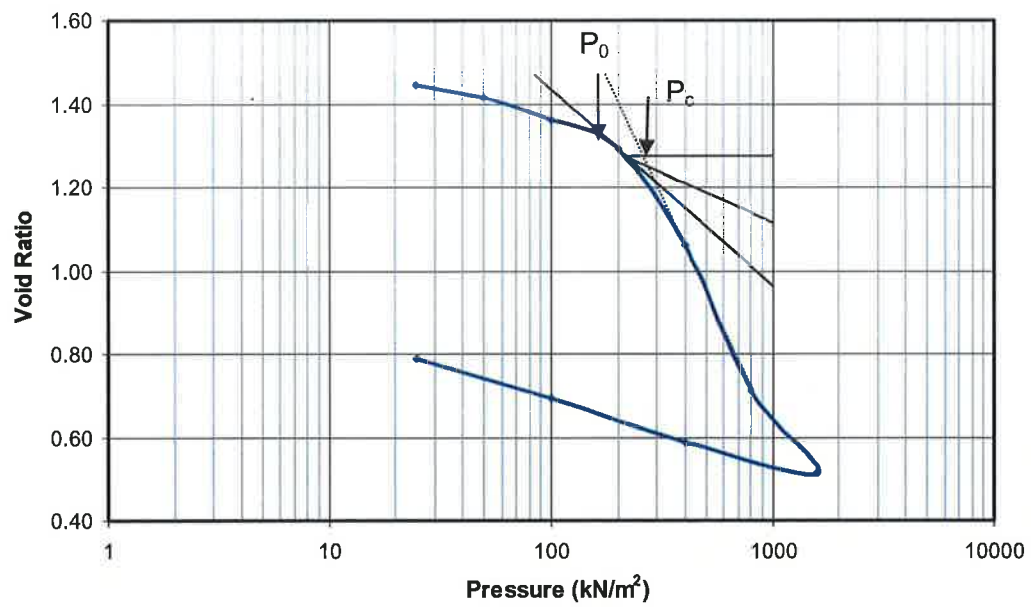
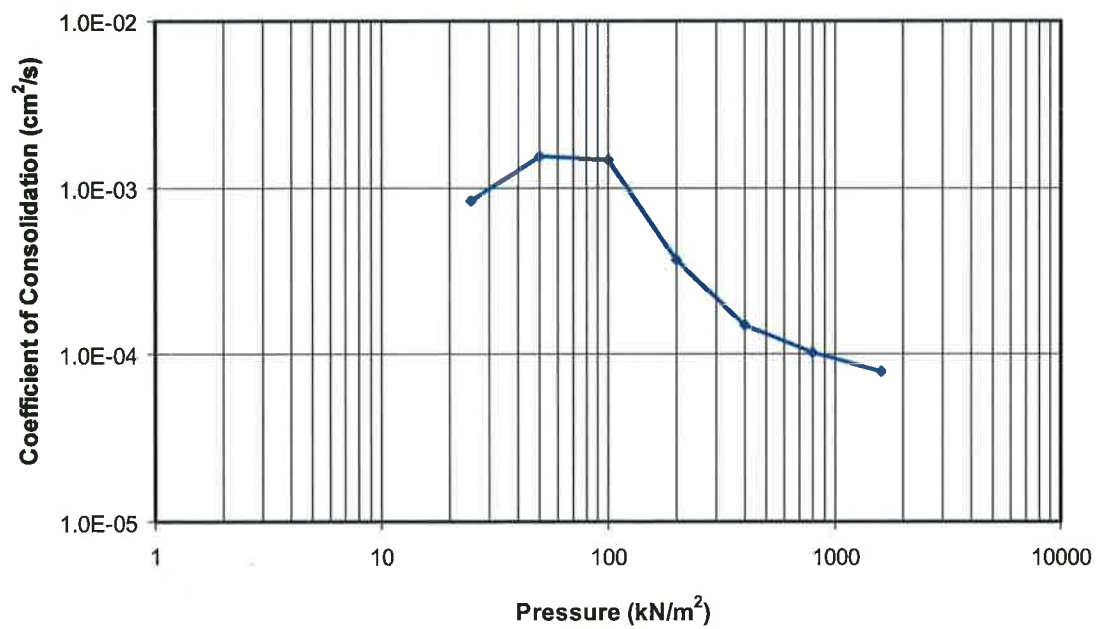
FIG No B-10

G.W.P. 5290-04-00

REF No SPT 1156A

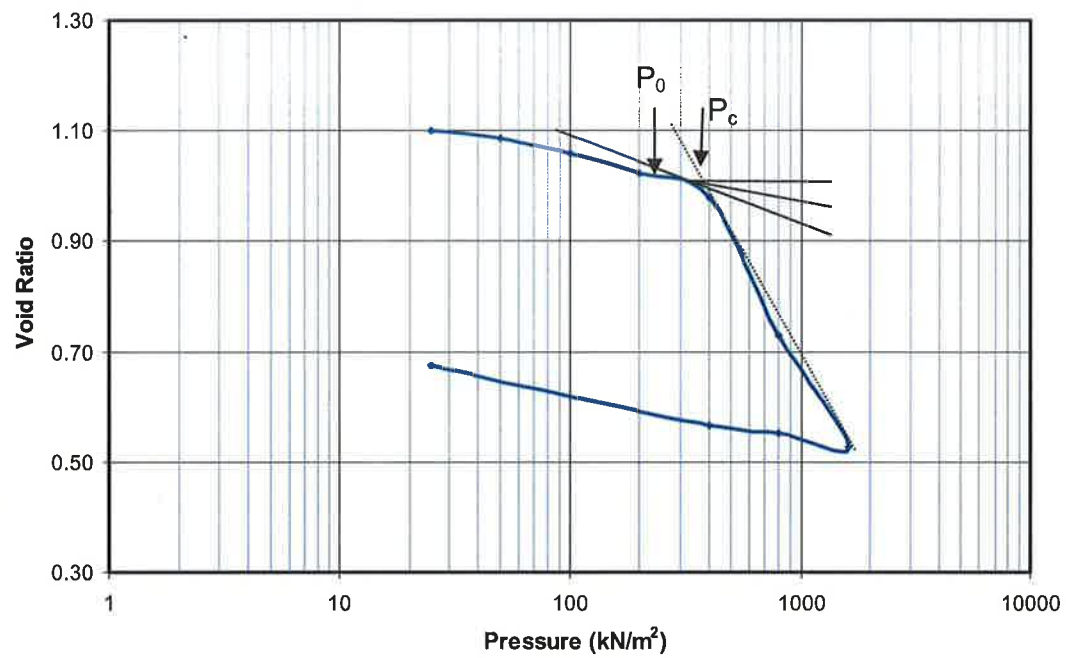
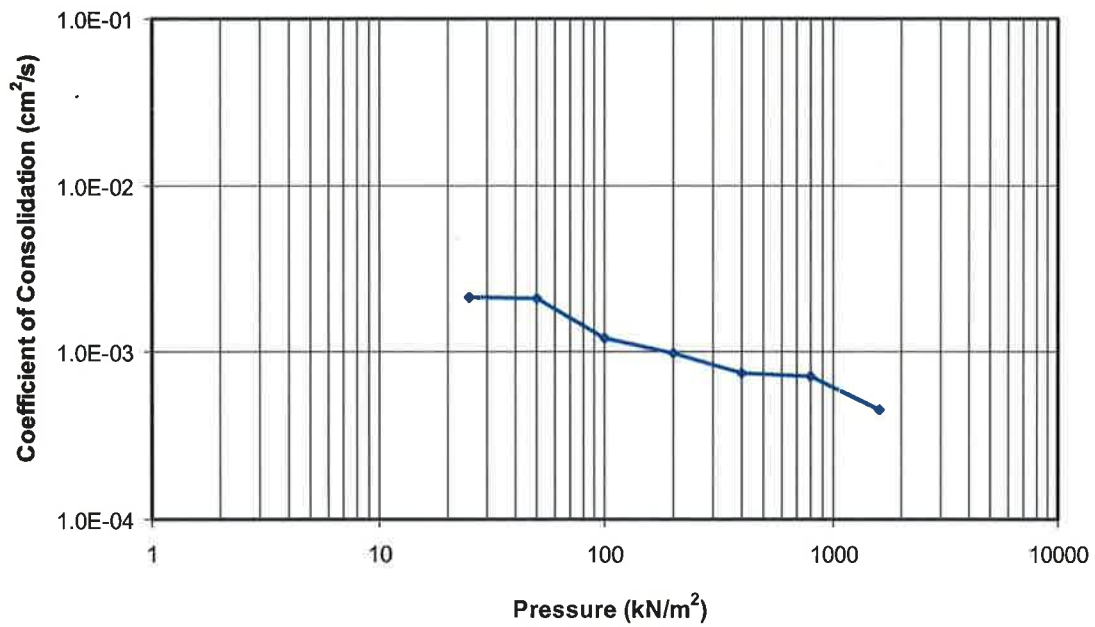
Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

BH 102 TW 11

Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

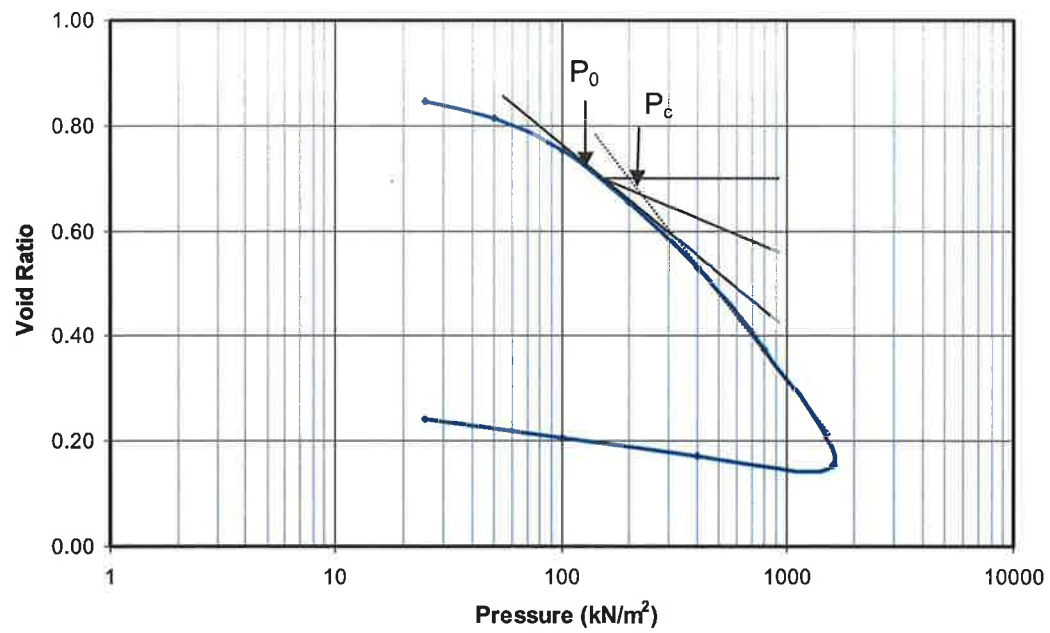
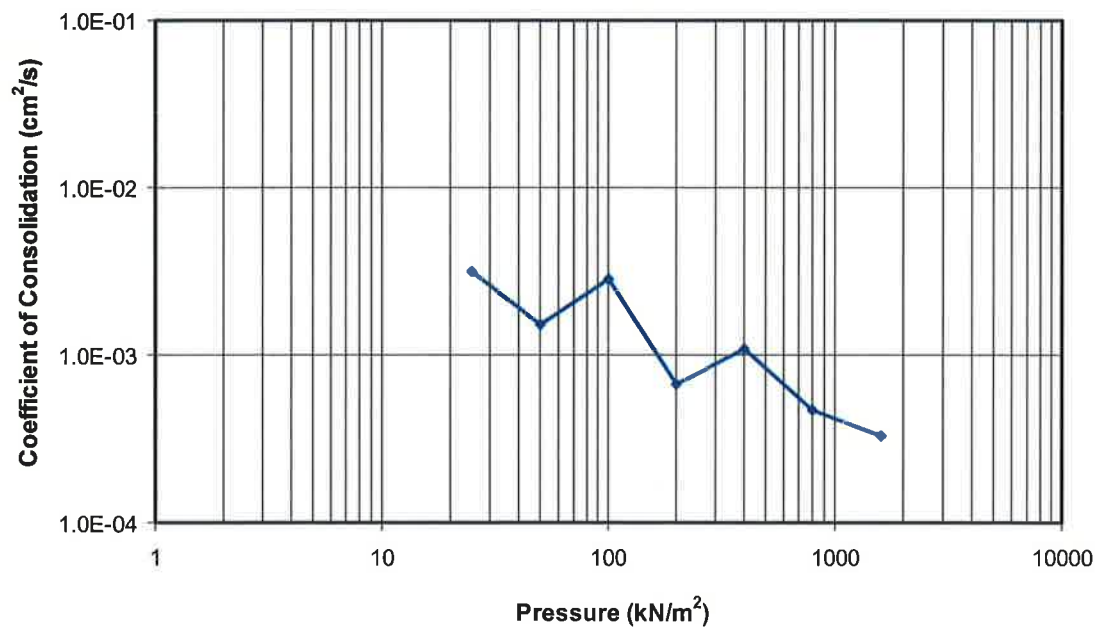
BH 102 TW 15

Figure B-12

Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

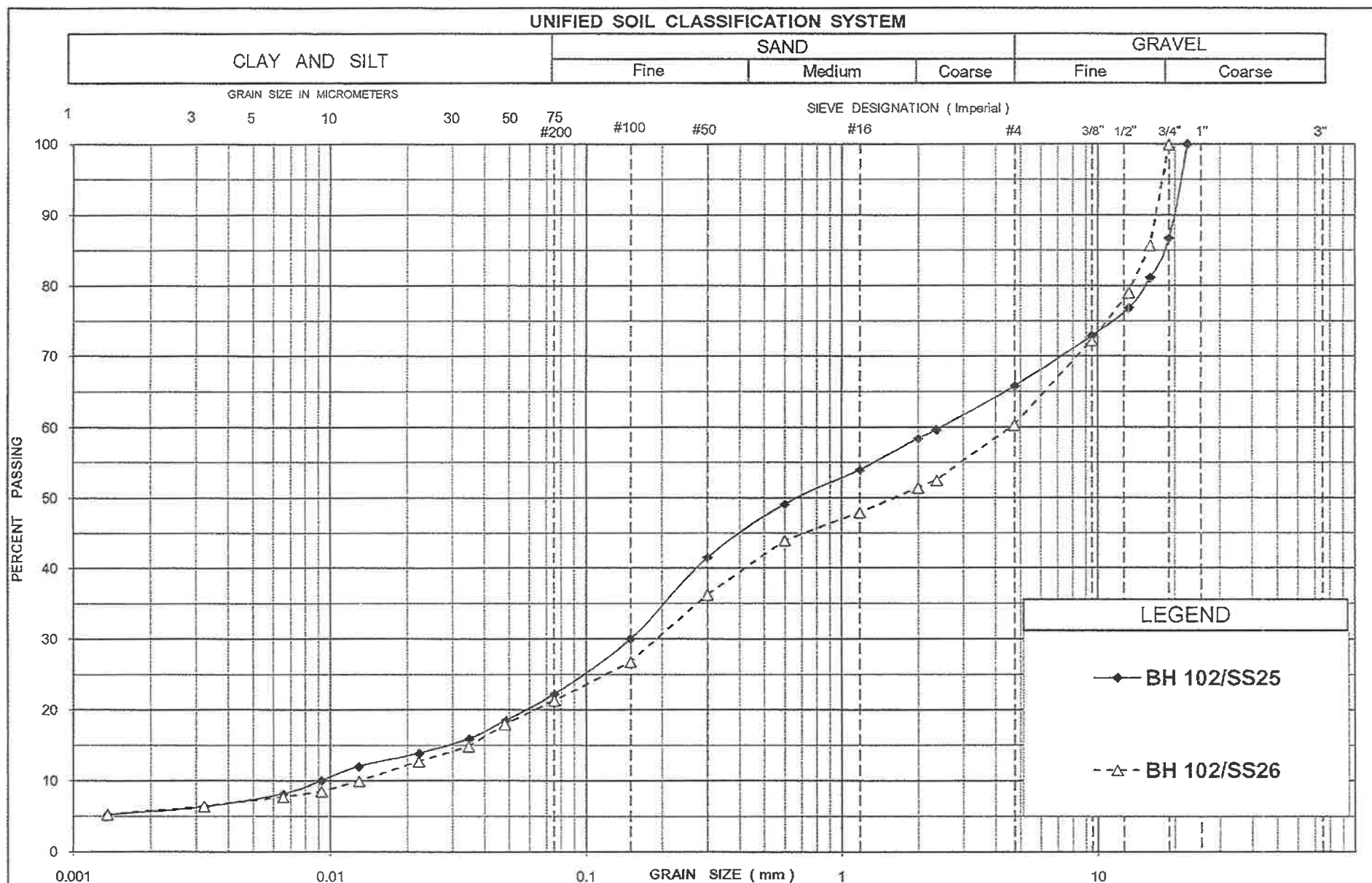
BH 102 TW 19

Figure B-13

Void Ratio versus Pressure**Coefficient of Consolidation vs. Pressure**

BH 103 TW 12

Figure B-14



Appendix C

Undrained Shear Strength Plots

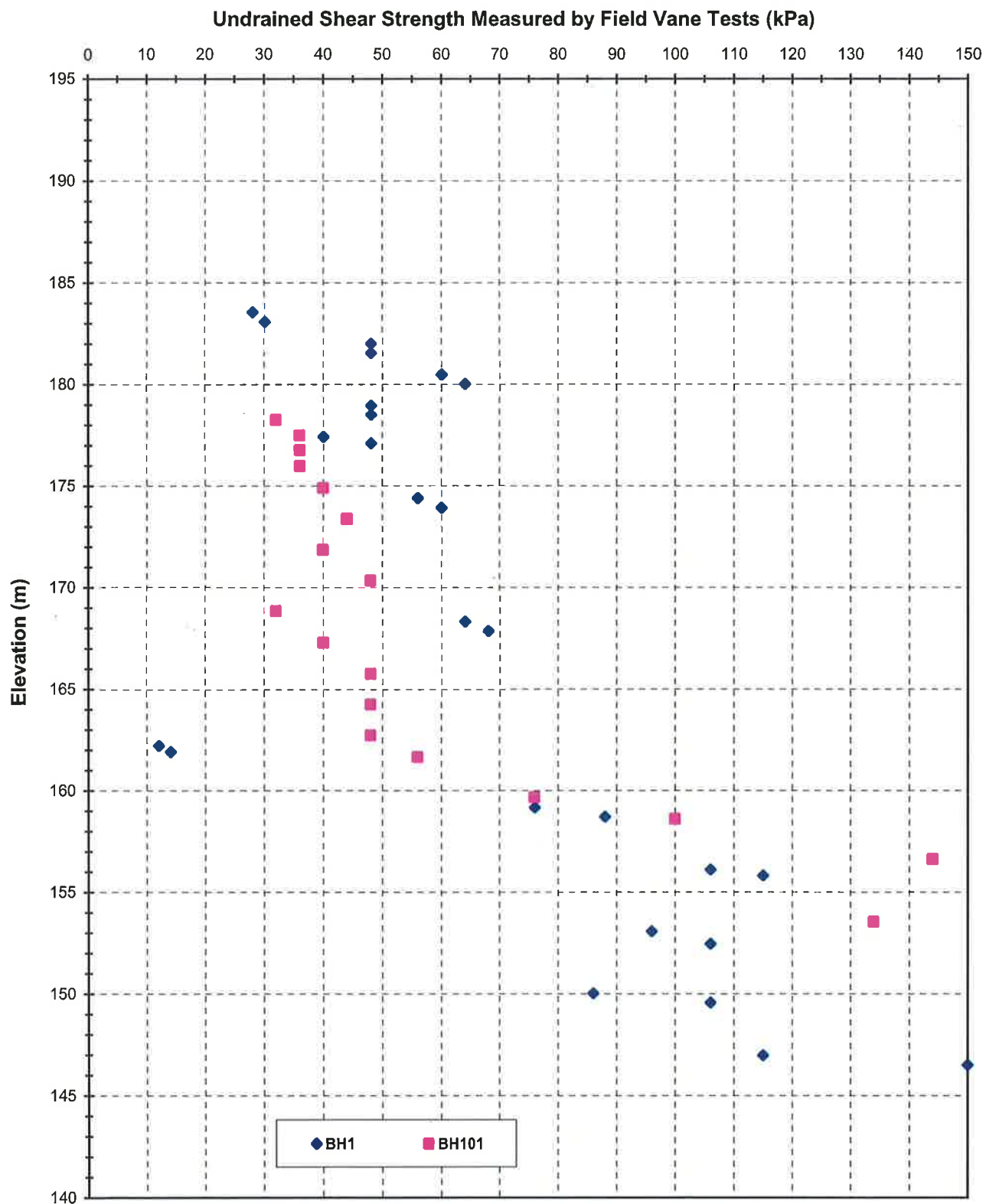


Figure C1 Plot of Undrained Shear Strength with Elevation for Boreholes 1 and 101

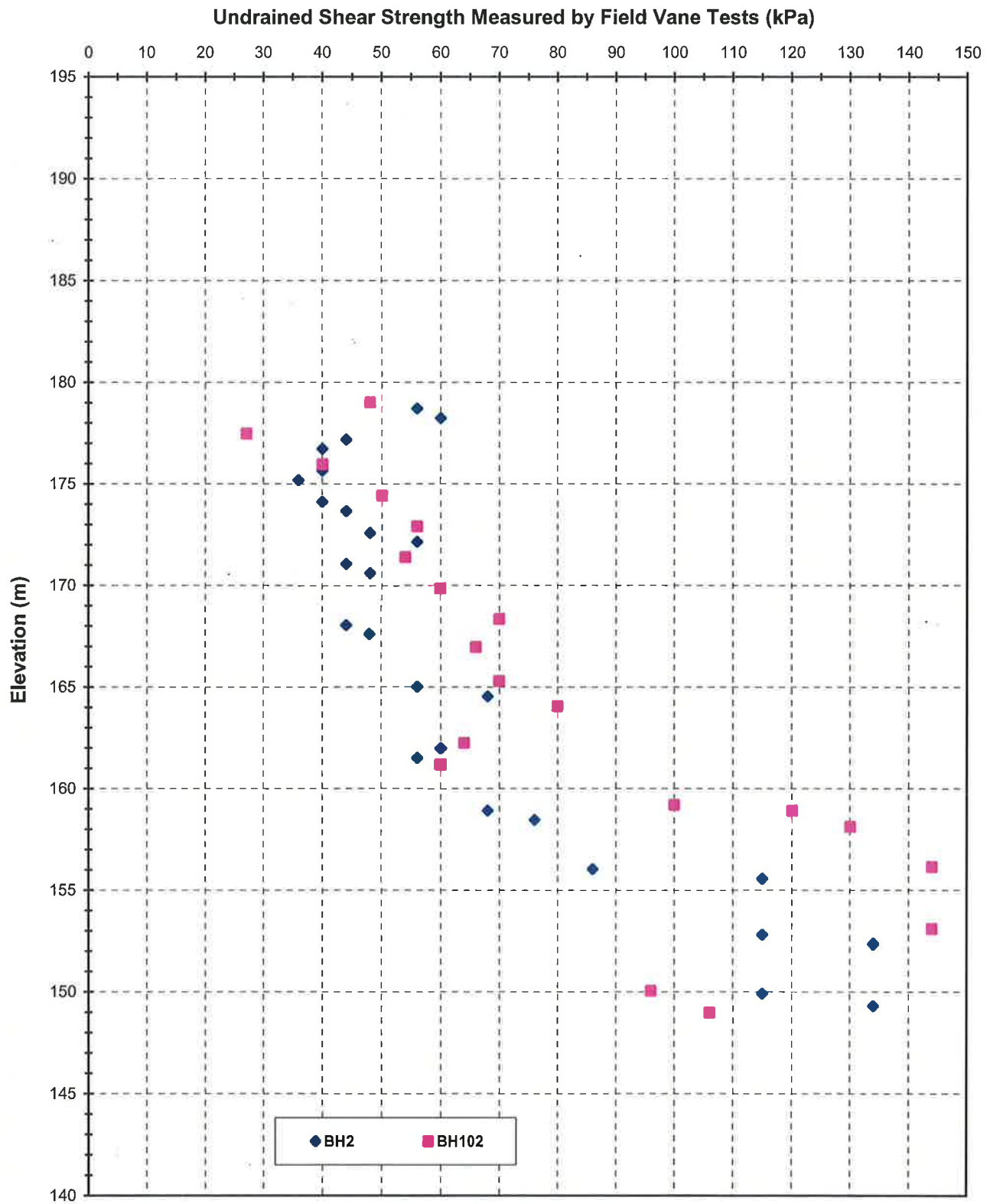


Figure C2 Plot of Undrained Shear Strength with Elevation for Boreholes 2 and 102

Appendix D

Site Photographs



Photograph D-1 Fill section (looking west)



Photograph D-2 West bank of Goulais River



Photograph D-3 East bank of Goulais River



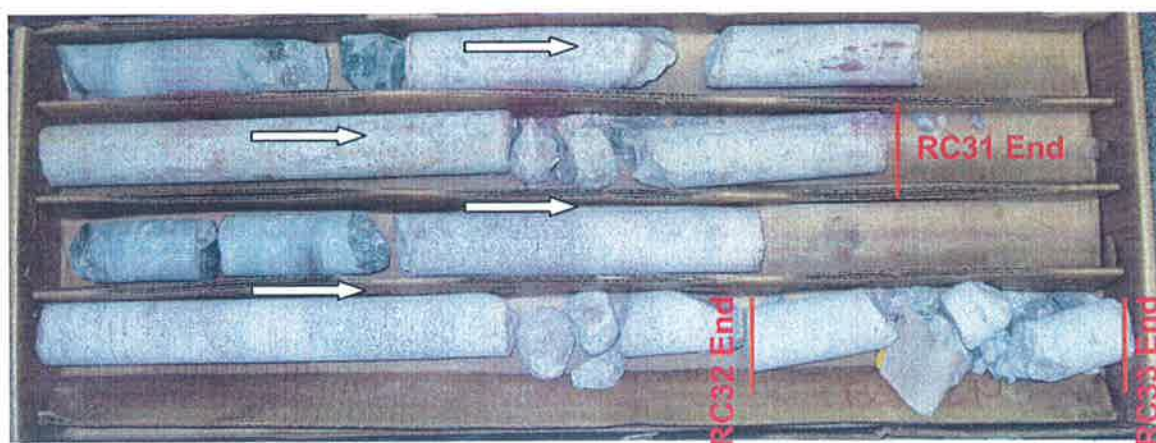
Photograph D-4 Cut section (looking east)A

Appendix E

Rock Core Photographs



Borehole 101 RC28, RC30 and RC32



Borehole 102 RC31, RC32 and RC32

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 \bar{N} .

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

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

**FOUNDATION DESIGN REPORT
SHEWFELT BRIDGE REPLACEMENT
GOULAIS BAY ROAD, 3 KM WEST OF
HIGHWAY 17,
DISTRICT OF ALGOMA, ONTARIO
G.W.P. 5290-04-00, SITE 38S-031
GEOCRES NO. 41K-82**

LEA Consulting Limited

Project: SPT1156
December 09, 2009

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Appendices

Appendix G: Profiles and Cross Sections of the Goulais Bay Road

Appendix H: Typical Stability Analyses (Bridge Abutment Locations)

Appendix I: Surcharge Plan and Stability Analyses with Surcharging

Appendix J: Typical Settlement Analyses

Appendix K: Typical Stability Analyses (Cut Section, STA 60+010 to 60+120)

Appendix L: Design Undrained Shear Strengths Plot

Appendix M: MTO Procedure for EPS Design

Appendix N: OPSD (pile reinforcement removed)

Appendix O: Limitations of Report

**FOUNDATION DESIGN REPORT
SHEWFELT BRIDGE REPLACEMENT, GOULAIS BAY ROAD
3 KM WEST OF HIGHWAY 17, DISTRICT OF ALGOMA
ONTARIO, G.W.P. 5290-04-00, SITE 38S-031**

5. DISCUSSION AND RECOMMENDATIONS

5.1 Proposed Bridge Structure

The proposed Shewfelt Bridge which will carry new Goulais Road over the Goulais River will be a two-span 91.0 m long structure. It will incorporate a pier in the river. The west span will be 36.5 m while the east span will be 54.5 m long. Virtually no grade change is proposed at the east abutment location but a 4.0 m grade raise is expected at the west abutment location, gradually decreasing to zero some 150 m further west. The present design incorporates integral type abutments.

The Goulais River at the proposed bridge location is some 55 m wide (at water level at the time of investigation). The water level in the River in December 2005 was at El. 183.75 m while during our investigation (Sept 5 – Oct 10, 2008) the water level fluctuated between El. 183.4 and 183.2 m (about 1.2 m deep). The anticipated 1:50 and 1:100 year high water levels are about 188.6 m and 188.7 m, respectively (i.e. about 1 m higher than the ground elevation at the west bank).

The investigation has shown below some surficial granular soils ranging from silt to sand, the site is underlain, at about El. 187-184 m at the east bank and at about El. 180 m under the river and the west bank, by an extensive silty clay deposit, with some silt and clayey silt seams, particularly near the top and the bottom of the deposit. In the deep boreholes this deposit extends to about El. 146 m (i.e. about 34 m thick). Its consistency as measured by field vane and SPT field tests is described as soft to very stiff. This cohesive deposit is underlain by granular soils, with frequent cobbles and boulders. The presence of bedrock was surmised in Borehole 101 at El. 137 m, while Borehole 102 was extended 6 m below this elevation to El. 131 m, where no bedrock was contacted. For the conditions encountered, integral abutment with steel H-piles will need to be utilized.

The water level observations made while drilling and in the various piezometers installed at the site indicate that two separate water/piezometric levels exist at the site namely, an upper level and a lower level. The upper level which is in the upper granular soils at a depth of about 3.4 to 4.5 m below the ground surface and the lower one emanates from the granular deposits underlying the massive clay deposit and from the bedrock which exhibited an artesian condition in the river and on the west bank area, where the grade is about 5 m lower than the east bank.

5.1.1 Foundations

The very loose to loose sand to sandy silts are considered unsuitable to support normal shallow spread footing foundations, including the use of spread footings on engineered fill. The relative density of these soils can be improved by means of in-situ densification but such operations are considered impractical immediately adjacent to some structures (e.g. residential houses) and the river. As well excessive long

term settlements can be expected due to the consolidation of the underlying weak clay deposit, especially on the west side where the grade is proposed to be raised by up to 4.0 m.

The bridge will, therefore, need to be supported on deep foundations and measures to reduce differential settlement 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 clay.

Auger press piles, also known as auger cast piles, can be extended in cohesionless soils below the groundwater table. Auger press piles are installed by rotating a continuously flighted hollow shaft auger into the soil to a specified depth. High strength concrete grout is poured under pressure through the hollow shaft as the auger is slowly withdrawn. The resulting grout column hardens and forms the auger press pile. Reinforcing when required, can be installed while the cement grout is still fluid, or in the case of full length single reinforcing bars, through the hollow shaft of the auger prior to the withdrawal and grouting press. Auger press piles are unlikely to be economical and therefore not recommended based on cost.

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), would be better suited than other pile types such as steel tube piles or steel H-piles with lighter sections or precast concrete and Franki type piles.

Consideration was given to the use of steel H-piles with a lighter cross section (e.g. HP 310 x 74) or steel tube type piles utilizing friction and adhesion. Axial resistances provided by this approach are considered to be unsuitable (i.e. too low) for the bridge under consideration, especially for the central pier. As well, a very soft zone was found in Borehole 1 of the advance investigation.

The most practical option appears to drive the piles into the granular soils underlying the clay deposit, thus utilizing both friction/adhesion and end resistance, while being cognizant of the prevailing artesian conditions at the site. The following table summarizes the recommended pile tip elevations and resistances for HP 310 x 110 steel H-piles.

Table 5.1.1.1 Recommended Pile Lengths and Resistances

Borehole No./Location	Existing Ground Elevation (m)	Recommended Tip Elevation (m)	Corresponding Pile Length Below Existing Ground Surface (m)	Corresponding Approximate Pile Length Below Pile Cap (m)	Recommended Pile Resistances for HP 310x110 H-Piles	
					ULS (kN/pile)	SLS (kN/pile)
East abutment BH1	193.3	140.5	52.8	48.6	1300	900
Pier BH101	182.1	140.8	41.3	39.5	1450	1050
West abutment BH102	187.7	135.7	52.0	51.1	1200	800

Higher resistances would be available if the piles were driven to greater depths but this may not be possible because piles are likely to “hang-on” boulders, the frequency of which increases with increasing depth. As well, driving the piles to greater depths may create problems due to the presence of artesian conditions.

At the west abutment location, when recommending these resistances, it is assumed that the grade will be raised to the final grade levels at least six months prior to driving the piles, in order to effect all of the settlements in the granular deposits and some of the settlements in the underlying silty clay.

Considering the length of the piles and 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. However in this case, the reinforcing will reduce adhesion, as well as increasing the possibility of upward migration of soil due to the prevailing artesian conditions. Consideration can be given to placing the reinforcing on the inside of the flange rather than outside but this is costly as well as being only partially effective. For this reason it is recommended that piles be driven under supervision so as not to damage the pile tips and that the piles be reinforced using Titus Point or equivalent, which do not protrude beyond the pile perimeter, so that adhesion will not be adversely affected. For this reason, the use of a heavier section, such as HP 310X125, may also be considered.

The piles will need to be driven using a heavy hammer capable of delivering a rated energy of at least 55 kilojoules/blow, but not more than 70 kilojoules/blow. 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. 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 6 m above the design elevation and driving then monitored by employing the Hiley Dynamic Formula in accordance with MTO Standard Drawing SS103-11.

We recommend that piles be driven to not less than 2.0 m of the final tip elevation given in Table 5.1.1.1. It is furthermore recommended that the piles not be driven more than about 1.0 m below the given pile tip elevations before notifying the QVE, due to the significant upward hydraulic gradients encountered in the boreholes.

During the driving process, piles which have already been driven will need to be monitored to determine if they are heaving due to the effects of driving of adjacent piles. If this phenomenon occurs, the affected piles will need to be re-driven. Retapping, to check that relaxation has not occurred, will be necessary in accordance with MTO procedures. Furthermore, it may be necessary to stagger the driving of the piles.

The minimum spacing between the piles should be chosen in consideration of the longer than usual pile lengths. The high slenderness ratio of the piles may need to be considered. A heavier section, such as HP 310X125 may be useful in this respect. The design should be in accordance with the Canadian Highway Bridge Design Code.

We recommend that an NSSP be provided in the contract to warn the contractor of the presence of cobbles and boulders below about El. 146 m, as well as the hydrostatic uplift and artesian conditions.

Consideration can also be given to pile load test(s) to verify the pile resistances.

Due to the presence of a 4 m grade raise adjacent to western abutment some downdrag settlement of the abutment piles can be expected. The magnitude of this downdrag settlement will depend upon the construction sequence and the nature of preloading of the approach embankment. Downdrag settlement will be less than the adjacent embankment settlement occurring after installation of the western abutment piles. As there will be no significant loading of the soil profile at the eastern abutment and at the central pier location, no significant downdrag settlement is expected for piles at these locations.

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

$$k_s = n_h z / d$$

where k_s = coefficient of horizontal subgrade reaction

z = depth

d = pile width

n_h = coefficient related to soil density as given in Table 5.1.1.2

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 k_s = coefficient of horizontal subgrade reaction

c_u = undrained shear strength

d = width of pile

Table 5.1.1.2

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)
East Abutment/BH1	193.1-192.7	Silty fine sand	19.5	27	2.0	40 40 80 110
	192.7-188.8	Sand	20.0	30	2.5	
	188.8-184.3	Silty sand to sandy silt	19.0	29	1.5	
	184.3-179.6	Clayey silt/Silty clay	17.5			
	179.6-159.0	Silty clay	16.0			
	159.0-147.6	Silty clay	16.5			
	147.6-146.1	Clayey silt/Silty clay	17.5			
	146.2-145.0	Gravel and cobbles	20.5	34	11.0	
Central Pier/BH 101	182.1-179.4	Sand	20.0	29	1.3	

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)
	179.4-160.0 160.0-157.0 157.0-149.3 149.3-146.3 146.3-143.3 143.3-140.1 140.1-137.3	Silty Clay Silty Clay Silty Clay Clayey Silt Silty sand with gravel Silty fine sand Gravel and cobbles with sand infill	16.0 16.0 16.5 18.0 20.5 20.0 21.0	 33 31 34	 7.0 5.0 11.0	40 80 130 150
West Abutment/BH102	187.7-187.0 187.0-183.6 183.6-180.7 180.7-178.0 178.0-175.0 175.0-159.0 159.0-148.4 148.4-145.4 145.4-137.7 137.7-131.3	Silty fine sand Sandy silt to silty sand Sand Silty clay Silty clay Silty clay Silty clay Clayey silt Silty sand with gravel Gravel and cobbles with sand infill	18.0 19.0 19.5 18.0 16.5 16.0 17.0 18.0 20.5 21.0	28 28 29 33 34	2.0 2.2 3.0 7.0 11.0	 40 40 50 100 120

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. = 130 kN/pile

Horizontal Resistance at S.L.S. = 50 kN/pile

At the central pier, the lateral resistance of the piles can be supplemented, if desired, by horizontal components of battered piles. In this instance, we recommend that the batter be limited to no more than 8:1, as in practice greater batter would be difficult to install in the river setting. At the abutments, if integral type abutments are not to be constructed, then the use of battered piles can be considered. 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. In essence, the current MTO standard for the flex zone consists of an annular space in between two concentric corrugated steel pipes (CSP's). One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. In accordance with current MTO practice, this space between the CSP's can be left void. After

the pile is driven, the space between the H-pile and the inner CSP is filled with sand. This double CSP scheme is typically used for false abutments.

Alternatively, if a false abutment is not provided which is likely to be for this project, in accordance with MTO structural office requirements (Report SO-96-01), the flex zone can be provided by augering a 600 mm diameter hole 3000 mm deep and filling with uniform sand. A special provision should be included in the contract specifying the gradation of the sand as follows:

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

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

5.1.2 Lateral Earth Pressures

Backfill behind abutments should consist of non-frost susceptible, free-draining granular materials in accordance with the Ontario Ministry of Transportation Standards and the requirements of OPSD 3101.150 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, $\phi = 35^\circ$ (unfactored)

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressure:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

$$K_a = 0.31$$

$$K_o = 0.47$$

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

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used in accordance with C.H.B.D.C. 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 C.H.B.D.C.

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 C.H.B.D.C. commentary can be consulted. We understand, however, that the present design of the bridge structures does not incorporate any wing walls.

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

5.1.3 Settlement at the West Abutment Location

We understand that the project incorporates a grade raise of up to 4.0 m at the west abutment area. A settlement of about 280 mm was calculated due to the stresses imposed by up to about 4.0 m high embankment on the west side while on the east abutment area minor settlement will be required and as such settlements should be negligible.

At the west abutment location, the settlement of the surficial sand and the sandy silt to silty sand deposits can be expected to take place rapidly while the consolidation of the underlying silty clay deposit can be expected to proceed at a much slower pace. The upper and lower zones of the clay deposit itself are expected to exhibit different time rate of consolidation characteristics in comparison with the middle (thicker) zone, due to the presence of frequent silt and clayey silt interbeds in the upper and lower zones.

Consolidation characteristics of the clay deposit were investigated by means of four one-dimensional consolidation (oedometer) tests. The results of these tests are presented in Appendix B. The tests were performed from samples recovered from Borehole 102 (three tests) and Borehole 103 (one test), both located on the west bank, where the existing grade (o.g. levels) are lower than the east bank by about 5 m. The test results indicate a pre-consolidation pressure (p'_c) in excess of the existing overburden in-situ pressure (P'_o) of about 80 to 130 kPa. A $P'_c - P'_o$ value of 80 kPa would be a reasonable figure if the previous height of the west bank was nearly equal to that of the east bank.

The thickness of the surficial sandy silt to silty sand deposits and of the underlying sand on the west side of the river, where the grade will be raised by up to about 4.0 m, is 7.0 m while the thickness of the underlying clay deposit is about 35 m. The upper 3 m and the lower 4 m of this 35 m thick cohesive deposit contains frequent silt interbeds and as mentioned before would likely consolidate at a significantly shorter time frame than the middle 28 m. Based on the consolidation tests, the following values were used in the settlement analyses for the middle 28 m zone.

$$e_o = 1.0$$

$$C_c = 0.80$$

$$C_r = 0.16$$

The embankment fill is expected to impose a maximum stress increase of about 84 kPa. Based on the available data, the anticipated settlement of the upper granular (7.0 m thick) soils, which are typically very loose to loose, is about 50 mm under this stress. This settlement would in our opinion, be substantially completed during the construction and within a period of about four weeks thereafter.

Owing to the presence of the surficial granular soils, in the underlying silty clay deposit, the stresses due to the embankment fills would be distributed to below 80 kPa (i.e. the measured lowest $P_c' - P_o'$ value in the consolidation tests).

Using a value of 80 kPa for pre-consolidation in excess of the existing overburden, the calculated settlement of the 35 m thick clay deposit is about 230 mm, bringing the total settlement figure to 280 mm (i.e. including the settlement of the surficial sand deposits). About 25 mm of this 230 mm settlement, which can be attributed to 35 m thick silty clay deposit, belongs to the upper 3 m thick silty zone, while another 5 mm is assessed for the lower 4 m thick silty zone, leaving a settlement of 200 mm for the middle 28 m thick zone. Based on our experience, the settlement of the upper and lower zones (i.e. total of 25+5 = 30 mm) can be expected to be substantially completed within about six months after the application of full embankment loads, while the consolidation settlement of the middle section can be expected to take a very long period of time.

To estimate the rate of settlement of the 28 m thick middle clay zone, C_v values obtained from the laboratory consolidation tests can be utilized. These typically range from 2×10^{-3} to 5×10^{-4} cm²/sec. However, experience show that C_v values obtained from laboratory tests are typically 5 to 10 times lower than values in the field. Using a value of $C_v = 1 \times 10^{-3}$ cm²/sec, the time required to obtain a 90% consolidation is decades (i.e. about 50 years). The clay contains occasional silt and clayey silt zones which may speed up to process. This aspect was not accounted for. On the other hand, the 28 m thick clay was assumed to drain both ways (i.e. towards the top and the bottom thus using a drainage path of $28/2 = 14$ m). However, since there is an upward gradient underlying the clay, downward dissipation of the pore pressure may not fully materialize.

Three treatment options were assessed to provide alternative construction approaches for consideration:

- Use of surcharging and wick drain to promote early embankment settlement
- Use of surcharging with future maintenance
- Use of surcharging and light weight fill

For all options, monitoring will be required to measure settlement response to check design assumptions and forecast settlement performance during operation.

These options are described below.

Wick Drain with Surcharge Option

Since settlements of this 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 downdrag on the piles, surcharging with or without wick drains may be required to speed up the rate of consolidation of the clay. Since only limited surcharging can be applied immediately adjacent to the already somewhat unstable river bank and the time period for surcharging (i.e. many years), wick drains may need to be used. For preliminary estimating purposes, wick drains can be assumed to extend to about El. 151 m, and installed at a horizontal spacing of about 1.5 m, in a triangular fashion. The recommended height of the surcharge is 2.0 m and the surcharging period would be about six months (we understand a surcharging period of only six months is available). These and some other details of this approach would be further looked into if a wick drains option is to be adopted. In particular, further investigation would be required of the existing vertical pore pressure profile under the influence of the pressurized lower sand layer (below about El. 145 m).

Surcharge Option

Another option would be to accept after surcharging the future settlements with periodic, future maintenance (e.g. initially after a six month surcharge period every year or so and subsequently at decreasing frequency) immediately adjacent to the bridge abutment on the west side.

Figures 1 and 2 in Appendix J show the anticipated settlements in the absence of wick drain immediately adjacent to the bridge on the west side, based on available data. Figure 1 indicates that of the predicted 280 mm long term settlements about 100 mm would take place within the first six months. If a 2.0 m surcharge is placed for six months prior to the construction, in addition to the predicted 100 mm settlement, the surcharge would induce another 50 mm settlement in the silty clay deposit, bringing the total settlements before paving the roadway to 150 mm, thus leaving a long term settlement of about 130 mm to take place over the next 35± years, as shown in Figure 2. Figure 2 also shows that the magnitude of the anticipated settlement immediately adjacent to the bridge abutment, based on 4.0 m high fill and a 2.0 m high surcharge with a minimum surcharge period of six months, is about 50 mm over the next five years, with another 50 mm settlement over the following fifteen years. This may be acceptable to MTO since this is a secondary highway, provided that an on-going inspection and maintenance programme can be implemented. This approach would however have some implications on the piles supporting the abutment (i.e. downdrag).

Settlement of the piled western abutment will depend on the nature of the founding conditions encountered. It is expected that settlement of the piled western abutment would be between zero and the maximum settlement of the embankment occurring after installation of the piles.

Figure 1 in Appendix I shows the recommended surcharge configuration. As shown, the surcharge extends from the west abutment to Station 60+410 gradually decreasing to zero at Station 60+420.

Figures 2 and 3 in Appendix I several surcharge cross sections. We tried a 1.5H:1V (side slope) temporary surcharge cross section but theoretically (as shown in figure 2 in Appendix I), the calculated factor of safety was below 1.3. The use of a mid-height berm with flatter (2H:1V) side slopes can be considered (figure 3 in Appendix I) or alternatively conventional 2H:1V side slopes can be used (figure 4 in Appendix I). The erosion of the side slopes should be prevented.

Light Weight Fill Option

If the anticipated embankment and the abutment settlement is not considered acceptable, another approach would be use light weight fill in conjunction with 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 choice, as it is probably more cost effective.

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 west abutment, the thickness of the EPS would be greater, gradually decreasing further west. The presently proposed vertical and horizontal alignments are given in Appendix G. As shown, the road is at a super-elevation at this location and it may be necessary to adjust the EPS usage to reflect this (i.e. thicker EPS may need to be placed on the south side).

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 M, 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 1:100 year storm elevation water elevation is 188.7 m. The bottom of EPS should be therefore not be extended below El. 186.8 m, to prevent an uplift condition.
- ❖ 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 are recommended.

The site to receive the EPS should be surcharged by 2.0 m over the full foot-print of the proposed embankment. Since, for example, immediately adjacent to the proposed abutment location the proposed grade for the finished roadway is at 191.4 m, the fill would be placed to El. 193.4 m (i.e. 191.4+2.0 m surcharge), while at Station 60+335, the proposed elevation of the embankment is 190.4 m and therefore the top of surcharge would be at 192.4 m. The surcharge would be placed at least six months prior to driving of the piles and the placement of the EPS, in order to effect the anticipated settlements in the upper sand and silt layers and the upper zone of the silty clay/clayey silt. Before placing the surcharge fill the site under the foot print of the embankment should be stripped of all the topsoil and otherwise unsuitable soils, as per standard MTO procedures. After stripping, the exposed subgrade should be compacted from the surface using a suitable heavy compactor. The fill will need to be compacted to at least 95% of its Standard Proctor Maximum Dry Density (SPMDD) only to the underside of the proposed EPS elevation, above this elevation the requirements for compaction can be relaxed (e.g. 90% SPMDD)

After the required surcharge period, the fill would be removed to the proposed underside EPS elevation after which the piles would be driven and EPS could be placed. As mentioned before, the 0.15 m zone of the fill immediately below the EPS should consist of sand (i.e. no gravel) in order to prevent damaging.

With these design conditions and cross sections provided to us, the following EPS thickness may be used for preliminary design and costing purposes.

Suggested EPS Thickness

EPS scheme 'A'			
Location (Station)	Existing Grade (o.g.) Elevation (m)	Proposed Finished Road Grade (m)	EPS Thickness (m)
To 60+315	187.7	191.2	1.6
60+315 - 60+325	187.7	191.0	1.3
60+325 - 60+335	187.5	190.7	1.0
60+335 - 60+345	187.5	190.4	0.8
60+345 - 60+355	187.4	190.1	0.5
60+355 - 60+370	187.4	189.7	Gradually reduce to zero
EPS scheme 'B'			
Location (Station)	Existing Grade (o.g.) Elevation (m)	Proposed Finished Road Grade (m)	EPS Thickness (m)
To 60+315	187.7	191.2	1.2
60+315 - 60+325	187.7	191.0	0.9
60+325 - 60+335	187.5	190.7	0.7
60+335 - 60+345	187.5	190.4	0.5
60+345 - 60+360	187.4	190.1	Gradually reduce to zero

It is also recommended that surcharging (i.e. 2.0 m surcharge) be applied to Station 60+410 gradually reducing to zero at Station. 60+420, as shown on Figure 1 in Appendix I. EPS blocks will need to be protected from ultra-violet light exposure during construction. Figures 3 and 4 in Appendix J show the anticipated settlements vs time for both EPS schemes (i.e. 1.6 m and 1.2 m). The figure shows that the total settlement for 4 m high embankment with 1.6 m thick EPs will be 210 mm. Since approximately 140 mm of this total settlement will have taken place during surcharging (i.e. during the first six months) the total remaining settlement after 6 months will be about 70 mm. Of this figure, approximately 40 mm is expected to take place within the first five years and the remaining 30 mm within the next 20± years.

If a 1.2 m thick EPS is used, the calculated approximate total settlement is 230 mm and thus the anticipated settlement after the surcharging period is 90 mm. Again, approximately 50 mm of this 90 mm settlement is expected to take place within the first five years, with the remaining 40 mm settlement during the following 25± years.

As was mentioned, monitoring will be required for all options above to measure settlement response to check design assumptions and forecast settlement performance during operation.

5.2 Stability of Forward Slopes

Slope stability analysis was carried out using the information provided to us by LEA, as given in Appendix G. The stability of the forward slopes 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

simplified Bishop 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. Borehole 1 and 107 on the east side and Boreholes 2, 102 and 103 on the west side). The soil parameters adopted in the analysis are summarized in Table 5.2.1.

Table 5.2.1

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)
Embankment Fill	20	-	32	-	32
Sand	20	-	31	-	31
Sandy Silt/Silty Sand	19	-	29	-	29
Clayey Silt	17.5	45	-	2	30
Silty Clay	16.5	45 to 65 and increase linearly with depth (between El. 179 m to 160 m)	-	3 to 4	26

Based on the information provided to us, the grade raise of east approach embankment will be minimal, while a 4.0 m high embankment will be constructed on the west side of the river (west approach embankment).

During our previous investigation in 2006 it was determined that the permanent slopes should be no steeper than 3H:1V and the present design of the bridge was based on this, as the banks of the river in the general area are known to be highly unstable.

The present analysis confirmed the findings, with typical results given in Appendices H and I. The analysis was carried out using the profile shown in Appendix G, which was provided to us by LEA.

In summary, it is recommended that forward slopes be constructed to 3H:1V slopes for both abutments, including the fill to be used to raise the grade on the west side.

As shown in Figure 4 in Appendix I, the west forward slope should be cut to its final slope after the removal of the surcharge. This is because the relative level portion of the valley slope near the bottom is beneficial for maintaining stability during the surcharge period. The erosion of the filled portion of the forward slope must be prevented during the construction period. This can probably be achieved by prompt seeding depending on the time of year.

5.3 Scour Protection

The Goulais River is actively eroding its shores and large scale slumping and slope failures are evident in the general area.

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 10 m into the river bed, in view of the fact that the river is aggressively eroding its banks. We will be pleased to further discuss these aspects, if you wish us to do so.

5.4 Cut Slopes Between Stations 60+010 and 60+120

At approximately 80 m east of the east abutment of the bridge structure (Station 60+200), the grade of the proposed Goulais Bay Road drops to below the existing ground elevation, starting at about Station 60+120 and continuing easterly by about 110 m to about station 60+010. This will necessitate a cut of about 2 m at Station 60+120, increasing to a maximum of about 4 m at Station 60+040 and decreasing to less than 1 m at about station 60+100, as shown in the cross section drawings provided in Appendix G.

Boreholes 108 and 109 were drilled at stations 60+080 and 60+040 respectively. These boreholes show, below a 0.2 to 0.3 m thick topsoil layer, the presence of very loose to compact sand to a depth of 2.2 to 2.9 m below the ground surface, underlain by loose to compact sandy silt to silty sand to 4.4 m, which is in turn underlain by a sand deposit to 6.0 m depth. The piezometers installed in the boreholes, at the time of the investigation the groundwater level was recorded at a depth of 4.3 to 4.4 m below the ground surface,

but can be expected to be higher at different times of the year, especially since a perched groundwater condition can occur due to the accumulation of surface water during rainy periods/spring thaw in the basically granular (i.e. relatively pervious) soils overlying the silty clay deposit which is known to underlie the site.

Based on the results of slope stability analysis (typical results presented in Appendix K) along with our experience with similar soils, we recommend 2.5H:1V side slopes. We also recommend that a minimum 200 mm thick Granular 'A' drainage layer be placed on the natural cut slope surface, overlain by a 300 mm thick rip-rap layer. The Granular 'A' layer should be free to drain into a ditch or a filtered drain pipe by means of which any collected water can be taken away and suitably discharged.

5.5 Approach Embankments

At present it is expected that the grade at the east abutment location will be essentially the same as the existing grade with minor cuts and fills, but the grade at the west abutment location will be raised by up to 4.0 m above o.g. levels.

Based on the borehole data, no foundation failures are anticipated for up to 5 to 6 m high embankments with normal 2H:1V site slopes, assuming that all organic or otherwise unsuitable materials will be removed as per MTO standards prior to placing the embankment fills (with EPS adjacent to the abutment and beyond without EPS). All organic and otherwise unsuitable materials should be removed within an envelope given by an imaginary slope not steeper than 1H:1V from the toe of the proposed embankment. Based on the available borehole data, for preliminary estimating purposes, on the average, thickness of unsuitable soils to be stripped can be assumed to be 0.3 m. However, the thickness of organic or otherwise unsuitable materials can be expected to be variable in between and beyond borehole locations.

After stripping, the exposed subgrade should be inspected and approved. The approved subgrade should be compacted from the surface using a suitable, heavy 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 under their own weight, prepared as described above, should not exceed 25 mm for embankment heights of up to about 4.0 m. This settlement is additional to foundation level settlement as discussed in section 5.1.3. The time-rate of settlement will depend on the materials used. For example, granular soils will settle more rapidly than finer soils. With the proposed surcharging, a significant portion of this 25 mm settlement can be expected to take place prior to the paving of the road.

In addition to settlement under self weight, considerable foundation settlements can be expected under the weight of the approach fills to be placed at the west abutment location. This was discussed in some detail in section 5.1.3 of this report.

5.6 Construction Comments

It is anticipated that the bulk of construction, including stripping operations for embankment construction will take place in sand or silty sand to sandy silt soils above the groundwater level and therefore, no major problems are foreseen during earthworks due to groundwater. Surcharge construction on the west side, including the cross sections for the embankments during the surcharging, was discussed in earlier sections. As mentioned before, erosion of the embankments, including the forward slopes, must be prevented during the surcharging period. Any cutting of the west slope near the toe should only be implemented after the surcharging period was over and the surcharge was removed.

Should wick drains be employed further assessment of the influence of elevated piezometric levels in the sand and gravels below about El. 145 m will be required.

The construction of the central pier foundations in the river will require a suitable scheme for constructability and to properly guide the piles to be driven. For example, a partially submerged floating steel form with CSP inserts (for guiding the piles to be driven) has been used on another project. In this respect, MTO experience includes the use of large diameter steel tube piles which are driven a short but sufficient distance into the river bed to provide proper guide in driving any battered piles. As was mentioned before, the batter should be minimized for ease in driving the piles.

5.7 Frost Protection

Design frost protection depth for the general area is about 1.9 m. Therefore, a permanent soil cover of about 1.9 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 O, are integral part of this report.

For and on behalf of Coffey Geotechnics Inc.


Ramon Miranda, P.Eng.



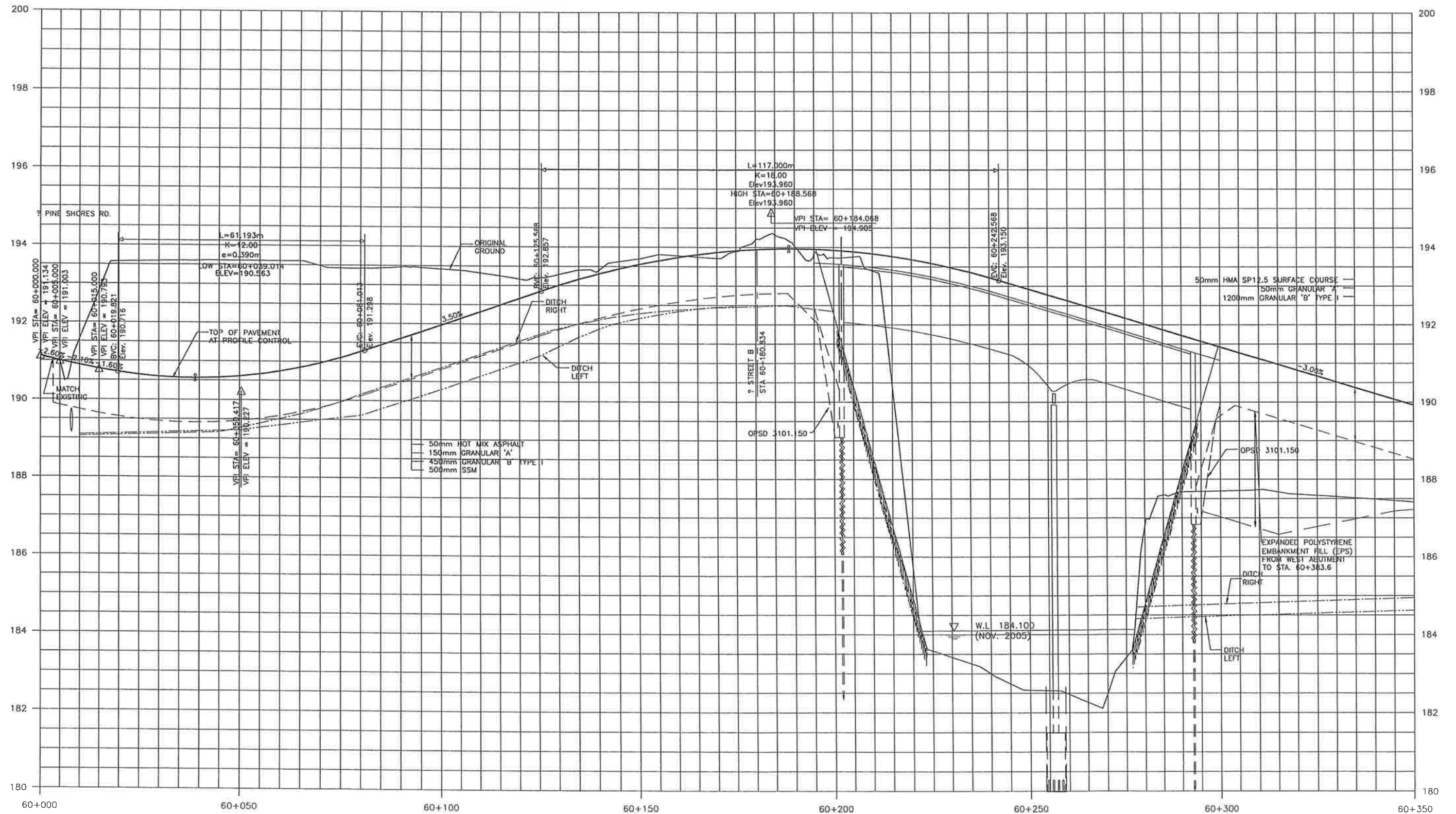

Zuhtu Ozden, P.Eng.



Appendix G

Profile and Cross-Sections of the Goulais River Road

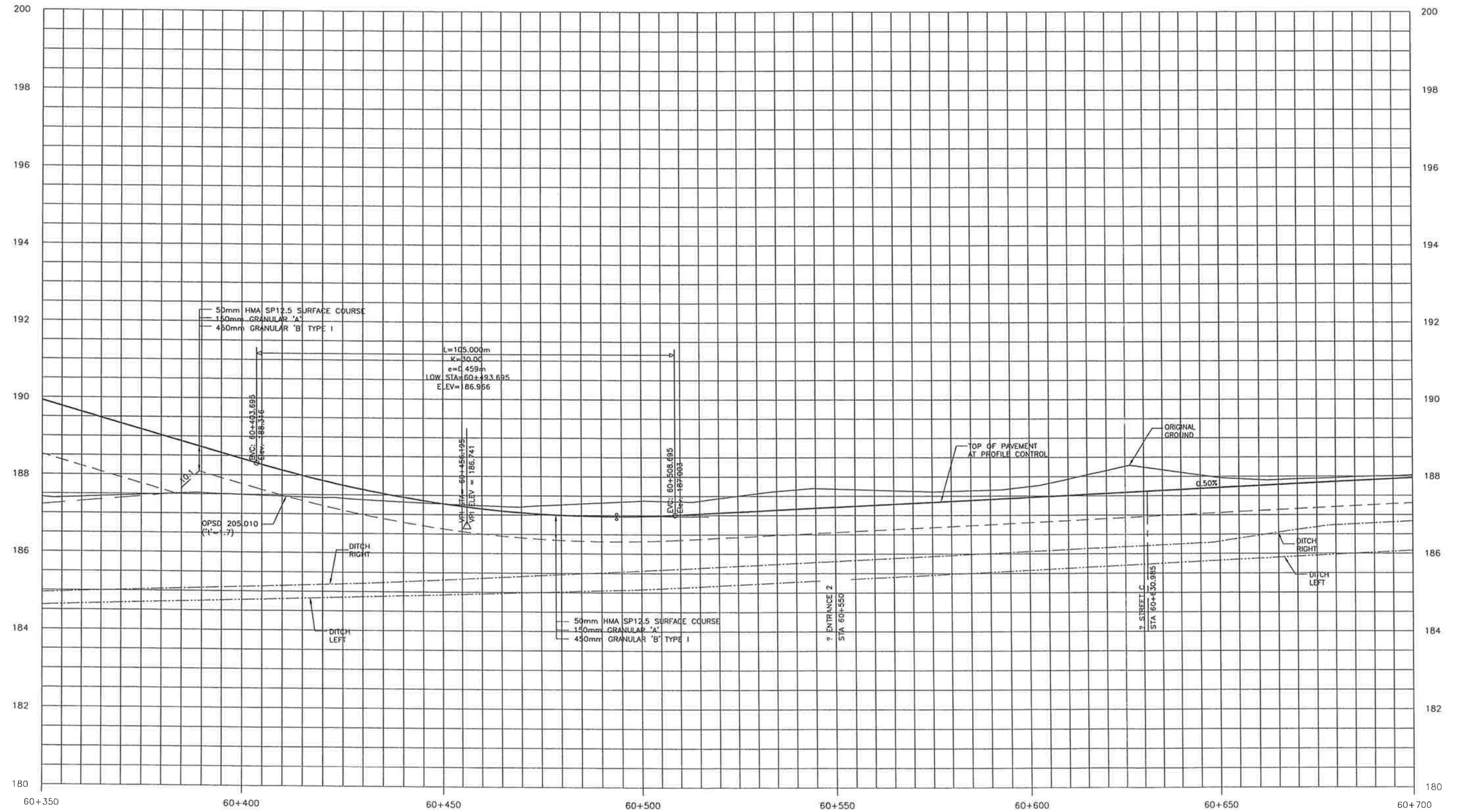
PROFILE GOULAIS BAY ROAD STA. 60+000 TO STA. 60+350



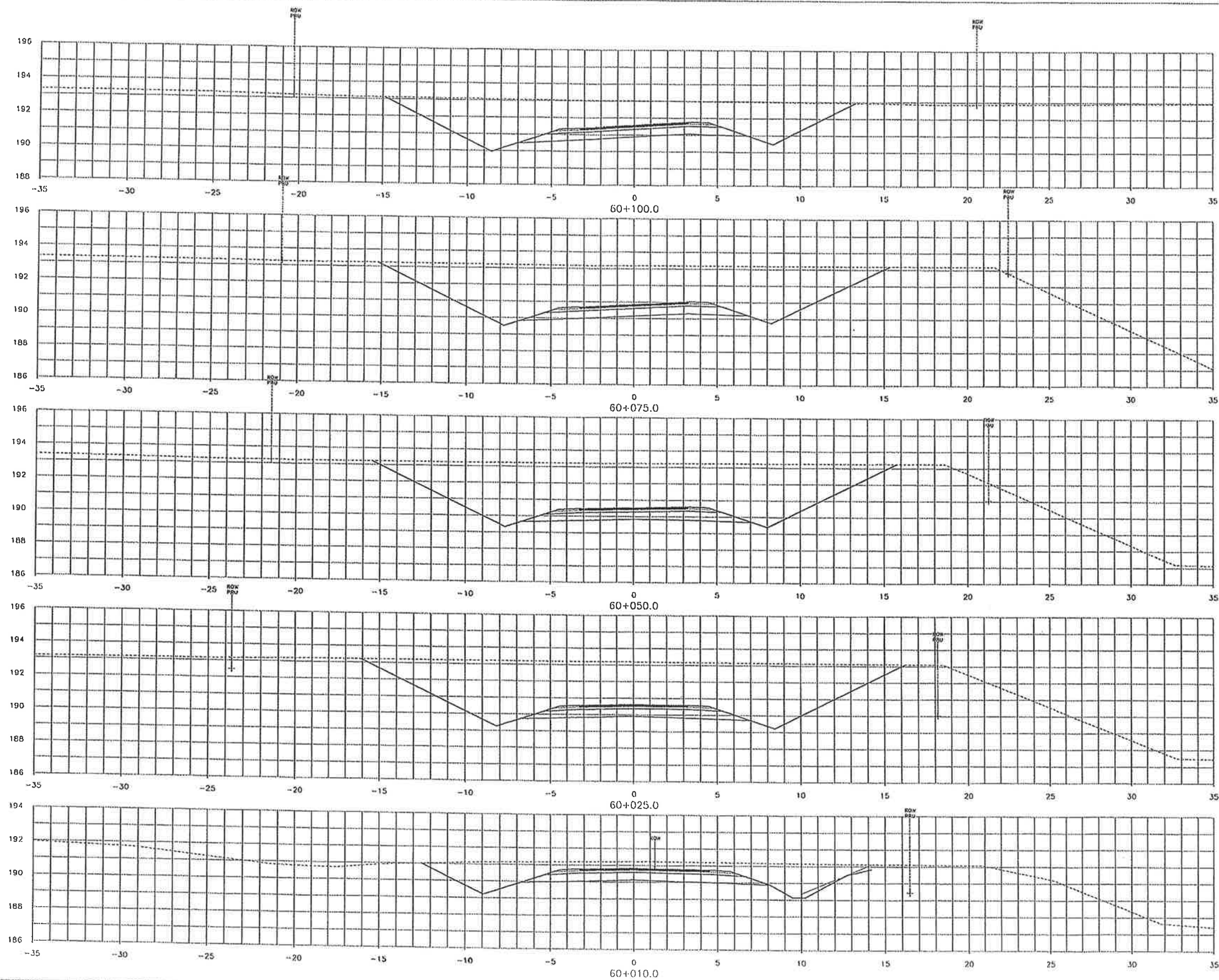
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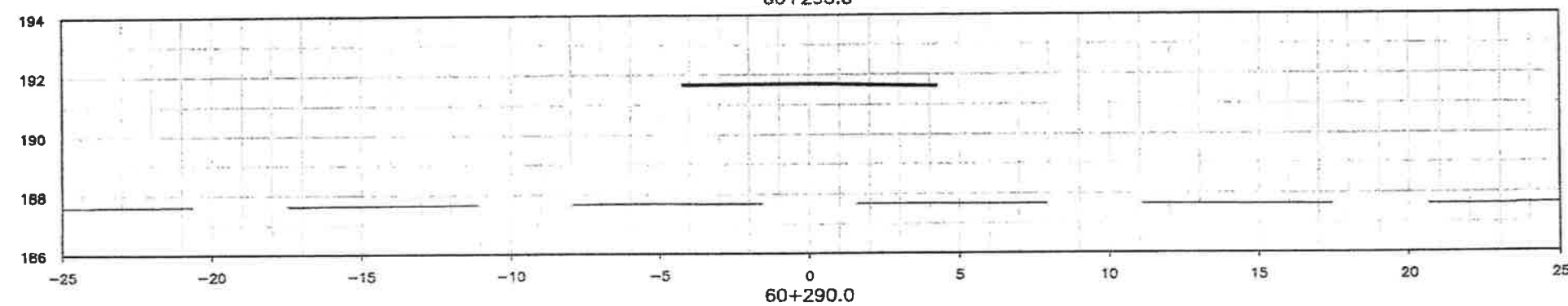
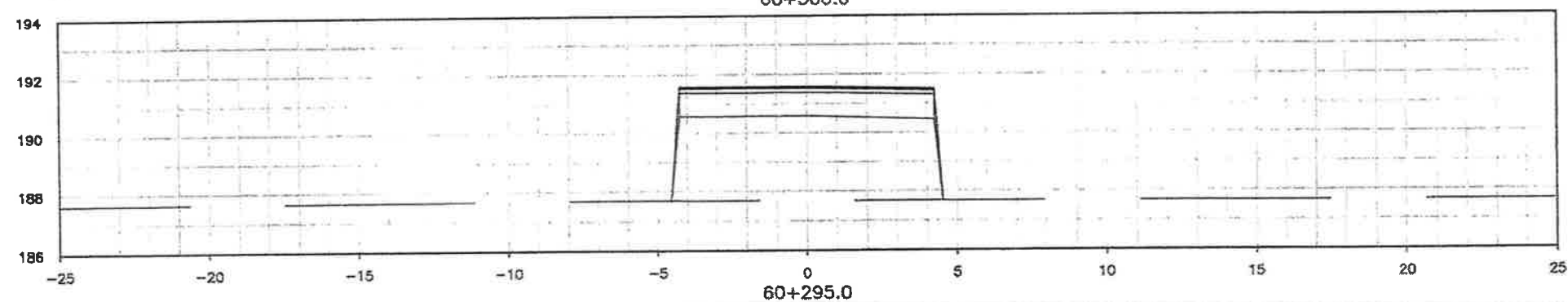
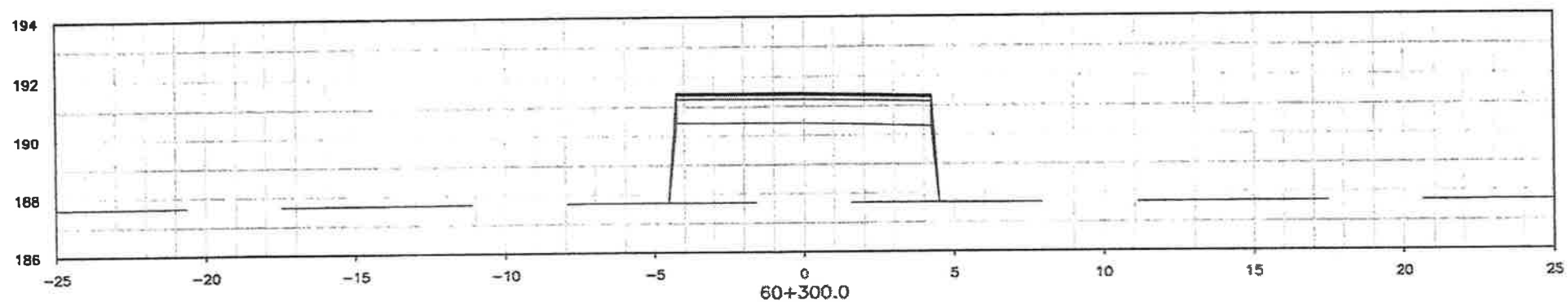
GOULAIS BAY ROAD

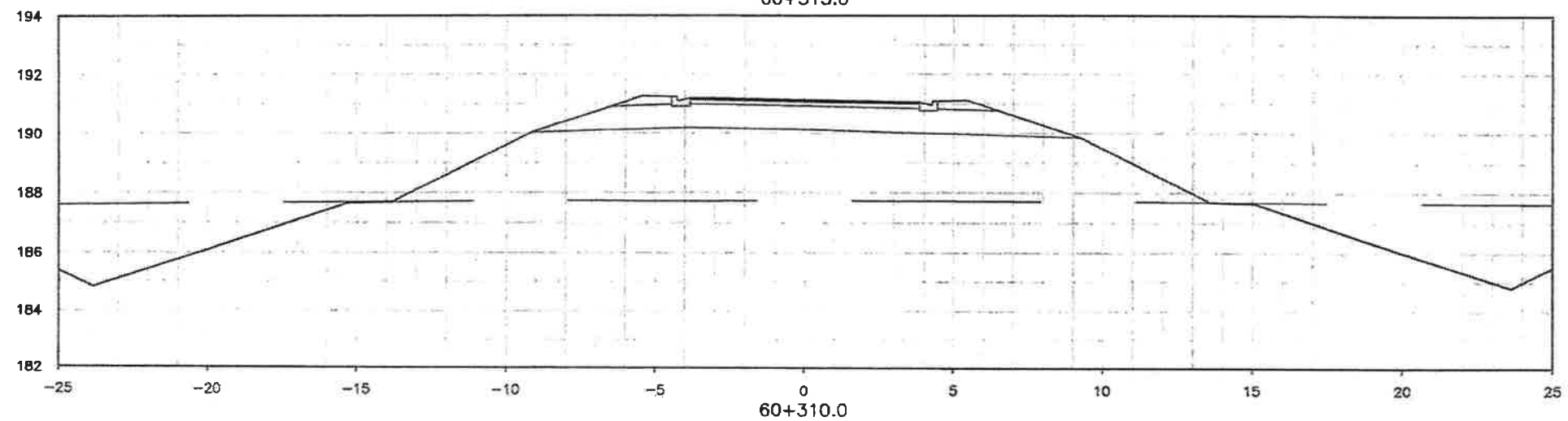
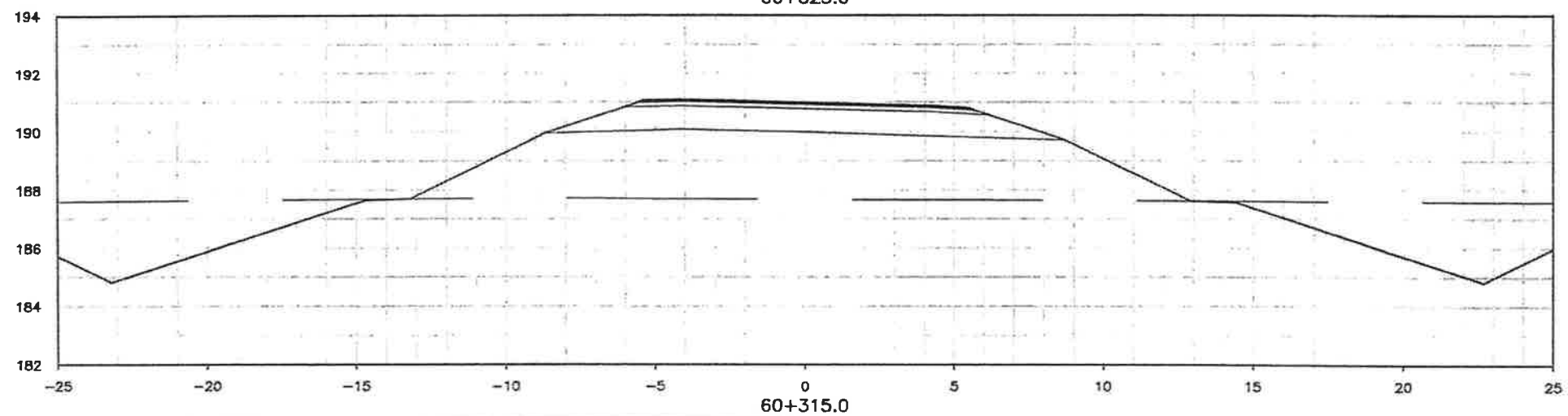
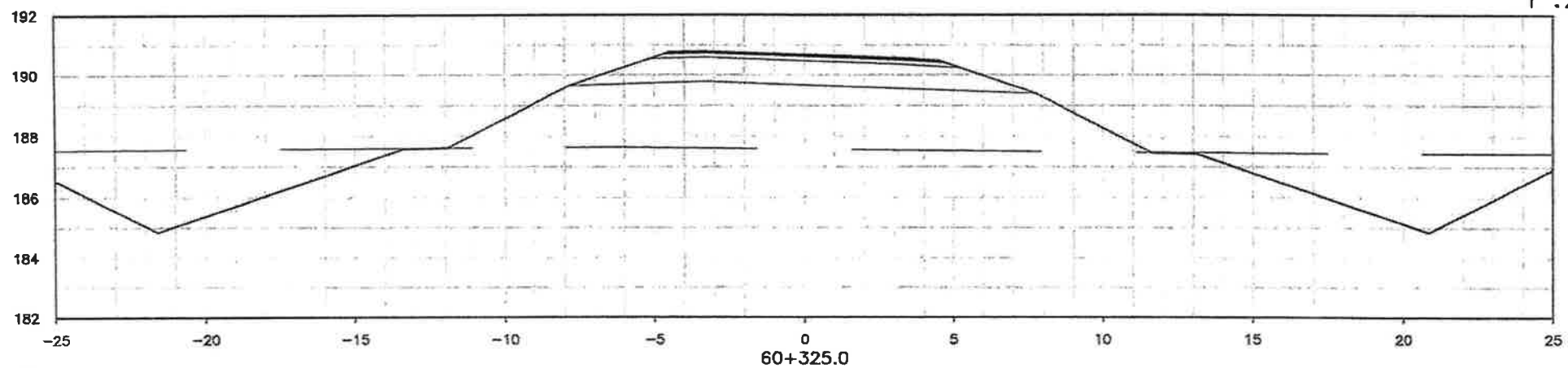
STA. 60+350 TO STA. 60+700

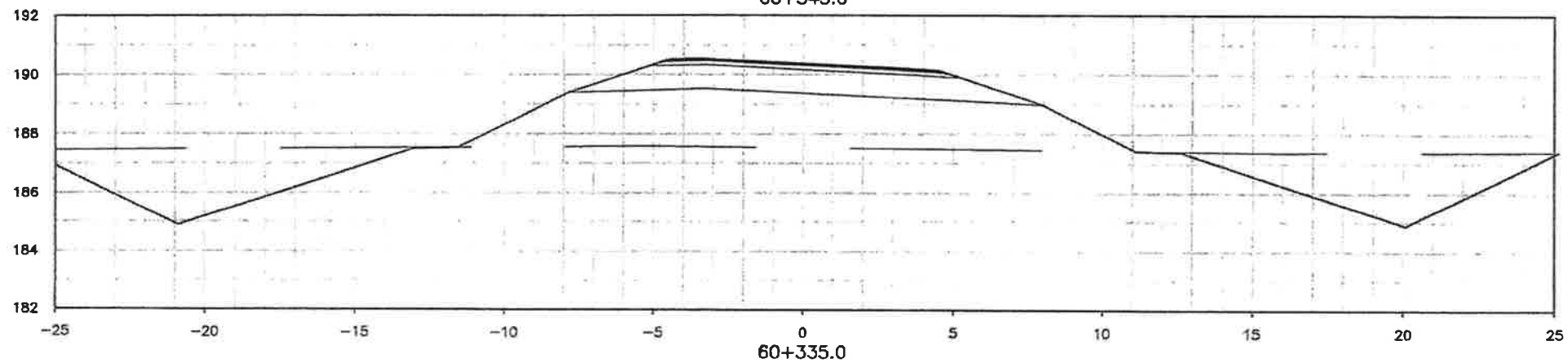
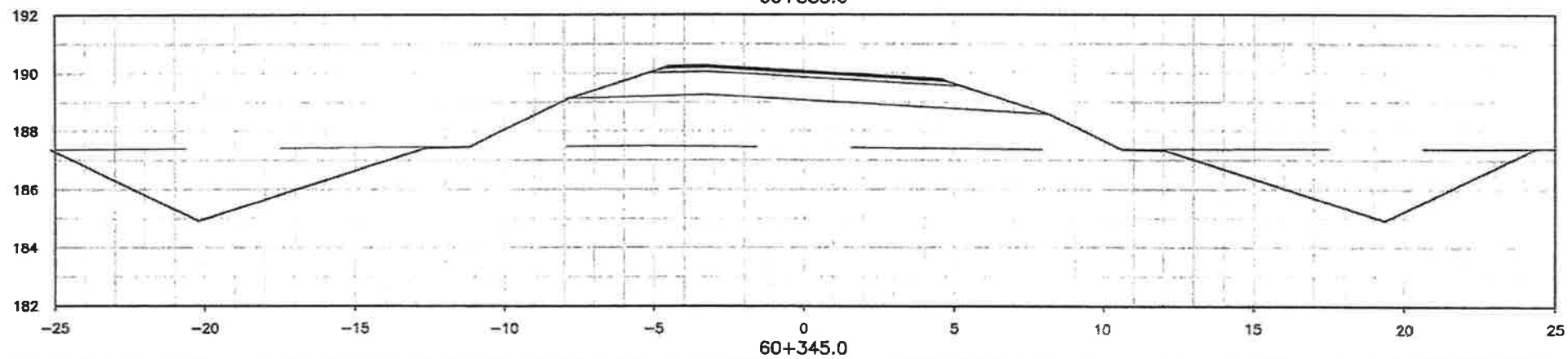
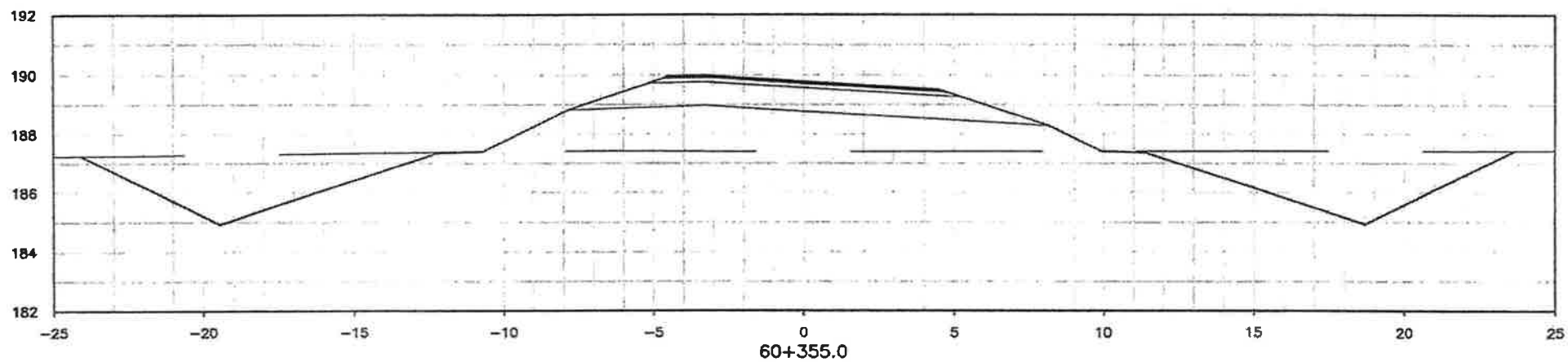


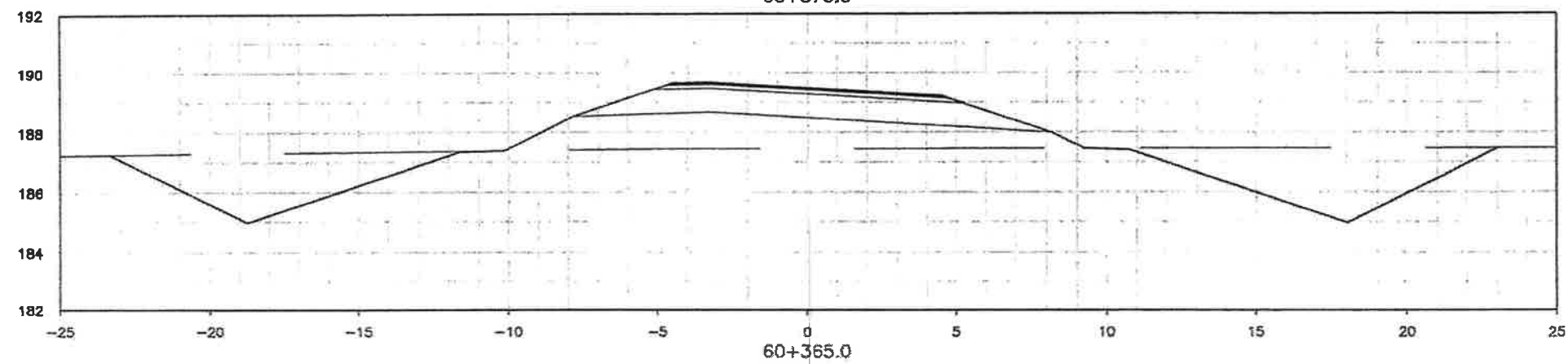
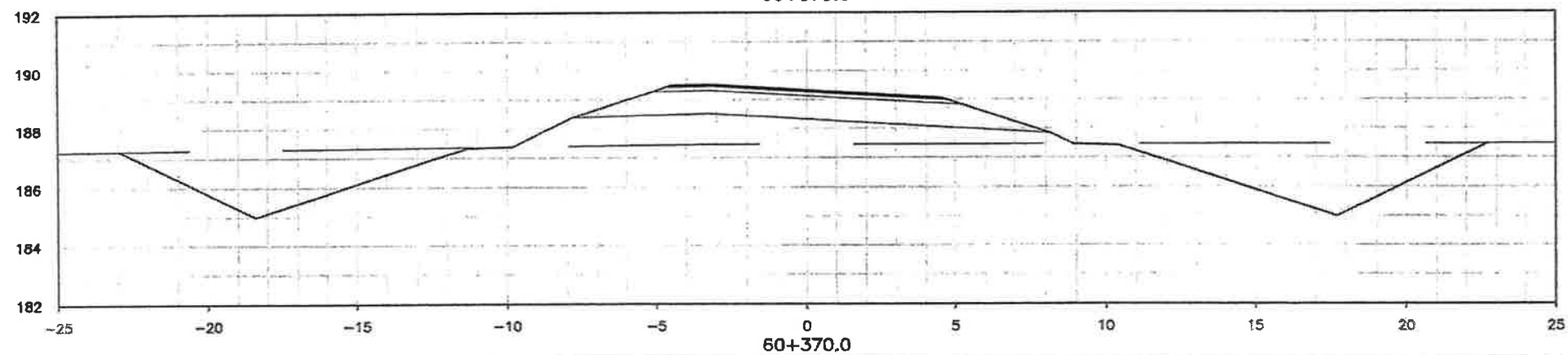
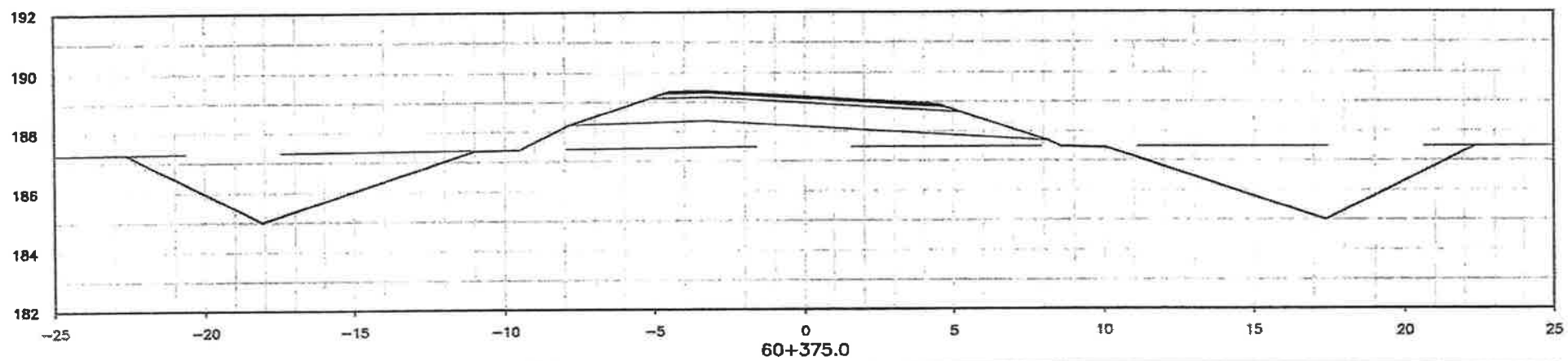
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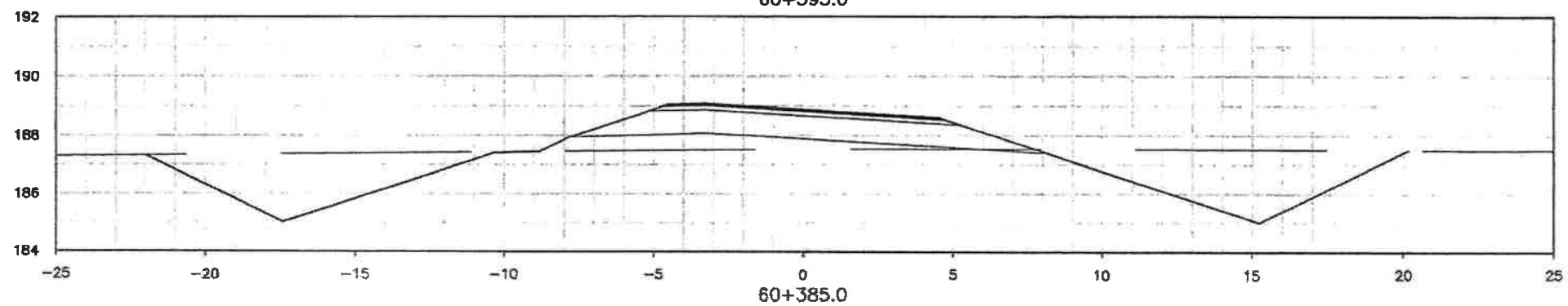
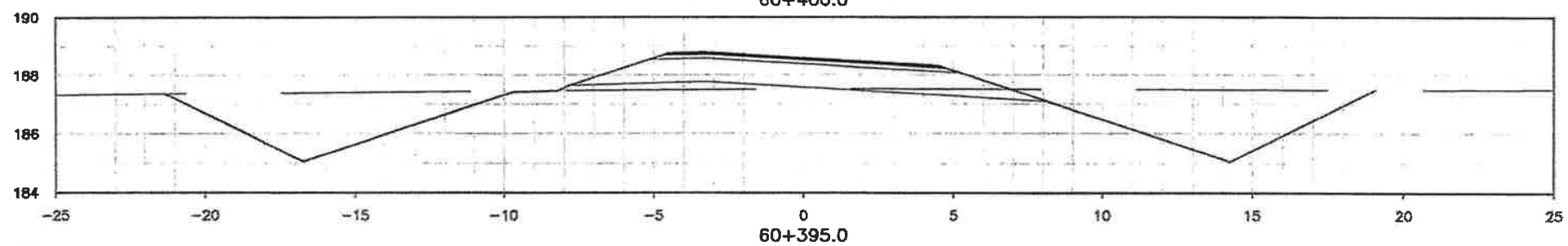
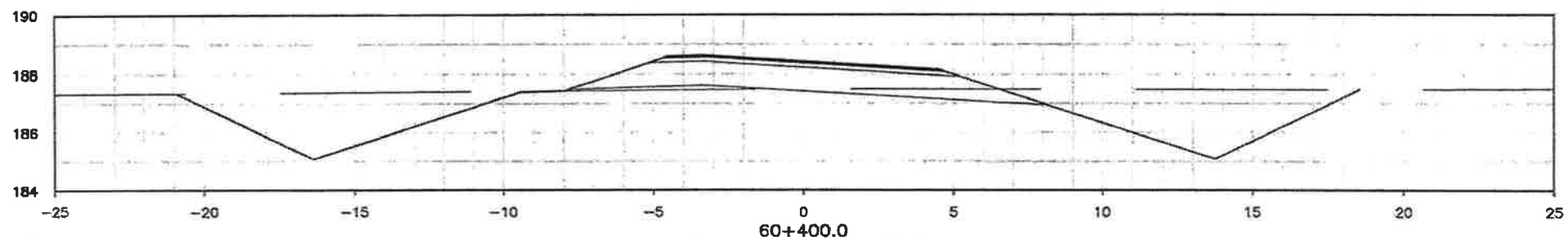






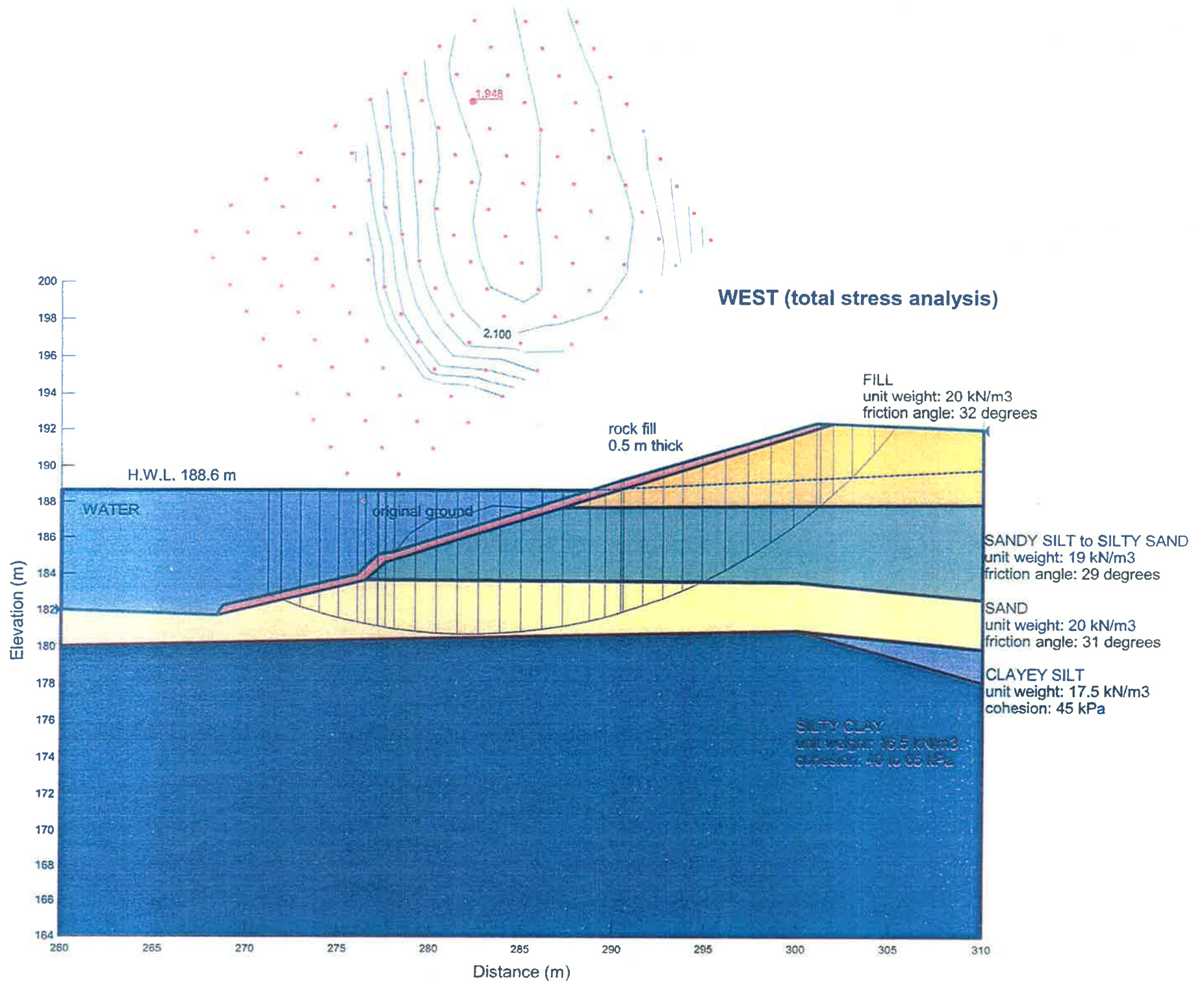


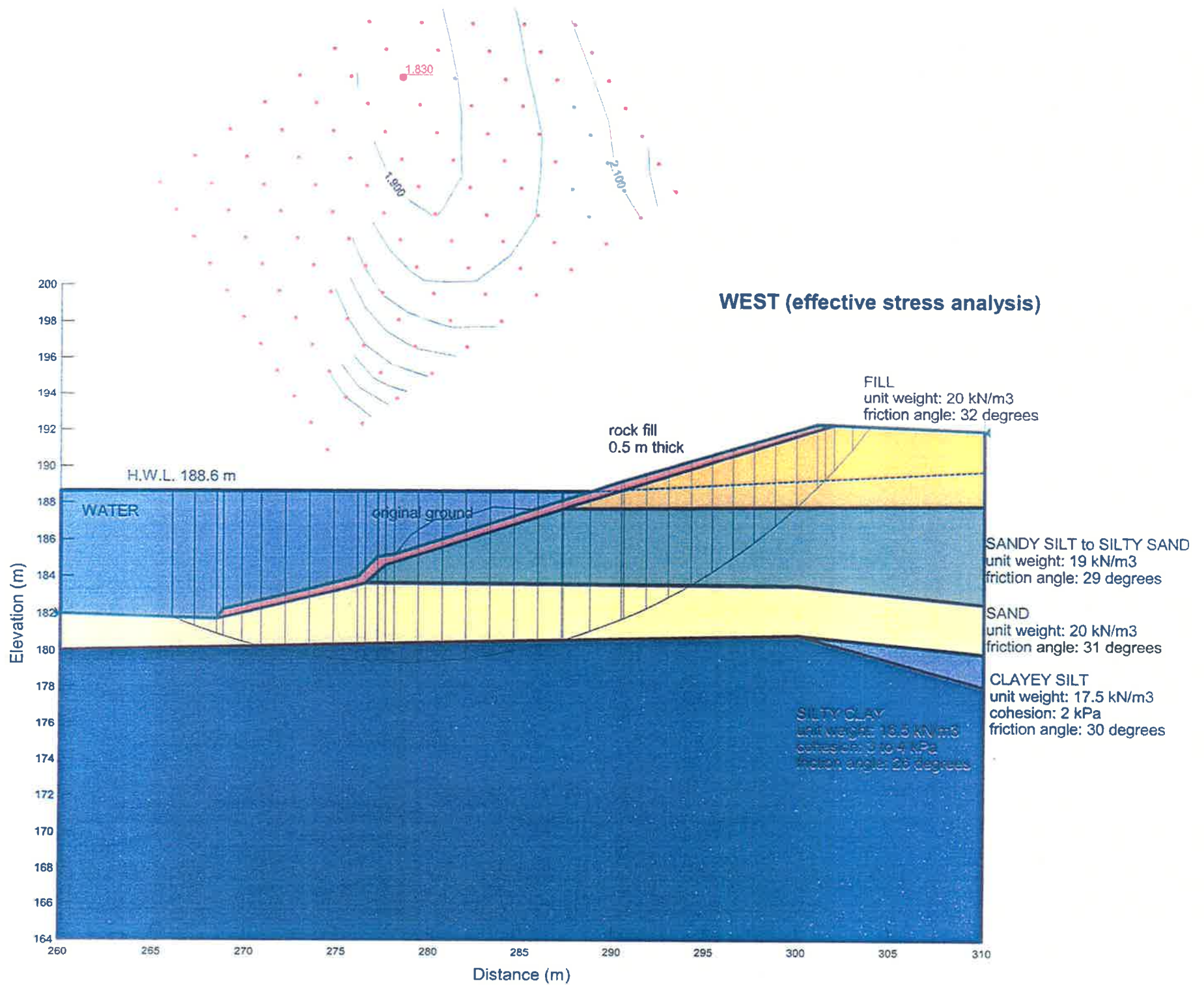


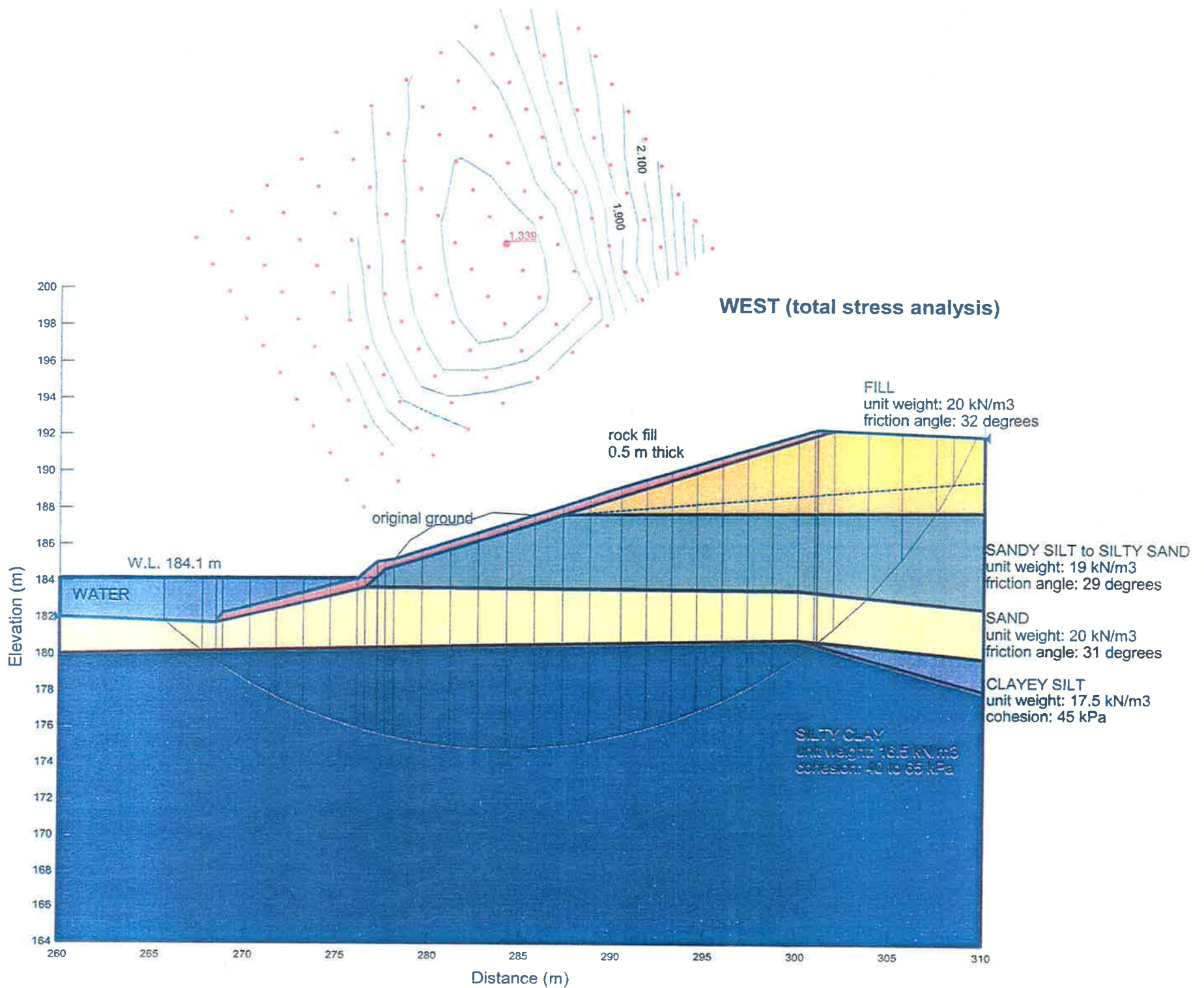


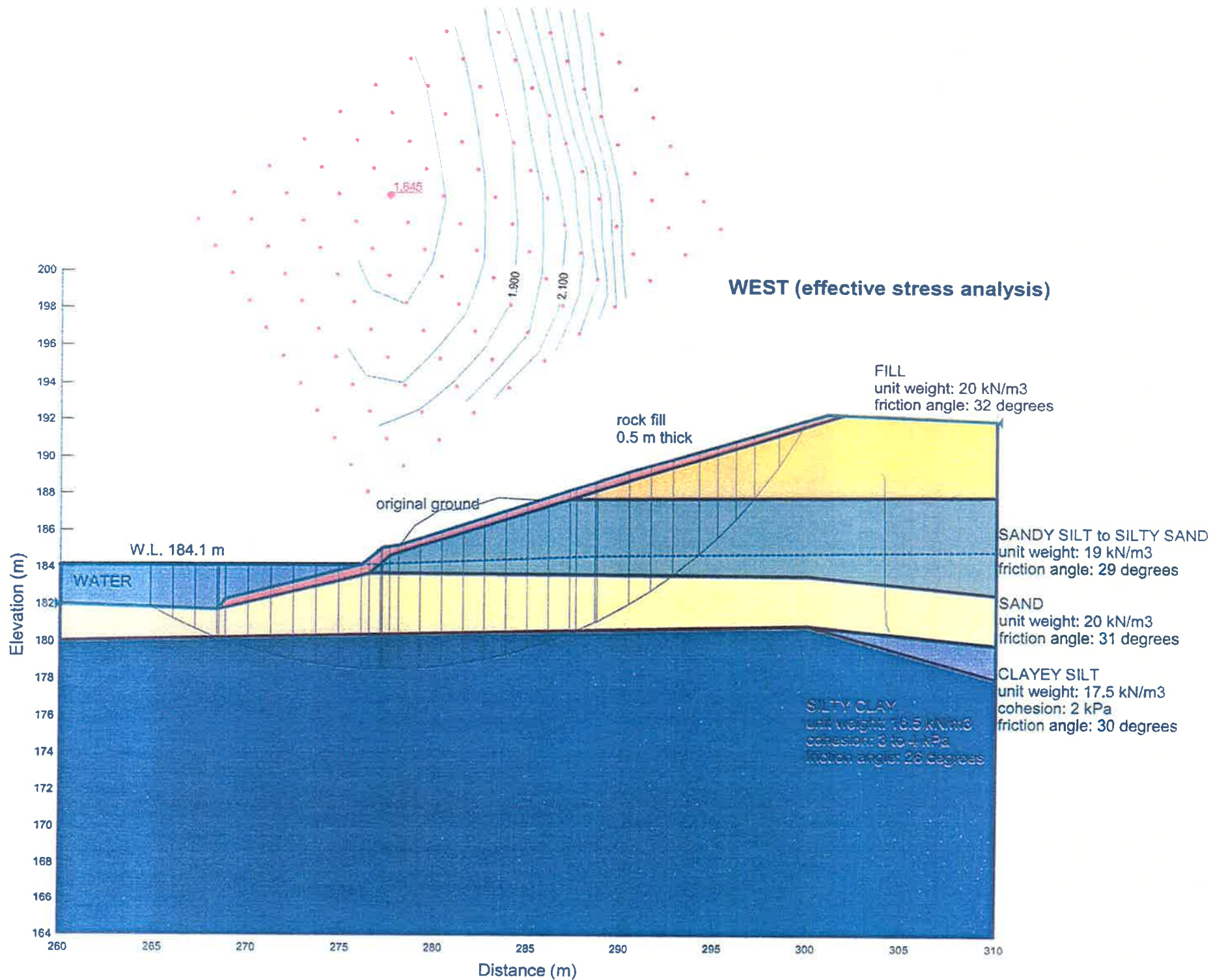
Appendix H

Typical Stability Analyses (Bridge Section)









EAST (total stress analysis)

SANDY SILT to SILTY SAND
unit weight: 20 kN/m³
friction angle: 29 degrees

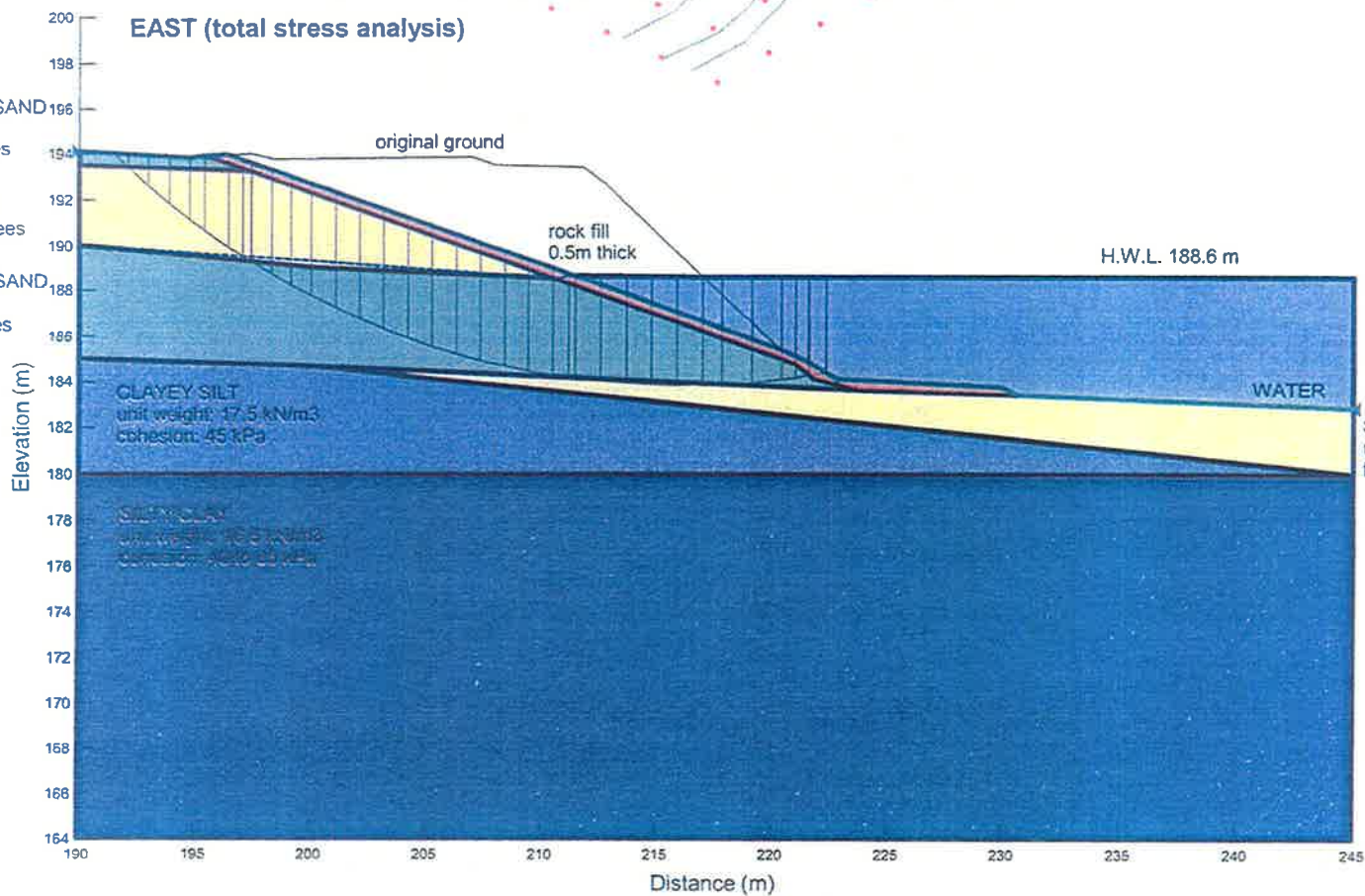
SAND
unit weight: 20 kN/m³
friction angle: 30 degrees

SANDY SILT to SILTY SAND
unit weight: 19 kN/m³
friction angle: 29 degrees

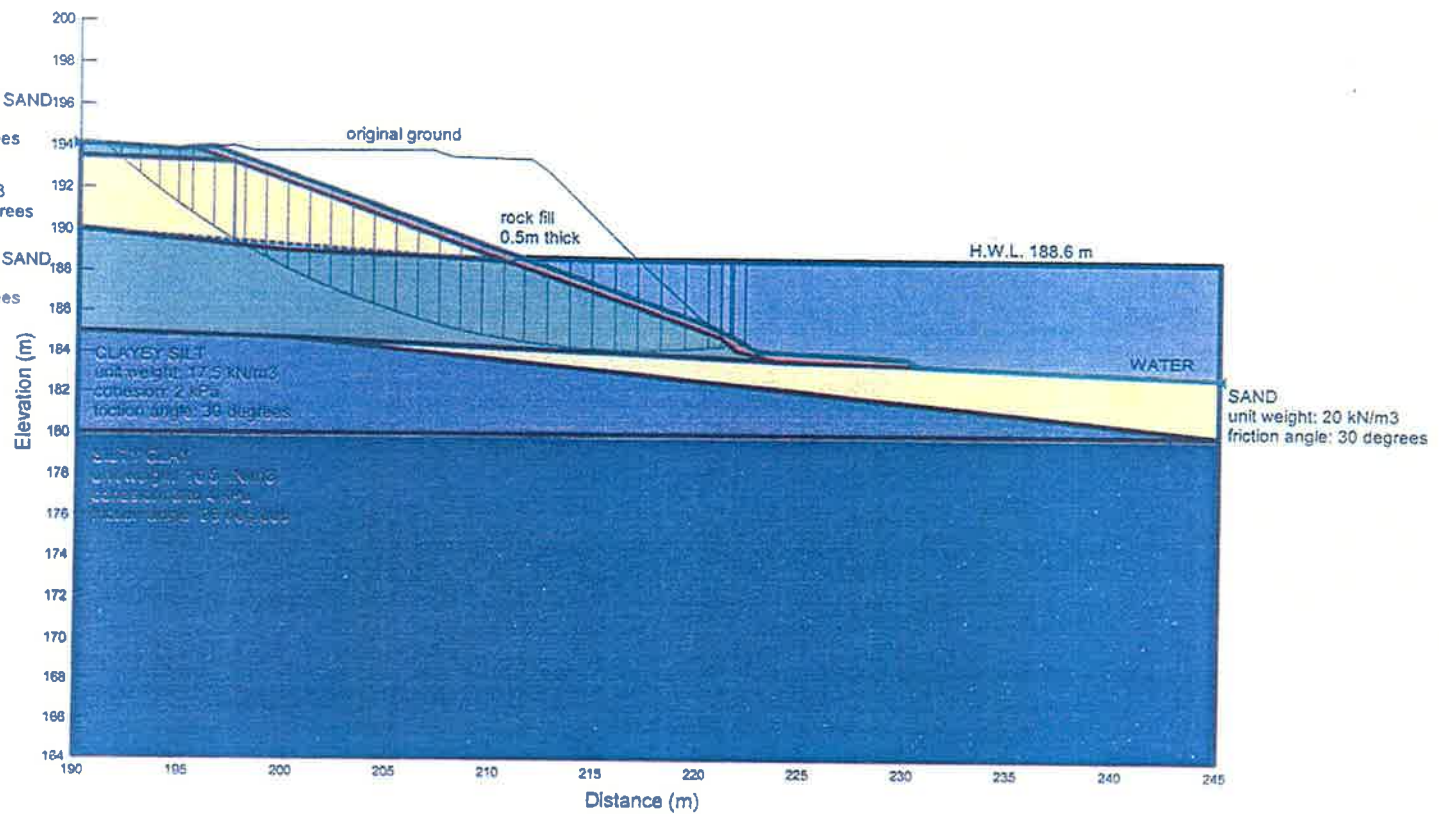
CLAYEY SILT
unit weight: 17.5 kN/m³
cohesion: 45 kPa

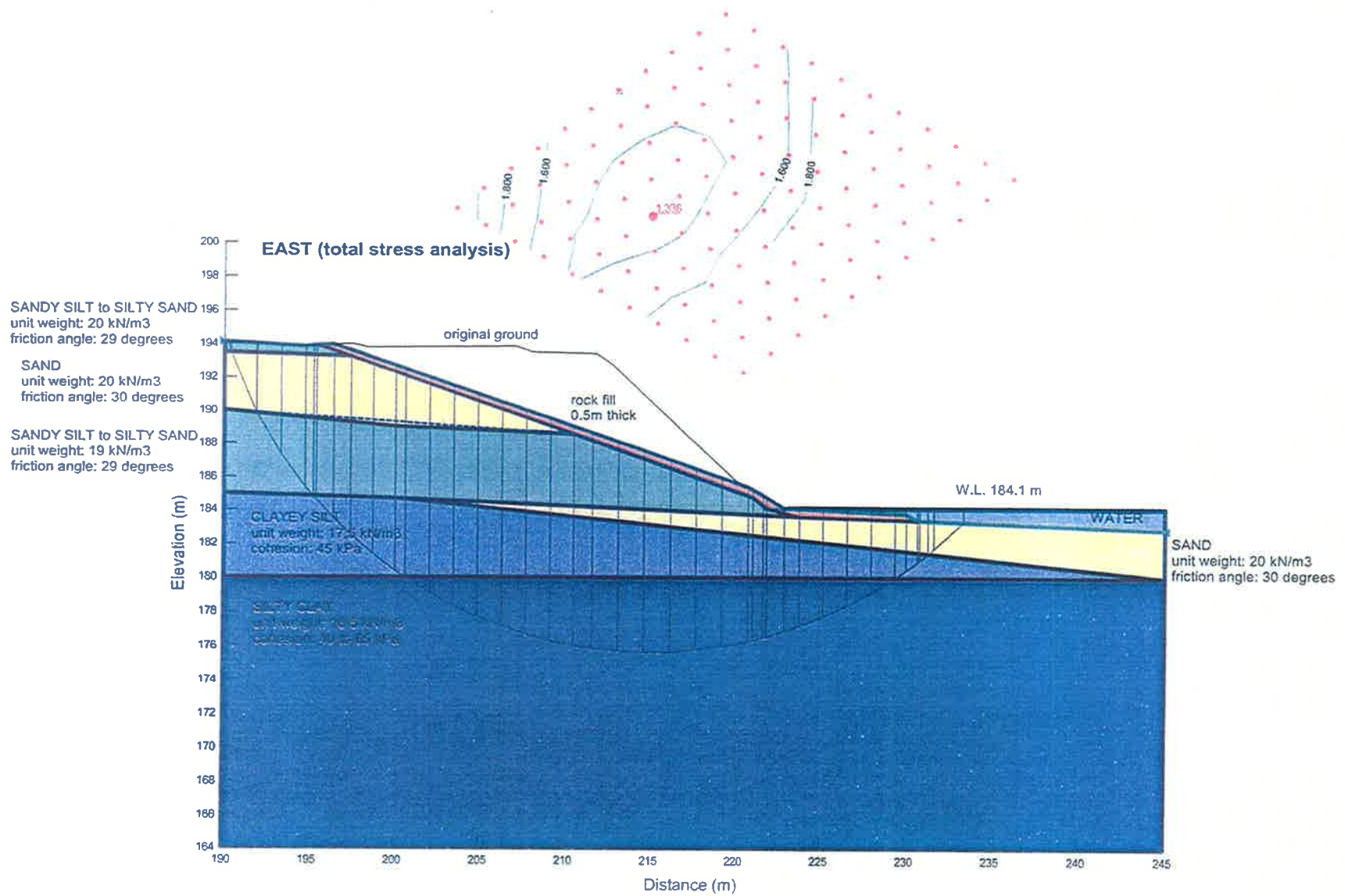
SILTY CLAY
unit weight: 16.5 kN/m³
cohesion: 40 kPa

SAND
unit weight: 20 kN/m³
friction angle: 30 degrees

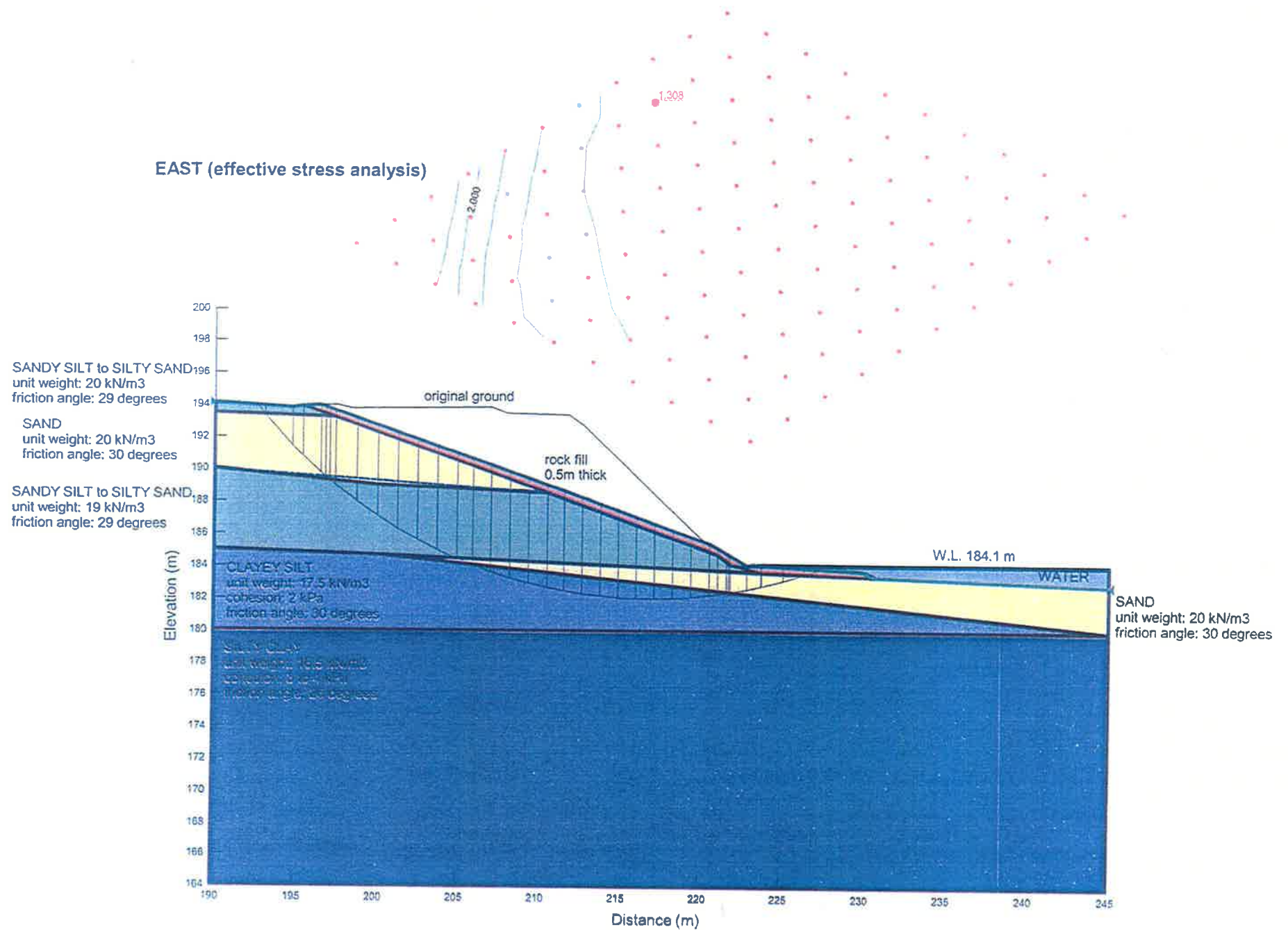


EAST (effective stress analysis)



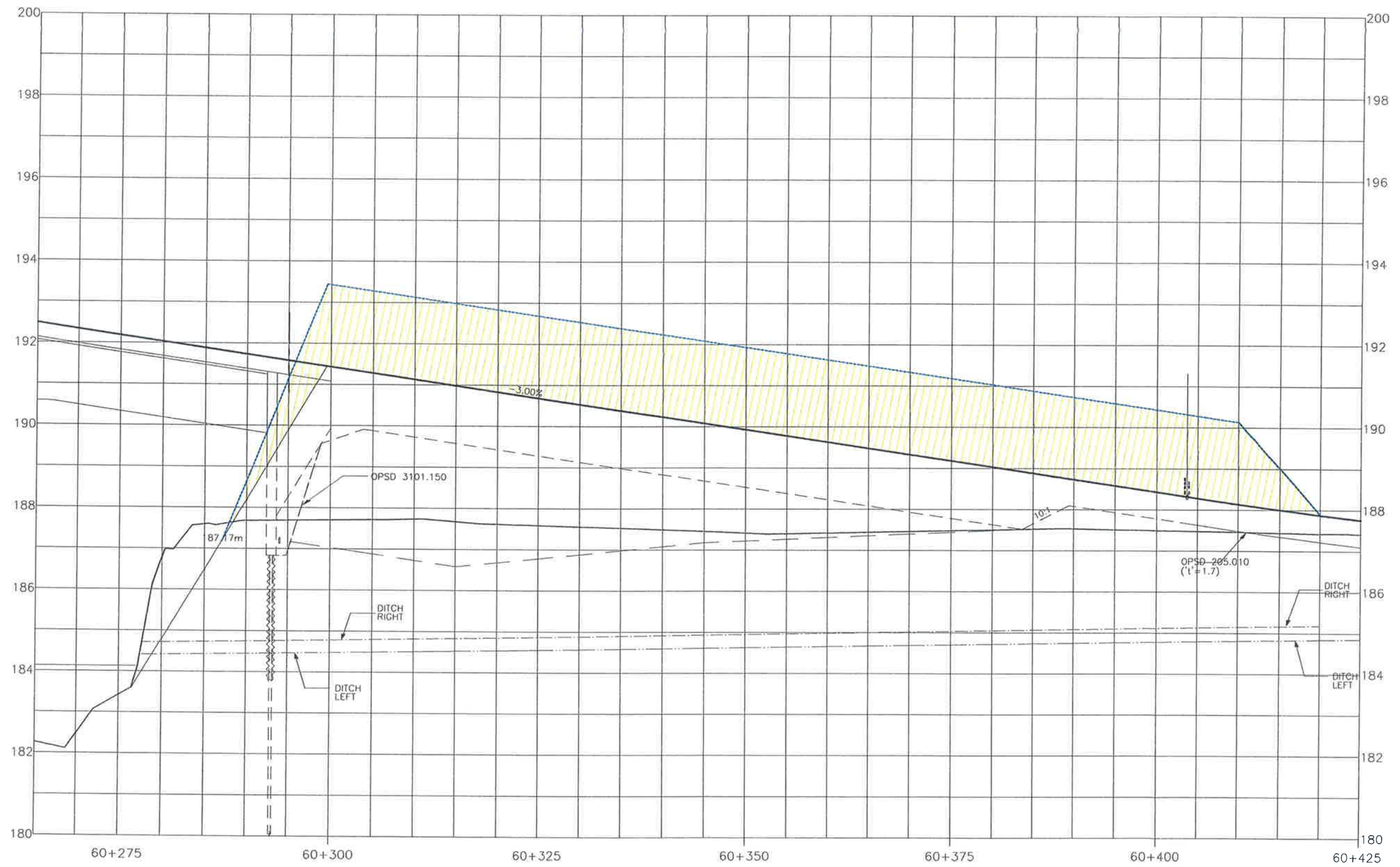


EAST (effective stress analysis)



Appendix I

Surcharge Plan and Stability Analyses



SURCHARGE PLAN
Figure 1

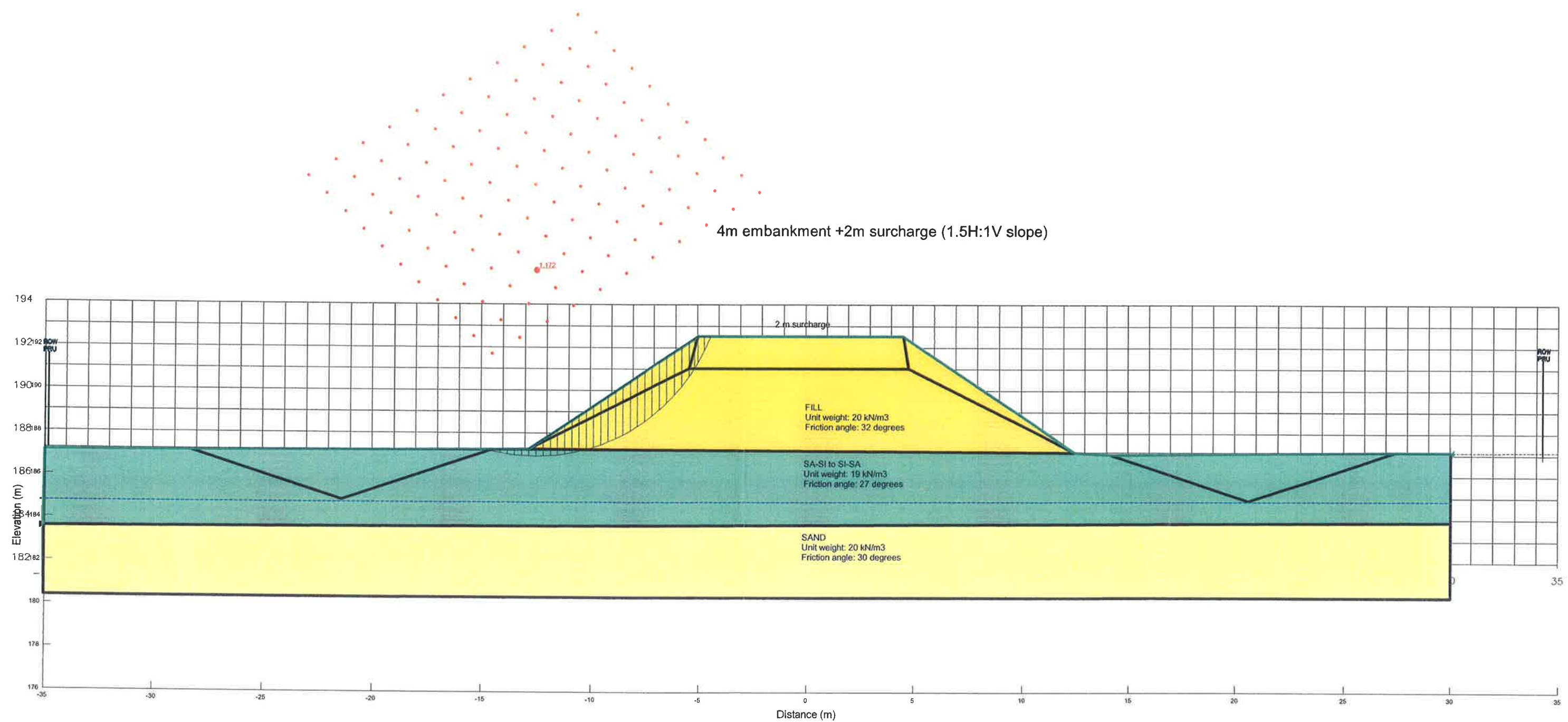


Figure 2. 1.5H:1V slope (2 m surcharge)

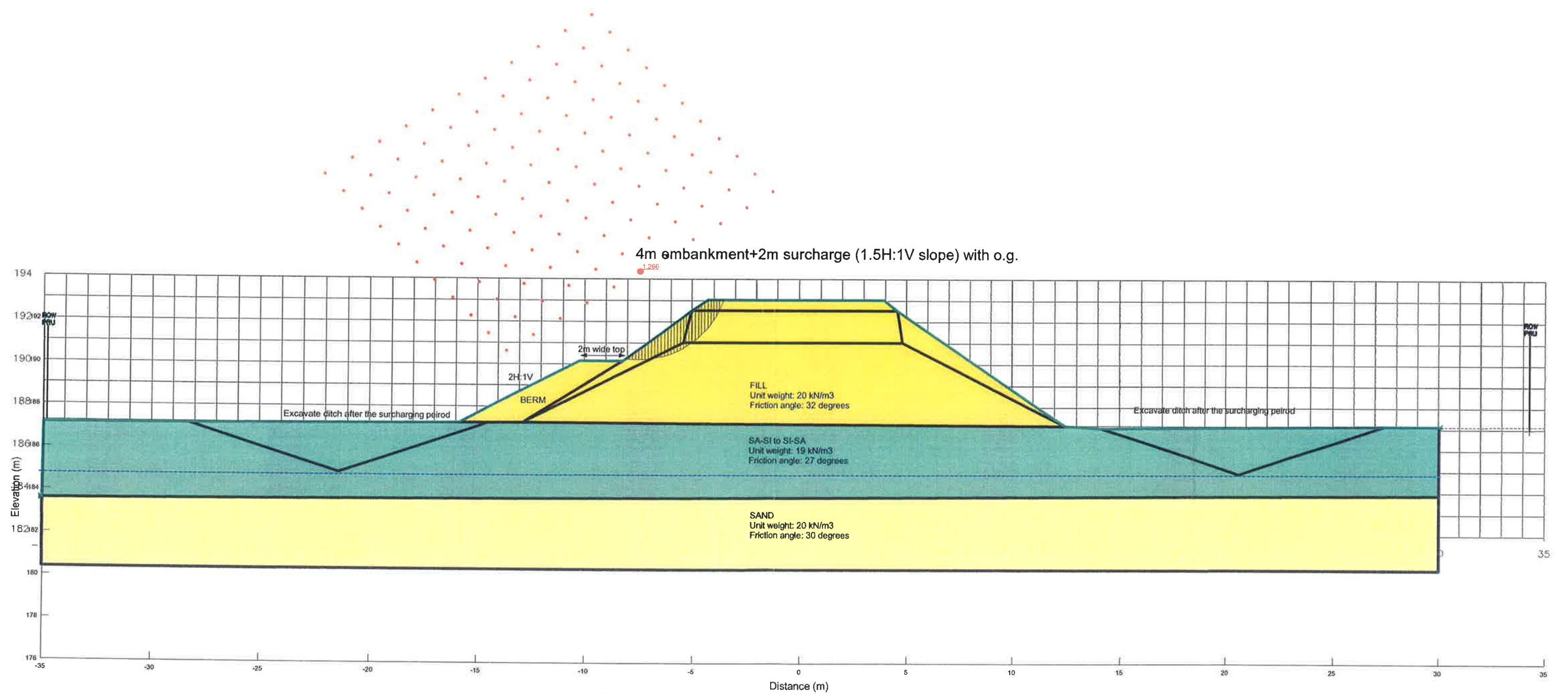


Figure 3. 1.5H:1V slope (2 m surcharge with berm)

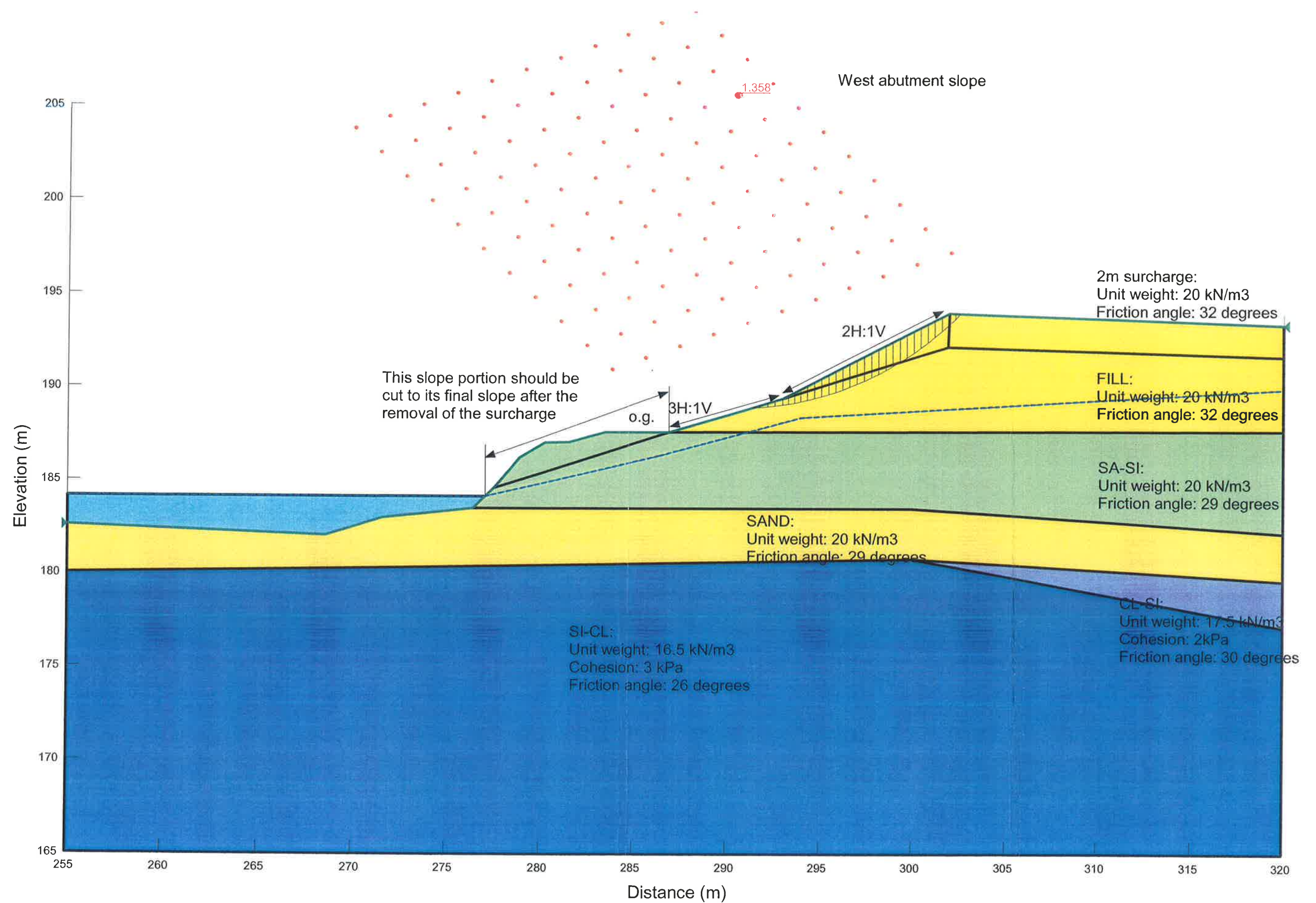


Figure 4. Stability of the forward slope on the west side during surcharging

Appendix J

Typical Settlement Analyses (Surcharge Section, STA 60+300 to 60+420)

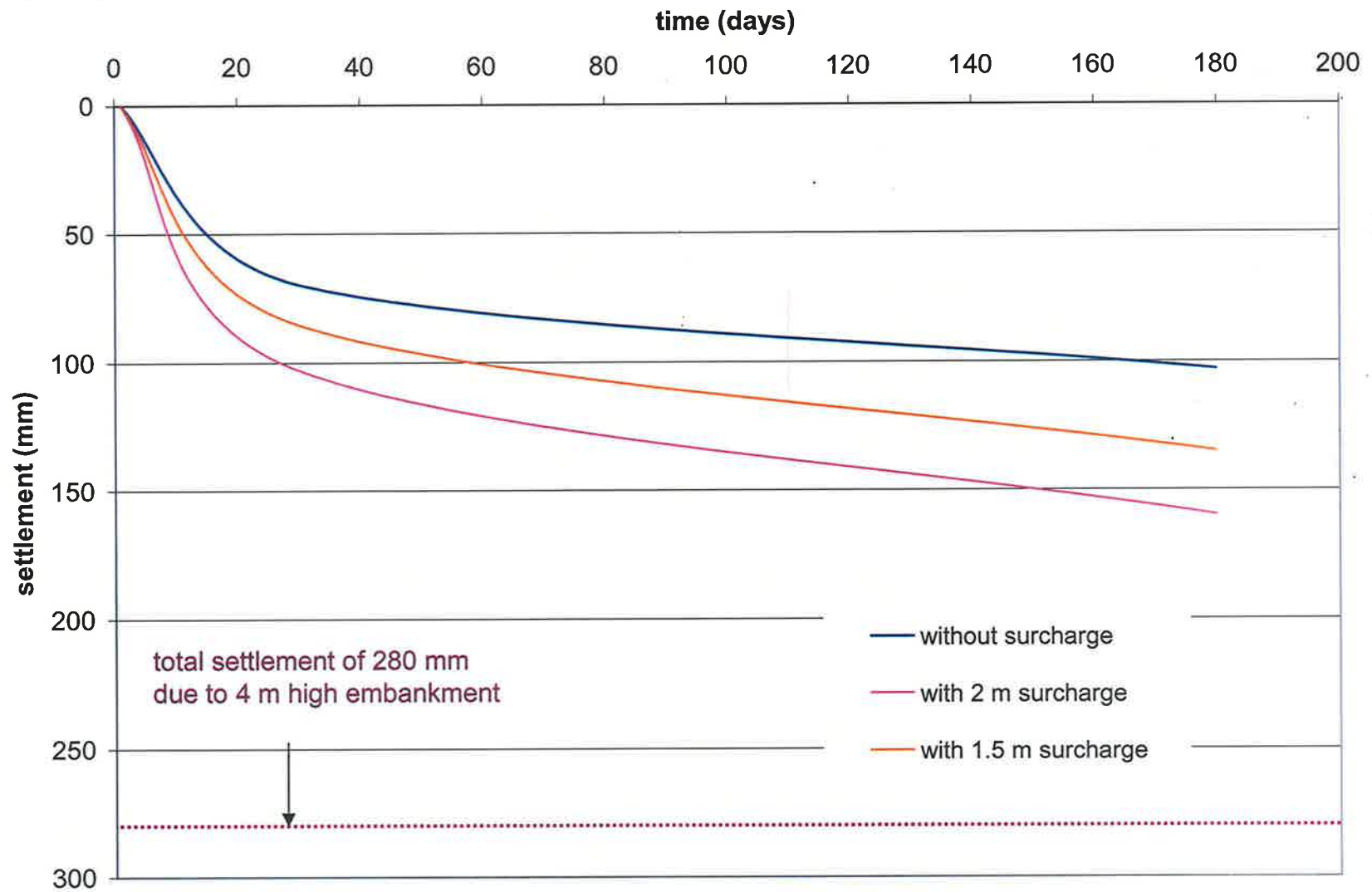


Figure1. Settlement of 4m high embankment with surcharge (adjacent to west abutment)

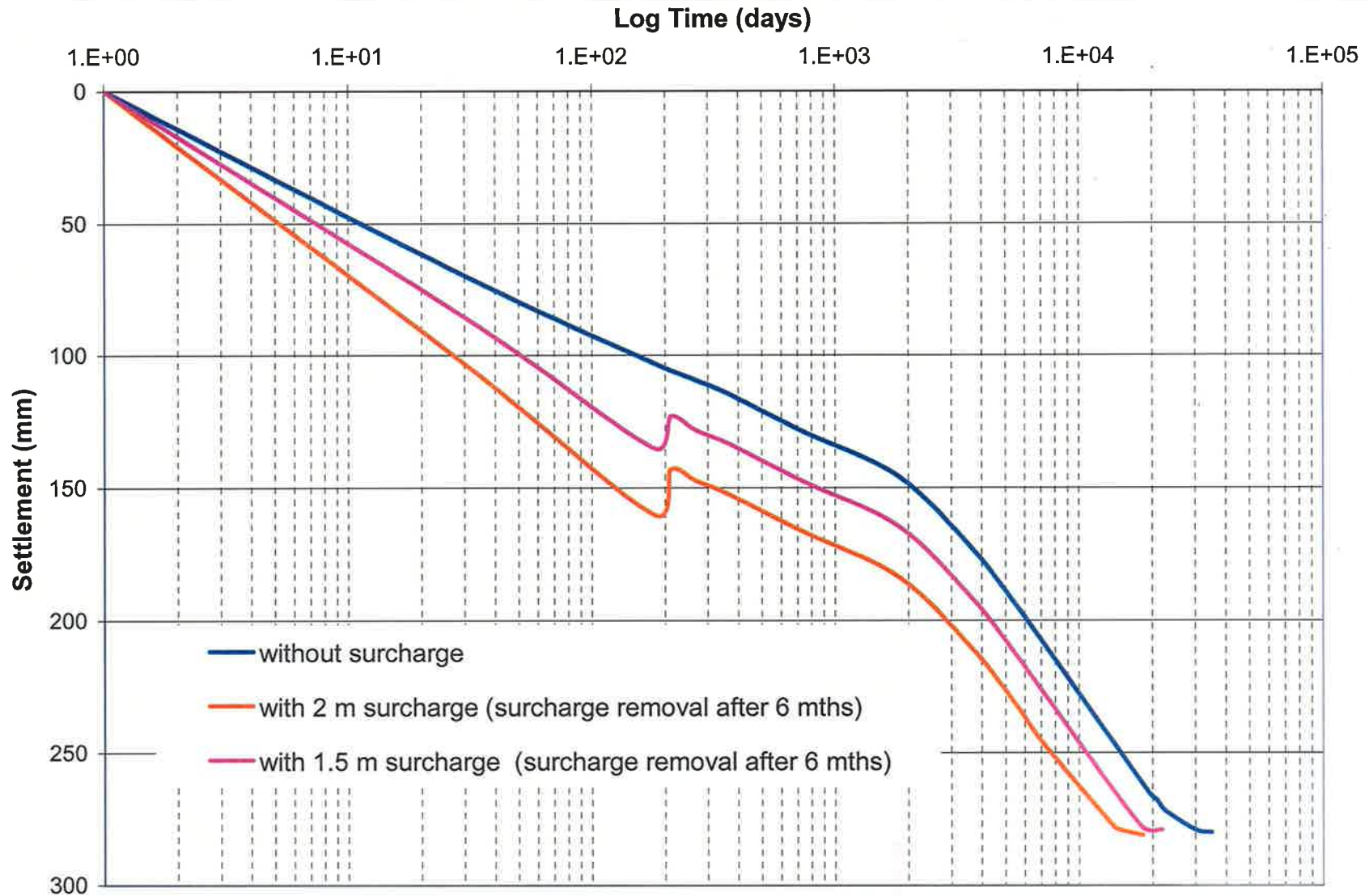


Figure 2. Settlement of 4m high embankment with surcharge (adjacent to west abutment)

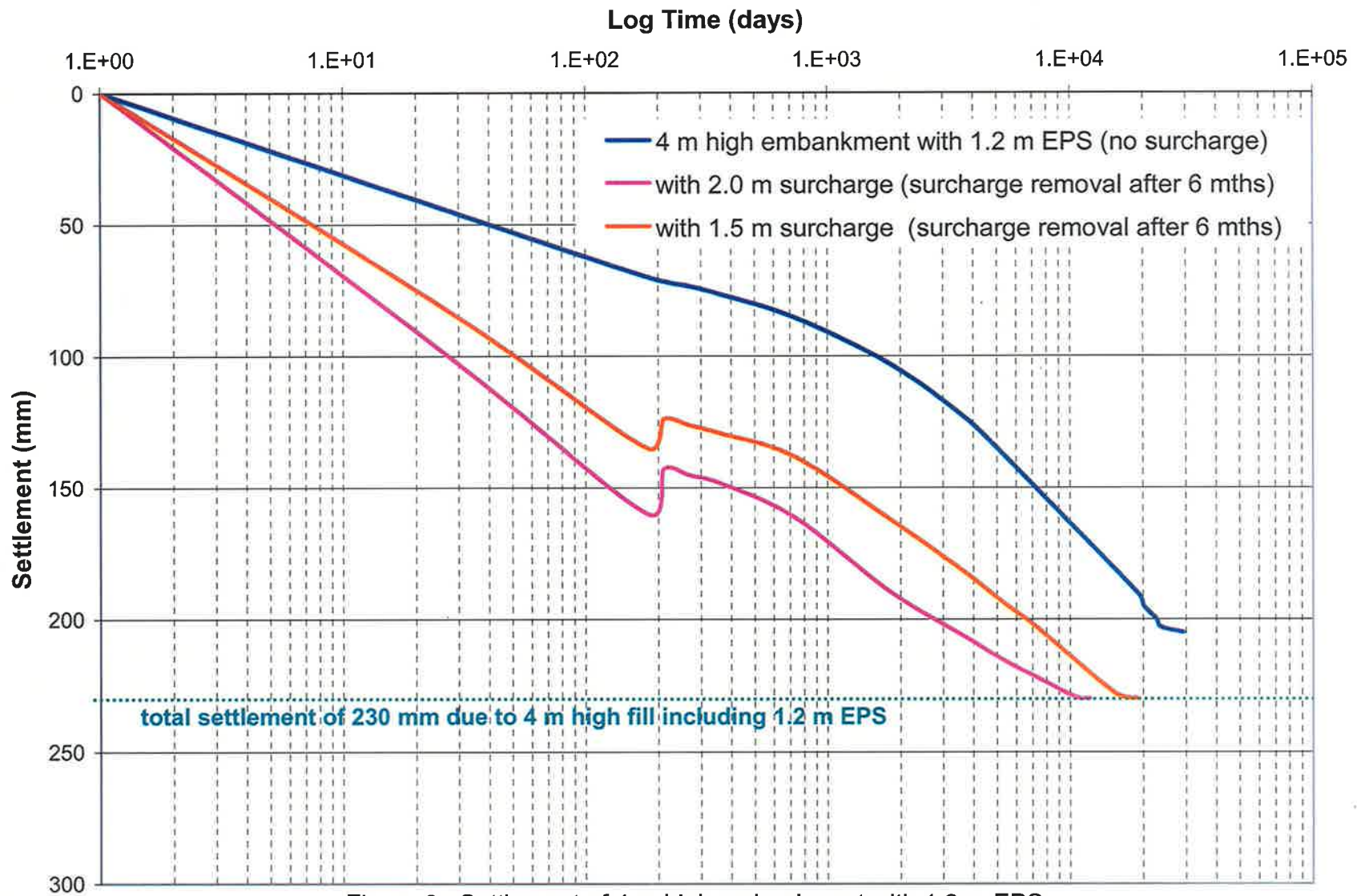


Figure 3. Settlement of 4 m high embankment with 1.2 m EPS

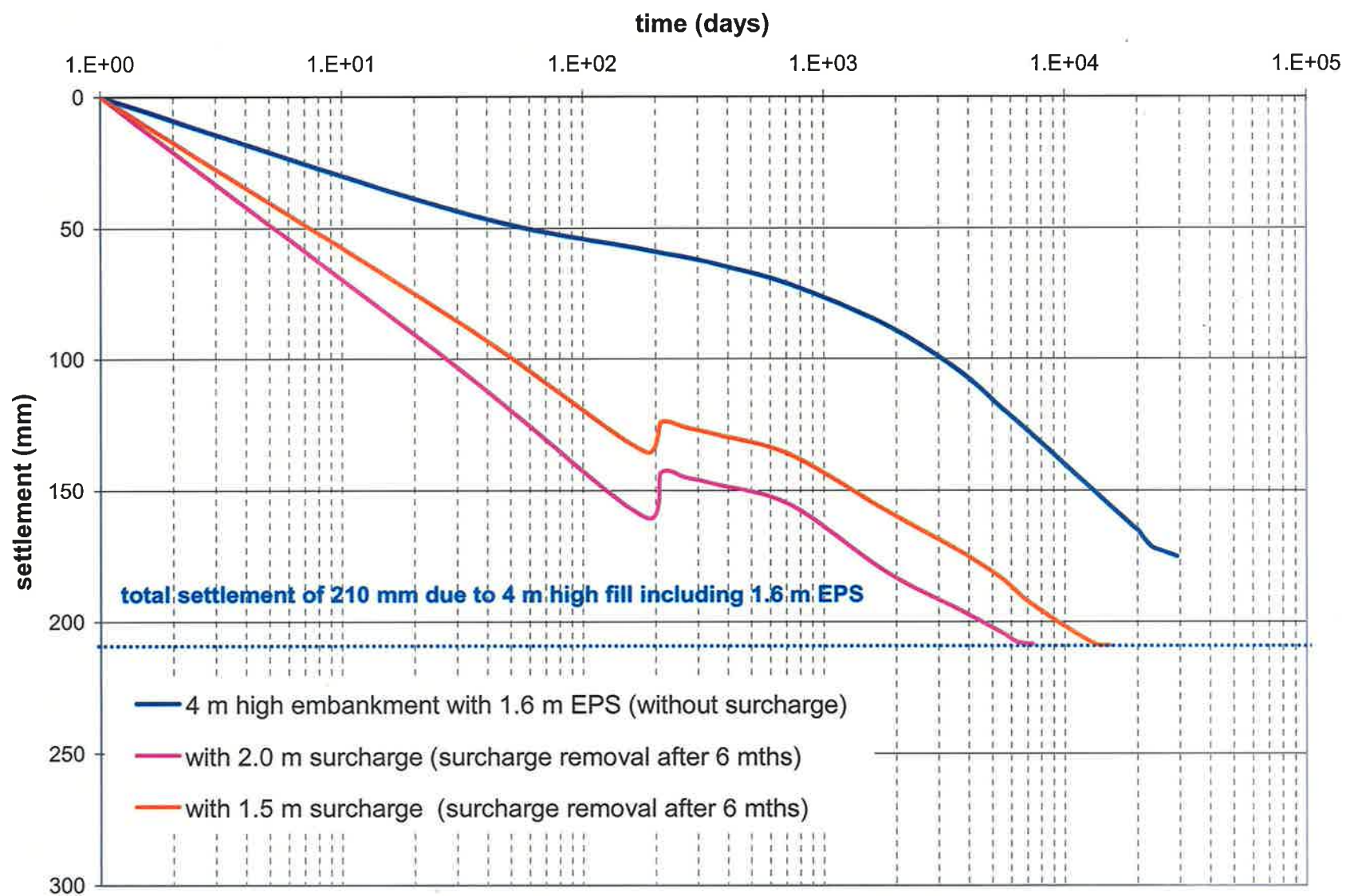
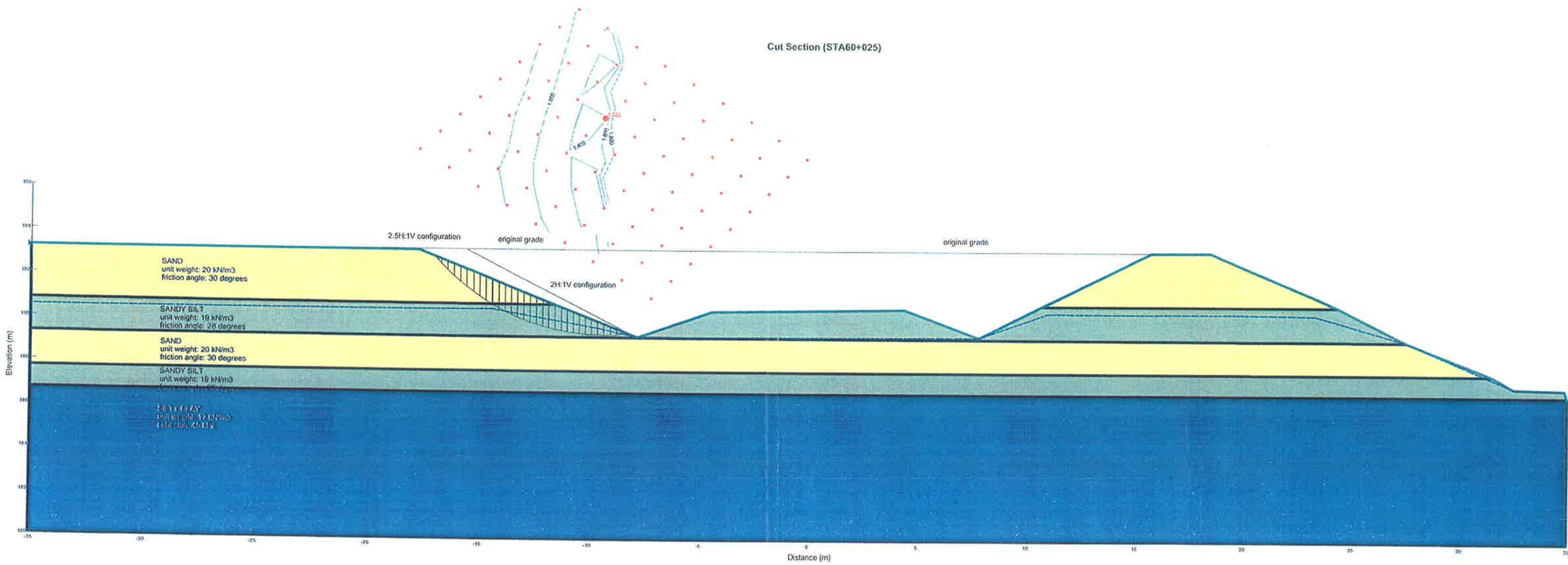


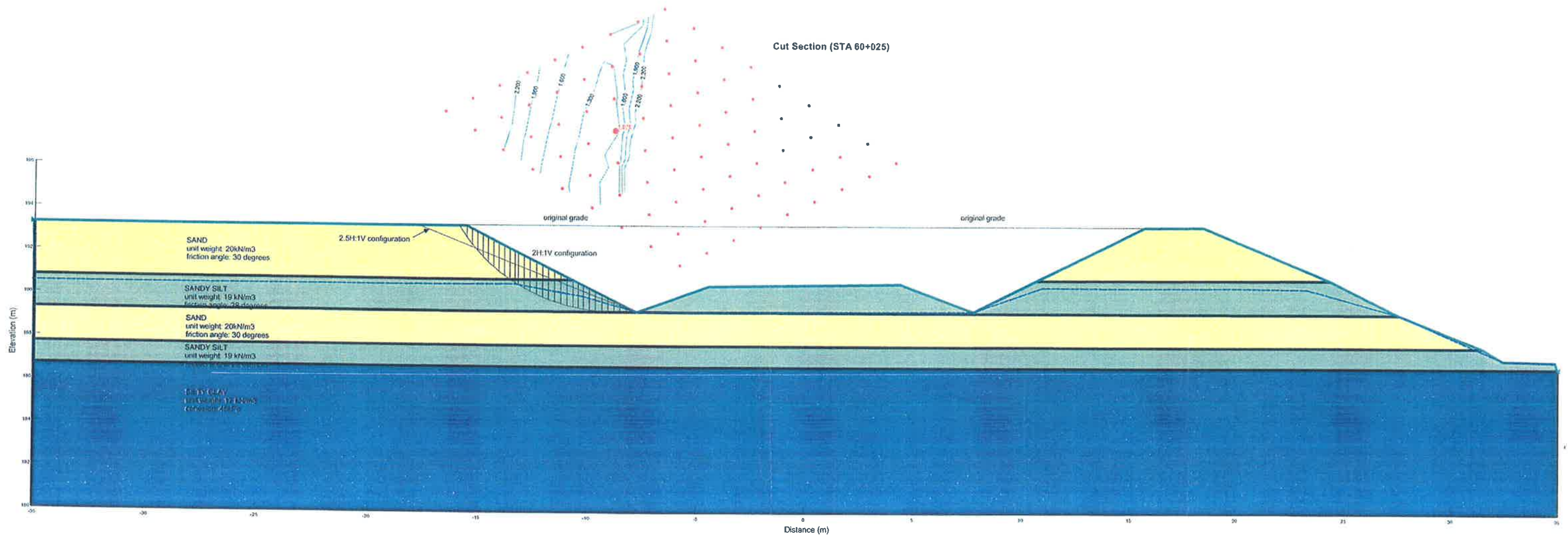
Figure 4. Settlement of 4 m high embankment with 1.6 m EPS

Appendix K

Typical Stability Analysis (Cut Section, STA 60+010 to 60+120)

Cut Section (STA60+025)





Appendix L

Design Undrained Shear Strength Plot

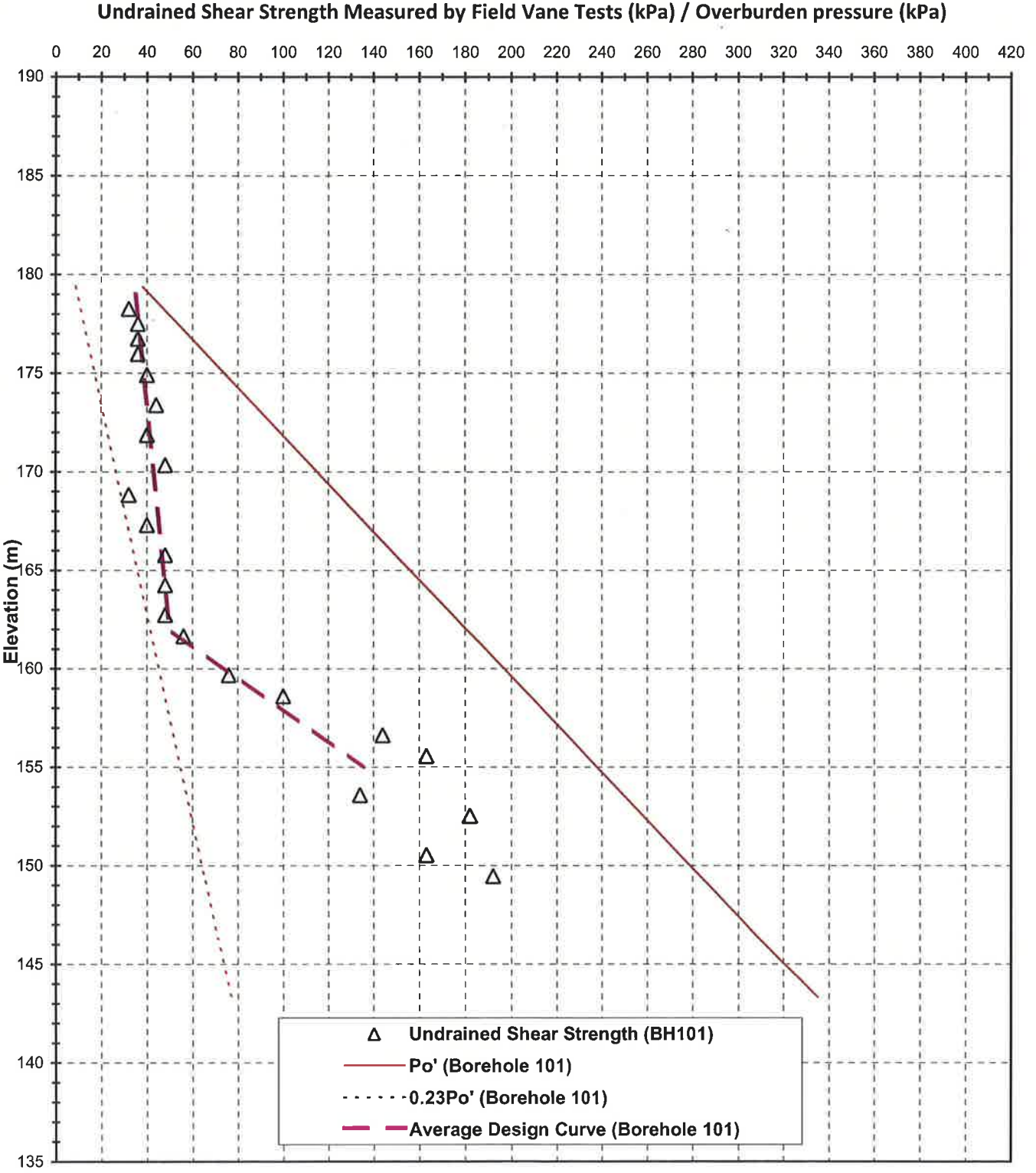
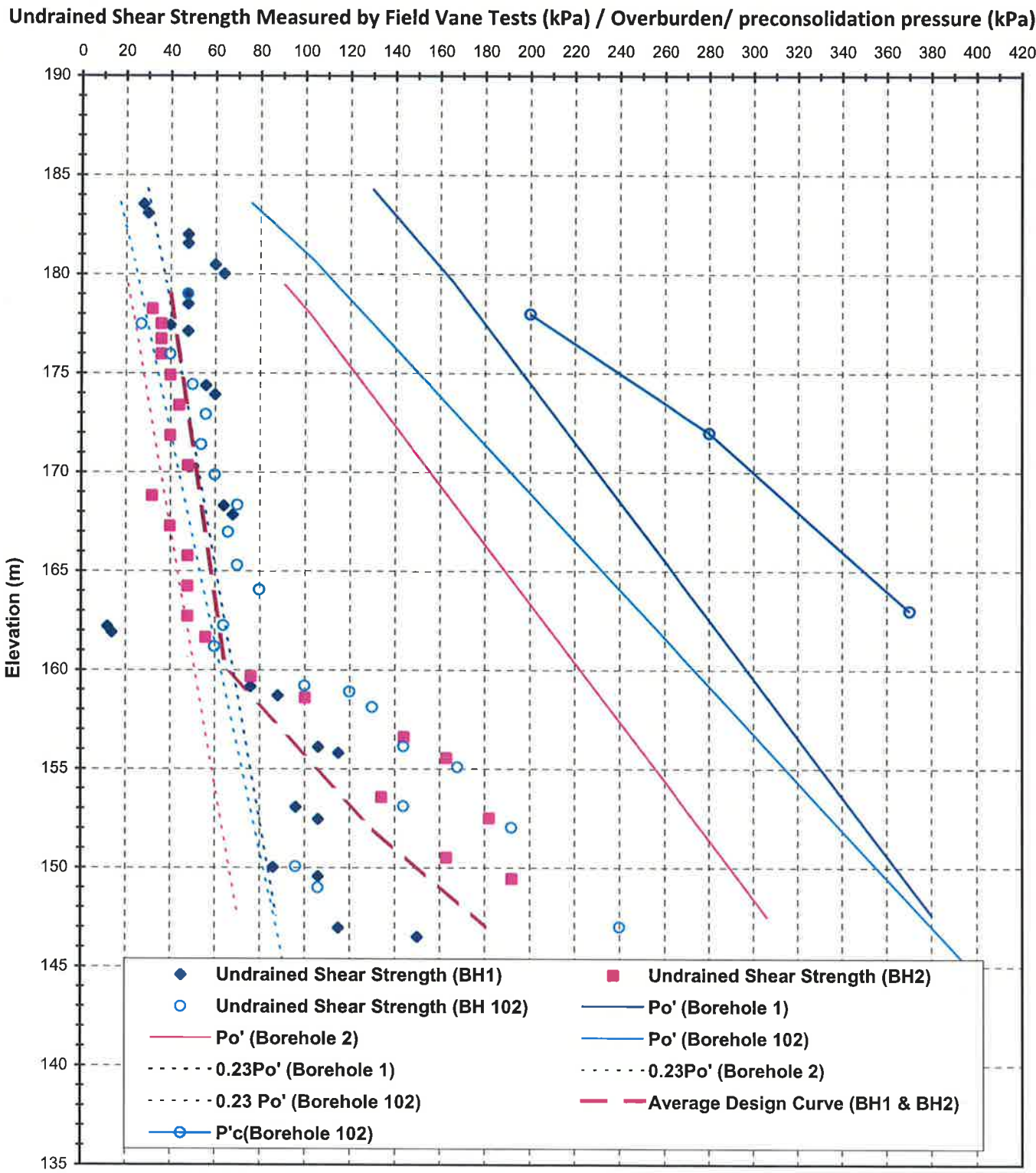


Figure 1 Plots of Undrained Shear Strength and Average Design Curve vs Elevation for Boreholes 1, 2, 101 and 102

Appendix M

MTO Procedures for EPS Design

EXPANDED POLYSTYRENE EMBANKMENT – Item No.

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 $\pm 1\%$ 10 mm in 3 m $\pm 0.5\%$ -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 $\pm 1\%$. 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 $0.7 \text{ m}^2 \cdot ^\circ\text{C}/\text{W}$ for a 25 mm thickness using the following equation and using the average value from three specimens:

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

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 than 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.

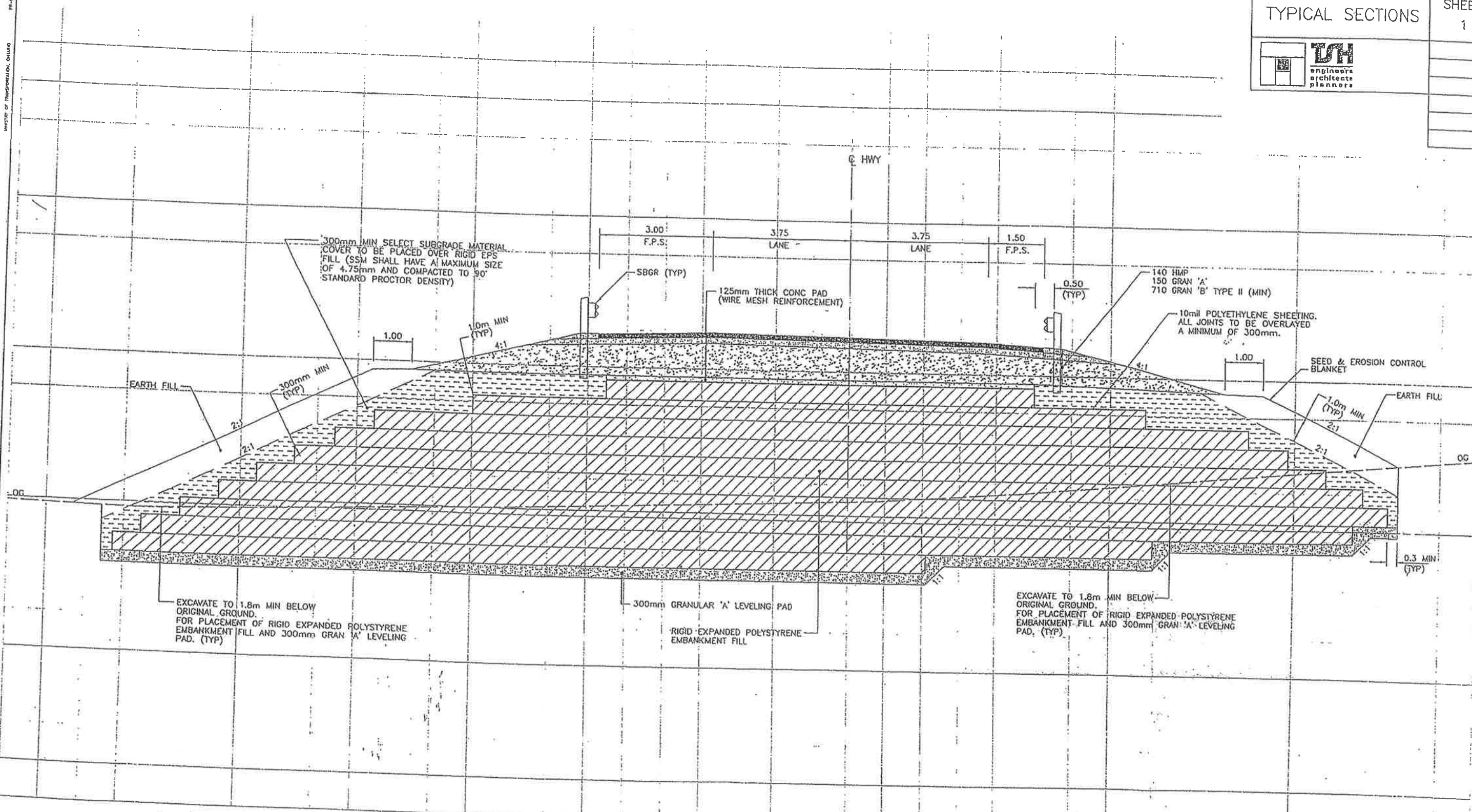
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT
WP

TYPICAL SECTIONS



SHEET
1



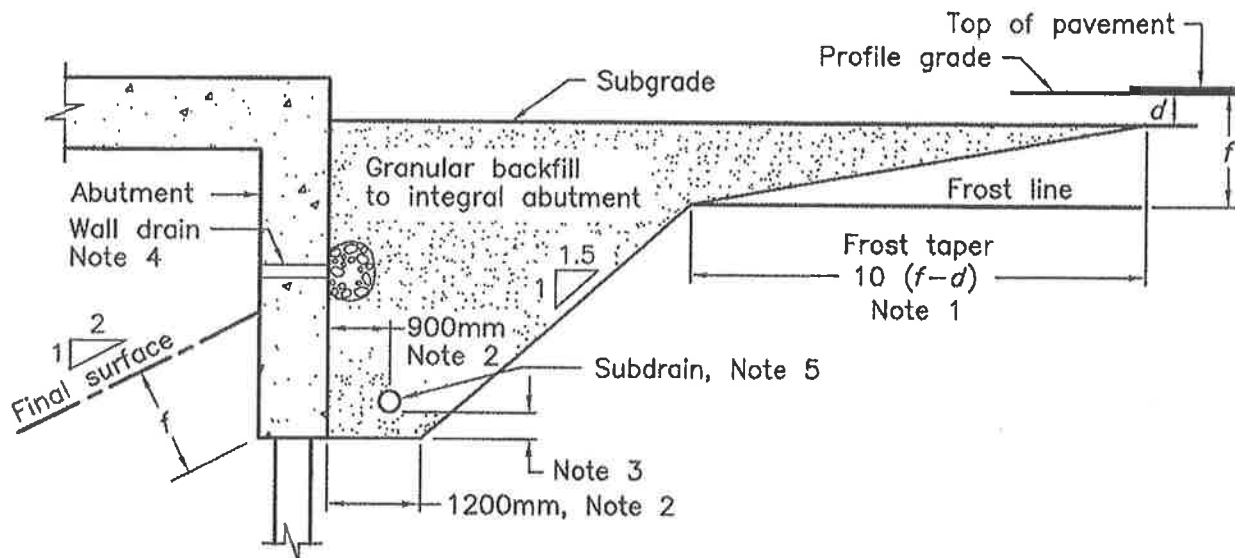
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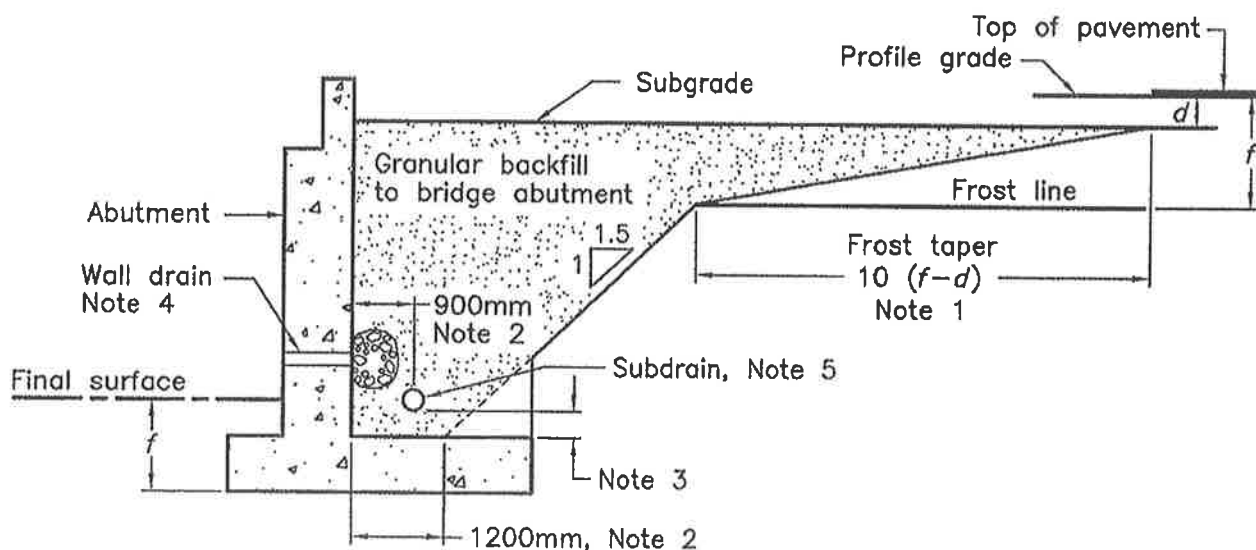
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982842

Appendix N

OPSD



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.

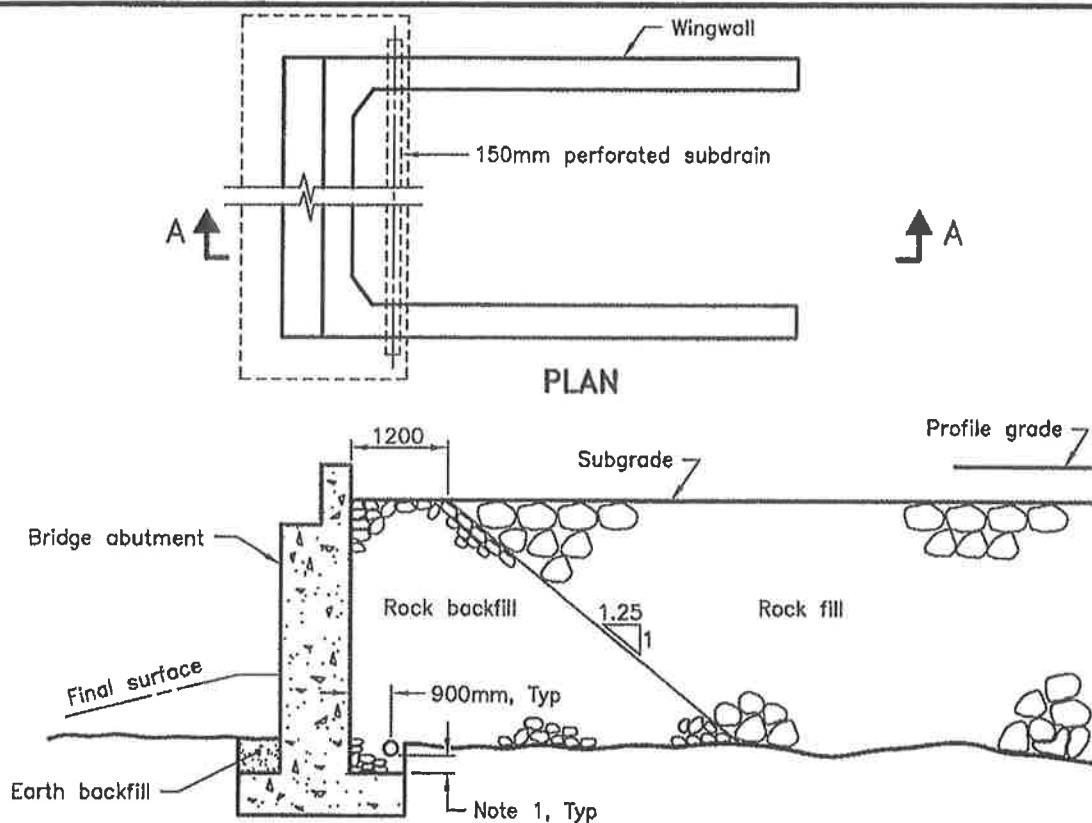
ONTARIO PROVINCIAL STANDARD DRAWING

Nov 2005 Rev 0



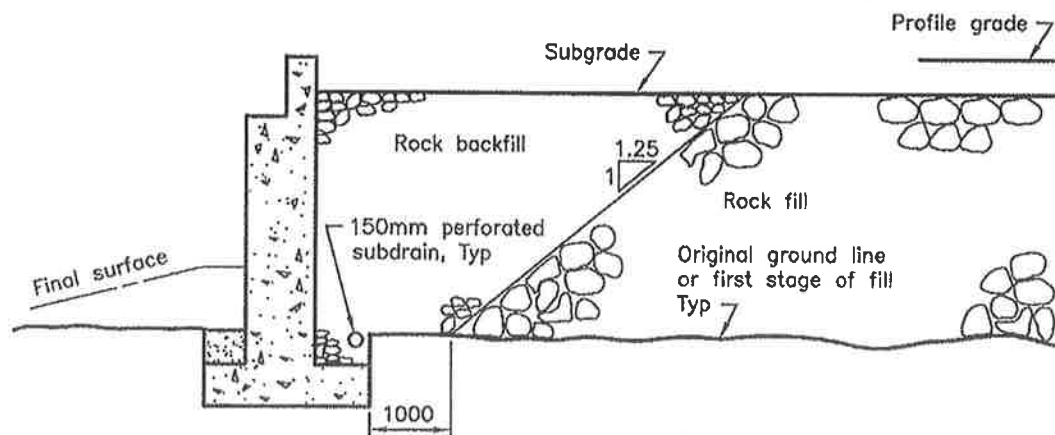
WALLS
ABUTMENT, BACKFILL
MINIMUM GRANULAR REQUIREMENT

OPSD - 3101.150



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

WALLS
ABUTMENT, BACKFILL
ROCK

Nov 2005 Rev 0



OPSD - 3101.200

Appendix O

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.