

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORTS
REPLACEMENT OF HWY 577 BRIDGE
OVER MEADOW CREEK
IROQUOIS FALLS, ONTARIO
G.W.P. 181-92-00; SITE 39E-077**

GEOCRES NO. 42A-66

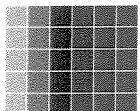
Prepared For:

MCCORMICK RANKIN CORPORATION

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1167
October 16, 2007**



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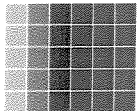
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DRAWING NO.

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**PRELIMINARY FOUNDATION INVESTIGATION REPORT
REPLACEMENT OF HIGHWAY 577 BRIDGE OVER
THE MEADOW CREEK, IROQUOIS FALLS, ONTARIO
G.W.P. 181-92-00
SITE: 39E-077**

1. INTRODUCTION

Shaheen & Peaker Limited (S&P) was retained by McCormick Rankin Corporation (MRC) to carry out a preliminary foundation investigation at the site of the proposed replacement of Highway 577 Bridge over the Meadow Creek in Iroquois Falls. The existing structure over the Meadow Creek, which consists of a twelve-span steel girder bridge with a concrete deck and a timber sub-structure, will be replaced with a new structure.

The purpose of this investigation was to obtain preliminary subsurface information at the site by means of boreholes and to determine the preliminary engineering characteristics of the subsurface soils by conducting field tests in the boreholes and carrying out laboratory testing.

The findings of investigation are presented in this report.

2. GEOLOGY

In general, the ground at the site falls from south to north from a high elevation of about 272 m along the existing Highway 577 about 500 m south of the existing Meadow Creek bridge to about Elevation 252 m at the Creek banks. The grade then rises to about Elevation 266 m about 350 m further north. The Creek valley is approximately 75 m wide at the top near the existing bridge, with a creek bottom elevation of about 243 m. At the time of our investigation, the water elevation in the Creek was at about 248 m.

Physiographic information from the Surficial Geological Map S365 for Algoma-Cochrane area by the Ontario Department of Lands and Forests and Quaternary Geology of Ontario Map 2555 (East-Central Sheet) by the Ministry of Northern Development and Mines (MND&M) indicates that the study area is part of the upper Moose River Basin in the Precambrian Shield physiographic region of the Northern Ontario. The study area is generally covered by lacustrine (glacial lake) deposits which are found between the clay till ground moraine of Wisconsin Glaciations period in the north and the sand till ground moraine in the south parts of this region. The lacustrine deposits generally consist of varved clay and silt deposits, which are widespread in the southeast part of this region. The thickness of lacustrine clay sediments in the area averages 20 m. Lacustrine fine sand and sand occur mainly along the southern periphery of the lacustrine plain. The Lacustrine clay

plain is generally underlain by sand followed by sand & gravel and crystalline bedrock at considerable depth.

3. INVESTIGATION PROCEDURES

The initial phase of this investigation was conducted during the period of August 10 through August 25, 2006 when boreholes were drilled from the land using a truck-mounted drilling rig. During this period, eight boreholes were put down to depths ranging between 12.8 and 34.9 m below the ground surface.

Subsequently, three boreholes (Boreholes 2, 4 and 6) were drilled from a raft during the period of September 5 through September 13, 2006. These boreholes were extended to depths ranging from 25 to 29 m below the level of water in the Creek.

The plan of the borehole locations are shown on Drawings No. 1 and 2. Boreholes 8, 9, 10 and 11 were drilled using hollow-stem augers to depths ranging from 18.3 to 20.4 m below the ground surface. Below these depths, the boreholes were further advanced using N-size casing and washboring. Bi-cone and tri-cone were also used where necessary to extend the holes. Boreholes 2, 4 and 6 (which were drilled from a raft) were advanced using N-casing and washboring techniques, including bi-cone and diamond drilling/coring through boulders towards the bottom of the borehole.

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

In cohesive (clayey) deposits, where the consistency of the soil permitted, relatively undisturbed samples were taken with thin-walled (TW) Shelby tubes which were pushed into the borehole by the application of static weight or hydraulic pressure. The undrained shear strength of the soil was also measured in-situ field vane tests. Where consistency permitted, MTO Field Vane was used to conduct the tests but when the soil became stiffer or inside the casing, this was changed to small field vane.

In addition, dynamic cone penetration tests (DCPT) were performed in Boreholes 1, 3 and 5 at various depths, as well as from the bottom of Boreholes 2, 3, 4, 6 and 7. This test consists of driving a 60° - point, 50 mm diameter cone attached to the drill rod continuously

into the undisturbed ground with a driving energy of 475 j (63.5 kg hammer dropping freely a distance of 76 cm) per blow, similar to SPT. The number of blows for each 30 cm of penetration is recorded and this provided an indication of the relative changes in the soil density with depth.

Groundwater conditions in the boreholes were observed during and on completion of drilling in the open boreholes. Upon their completion, most of the boreholes were grouted using a cement/bentonite mixture as per MTO procedures.

Piezometers were installed in Boreholes 3, 5, 8, 9, 10 and 11, to enable us to monitor groundwater levels over a prolonged period of time, without interference from surface water. These piezometers were not decommissioned so that they can be utilized during the detailed investigation phase of the project.

The soil samples were transported to our laboratory in Toronto for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations, grain-size analyses, Atterberg limits tests of selected soil samples, bulk unit and one-dimensional consolidation (oedometer) tests, was performed.

Ground surface elevations at the borehole locations were provided to us by MRC, as well as the coordinates of the boreholes.

4. SUBSURFACE CONDITIONS

The subsurface conditions were explored at eleven borehole locations. The plan locations of the boreholes, along with the inferred stratigraphic profile are shown on Drawing Nos. 1 and 2. Details of the subsurface conditions encountered at each borehole location, including the results of in-situ testing, groundwater observations and laboratory test results are presented on the Record of Borehole Sheets in Appendix A. Detailed laboratory test results are given in Appendix B.

An investigation was carried out in 1970 by Department of Highways of Ontario (currently known as MTO), for the then proposed realignment of Highway 577, between Monteith and Ansonville. Of the several alignments chosen, alignment 'A' appears to be closest to the present (this may need to be verified for the detailed investigation, if necessary) creek crossing. The Records of Boreholes from that investigation, along with the factual portion of the foundation investigation report, are given in Appendix D.

The boreholes put down during this present investigation show the presence of some fill and organic soils in the boreholes drilled within the Creek valley, near the existing bridge structure (i.e. Boreholes 1 through 7). These soils typically extend 3 to 8 m below the ground surface (or below the water level in the Creek) and are underlain by an extensive

clay deposit, while boreholes drilled at higher elevations, further away from the existing bridge encountered little or no fill and clay was contacted at very shallow depths.

The clay deposit is typically 12 to 22 m thick and is underlain by granular soils ranging in composition from relatively fine-grained silts and fine sands to coarse sands with some gravel or cobbles or boulders in a sand and silt matrix. Boreholes 1 through 7 (i.e. deep boreholes) were terminated in these granular deposits upon encountering refusal to further penetration by normal washboring methods at depths ranging between 25 and 35 m below the ground surface or creek water surface, after penetrating them for a vertical distance of between about 3 and 15 m.

The various soil strata encountered in the boreholes and their geotechnical properties are briefly described in the following paragraphs of this section of this report.

4.1 TOPSOIL

Topsoil was contacted in Boreholes 5, 7, 8 and 9. The thickness of the topsoil at the borehole locations ranged from 0.15 m (Boreholes 5 and 8) to 0.45 m (Borehole 9).

4.2 PAVEMENT

Borehole 3 was drilled from the paved road surface adjacent to the south end of the existing bridge structure and encountered a pavement structure (i.e. asphalt, granular base and sub-base courses) extending to a depth of 1.0 m below the ground surface. It is noted that two layers of asphaltic concrete were encountered with a granular layer in between. This is likely due to the fact that padding was required to raise the grade adjacent to the bridge structure, due to ground settlements.

4.3 FILL

4.3.1 GRANULAR FILL

Boreholes 1, 5, 7, 10 and 11 encountered granular fills, which extend to depths ranging between 0.35 m (Borehole 1) and 1.1 m (Borehole 10) below the ground surface. The granular fill in the remaining three boreholes was measured to extend to 0.8 m below the ground surface.

Standard Penetration tests performed in the granular fill yielded N-values which range from 2 to 11 blows/0.3 m which indicate a very loose to compact but generally a loose condition.

In addition, in Borehole 2, which was drilled from a raft, a 0.3 m thick river bottom sediment was encountered which consisted of primarily sand.

4.3.2 ROCK FILL

In Borehole 4, which was drilled from a raft, an approximately 0.7 m of rock fill with silt and sand infill was contacted at the creek bottom.

4.3.3 CLAYEY FILL

Boreholes 1, 2, 3, 5 and 7, which were drilled in the close proximity of the existing bridge structure, contacted silty clay fill deposits at depths ranging from 0.4 to 1.8 m below the ground or water surface or between elevations ranging from El. 252.9 to El. 246.0 m. The thickness of these basically cohesive fill deposits ranged from 2.2 m to 6.7 m and they extended to depth varying from 2.9 to 7.5 m below the existing ground or water surface or to Elevations ranging from El.250.6 to El.241.6 m.

These cohesive fills were found to consist of basically indigenous silty clay soils but at most locations they were found to be mixed with some organic soil.

N-values recorded in these clayey fills ranged from 1 to 11 blows/0.3 m, but typically from 2 to 9 blows/0.3 m, indicating a generally very soft to stiff consistency. These results lead us to believe that these fills did not receive a systematic compaction effort when they were first placed.

4.4 ORGANIC SOILS

Beneath the fill (Boreholes 1, 2, 3 and 5) or creek bottom deposits (Boreholes 4 and 6), Boreholes 1 through 6 contacted organic soils at depths ranging from 2.9 to 7.5 m below the ground or water surface or at Elevations ranging from 250.4 m (Borehole 1) to 241.6 m (Borehole 2). The thickness of the organic soils ranged from 0.3 to 2.5 m (Borehole 4) and they extended to Elevations 249.1 to 240.2 m. The composition of these organic soils ranged from somewhat organic silty clays to completely organic peat deposits. In some cases, layers of somewhat organic silty clay and inorganic silty clay were found to be interbedded with peat, while in others, the presence of thin peat interbeds (seams) was noted in the silty clay or in somewhat organic silty clay. These materials are considered to be basically cohesive soils.

Based on the recorded N-values which range from 1 to 19 blows/0.3 m (typically 2 to 7 blows/0.3 m), these deposits can be classified as very soft to very stiff, but generally very soft to firm. Owing to their organic nature, these materials can be expected to be weak and highly compressible. As well, they can be expected to present secondary consolidation. The degree to which this is applicable, however, depends on the organic content which at the site ranges from low to high. For example, somewhat organic silty clay can be expected to be less compressible and exhibit less secondary consolidation in comparison with highly organic layers, such as peat.

The measured natural moisture contents range from 25 to 75 %

4.5 SILTY CLAY

Underlying the topsoil, fill or organic soils, the site is underlain by a major silty clay deposit.

This cohesive soil deposit was contacted in all of the eleven boreholes drilled at the site at depths/elevations ranging from 0.15-0.5 m (immediately below the topsoil in Boreholes 8 and 9) to 8.4 m (underlying the fill and organic soils in Borehole 5) or at Elevations varying from El. 265.6 m (Borehole 10) to El. 240.2 m (Borehole 4). In the deeper boreholes (i.e. Boreholes 1 through 7) where the vertical extent of the deposit was fully explored, the silty clay deposit was found to extend to depths of 17.0 m (Borehole 3) to 26.0 m (Borehole 7) below the ground surface or to between Elevations 234.5 m (Borehole 3) and 227.6 m (Borehole 7).

Boreholes 8, 9, 10 and 11 were terminated in this deposit at depths ranging between 12.8 and 19.5 m below the ground surface, or between El. 252.2 and 241.3 m.

At most locations, the silty clay consists of a layered material with irregular layers/interbeds of silty clay and clay with some clayey silt and occasional silt seams. In some zones, the deposit exhibits more uniform structure while in others it contains more frequent interbeds of clayey silt and silt within a silty clay to clay layering. As well at some borehole locations, the upper zones (i.e. the top $2\pm$ m) of the deposit consists primarily of clayey silt with silty clay and silt seams (e.g. Boreholes 9 and 10). In general, however, the deposit attains a more silty nature near its bottom zones, changing to a basically clayey silt with silt and some silty clay to clay seams to silt with clayey silt zones (e.g. Borehole 3).

The results of grain-size analyses carried out on two selected samples from the deposit near the creek (BH3/TW12 and BH5/TW16) are given in Figure B-1 in Appendix B. The results indicate 0 to 1 % sand, 23 to 57 % silt and 42 to 77% clay size particles.

Figure B-2 shows the grain size distribution of samples from the deposit in the relatively higher ground away from the creek (Boreholes 8 and 9). The results indicate 3 to 4 % sand, 62 to 64% silt and 32 to 35% clay size particles. Therefore, the behaviour of this deposit is expected to be governed by its predominantly high clay content (i.e. cohesive soil).

From the grain-size distribution curves, the deposit can be expected to be practically impermeable. As well, because of the interbedded structure, its permeability can be expected to be variable, especially in the horizontal permeability in relation to the vertical permeability.

Atterberg limits tests carried out in the laboratory on twelve selected samples from the deposit yielded the following ranges of index values.

Liquid Limit: 26.0 – 47.0%
Plastic Limit: 15.4 – 28.7%
Plasticity Index: 8.0 – 27.3%

Table 1 – Results of Laboratory Atterberg Limits Tests

Borehole No./ Sample	Depth (m)	El. (m)	PL	LL	PI	Classification
BH1/SS9	7.6 – 8.0	245.7 – 245.3	28.7	53.3	24.6	CH
BH2/SS9	6.1 – 6.5	241.7 – 241.3	26.0	44.7	18.7	CI
BH2/SS12	10.7 – 11.0	237.1 – 236.7	15.4	30.7	15.3	CI
BH3/ TW12	10.7 – 11.1	240.8 – 240.2	20.0	47.0	27.0	CI
BH5/TW16	15.2 – 15.7	236.4 – 235.9	18.0	26.0	8.0	CL
BH7/SS8	6.1 – 6.5	247.5 – 247.1	21.5	41.7	20.2	CI
BH8/SS4	2.3 – 2.7	255.0 – 254.6	16.7	44.0	27.3	CI
BH8/SS8	6.1 – 6.5	251.2 – 250.8	20.1	43.0	22.9	CI
BH9/SS4	2.3 – 2.7	261.3 – 260.9	19.4	41.5	22.1	CI
BH9/SS8	6.1 – 6.5	257.5 – 257.0	21.3	43.3	22.0	CI
BH10/SS5	3.1 – 3.5	254.45 – 254.0	17.1	36.0	18.9	CI
BH11/SS6	3.8 – 4.2	257.4 – 257.0	18.1	43.1	25.0	CI

As shown in Figure B-3, Appendix B, these values are characteristic of clayey soils of low to high plasticity, but generally of intermediate plasticity (CI). The measured material moisture contents of the samples tested range from 26 to 56% that is generally closer to the measured liquid limit values rather than the measured plastic limit values. These values are indicative of relatively weak and compressible soils.

It should, however, be emphasized that the samples tested, in some cases, were comprised of various interbeds, which may range from high to low plasticity clays to high plasticity silt.

From a visual examination of the soil samples (which showed some dilatency) as well as a comparison of the clay content of the soil (from grain-size analyses) and the Atterberg limits test results, the clay particles do not appear to be 'active.'

Two one-dimensional consolidation (oedometer) tests were performed on Shelby tube samples recovered from the deposit and the results are presented in Figures B-4 and B-5, in Appendix B. The test results indicate a probable pre-consolidation pressure (P_c' - P_o') of between 60 and 80 kPa.

As was mentioned before, in 1970 MTO carried out a foundation investigation for the then proposed realignment of Highway #577. It is likely that of the various alignments discussed in this report, Alignment A is near the existing bridge site. Along this alignment (i.e. Alignment designated as Alignment A in the 1970 MTO report) five boreholes were drilled and considerable laboratory testing was performed. The factual portion of the MTO report,

including Record of Borehole Sheets along with laboratory testing performed on samples from these five boreholes is given in Appendix D of this present report. It is noted that of the consolidation (oedometer) test one was from Borehole 2 (i.e. along Alignment A) and this test shows a $P_c'-P_o'$ value of in excess of 200 kPa, while in others (i.e. along other alignments) values as low as zero were obtained.

Standard Penetration tests performed in this deposit in the eleven boreholes drilled for this present investigation gave N-values between 1 and 26 blows/0.3 m with typical results ranging from 2 to 15 blows/0.3 m. These values indicate a generally very soft to stiff consistency. The relatively higher N-values were recorded in the upper 'crust' zone (i.e. within the upper $3\pm$ m of the deposit) and also, in the lower zones of the deposit. Within the mid-section (i.e. below the upper crust where the crust exists, and above the lower zones), typical N-values range from 2 to 10 blows/0.3 m.

Field vane tests yielded undrained in-situ shear strength values from 34 to in excess of 100 kPa, which indicate a firm to very stiff consistency. Again, however, in the typical silty clay to clay zones below the crust (where it exists) and above the lower silty (i.e. less plastic) zones, typical undrained in-situ shear strength values range from 40 to 80 kPa which indicate a generally firm to stiff consistency.

The variation of measured undrained shear strengths with elevation from various boreholes is presented in Figures C-1, C-2 and C-3 in Appendix C.

4.6 LOWER CLAYEY SILT

As mentioned before, the silty clay attains at lower depths a somewhat siltier (less plastic) nature and contains more frequent thin clayey silt/silt interbeds. In addition, in Boreholes 1, 3, 5 and 7 the silty clay was found to be underlain by clayey silt with some silt seams at depths ranging from 17.0 m/El. 234.5 m (Borehole 3) to 26.0 m/El. 227.6 m (Borehole 7). The thickness of this transition zonal deposit was found at the borehole locations to range from 2.5 to 5.5 m and the deposit extended to depths of 20.0 (El. 231.5 m) to 29.5 m (El. 224.1 m).

Standard Penetration Tests conducted in this cohesive deposit yielded N-values which range from 10 to 30 blows/0.3 m. These values indicate a stiff to very stiff consistency.

In addition, in Borehole 6 a 0.6 m thick clayey silt till layer was contacted, underlying the silty clay deposit at 22.3 m depth (El. 225.8 m). From a recorded N-value of 82 blows/0.3 m, the consistency of this cohesive glacial till material is described as hard.

4.7 BASAL GRANULAR SOILS

Underlying the silty clay and the clayey silt deposits described in the previous section of this report, Boreholes 1 through 7, which were extended deeper, contacted fine to coarse grained granular soils which range from sandy silt to silty fine sand (e.g. Borehole 1) to sand and gravel with cobbles (e.g. Borehole 4) or cobbles and boulders in a matrix of sand and gravel (e.g. Borehole 2). In some cases, these soils were identified as probable glacial till deposits (e.g. Borehole 6).

These granular (i.e. non-cohesive) soils were encountered at depths/elevations which range from 19.0 m (Borehole 4) or El. 231.5 m (Borehole 3) to 29.5 m or El. 224.1 m (Borehole 7). The boreholes were extended into these water bearing granular deposits by a distance which range from 3.2 m (BH1) to 14.9 m (BH3), and terminated at depths ranging from 25.2 m (Borehole 4) to 34.9 m or El. 216.6 m (Borehole 3) below the ground surface. In general, the boreholes were terminated at refusal depths which appear to be due to boulders. This, however, was not verified by coring.

The grain-size distributions for five selected samples from the deposit are given in Figures B-6, B-7, B-8 and B-9 in Appendix B.

Standard Penetration tests performed in these basal granular deposit yielded N-values which range from 29 to in excess of 175 blows/0.3 m. These indicate a generally dense to very dense condition.

5. GROUNDWATER CONDITIONS

Groundwater conditions were observed in the open boreholes during the drilling and upon completion of each borehole, as well in sealed piezometers installed in Boreholes 3, 5, as summarized in Table 2 below.

Table 2 – Groundwater Monitoring Results

Borehole No	Ground El (m)	Borehole Depth (m)	Measured Groundwater Level			
			August 2006		Mid-September, 2006	
			Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
BH1	253.3	28.7	9.7*** (water level in open borehole upon completion)	243.6***	-	-
BH2**	247.8 (water level in creek)	26.0	-	-		

Borehole No	Ground El (m)	Borehole Depth (m)	Measured Groundwater Level			
			August 2006		Mid-September, 2006	
			Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
BH3*	251.5	34.9	5.2	246.3	5.3	246.2
BH4**	247.8 (water level in creek)	25.2	-	-	-	-
BH5*	251.6	32.9	5.2	246.4	5.2	246.4
BH6**	248.1 (water level in creek)	28.8	-	-	-	-
BH7	253.6	33.9	6.0 (water level in open borehole upon completion, possibly still rising)	247.6	-	-
BH8*	257.3	16.0	11.4***	245.9***	6.8 (possibly still rising)	250.5
BH9*	263.6	19.5	17.6***	246.0***	13.2 (possibly still rising)	250.4
BH10*	266.7	14.5	13.4***	253.3***	7.2 (possibly still rising)	259.5
BH11*	261.2	12.8	0.6***	260.6***	2.7	258.5

* Borehole with Installed Piezometers.

** Borehole drilled on water (from a raft in the creek)

*** Water Level Not Stabilized.

The groundwater table in the Creek Valley would be primarily influenced by the water level in the Creek, as well as major weather events. At the time of our investigation in September 2006 the water level in the Creek was recorded at about El. 248.0 m. We understand that the water level in the Creek is controlled by dams located both upstream and downstream at about the same elevation (i.e. El. 248.0± 0.2 m) and that in a Regional Storm event this level may rise to 248.5 m. Based on this, it can be expected that the groundwater level in the immediate vicinity of the existing bridge location would be about El. 248.0 m, rising gradually further north and south, reflecting the rise in the ground surface up the valley. For example in Borehole 10 (ground surface El. 266.7 m) and Boreholes 11 (ground surface El. 261.2 m), which are some 300 m to the south and 200 m to the north of the existing bridge structure respectively, the water level was recorded at El. 259.5 and 258.5 m, respectively (possibly still rising, especially at Borehole 10 location). For this reason, the piezometers in all the boreholes were left in place to enable the monitoring of the groundwater levels over a prolonged period of time.

It should be pointed out that the groundwater table would be subject to seasonal fluctuations and in response to major weather events.

SHAHEEN & PEAKER LIMITED



Ramon Miranda, P.Eng.

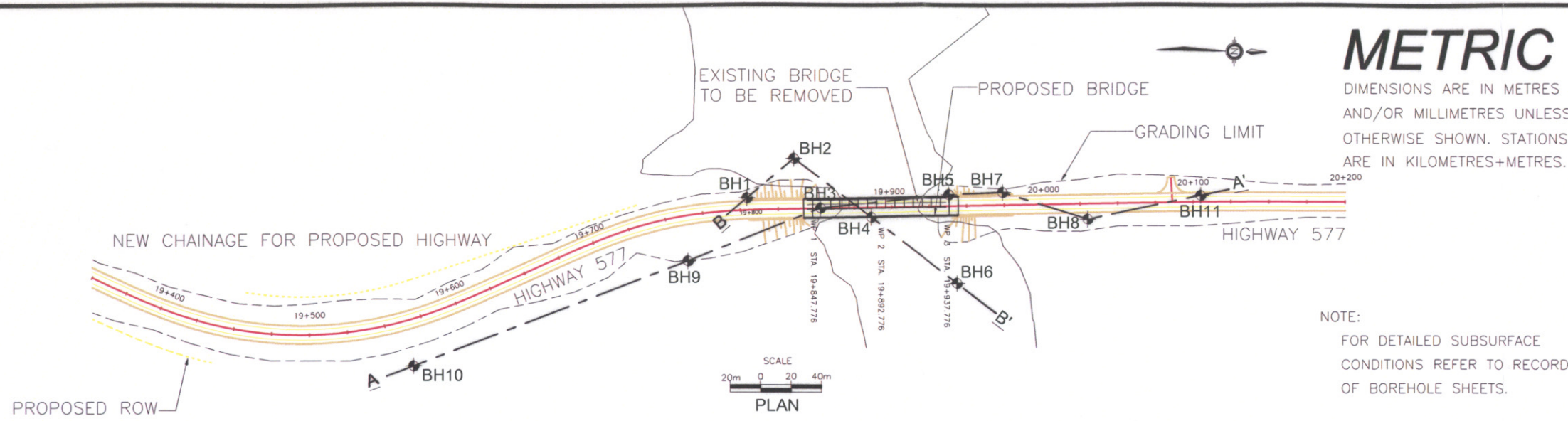


Z.S. Ozden, P.Eng.

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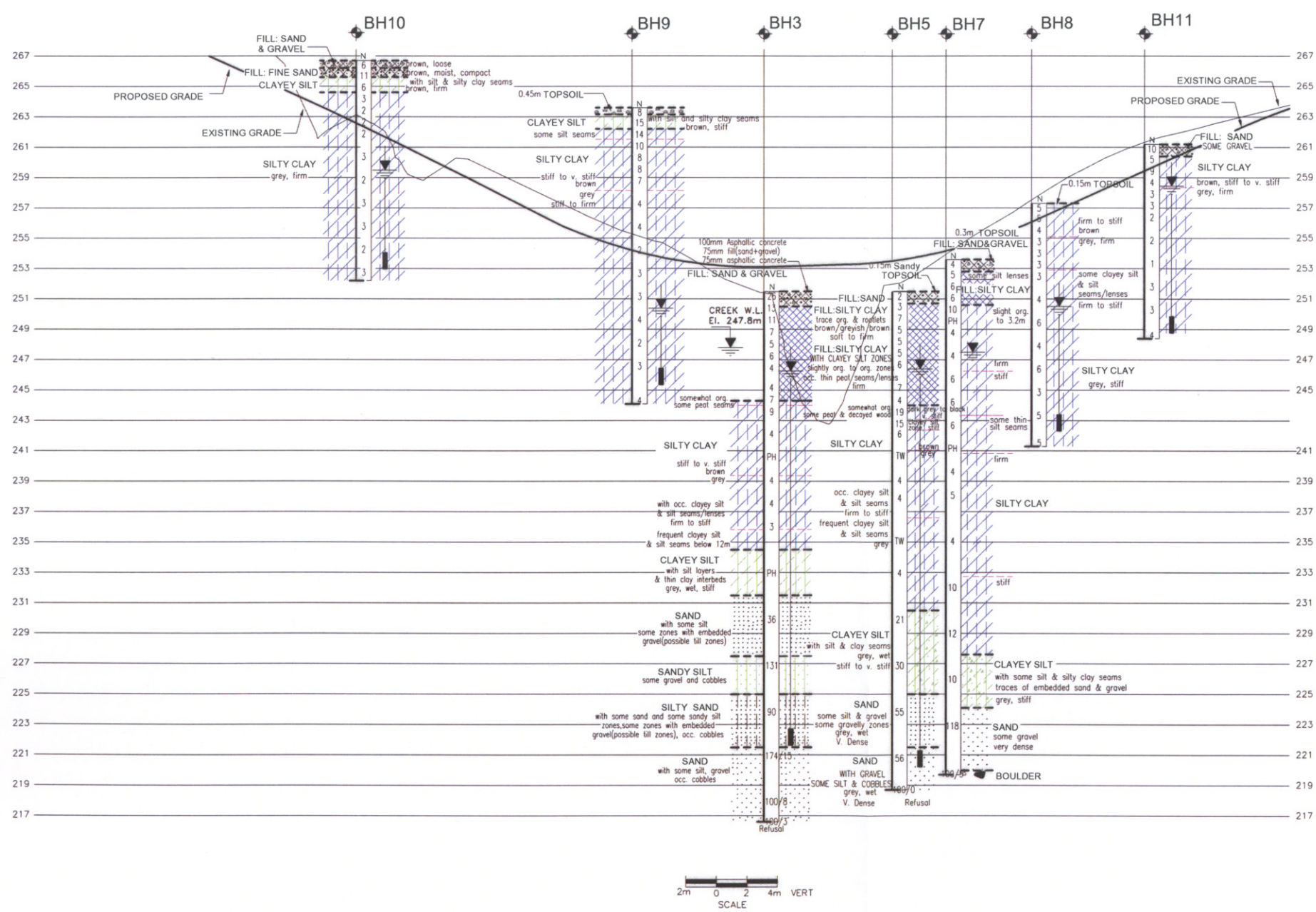
Drawings



METRIC

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

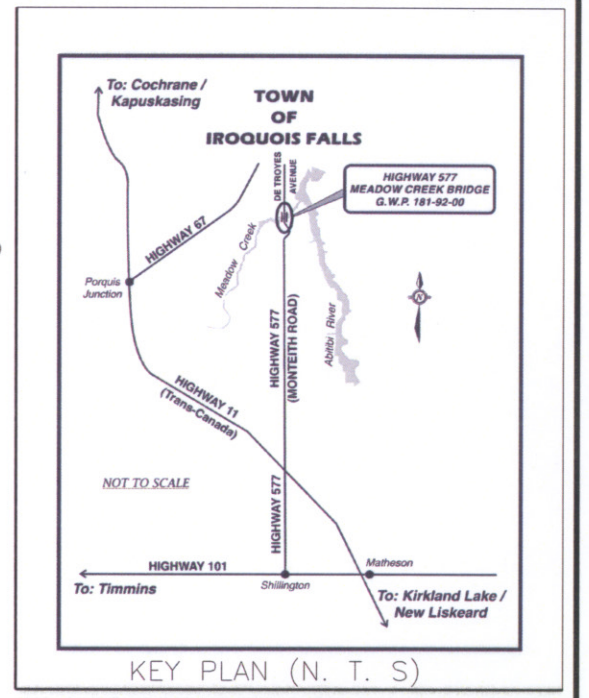
NOTE:
FOR DETAILED SUBSURFACE CONDITIONS REFER TO RECORD OF BOREHOLE SHEETS.



CONT No.
GWP: 181-92-00

Meadow Creek Bridge, Iroquois Falls
SECTION AT STA. 19+600 TO 20+150
BORE HOLE LOCATIONS & SOIL STRATA

SHAHEEN & PEAKER LIMITED



LEGEND

- Borehole
- Blows/0.3m (Std. Pen. Test, 475 J/blow)
- Water Level at Time of Investigation Sept, 2006 (Not Stabilized)
- Water Level in Piezometer
- Piezometer

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
BH 3	251.5	5 401 293.3	328 274.1
BH 5	251.5	5 401 378.6	328 263.9
BH 7	253.6	5 401 414.2	328 262.0
BH 8	257.3	5 401 470.6	328 278.8
BH 9	263.6	5 401 206.7	328 310.6
BH 10	266.7	5 401 025.4	328 382.8
BH 11	261.2	5 401 545.4	328 261.7

NOTE

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

LICENSED PROFESSIONAL ENGINEER
1. L
RAMON MIRANDA
Oct. 18/07
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
Z. S. OZDEN
Oct. 18/07
PROVINCE OF ONTARIO

REV.	DATE	BY	DESCRIPTION

Geocres No. 42A-66

MEADOW CREEK BRIDGE-HWY577			DIST
SUBM'D ZO	CHECKED RM	DATE Nov, 2006	SITE 39E-077
DRAWN XS	CHECKED FS	APPROVED ZO	DWG 1



MEADOW CREEK BRIDGE-HWY 577			DIST
SUBM'D ZO	CHECKED RM	DATE Nov,2006	SITE 39E-077
DRAWN XS	CHECKED FS	APPROVED ZO	DWG 2



Appendix A

Record of Borehole Sheets

SPT1167

1 OF 3

METRIC

DATUM Geodetic DATE 8/10/2006 CHECKED BY RM

238.3

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

SPT1167

RECORD OF BOREHOLE No 1

2 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 244.8; E 328 268.2 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augering; N-casing and Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/10/2006 CHECKED BY RM

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 ● POCKET PENETR.	+ FIELD VANE × LAB VANE						
							20 40 60 80 100	20 40 60 80 100	10 20 30						
15.0	SILTY CLAY grey, damp to moist stiff		14	SS	6										
				15	SS	3									

Continued Next Page

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

SPT1167

3 OF 3

METRIC

SOIL PROFILE					
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES <div>NUMBER</div> <div>TYPE</div> <div>"N" VALUES</div>	GROUND WATER CONDITIONS	ELEVATION SCALE
	Dynamic Cone Penetration Test performed below 225.3m, yielded 200 blows for 23 cm penetration.				

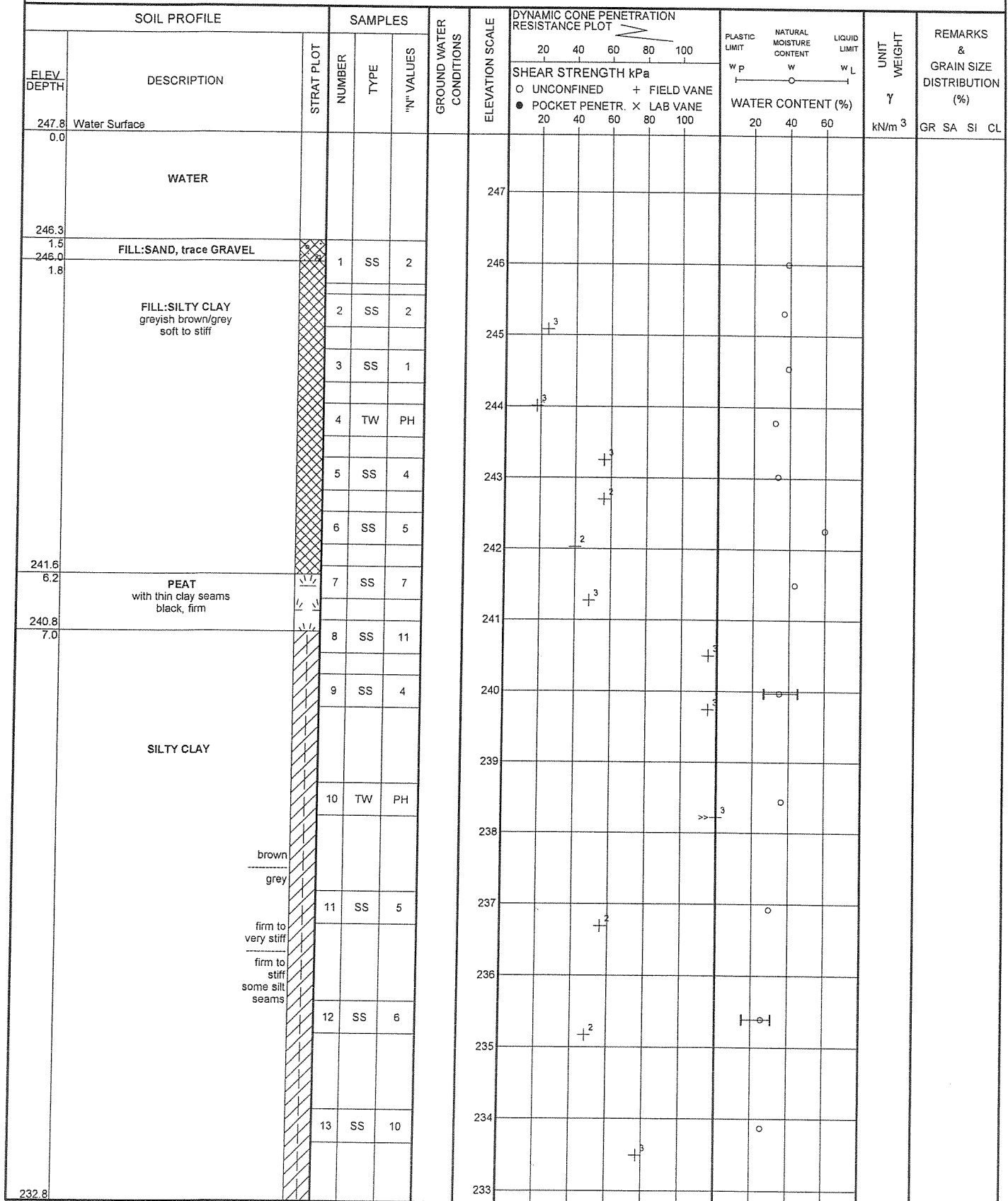
SPT1167

RECORD OF BOREHOLE No 2

1 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 275.1; E 328 241.6 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE N- casing, Wash Boring and Rock Coring COMPILED BY HL
DATUM Geodetic DATE 9/5/2006 CHECKED BY RM



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






SPT1167

RECORD OF BOREHOLE No 2

2 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 275.1; E 328 241.6 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE N- casing, Wash Boring and Rock Coring COMPILED BY HL
DATUM Geodetic DATE 9/5/2006 CHECKED BY RM

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						
								20 40 60 80 100						
15.0	SILTY CLAY grey, firm to stiff		14	SS	12		232	3.0						
			15	SS	9		231	2.0						
							230							
							229							
228.3	SAND some gravel, cobbles and boulders grey, vrey dense, wet		16	SS	57		228							
19.5							227							
			17	SS	100/0		226							
		Boulder 	18	RC	---									
225.3	FINE SAND some silt grey, very dense, wet		19	SS	155		225							
22.5							224							
			20	SS	118									
223.3	COBBLES and BOULDERS in a matrix of sand and gravel grey, very dense, wet		21	SS	100/12		223							
24.5							222							
			22	SS	100/12									
221.8	End of Borehole at 26.0m DCPT performed from 26.0 to 26.35m 26.0----26.3 115 blows/30 cm 26.3----26.35 100 blows/5 cm													
26.0														

SPT1167

1 OF 3

METRIC

LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 293.3; E 328 274.1

ORIGINATED BY GI

BOREHOLE TYPE Hollow Stem Augers, N - Casing & Wash Boring

COMPILED BY JZ

DATE 8/23/2006

CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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	WATER CONTENT (%)			
						○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				
						20 40 60 80 100	20 40 60				
251.5 0.0	Ground Surface		1	SS	26						
250.5 1.0	FILL:SILTY CLAY trace organics and rootlets brown, stiff to firm		2	SS	13						
			3	SS	11						
			4	SS	7						
248.5 3.0	FILL:SILTY CLAY with clayey silt zones brown and grey, some dark grey to blackish slightly organic to organic zones, occasional thin peat seams/lenses firm to very stiff		5	SS	5						
			6	SS	6						
			7	SS	4						
			8	SS	4						
244.3 7.2	SILTY CLAY somewhat organic, some peat seams dark grey to black, firm		9	SS	7						
244.0 7.5			10	SS	9						
			11	SS	4						
	SILTY CLAY with occasional clayey silt and silt seams/ lenses firm to stiff		12	TW	PH						
	frequent clayey silt&silt seams below 12m		13	SS	4						
			14	SS	4						

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

SPT1167

RECORD OF BOREHOLE No 3

3 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 293.3; E 328 274.1 ORIGINATED BY Gi
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers, N - Casing & Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/23/2006 CHECKED BY RM

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							
								<div>○ UNCONFINED + FIELD VANE</div> <div>● POCKET PENETR. × LAB VANE</div>	<div>PLASTIC LIMIT</div> <div>NATURAL MOISTURE CONTENT</div> <div>LIQUID LIMIT</div>	<div>W_P</div> <div>W</div> <div>W_L</div>					
							20 40 60 80 100	WATER CONTENT (%) 10 20 30							
30.0	SAND some silt, gravel, occasional cobbles		20	SS	174/15	221							Sample 21: No recovery, bouncing on a cobble		
			21	SS	100/8	218							Sample 22: No recovery, bouncing (probably boulder)		
216.6	End of Borehole Sampler bouncing, refusal to casing advance and tricone. Dynamic Cone Penetration Test (DCPT) from 21.8 to 24.2m from 24.8 to 27.2m from 27.8 to 28.8m from 30.5 to 31.2m from 33.7 to 34.9m Water level at 5.2m(not stabilized) and hole open to 23.5m upon completion Piezometer installed to 29.8m Date W.L.m.Piezometer 8/25/06 5.2m(EL.246.3m) 8/26/06 5.2m(EL.246.3m) 9/04/06 5.6m(EL.245.9m) 9/06/06 5.4m(EL.246.1m) 9/07/06 5.5m(EL.246.0m) 9/09/06 5.5m(EL.246.0m) 9/09/06 5.3m(EL.246.0m) 9/12/06 5.3m(EL.246.2m) 9/14/06 5.3m(EL.246.2m)		22	SS	100/3	217									
34.9															

RECORD OF BOREHOLE No 4

1 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 327.1; E 328 279.7 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE N - casing and Wash Boring COMPILED BY HL
DATUM Geodetic DATE 9/7/2006 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● POCKET PENETR. × LAB VANE										
247.8 0.0	Water Surface						20 40 60 80 100				20 40 60		kN/m ³	GR SA SI CL				
	Water																	
243.4 4.4	ROCK FILL with silt & sand infill		1	SS	1													
242.7 5.1	SILTY CLAY somewhat organic dark grey, firm		2	SS	5													
242.4 5.4	PEAT with ORGANIC SILT/ CLAY some silty clay layers, dark grey/blackish soft to firm, wet		3	SS	4													
			4	SS	2													
240.2 7.6	trace of organic		5	SS	2													
			6	SS	2													
	darkish grey grey		7	SS	1													
	SILTY CLAY grey, firm to stiff		8	TW	PH													
			9	SS	3													
			10	SS	4													
232.8	some clayey silt layers																	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT1167

RECORD OF BOREHOLE No 4

2 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 327.1; E 328 279.7 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE N - casing and Wash Boring COMPILED BY HL
DATUM Geodetic DATE 9/7/2006 CHECKED BY RM

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
FLEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
15.0	SILTY CLAY some clayey silt layers grey, firm to stiff		11	TW	PH		232							
			12	SS	10		231							
	SAND with GRAVEL, COBBLES and BOULDERS grey, very dense, wet		13	SS	12		230							
228.8 19.0	SAND with GRAVEL, COBBLES and BOULDERS grey, very dense, wet						229							
			14	SS	145		228							
							227							
							226							
	Refusal to casing penetration and tricone (Possible bedrock or boulder). Dynamic Cone Penetration Test (DCPT) performed from 25.2m, 100 blows for 8cm penetration		15	SS	100/12		225							
							224							
							223							
222.6 25.2	Refusal to casing penetration and tricone (Possible bedrock or boulder). Dynamic Cone Penetration Test (DCPT) performed from 25.2m, 100 blows for 8cm penetration		16	SS	100/5									

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



SPT1167

RECORD OF BOREHOLE No 5

1 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 378.6; E 328 263.9 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers, N-casing and Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/14/2006 to 8/15/2006 CHECKED BY RM

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
251.6 0.0	Ground Surface		1	SS	2		251					
250.8 0.8	0.15m Sandy TOPSOIL FILL: SAND trace gravel brown, moist, very loose		2	SS	3		250					
	FILL: SILTY CLAY trace organics & rootlets brown/ greyish brown soft to firm		3	SS	7		250					
249.4 2.2			4	SS	5		249					
	FILL: SILTY CLAY with Clayey Silt Zones brown and grey some dark grey to blackish, slightly organic to organic zones, occasional thin peat seams/lenses firm		5	SS	5		249					
			6	SS	5		248					
			7	SS	6		247					
			8	SS	7		246					
			9	SS	4		245					
244.1 7.5	SILTY CLAY somewhat organic, some peat and decayed wood dark grey to black, very stiff		10	SS	19		244					
243.2 8.4	clayey silt zone stiff		11	SS	15		243					
			12	SS	6		242					
			13	TW	PH		241					
			14	SS	4		240					
			15	SS	4		239					
							238					
							237					

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

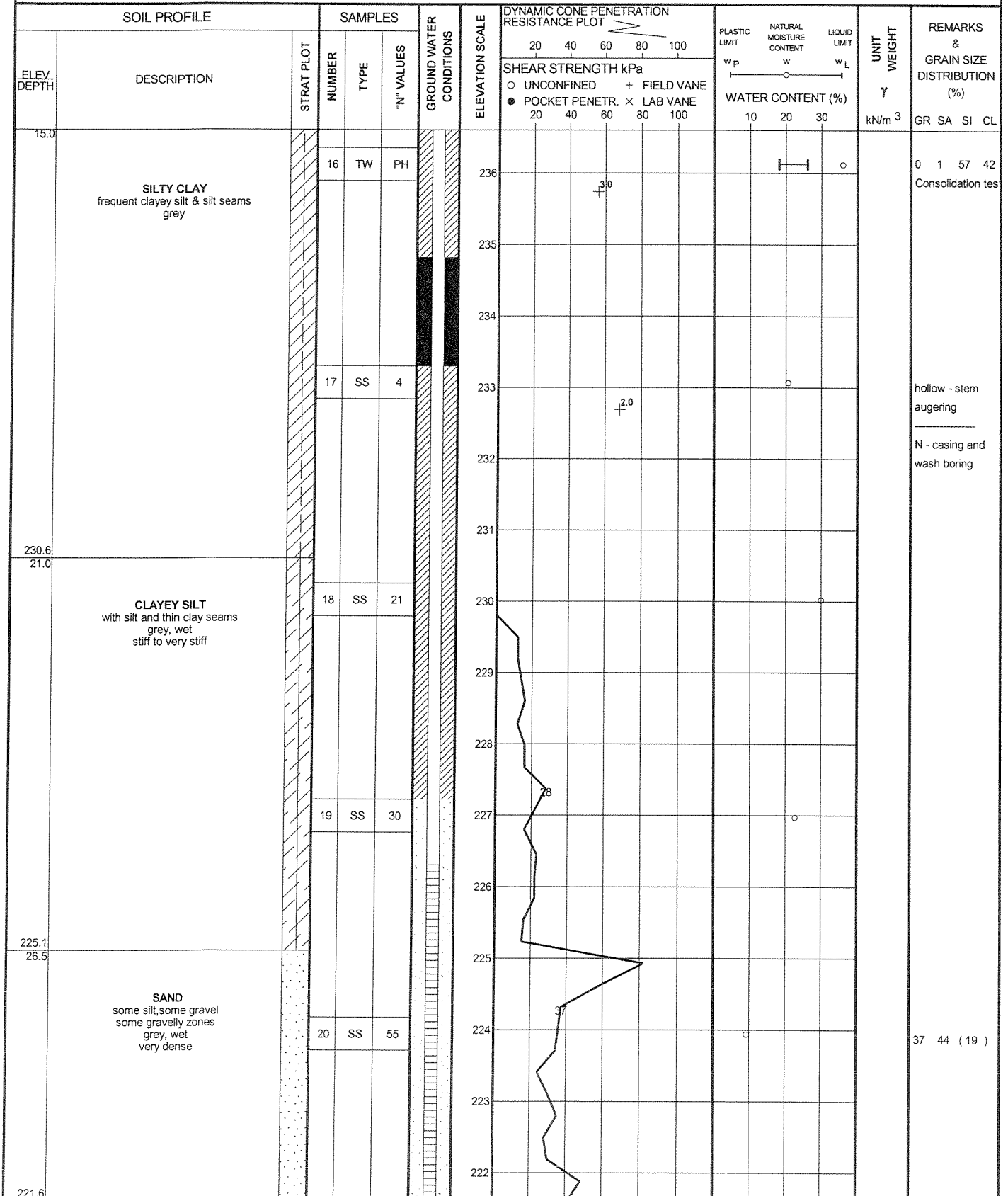
SPT1167

RECORD OF BOREHOLE No 5

2 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 378.6; E 328 263.9 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers, N-casing and Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/14/2006 to 8/15/2006 CHECKED BY RM



Continued Next Page

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

SPT1167

3 OF 3

METRIC

GWP	<u>181-92-00</u>	LOCATION	<u>Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 378.6; E 328 263.9</u>	ORIGINATED BY	<u>GI</u>
DIST	<u> </u>	HWY	<u>577</u>	BOREHOLE TYPE	<u>Hollow Stem Augers, N-casing and Wash Boring</u>
DATUM	<u>Geodetic</u>	DATE	<u>8/14/2006 to 8/14/2006</u>	COMPILED BY	<u>JZ</u>
				CHECKED BY	<u>RM</u>

+³, ×³: Numbers refer to Sensitivity

SPT1167

RECORD OF BOREHOLE No 6

1 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 385.1; E 328 322.5 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE N-coring, Wash Boring, and Rock Coring COMPILED BY HL
DATUM Geodetic DATE 9/11/2006 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE						
								● POCKET PENETR.	× LAB VANE						
248.1 0.0	Water Surface						20 40 60 80 100	20 40 60 80 100	20 40 60	kN/m ³	GR SA SI CL				
243.8 4.3	Logs at river bottom surface (river bottom sediments) SILT and CLAY very loose/very soft		1	SS	1										
242.8 5.3	ORGANIC SILTY CLAY		2	SS	1										
242.5 5.6	with peat seams, dark grey, very soft														
241.7 6.4	PEAT black, soft to stiff		3	SS	11										
240.8 7.3	SILTY CLAY somewhat organic some peat seams, dark grey stiff		4	SS	14										
			5	SS	8										
			6	TW	PH										
			7	SS	3										
			8	TW	PH										
			9	SS	5										
			10	SS	2										

Continued Next Page

+ 3, × 3: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

SPT1167

RECORD OF BOREHOLE No 6

2 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 385.1; E 328 322.5 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE N-coring, Wash Boring, and Rock Coring COMPILED BY HL
DATUM Geodetic DATE 9/11/2006 CHECKED BY RM

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
15.0	SILTY CLAY very frequent clayey silt and silt layers grey		11	SS	4		233					
							232					
			12	SS	8		231					
							230					
			13	SS	17		229					
							228					
			14	SS	20		227					
							226					
			15	SS	26		225					
225.8							224					
22.3	CLAYEY SILT TILL grey, hard		16	SS	82		223					
225.2							222					
22.9	SANDY SILT to SILTY SAND TILL trace of clay near top, occ. fine sand layers, frequent cobbles and boulders grey, very dense, wet		17	SS	155/22		221					
			18	RC	----		220					
			18	SS	177							
219.3			19	SS	100/12.7							
28.8	End of borehole DCPT conducted from 28.8m to 29.15m Blow counts: 28.8-----29.1m-----140blows/30cm 29.1-----29.15m-----100blows/5cm											

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

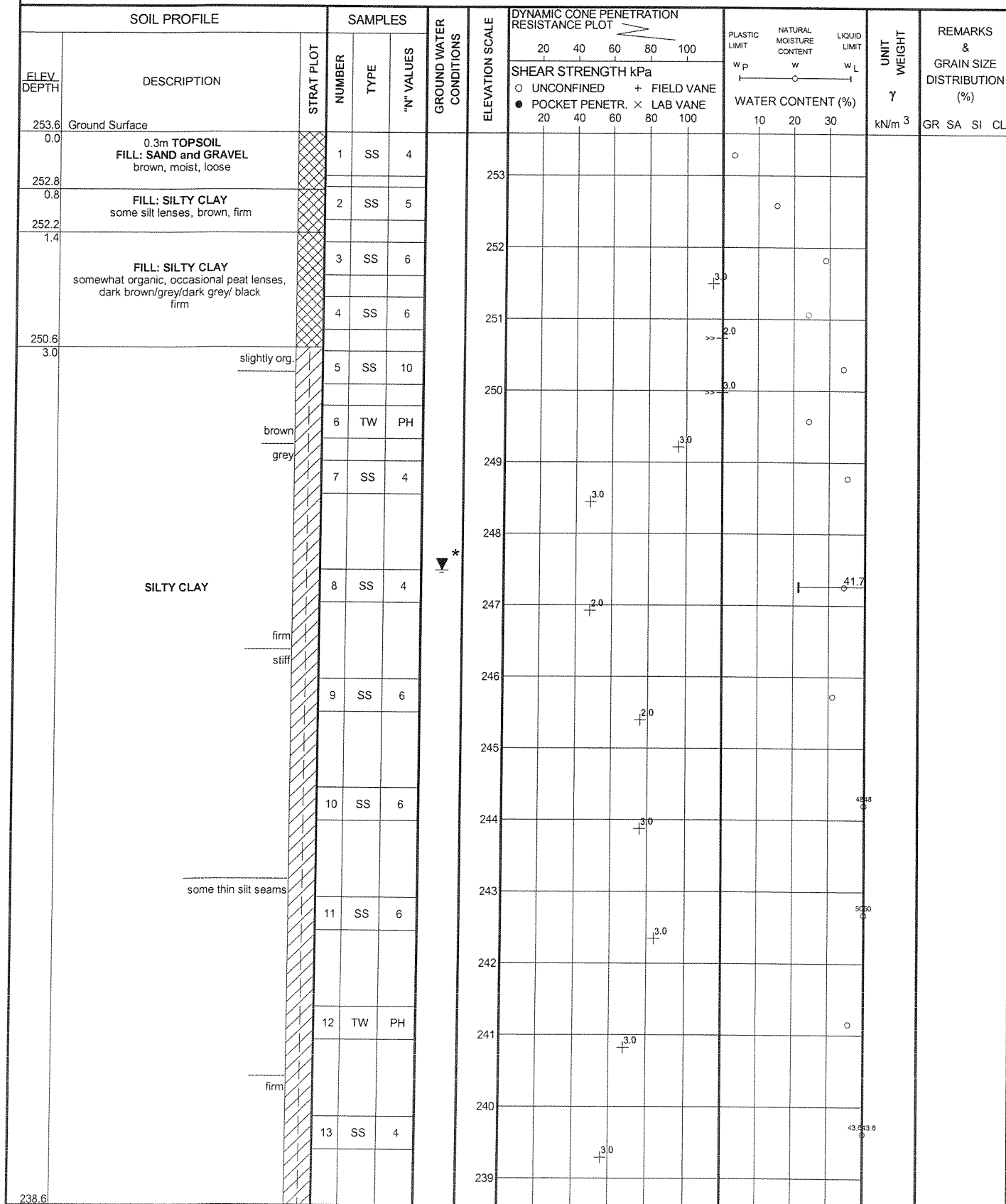
SPT1167

RECORD OF BOREHOLE No 7

1 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 414.2; E 328 262.0 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers & Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/16/2006 to 8/18/2006 CHECKED BY RM



+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

SPT1167

RECORD OF BOREHOLE No 7

2 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 414.2; E 328 262.0 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers & Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/16/2006 to 8/18/2006 CHECKED BY RM

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			20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
15.0	SILTY CLAY grey		14	SS	5		238	2.0				
							237					
							236					
			15	SS	4		235	3.0			43.5	
	CLAYEY SILT with some silt and silty clay seams, traces of embedded sand and gravel grey, stiff						234					
							233					
			16	SS	10		232	2.0				
							231					
	CLAYEY SILT with some silt and silty clay seams, traces of embedded sand and gravel grey, stiff						230					
							229	3.0				
							228					
							227					
227.6 26.0	CLAYEY SILT with some silt and silty clay seams, traces of embedded sand and gravel grey, stiff						226					
							225	2.0				
			18	SS	10		224					
							223					
224.1 29.5	SAND, some gravel						222					

Continued Next Page

+ 3, x 3. Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1167

RECORD OF BOREHOLE No 7

3 OF 3

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 414.2; E 328 262.0 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers & Wash Boring COMPILED BY JZ
DATUM Geodetic DATE 8/16/2006 to 8/18/2006 CHECKED BY RM

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								
	SAND with some silt and gravel, occasional cobbles grey, wet, very dense		19	SS	118		223						16 73 (11)
							222						
							221						
220.0							220						
33.6	Sampler bouncing on probable boulder.		20	SS	100/5								cored through a boulder from 33.6 to 33.9m
219.7			21	RC									
33.9	End of borehole												
	Dynamic Cone Penetration Test (DCPT) conducted from 28.0m to 30.5m. DCPT conducted from 30.9 to 31.5m. * Water level at 6.0m (not stabilized) and hole open to 27m upon completion												

+ 3, × 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1167

RECORD OF BOREHOLE No 8

1 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 470.6; E 328 278.8 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers COMPILED BY JZ
DATUM Geodetic DATE 8/18/2006 CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
257.3	Ground Surface							20 40 60 80 100	20 40 60 80 100				kN/m ³	GR SA SI CL
0.0	0.15m TOPSOIL		1	SS	5		257							
			2	SS	5									
	firm to stiff brown		3	SS	4		256							
	grey, firm		4	SS	3		255							
			5	SS	3		254							
	SILTY CLAY		6	SS	3		253							
	some clayey silt and silt seams/lenses firm to stiff		7	SS	3		252							
			8	SS	4		251							
			9	SS	6		250							
			10	SS	4		249							
			11	SS	6		248							
			12	SS	3		247							
			13	SS	5		246							
							245							
							244							
							243							
242.3														

Continued Next Page

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

SPT1167

2 OF 2

METRIC

GWP	181-92-00	LOCATION	Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 470.6; E 328 278.8	ORIGINATED BY	GI
DIST		HWY	577	BOREHOLE TYPE	Hollow Stem Augers
DATUM	Geodetic	DATE	8/18/2006	COMPILED BY	JZ
				CHECKED BY	RM

[illegible]

+³, ×³: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

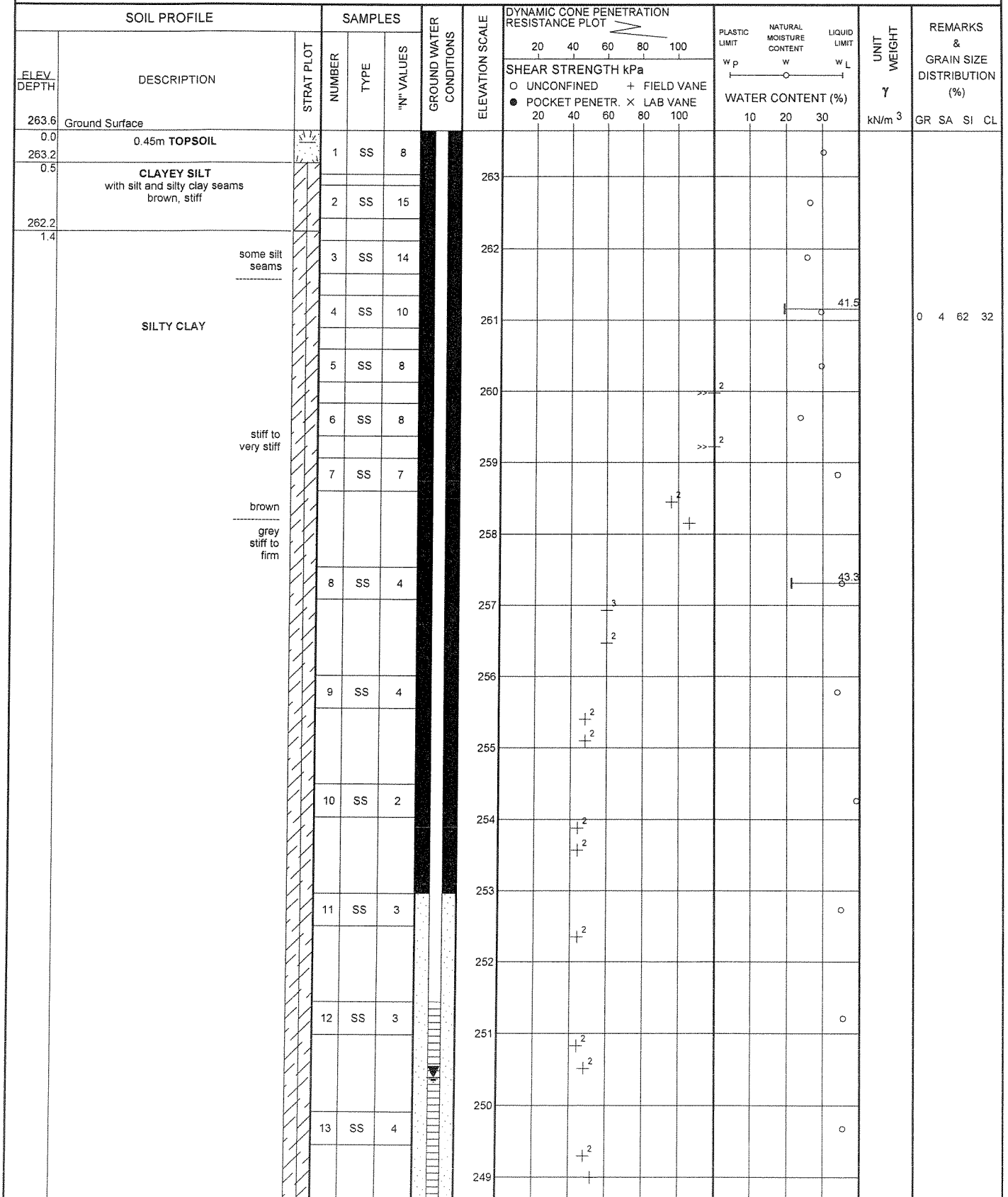
SPT1167

RECORD OF BOREHOLE No 9

1 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 206.7; E 328 310.6 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers COMPILED BY JZ
DATUM Geodetic DATE 8/12/2006 CHECKED BY RM



Continued Next Page

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

SPT1167

RECORD OF BOREHOLE No 9

2 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 206.7; E 328 310.6 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers COMPILED BY JZ
DATUM Geodetic DATE 8/12/2006 CHECKED BY RM

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			20 40 60 80 100	20 40 60 80 100	W _P W W _L	10 20 30			
244.1	SILTY CLAY grey, firm to stiff		14	SS	2		248	2.0						
							247							
							246	3.0	2.0					
							245	3.0	3.0					
19.5			16	SS	4									
<p>End of borehole.</p> <p>Borehole dry upon completion (not stabilized).</p> <p>Piezometer installed in 18.2m</p> <p>Date W. L. in Piezometer:</p> <p>8/16/06 17.6m(EI. 246.0m)</p> <p>8/18/06 17.0m(EI. 246.6m)</p> <p>8/19/06 16.9m(EI. 246.7m)</p> <p>8/22/06 15.9m(EI. 247.7m)</p> <p>8/24/06 15.6m(EI. 248.0m)</p> <p>8/26/06 15.4m(EI. 248.2m)</p> <p>9/04/06 13.9m(EI. 249.7m)</p> <p>9/08/06 13.8m(EI. 249.8m)</p> <p>9/11/06 13.5m(EI. 250.1m)</p> <p>9/13/06 13.3m(EI. 250.3m)</p> <p>9/14/06 13.2m(EI. 250.4m)</p> <p>* W. L. possibly not stabilized.</p>														



SPT1167

RECORD OF BOREHOLE No 10

1 OF 2

METRIC

GWP 181-92-00

LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 025.4; E 328 382.8

ORIGINATED BY GI

DIST HWY 577

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY JZ

DATUM Geodetic

DATE 8/13/2006 to 8/14/2006

CHECKED BY RM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
266.7	Ground Surface													
0.0	FILL: SAND and GRAVEL brown, loose		1	SS	6									
266.2														
0.5	FILL: FINE SAND brown, moist, compact		2	SS	11									
265.6														
1.1	CLAYEY SILT with silt and silty clay seams brown, firm		3	SS	6									
264.6														
2.1														
	SILTY CLAY grey, firm		4	SS	3									
			5	SS	2									
			6	SS	2									
			7	SS	2									
			8	SS	3									
			9	SS	2									
			10	SS	3									
			11	SS	3									
			12	SS	2									
			13	SS	3									
252.2														
14.5	End of borehole.													

Continued Next Page

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

SPT1167

RECORD OF BOREHOLE No 10

2 OF 2

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 025.4; E 328 382.8 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers COMPILED BY JZ
DATUM Geodetic DATE 8/13/2006 to 8/14/2006 CHECKED BY RM

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 (%)					
	No free-standing water in open borehole upon completion(not stabilized); Hole open to 14.0m, Piezometer installed to 14m.													
	Date 8/16/06 8/18/06 8/22/06 8/24/06 8/26/06 9/04/06 9/06/06 9/07/06 9/09/06 9/11/06 9/12/06 9/13/06 9/14/06													
	W. L. in piezometer: 13.4m(El. 253.3m) 13.1m(El. 253.6m) 12.4m(El. 254.3m) 12.2m(El. 254.5m) 12.0m(El. 254.7m) 10.2m(El. 256.5m) 10.15m(El. 256.55m) 9.2m (El. 257.5m) 9.1m (El. 257.6m) 8.2m (El. 258.5m) 7.45m(El. 259.25m) 7.15m(El. 259.55m) 7.2m(El. 259.5m)													

+ 3, x 3: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE



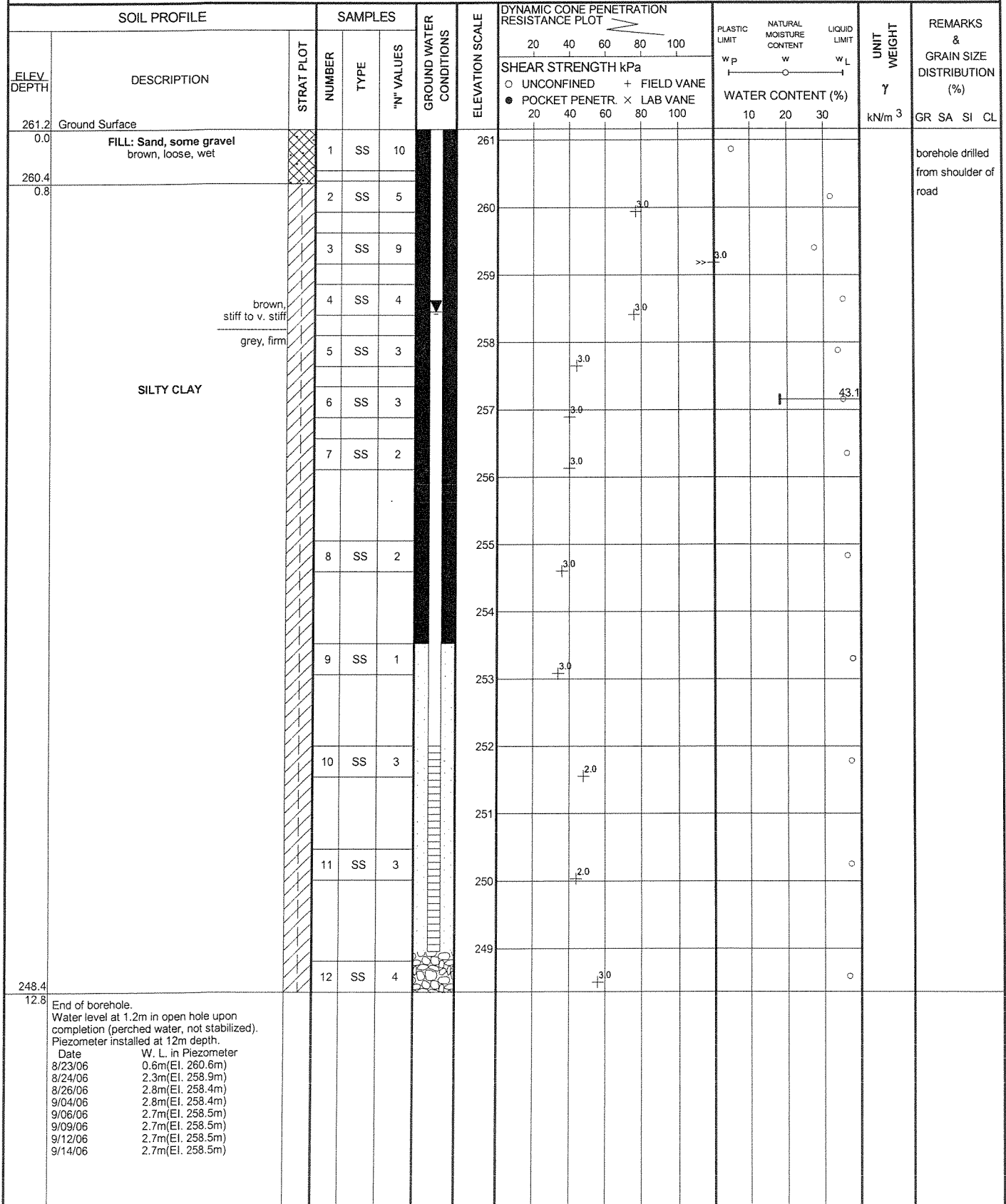
SPT1167

RECORD OF BOREHOLE No 11

1 OF 1

METRIC

GWP 181-92-00 LOCATION Meadow Creek Bridge, Iroquois Falls, ON, Coords: N 5 401 545.4; E 328 261.7 ORIGINATED BY GI
DIST HWY 577 BOREHOLE TYPE Hollow Stem Augers COMPILED BY HL
DATUM Geodetic DATE 8/22/2006 CHECKED BY RM



+ 3, x 3: Numbers refer to
Sensitivity

20
15
10
5
0

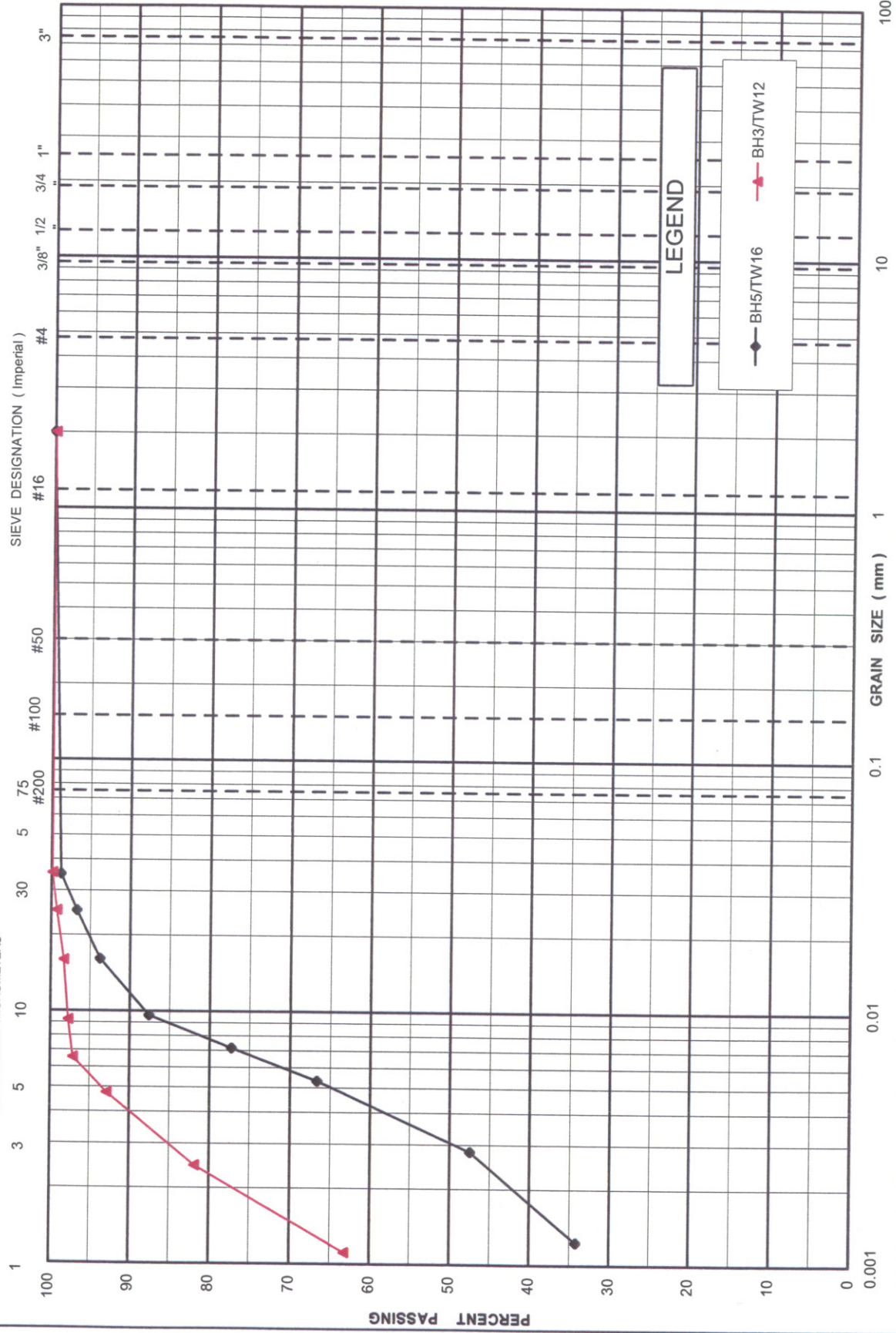
(%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	



SHAHEEN & PEAKER LIMITED

GRAIN SIZE DISTRIBUTION

SILTY CLAY

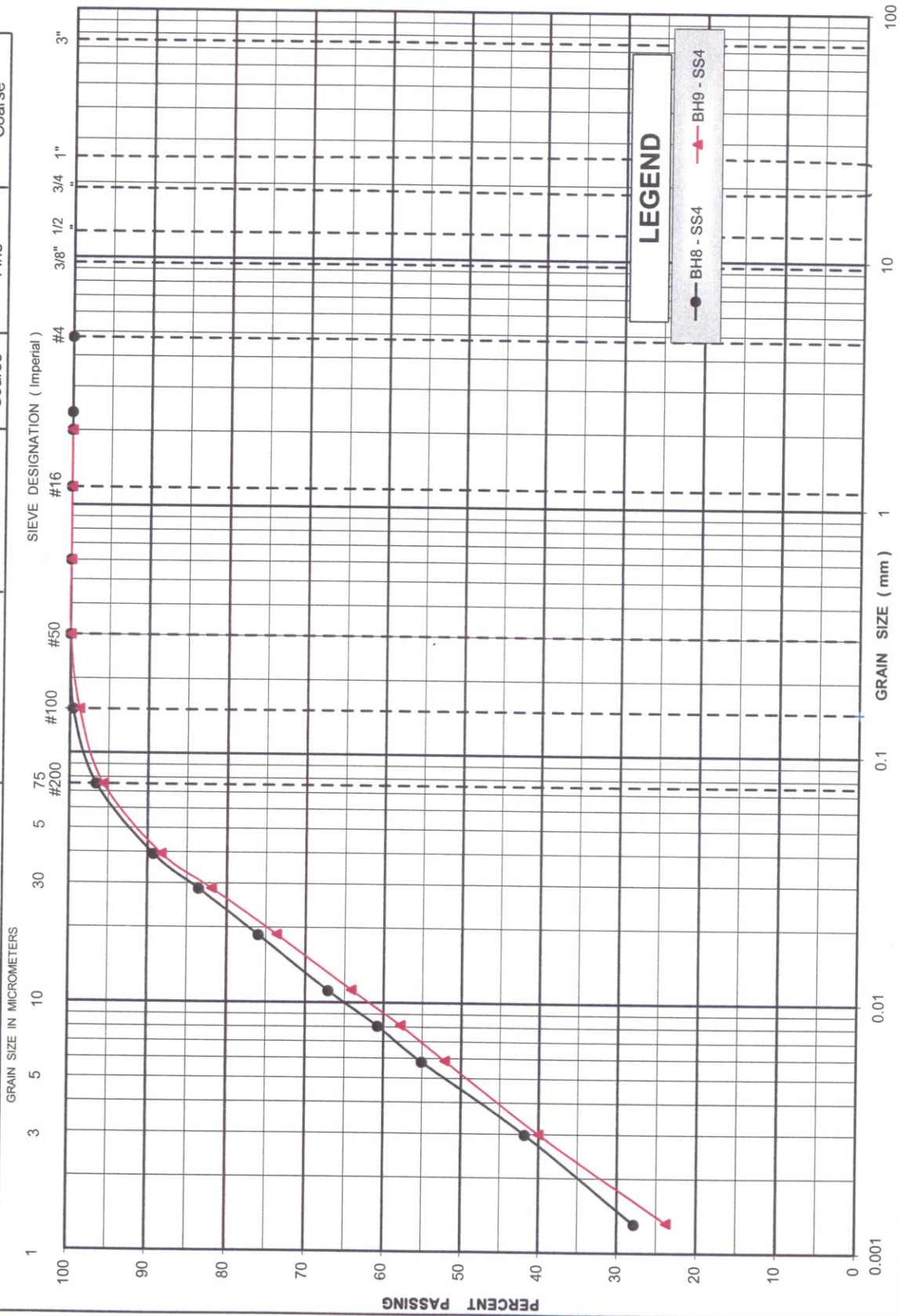
FIGURE No. B-1

REF. No. SPT 1167

DATE SEPTEMBER, 2006

UNIFIED SOIL CLASSIFICATION SYSTEM

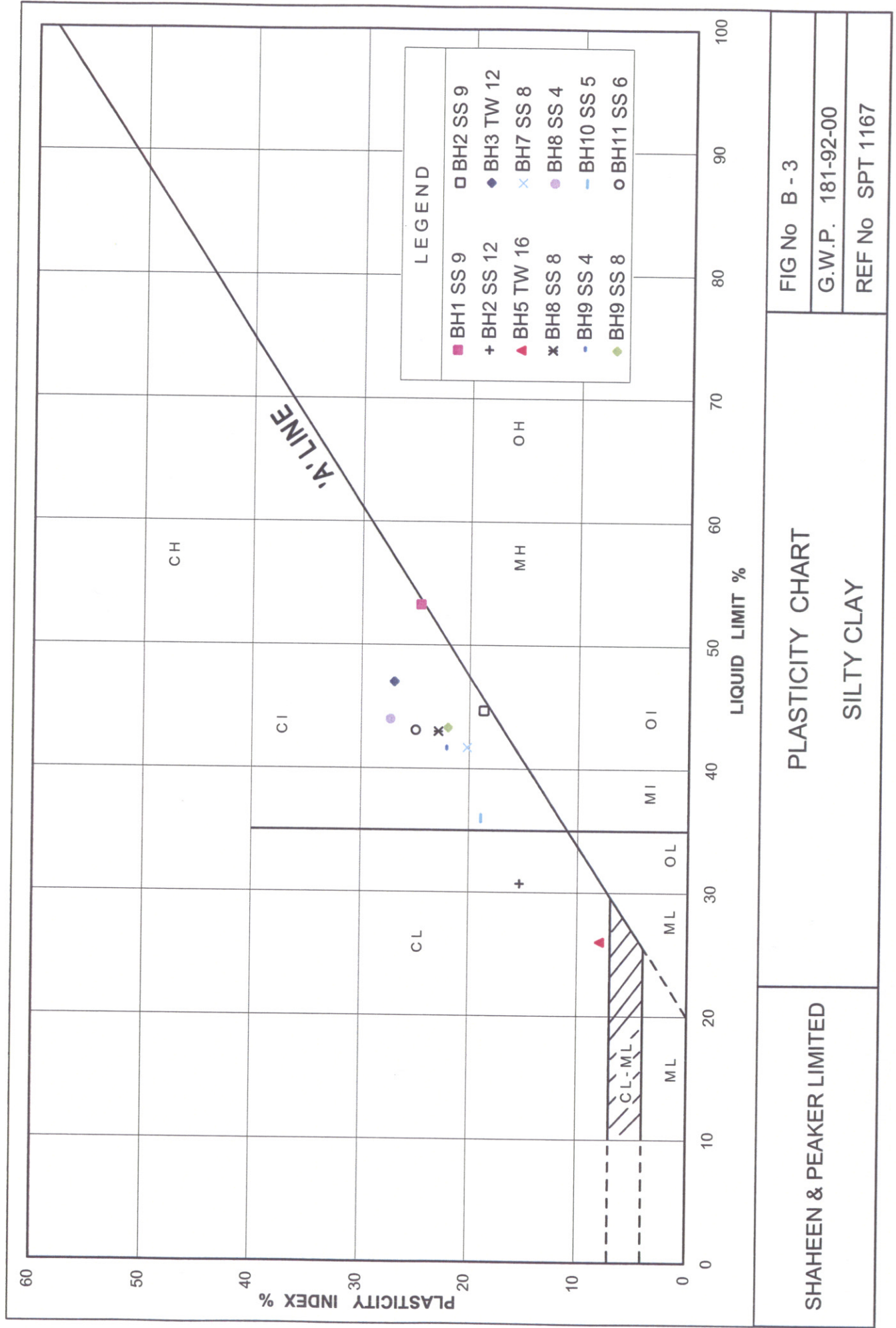
CLAY AND SILT		SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS		Fine	Medium	Coarse	Fine	Coarse	Coarse



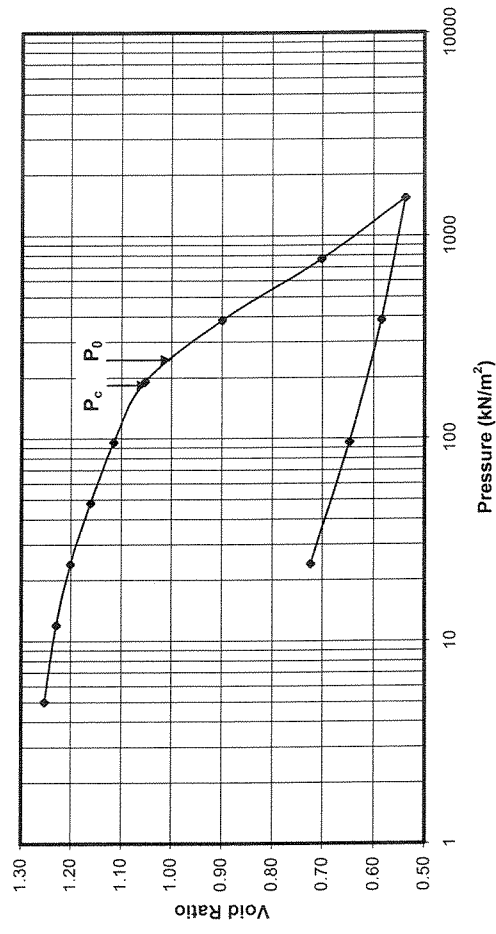
GRAIN SIZE DISTRIBUTION
SILTY CLAY

SHAHEEN & PEAKER LIMITED

SAMPLE No.: B - 2
PROJECT No: SPT - 1167
Date: October 20, 2006



**Figure B - 4 Consolidation Test Results for Sample
BH3 - TW12 From 10.7 - 11.1m Depth**
Void Ratio versus Pressure



Coefficient of Consolidation vs. Pressure

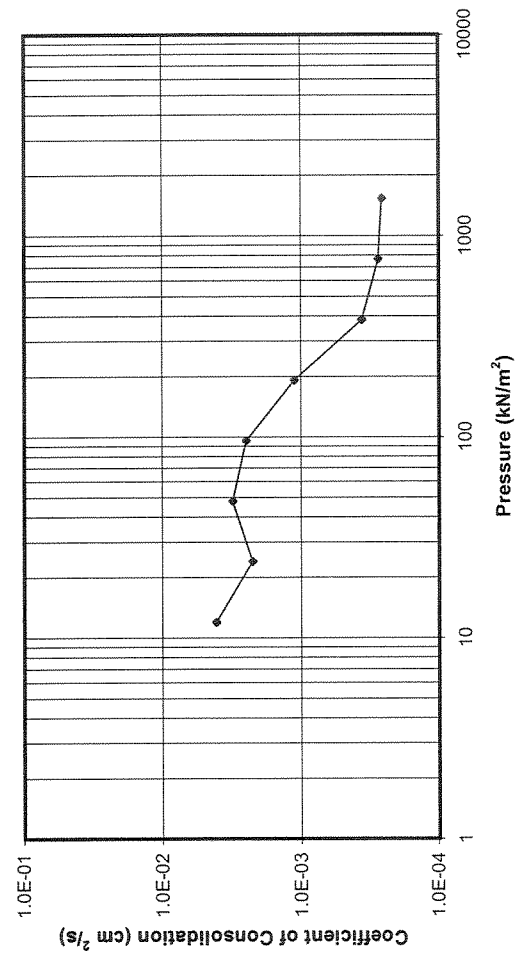
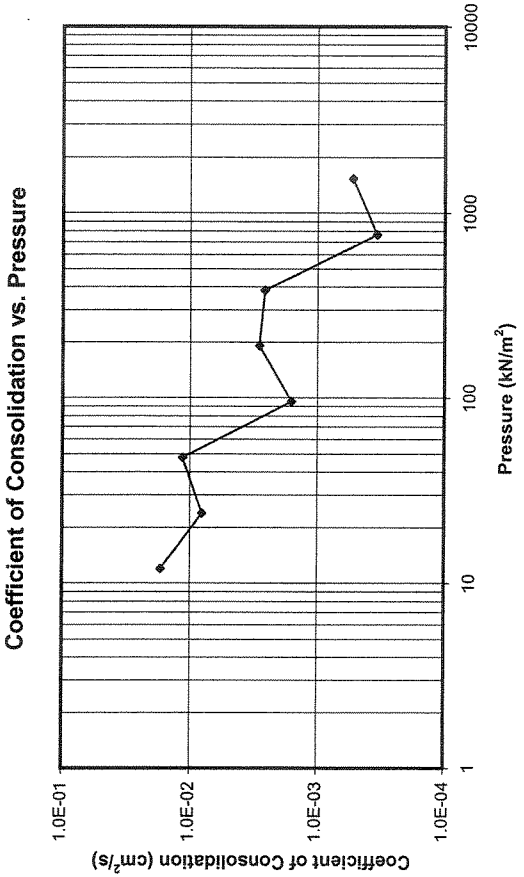
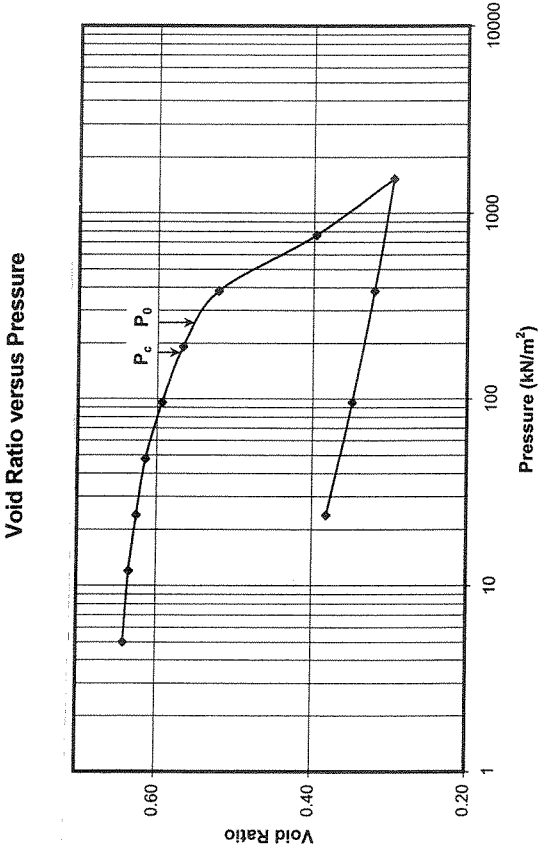
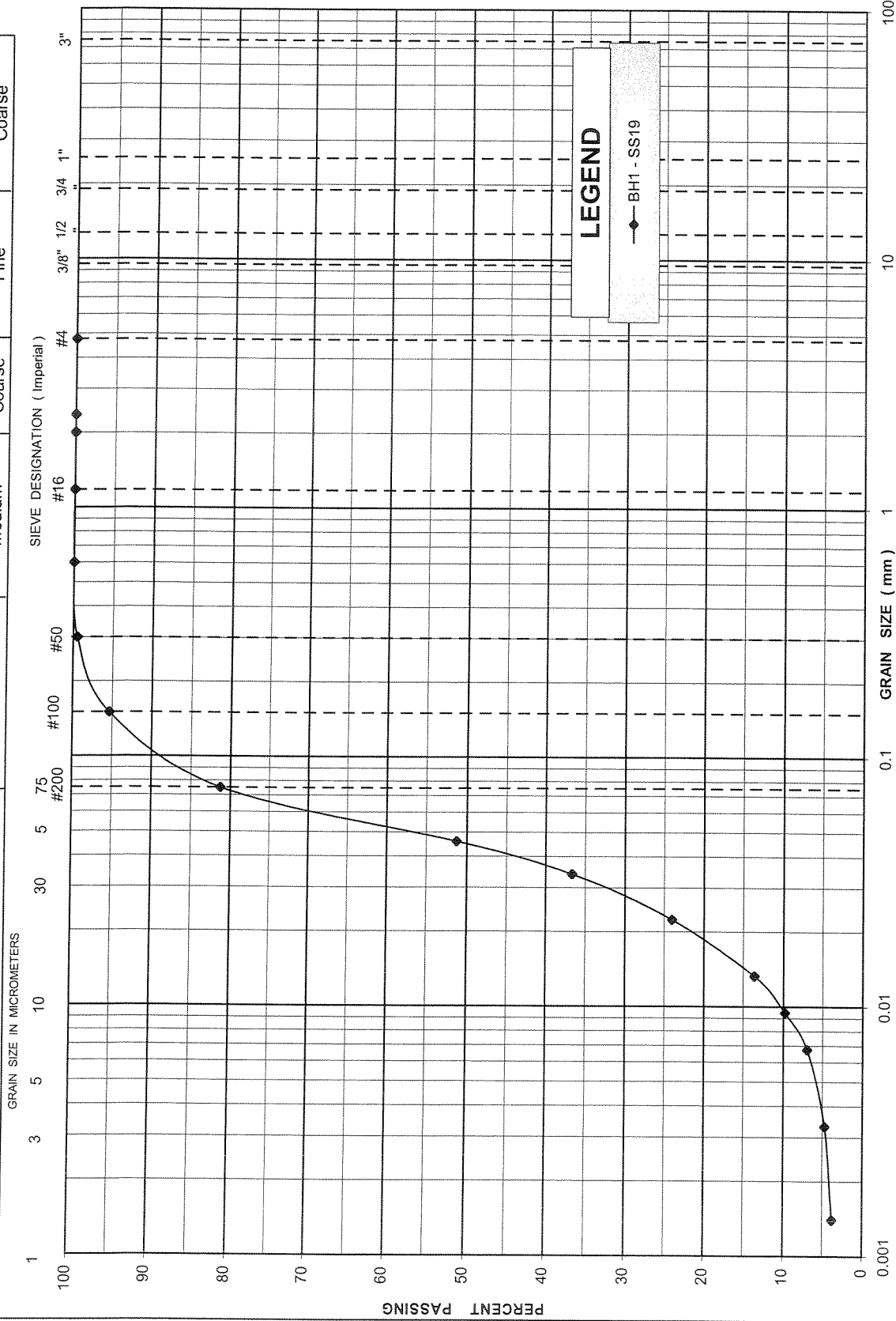


Figure B - 5 - Consolidation Test Results for Sample
BH5 - TW16 Depth 15.25 - 15.7m



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



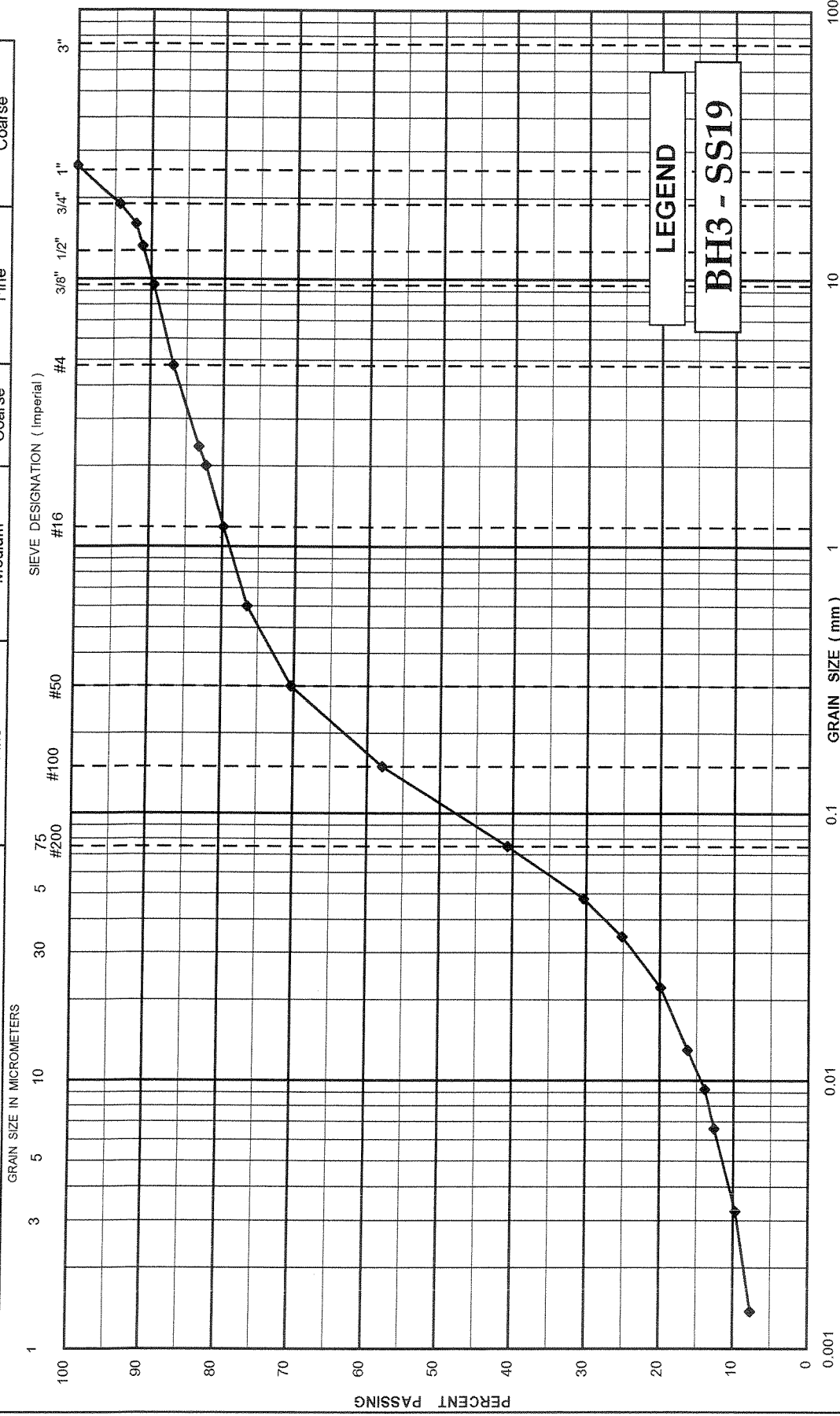
GRAIN SIZE DISTRIBUTION

SHAHEEN & PEAKER LIMITED

SAMPLE No.: B - 6
PROJECT No: SPT - 1167
Date: October 30, 2006

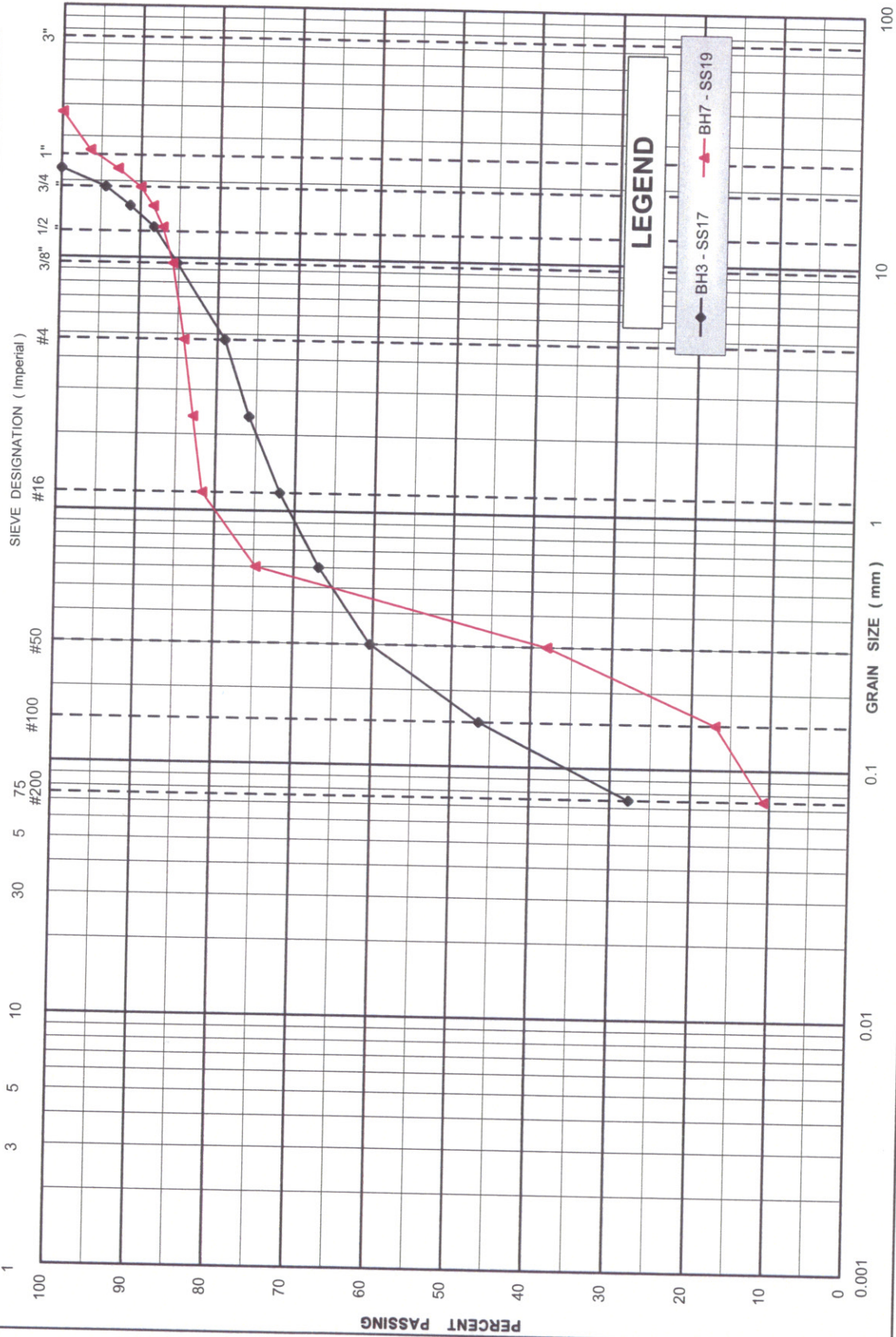
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



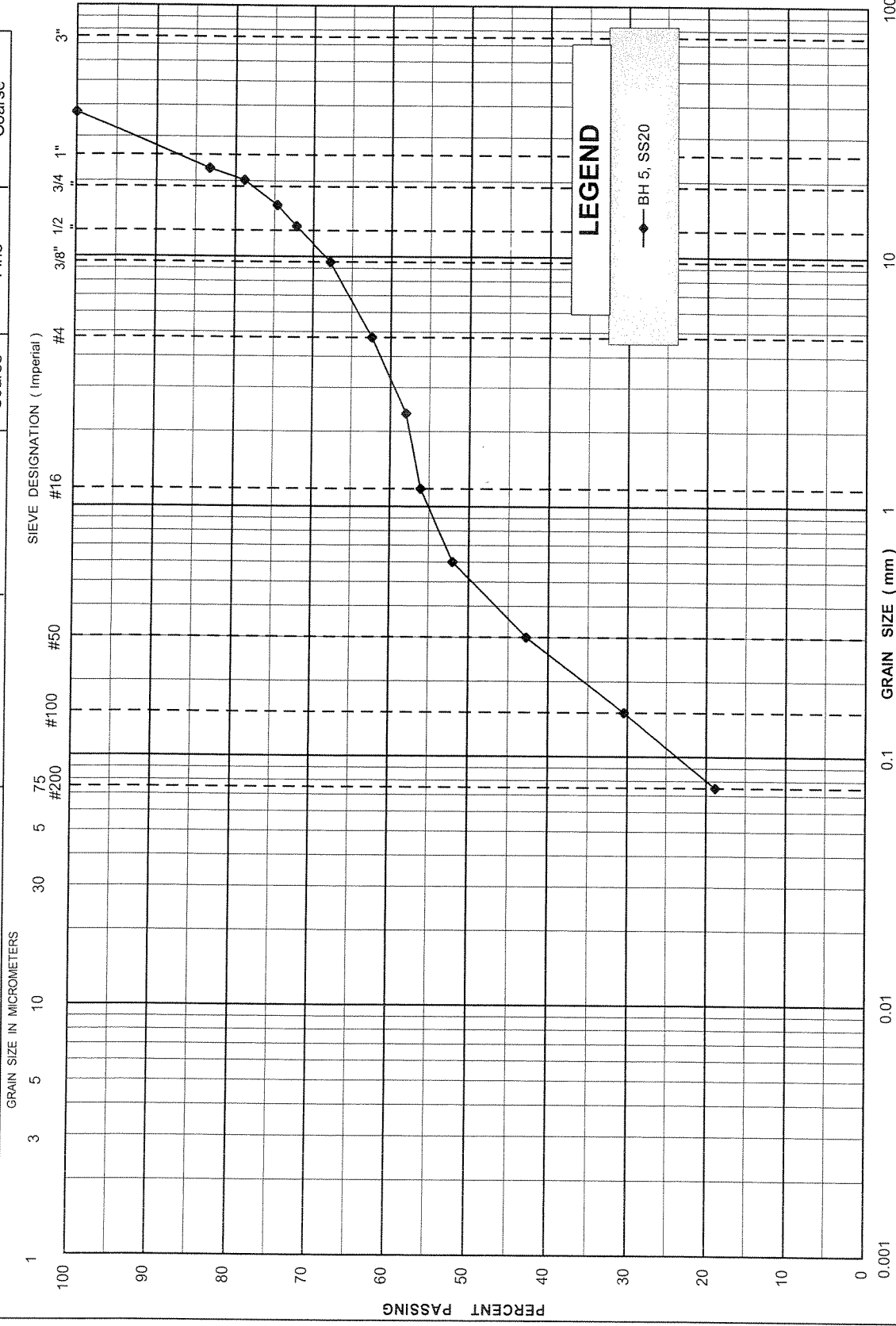
UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT			SAND			GRAVEL		
GRAIN SIZE IN MICROMETERS			Fine	Medium	Coarse	Fine	Coarse	



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT		SAND			GRAVEL		
		Fine	Medium	Coarse	Fine	Coarse	



GRAIN SIZE DISTRIBUTION
GRAVELLY SAND, some silt

SAMPLE No.: B - 9
PROJECT No: SPT - 1167
Date: October 20, 2006

SHAHEEN & PEAKER LIMITED

Appendix C

Undrained Shear Strength Plots

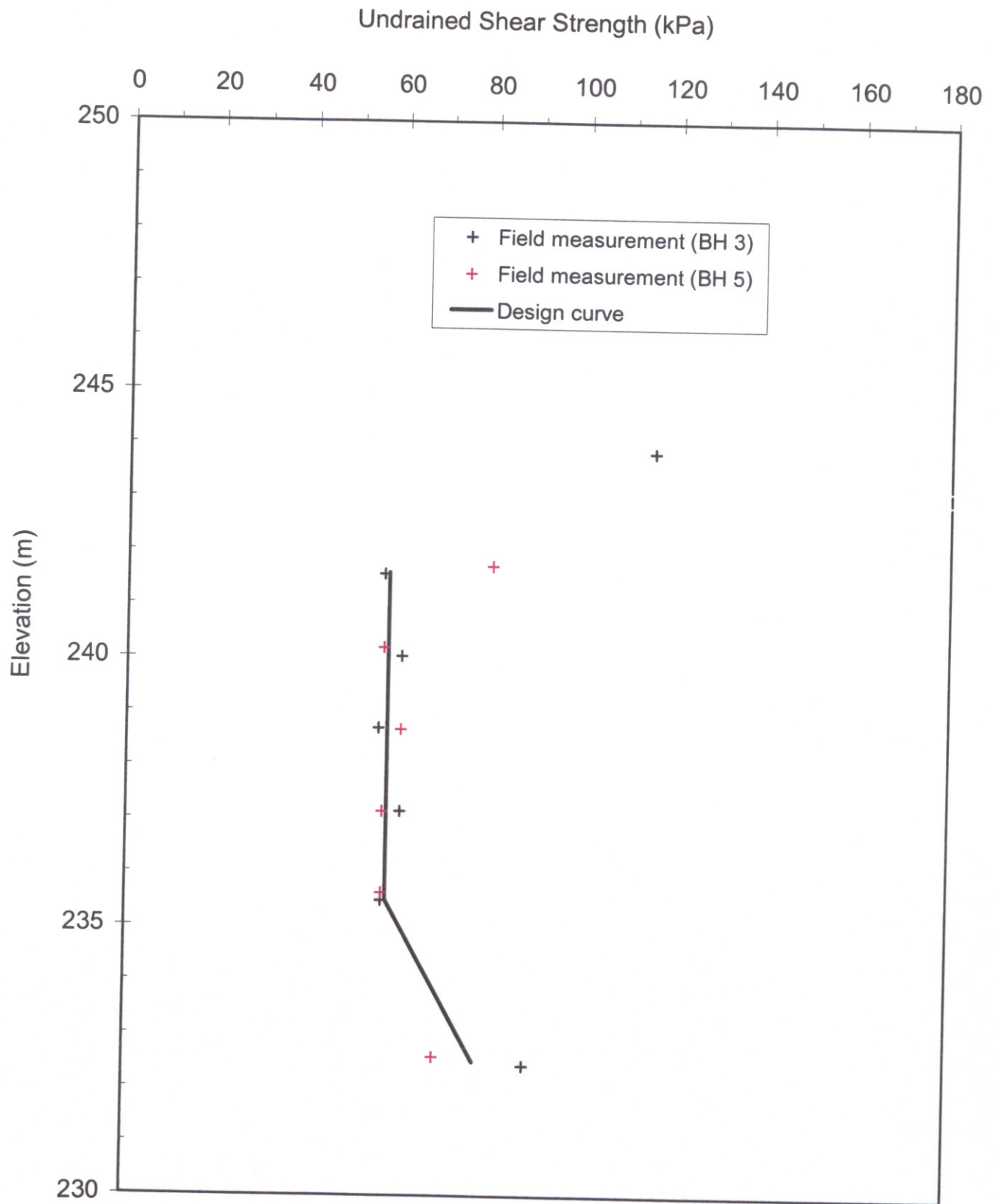


Fig. C1 - Variation of undrained shear strength (as measured by field vane tests) with Elevation - Boreholes 3 and 5

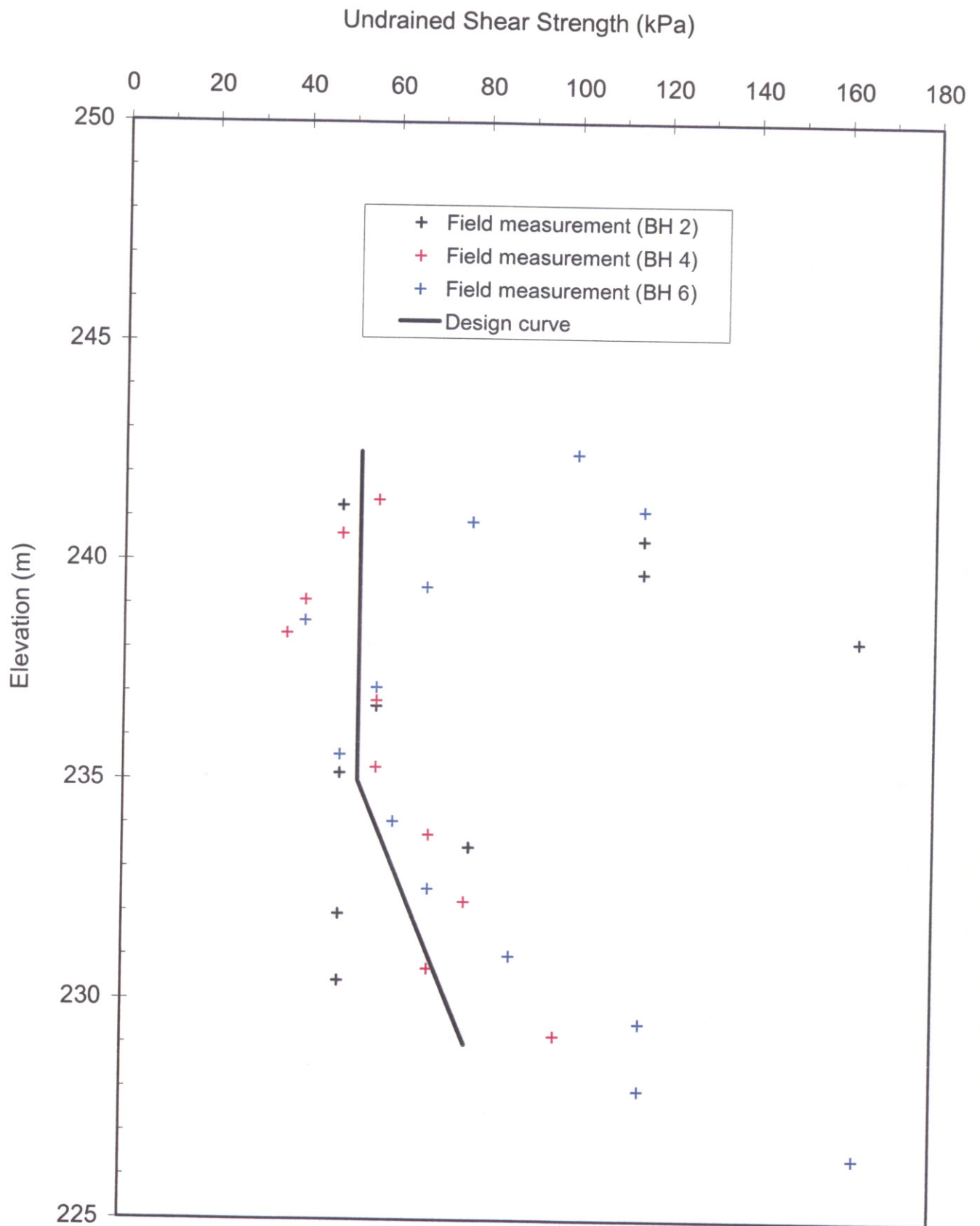


Fig. C2 - Variation of undrained shear strength (as measured by field vane tests) with Elevation - Boreholes 2, 4 and 6

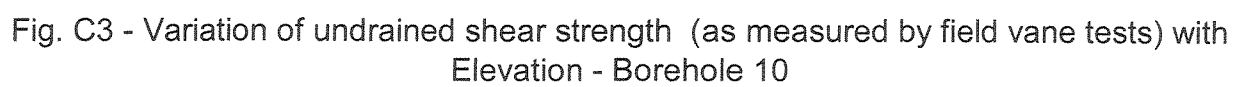
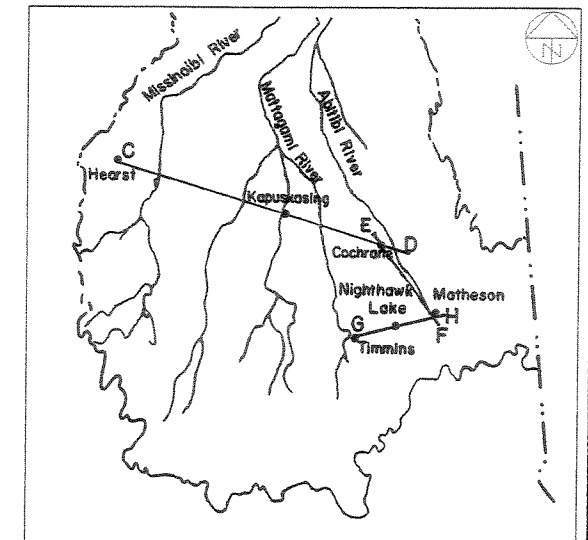


Fig. C3 - Variation of undrained shear strength (as measured by field vane tests) with Elevation - Borehole 10

Appendix D

Geology Map

SHAHEEN & PEAKER LIMITED



INDEX PLAN

LEGEND

Aquifers

Sand, gravel

Sand

Aquitards

Clay, silt

Clay till

Crystalline bedrock

Flowing well

Water well record number

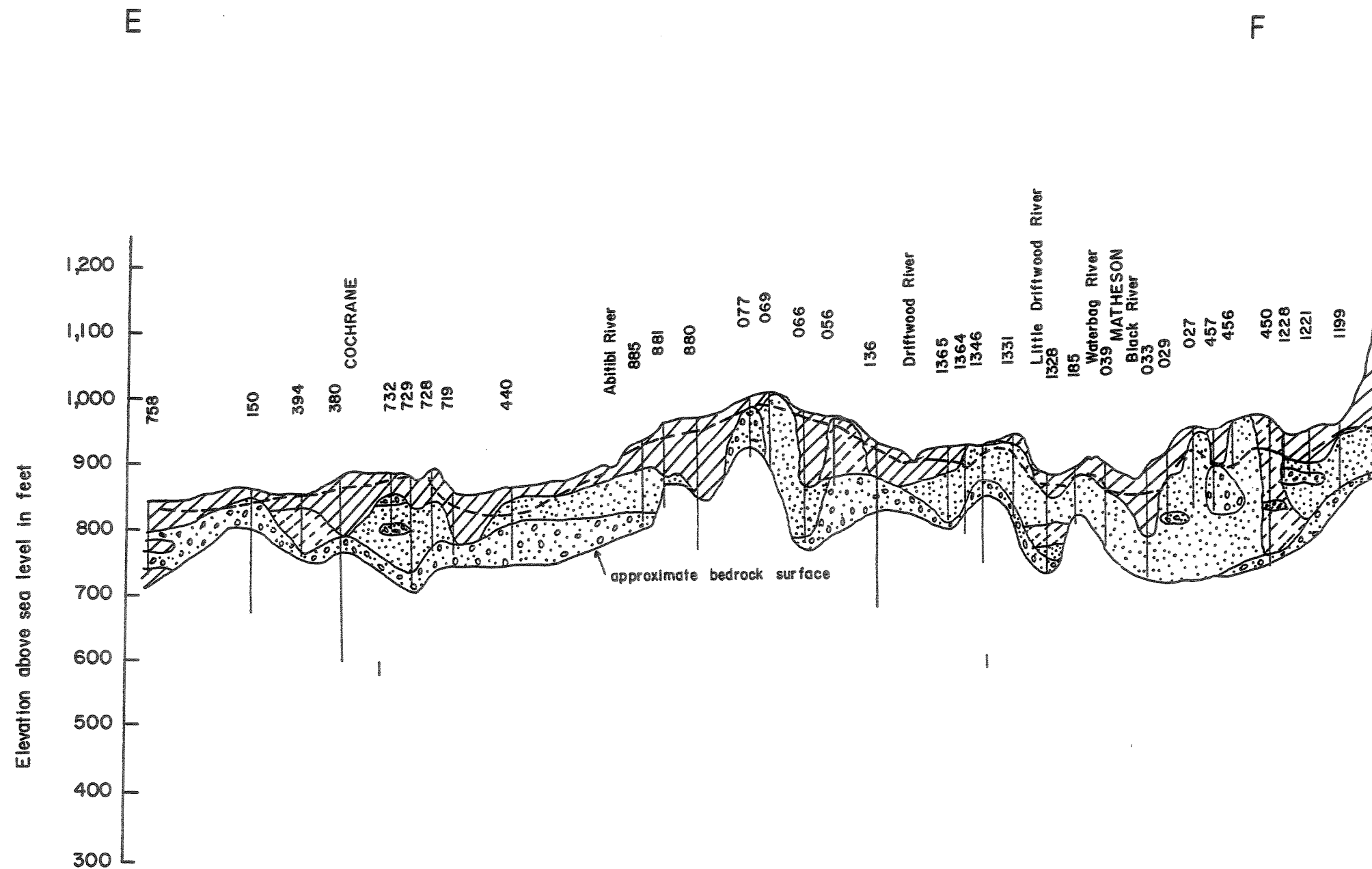
Piezometric surface

SOURCES OF INFORMATION

1. Ontario Water Resources Commission, 1972, Water well records in Ontario, northern area, 1946 - 1969; Water Res. Bull. 2 - 9.

2. Unpublished water well records on file with the Ontario Ministry of the Environment to the end of 1972.

SPT PROJECT 1167	DIST
SUBM'D ZO	CHECKED FS
DATE Dec. 2008	SHE
DRAWN HL	CHECKED FS
APPROVED	DWG



Appendix E

Excerpts from Previous 1971 Investigation Report By MTO

Department of Transportation and Communications

XXXXXXXXXXXXXXXXXXXX

MEMORANDUM

42 A - 10

TO: Mr. H. W. Hurrell,
Regional Road Design Supt.,
Northwestern Region,
THUNDER BAY, Ontario.

FROM: Foundation Section,
Room 107, Lab. Bldg.

ATTENTION:

DATE: June 25, 1971

OUR FILE REF.

IN REPLY TO

JUN 29 1971

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Proposed Realignment of Sec. Hwy. #577
Station 41+00 - Station 118+00 and
Station 406+00 - Station 422+00
Lines 'D', 'E' and 'F'
District No. 16 (Cochrane)
W.O. 70-11098 -- W.P. 100-65-02
CONT. 74-84

Attached, we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above structure site.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please feel free to contact our Office.

AGS/MdeF
Attach.

cc: Messrs. H. W. Hurrell (2)
F. G. Allen
D. W. Farren
D. S. Corneli
R. Morgenroth
L. P. Shorr
B. J. Gircux
B. A. Singh
Foundations Files
Gen. Files

A. G. Stermac
A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

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1. INTRODUCTION.
 2. DESCRIPTION OF THE SITE.
 3. FIELD AND LABORATORY WORK.
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 - 4.2) Varved Clay.
 - 4.3) Clayey Silt with some Sand.
 - 4.4) Silt with Traces of Clay.
 - 4.5) Silty Sand with Traces of Gravel and Clay.
 - 4.6) Sandy Silt to Sand with some Gravel.
 5. GROUNDWATER CONDITIONS.
 6. DISCUSSION AND RECOMMENDATIONS:
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 - 6.3) Site 'B' - Sta. 106+00 to Sta. 110+00.
 - 6.4) Site 'C' - Sta. 99+00 to Sta. 102+00.
 - 6.5) Site 'D' - Sta. 83+00 to Sta. 87+00.
 - 6.6) Site 'E' - Sta. 58+00 to Sta. 62+00.
 - 6.7) Site 'F' - Sta. 49+00 to Sta. 52+00.
 - 6.8) Site 'G' - Sta. 41+00 to Sta. 44+00.
 7. MISCELLANEOUS.
-

FOUNDATION INVESTIGATION REPORT
For
Proposed Realignment of Sec. Hwy. #577
Station 41+00 - Station 118+00 and
Station 406+00 - Station 422+00
Lines 'D', 'E' and 'F'
District No. 16 - (Cochrane)
W.O. 70-11098 -- W.P. 100-66-02

1. INTRODUCTION:

A request for a subsoil investigation at the site of several fill areas on the proposed realignment of sec. Hwy. 577 from Sta. 41 + 00 to Sta. 118 + 00 and from Sta. 406 + 00 to 422 + 00 was received from Mr. R. Morgenroth, Regional Materials Engineer in a memorandum dated October 9, 1970.

A field investigation was subsequently carried out by the Foundation Section to determine the subsoil conditions existing at the various sites. This report contains the results of this investigation and our recommendations pertaining to the design of the proposed embankments and cuts.

2. DESCRIPTION OF THE SITE:

It is proposed to reconstruct Hwy. #577 from Monteith to Ansonville, a total of some 7 miles in length - i.e. Sta. 0 = 00 at the intersection with Hwy. #11 just south of Monteith to Sta. 422 + 00 at Meadow Creek on the outskirts of Ansonville. This reconstruction will consist of complete realignment from Sta. 41 + 00 to Sta. 118 + 00 and Sta. 406 + 00 to Sta. 422 + 00.

2. DESCRIPTION OF THE SITE: (cont'd) ...

The topography generally is flat to rolling, although in the areas mentioned above this is punctuated by deep steep sided gulleys formed by natural drainage courses; it is these areas that are the subject of the investigation. Along the highway there are occasional houses and cleared areas for agricultural purposes though, generally the area is bush covered.

Physiographically, the site is located on a clay plain deposited in a glacial lake during the Wisconsin Glacial Substage.

3. FIELD AND LABORATORY WORK:

A total of twenty-six boreholes of which were accompanied by a dynamic cone penetration test, was put down during this investigation. Equipment used consisted of a continuous flight auger and a diamond drill rig adapted for soil sampling purposes. One borehole - i.e. No. 23 was put down by means of a hand auger as also were Nos 27 to 32 which were undertaken by Regional Soils Personnel and are included in this report.

Disturbed soil samples were obtained at the required depths in the overburden in a 2" O.D. split spoon sampler which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. Undisturbed samples were obtained by pushing a 2" I.D. Shelby tube into the cohesive portions of the overburden. Insitu field vane tests were carried out 18" below sample depths whenever possible.

3. FIELD AND LABORATORY WORK: (cont'd) ...

The locations and elevations of all boreholes were surveyed by personnel from Cochrane District Office and are shown on Drawing Nos. 7C-11098A & B which accompany this report.

All samples were subjected to careful visual examination in the field and subsequently in the laboratory. Selected samples were then tested in the laboratory in order to determine the following physical properties.

- Bulk Density
- Moisture Content
- Atterberg Limits
- Undrained Shear Strength
- Grain Size Distribution
- Consolidation Characteristics
- Effective Stress Parameters
- Organic Content

The results of the various tests are given on the individual Record of Borelog Sheets.

4. SUBSOIL CONDITIONS:

4.1) General:

The predominant stratum over the whole area of the investigation generally consists of soft to very stiff varved clay overlying either stiff to very hard clayey silt or very dense sand. The boreholes undertaken in the cut areas indicated some dessication of the upper zones of the varved clay while those boreholes in the valleys showed traces of organic material. The varved clay consists of layered silty clay and clay; in some areas seams of clayey silt to silt were found in the lower zones: the thickness of the latter seams increasing with depth.

4. SUBSOIL CONDITIONS: (cont'd) ...

4.1) General: (cont'd) ...

Several distinct sites were investigated; Site 'A' from Sta. 414 + 00 to Sta. 420 + 00 at the North end of Hwy. 577 and Sites 'B' to 'G' from Sta. 110 + 00 to Sta. 41 + 00 at the South end of Hwy. 577.

The predominant stratum over the whole area of the investigation generally consists of from 29 to 60 ft. of soft to stiff varved clay overlying either dense silts and sands or stiff to very hard clayey silt. The boreholes in the proposed cut sections indicated some dessication of the upper zones of the varved clay deposit while in the valleys the top 5-8 ft. of the deposit contained traces of organics, and was sometimes overlain by a shallow layer of muck.

The boundaries of the different deposits as determined in the boreholes are shown on the accompanying borelog sheets and the estimated stratigraphical profile contained in Drawing 70-11098A & B is based on this information. From ground level downward the various soil types are as follows: -

4.2) Varved Clay:

This material was found from ground level down or alternatively underlying surficial deposits of muck.

The thickness of the deposit over the area of the investigation generally varied between 29 to 60 ft.

The varved clay consists of layered silty clay and clay; in some areas seams of clayey silt to silt were found in the lower zones; the thickness of the aforementioned seams

4. SUBSOIL CONDITIONS: (cont'd) ...

4.2) Varved Clay: (cont'd) ...

increasing with depth.

Field and laboratory tests gave the following results: -

A. Silty Clay to Clay:

Plastic Limit (%)	20 - 30
Liquid Limit (%)	35 - 70
Moisture Content (%)	33 - 63
N values blows/ft.	1 - 18
Density P.C.F.	99 - 120

Undrained Shear Strength { Field vane tests p.s.f. 350 - over 2000
Unconfined Compression
& Triaxial Tests p.s.f. 250 - 2300

Sensitivity 2.8 to 13

Compressibility Characteristics

Void Ratio (eo)	1.0 to 2.1
Compression Index (Cc)	.27 to 1.09
Degree of Preconsolidation 0 to 4,800 p.s.f. (Pc - Po)	

Effective Stress Parameters

C' (p.s.f.)	0 to 740
ϕ (°)	24° to 30°

B. Clayey Silt to Silt Layers:

Moisture Content (%)	20 - 30
Plastic Limit (%)	14 - 22
Liquid Limit (%)	18 - 29

The Atterberg Limits ranged from non-plastic to the above values.

Plots of plasticity index vs. liquid limit for the deposit are included in the Appendix in Fig. #1 and grain size distribution curves in Fig. #2.

4. SUBSOIL CONDITIONS: (cont'd) . . .

4.3) Clayey Silt with some Sand:

This deposit was found underlying the varved clay in several locations with a minimum thickness of 9 ft. in borehole #20 where it presumably overlies bedrock. Elsewhere, the boreholes were terminated in this layer. The consistency of the material is stiff to very hard. Field and laboratory tests gave the following results: -

Moisture Content	(%)	20 - 26
Plastic Limit	(%)	9 - 18
Liquid Limit	(%)	18 - 28
Grain Size Distribution	(%)	Gra. 0 Sa. 4-39 Si. 41-67 Cl. 20-29
Bulk Density (p.c.f.)		124-126
Undrained Shear Strength (p.s.f.)	{ Field vane Unconfined Compression Tests	1,400 to over 2000 400 to 1,100

Typical grain size distribution curves are included in the Appendix in Fig. #3.

4.4) Silt with Traces of Clay:

This material also underlies the varved clay deposit in several locations. It has a minimum thickness of 8 ft. and overlies either bedrock or the dense sand deposit. 'N' values as determined by the Standard Penetration Test varied from 8 to 21 blows per foot indicating a 'loose' to 'compact' material.

Laboratory tests gave the following results: -

Moisture Content	(%)	18 - 30 (mean 20)
Grain Size Distribution	(%)	Gra. 0 Sa. 0-1 Si. 79.91 Cl. 8 - 20

4. SUBSOIL CONDITIONS: (cont'd) ...

4.4) Silt with Traces of Clay: (cont'd) ...

Typical grain size distribution curves are included in the Appendix in Fig. #4.

4.5) Silty Sand with Traces of Gravel & Clay:

This material was found to underlie either the varved clay deposit or the silt deposit in most locations. It has a minimum thickness of 4 feet in borehole #22 where it presumably overlies bedrock; elsewhere, the boreholes were terminated in this layer. 'N' values as obtained from the Standard Penetration Test varied from 53 blows/ft. to 93 blows/ft, indicating a very dense material.

Laboratory tests gave the following results:-

Moisture Content (%) 6 - 22

Grain Size Distribution (%) Gra. 0-9 Sa. 51-69
Si. & Cl. 22-49

Typical grain size distribution curves are included in the Appendix in Fig. #5.

4.6) Sandy Silt to Sand with some Gravel:

This deposit was found at Site 'C' only lying directly under the surficial muck deposit or beneath a shallow layer of silty clay. All boreholes done in this area were terminated in this layer. 'N' values as determined by the Standard Penetration Test varied from 7 blows/ft. to 60 blows/ft. indicating a loose to very dense material.

Laboratory tests gave the following results:-

Moisture Content (%) 6-21

Grain Size Distribution (%) Gra. 2-37 Sa. 20-66
Si. & Cl. 4-70

4. SUBSOIL CONDITIONS: (cont'd) ...

4.6) Sandy Silt to Sand with some Gravel: (cont'd) ...

Typical grain size distribution curves are included in the Appendix in Fig. #6.

5. GROUNDWATER CONDITIONS:

Groundwater observations were carried out at the period of the investigation in the open holes. The observations including any artesian conditions are recorded in the borelog sheets and summarized in Drawing #70-11098A & B.

It must be noted that due to the majority of boreholes being undertaken in natural drainage channels and the impermeability of the subsoil, the water levels given cannot be taken as accurate.

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 3

JOB 70-11098

LOCATION Sta. 415 + 55 O/S 100' ft.

ORIGINATED

W.P. 100-66-02

BORING DATE November 18 & 19, 1970

COMPILED BY

DATUM Geodetic

BOREHOLE TYPE Washboring, NX Casing

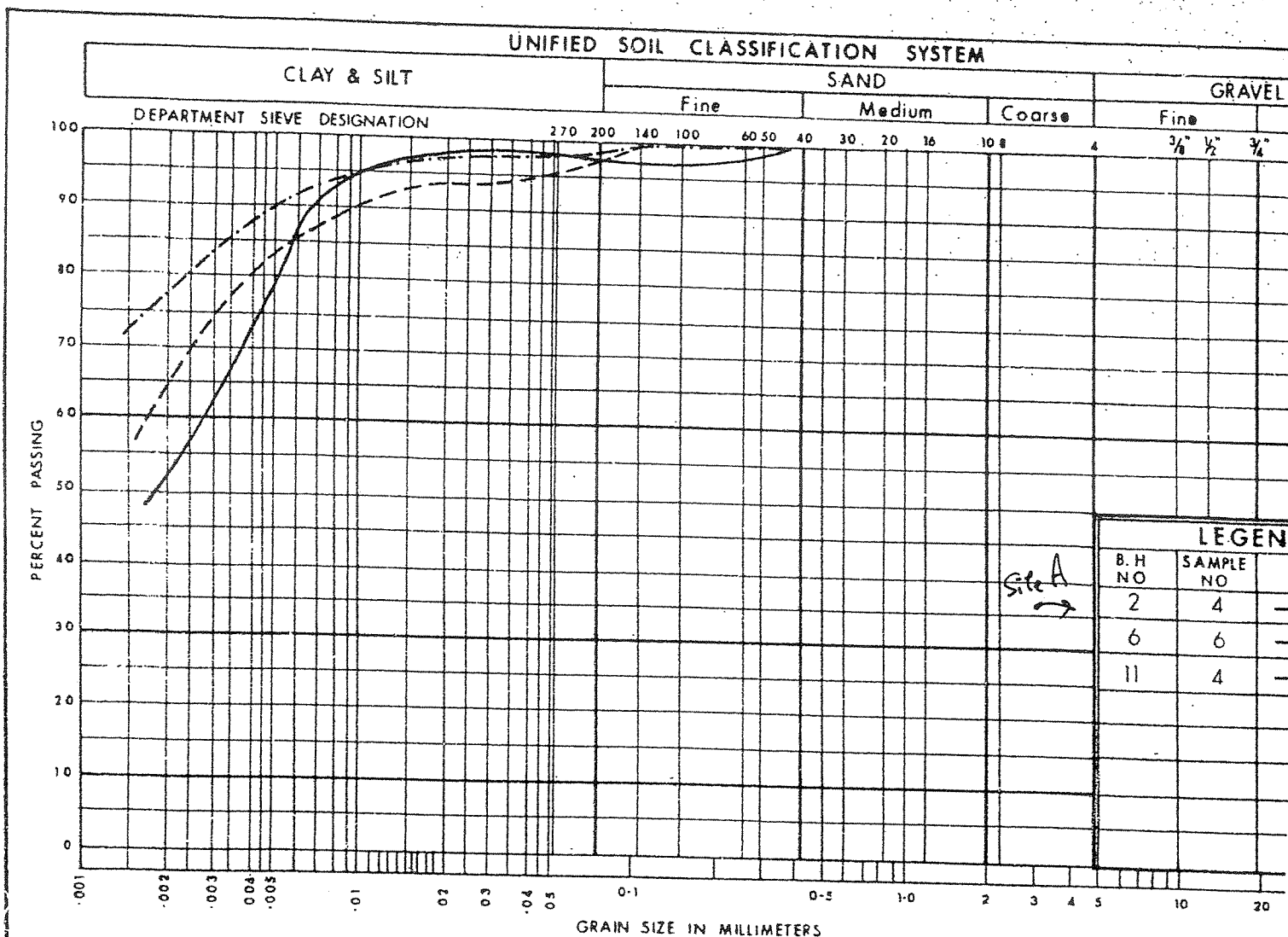
CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — W _L PLASTIC LIMIT — W _P WATER CONTENT — W	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.		W _P — W — W _L	
829.0	Ground Level									
0.0	Silty clay to clay Firm to stiff		1	TW PM						
			2	TW PM						
			3	TW PM		820	δ + S 4.0			
			4	TW PM			+ S 6.0			
			5	TW PM			+ S 3.7			
			6	TW PM			+ S 3.3			
			7	TW PM		310	+ S 4.0			
			8	TW PM			+ S 3.7			
			9	SS F		800	+ S 5.3			
			10	TW PM			+ S 4.3			
			11	SS 11		790	+ S 5.0			
779.0	Clayey silt, some sand Very stiff		12	TW PM		780	+ S 6.0			
50.0			13	SS 22		770	+ S			
762.5	End of Borehole									
66.5										

CHECKED BY

4

60.5 End of Borehole



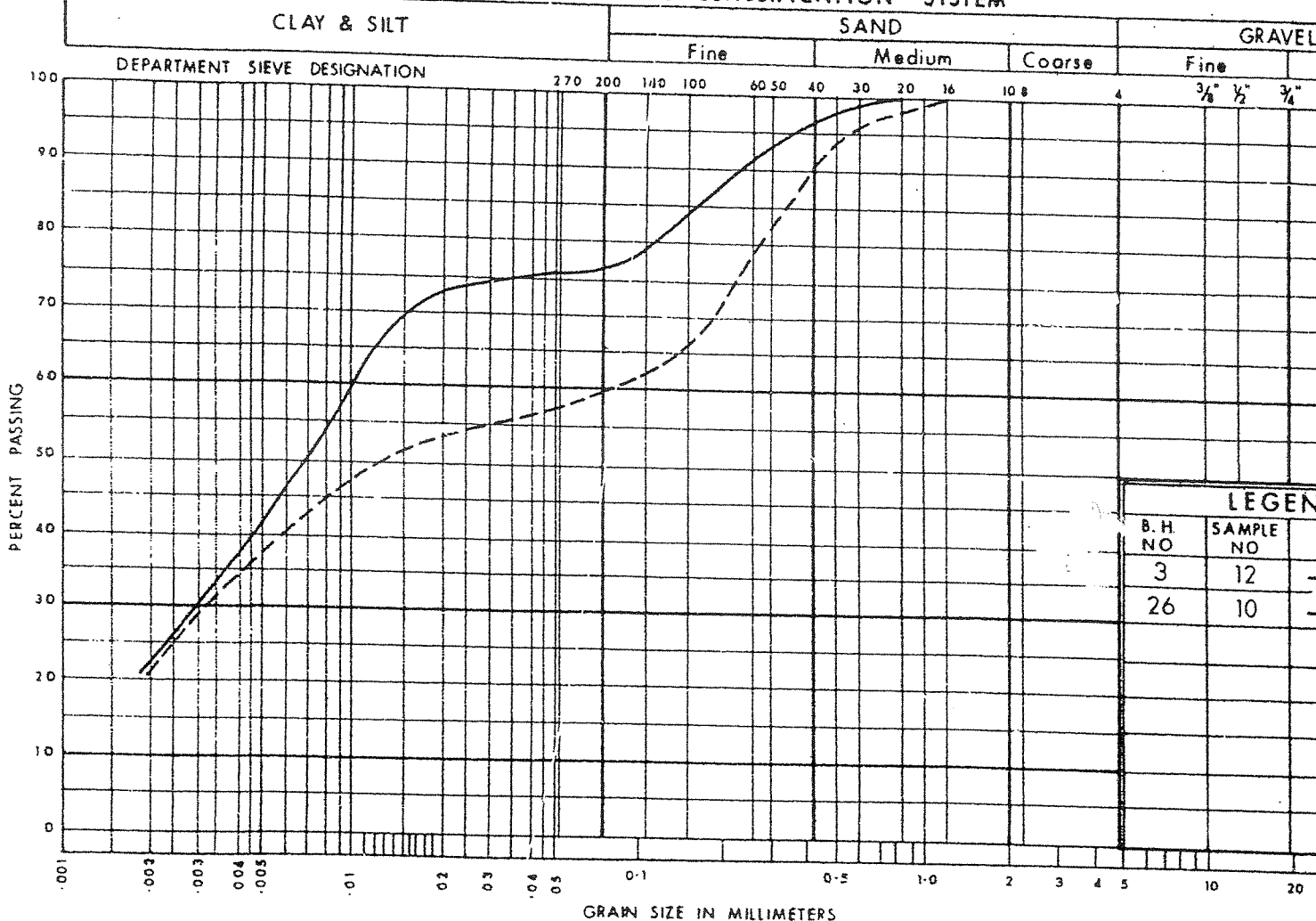
DEPARTMENT OF HIGHWAYS
**MATERIALS and
TESTING
DIVISION**

GRAIN SIZE DISTRIBUTION

VARVED CLAY

W.P. No. 100
JOB No. 70
FIG

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

GRAIN SIZE DISTRIBUTION

CLAYEY SILT WITH SOME SAND

W.P. No. 100

JOB No. 70

FIG

VOID RATIO - PRESSURE CURVES

JOB NO. 70 - 11098.

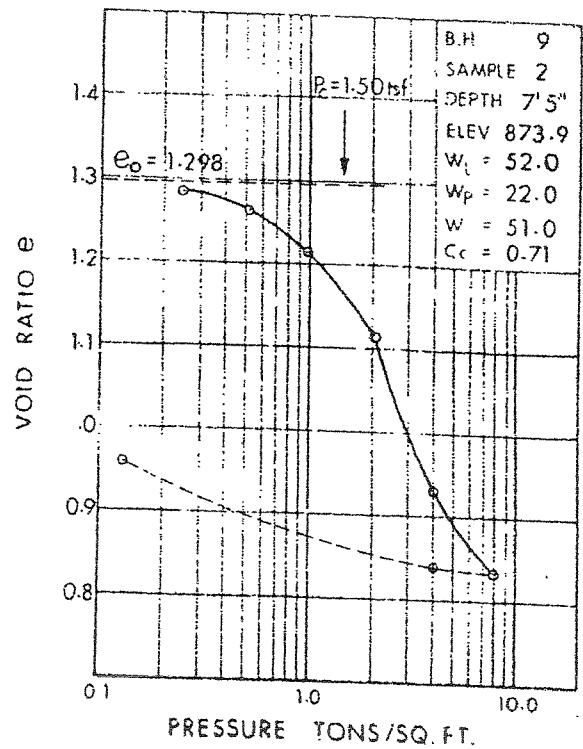
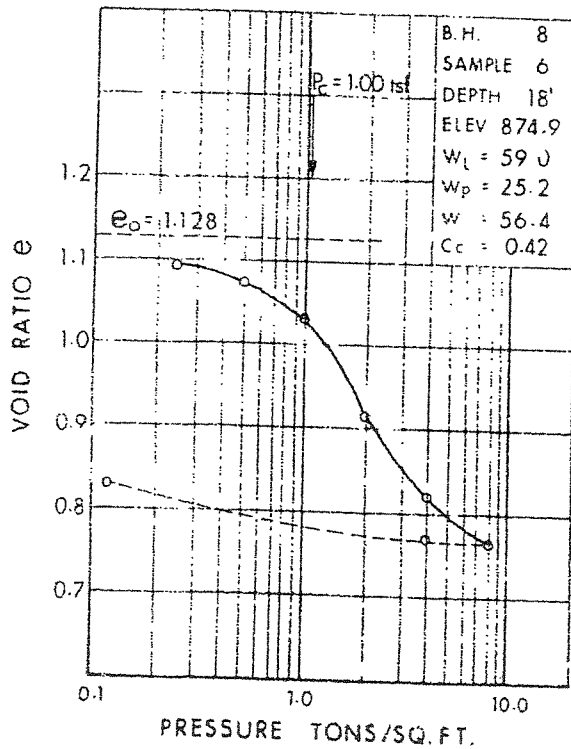
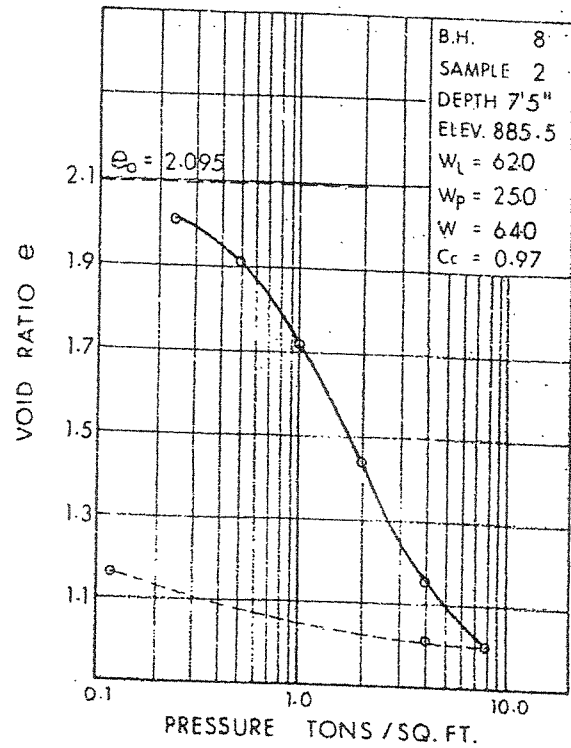
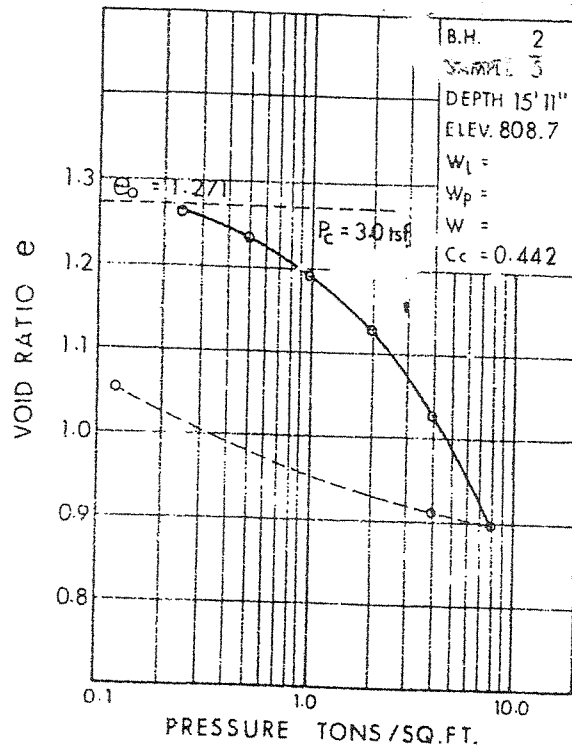


FIG. 7

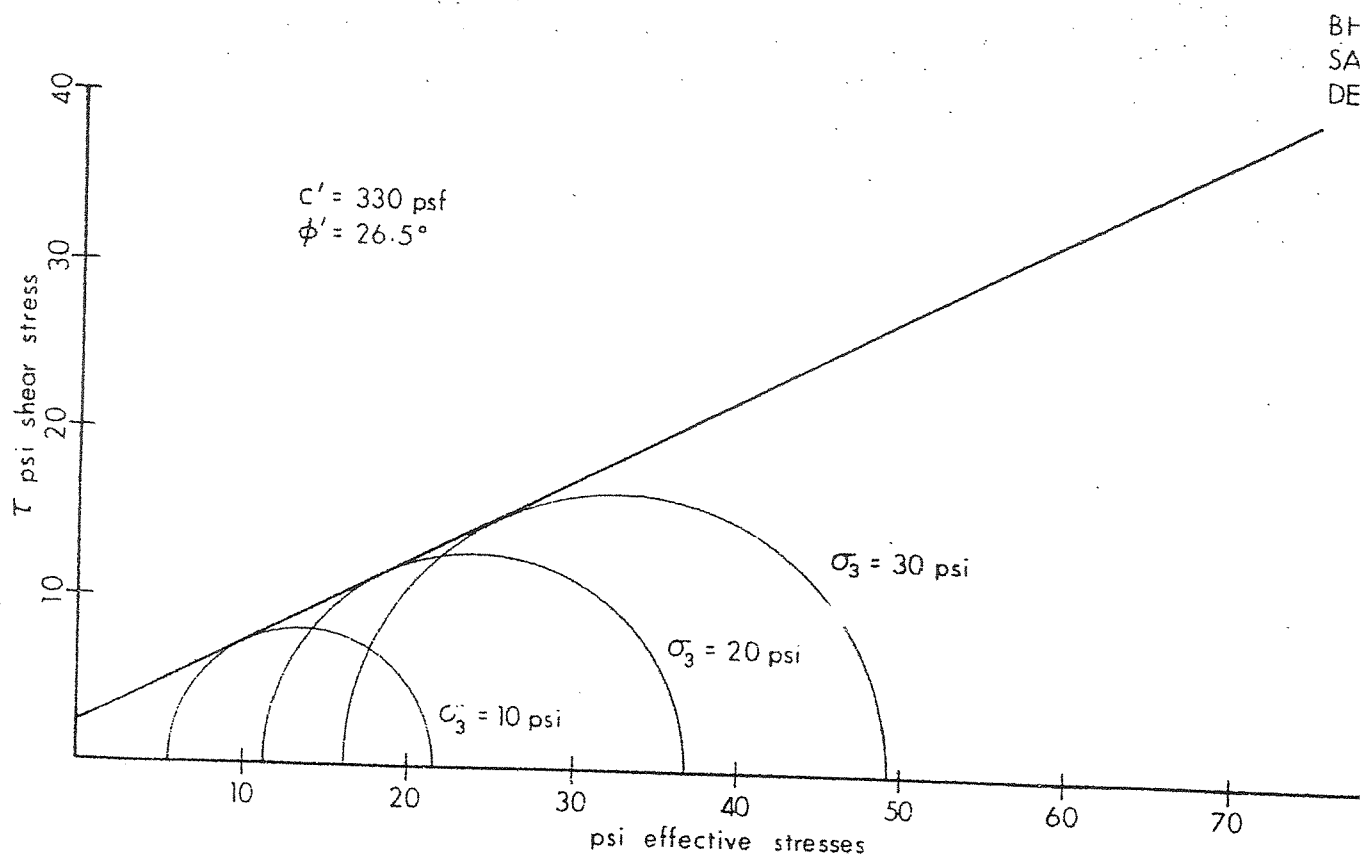
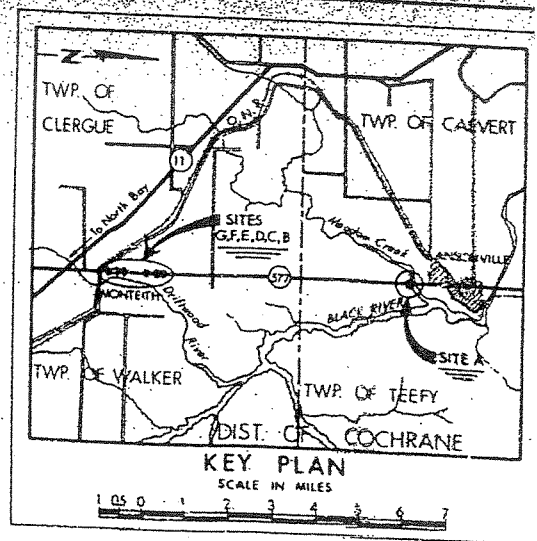
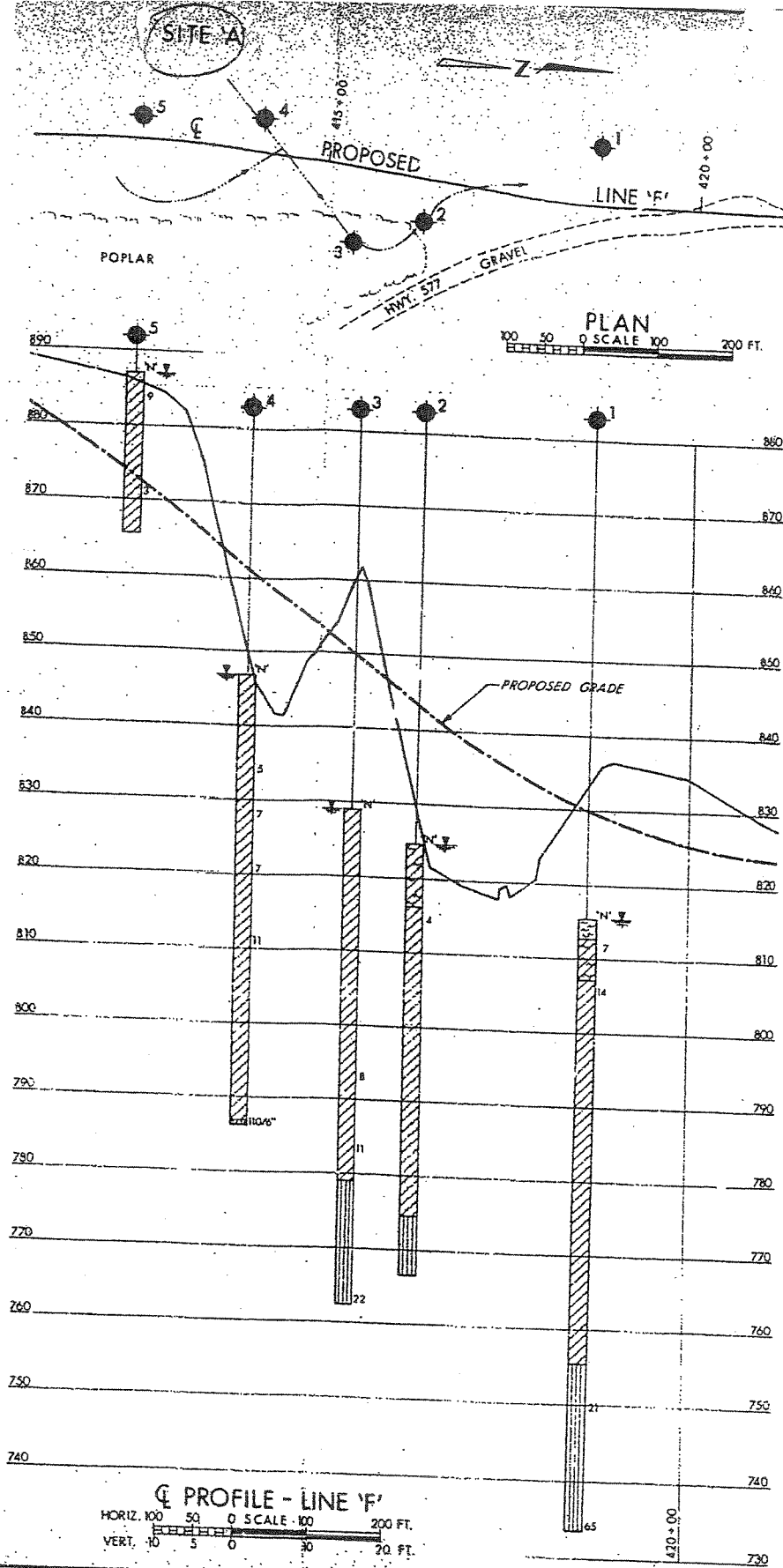


FIG. 10

JOB No. 70 -



LEGEND			
	Bore Hole		
	Cone Penetration Test		
	Bore Hole & Cone Test		
	Water Level established at time of field investigation NOV. 1970.		
	Head		
	Artesian Water Level		
	Encountered		
NO.	ELEVATION	STATION	OFFSET
1	815.4	418+65	75' LT.
2	824.7	416+40	55' RT.
3	829.0	415+55	100' RT.
4	846.9	414+14	43' LT.
5	887.0	412+60	23' LT.
6	913.4	108+63	Q LINE 'E'
7	902.7	106+80	55' RT.
8	892.9	106+20	50' RT.
9	881.3	101+07	53' LT.
10	877.2	101+07	100' RT.
11	860.4	86+55	50' LT.
12	856.0	85+60	55' RT.
13	854.5	84+40	7' LT.
14	853.0	84+60	60' RT.

— NOTE —
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS	DATE	BY	DESCRIPTION

DEPARTMENT OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATION SECTION

PROPOSED RE-ALIGNMENT

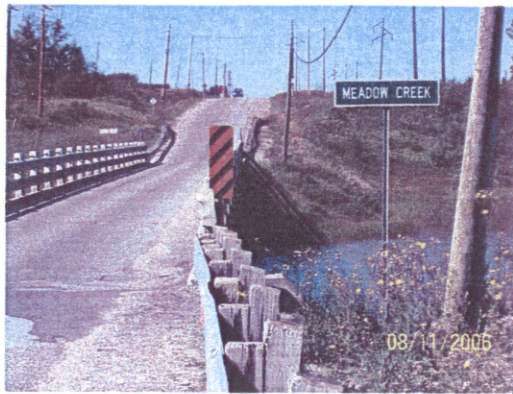
HIGHWAY NO. 577 LINE D,E & F DIST NO. 16
DIST. COCHRANE
TWP. CLERGUE, CALVERT, WALKER & TEEFY

BORE HOLE LOCATIONS & SOIL STRAT.

SUBM'D G.A. CHECKED <u>1/1</u>	W.P. NO. <u>100-66-02</u>	DRAWING NO.
DRAWN S.R. CHECKED <u>1/1</u>	JOB NO. <u>70-11098</u>	70-11098 A
DATE <u>JUNE 3, 1971</u>	SITE NO.	BRIDGE DRAWING NO.
APPROVED <u>[Signature]</u>	CONT. NO.	
PRINCIPAL FOUNDATION ENGINEER		

Appendix F

Site Photographs



Photographs 1 & 2 - Existing Highway 577 Bridge over Meadow Creek



Photographs 3 & 4 - Field boring & sampling via a drilling rig



Photographs 5 & 6 - Deployment of the drilling rig on the south side of the bridge with traffic control & safety measures in place along the highway



Photographs 7 & 8 - Drilling near the Proposed South Abutment and North Approach Embankment

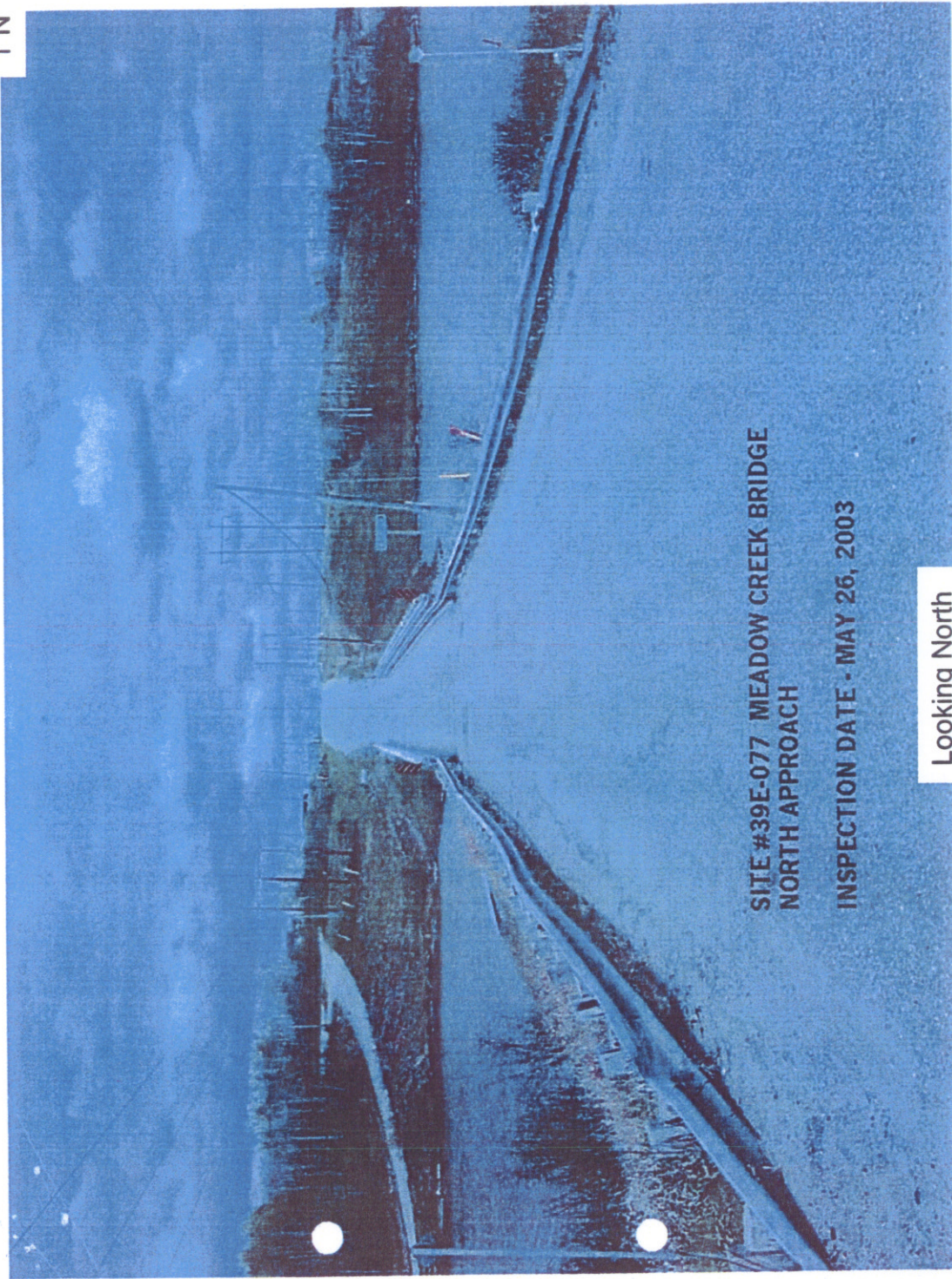


Photographs 9 & 10 - Existing pavement condition along the approach embankment



Photographs 11 & 12 – Overview of west and east sides of north approach embankment and existing bridge railing

↑ N



SITE #39E-077 MEADOW CREEK BRIDGE
NORTH APPROACH

INSPECTION DATE - MAY 26, 2003

Looking North

↓ S



Looking South



East elevation

Close up

West elevation



Meadow Creek Bridge Looking From East

EMBANKMENT EROSION



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 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 STRUCTURAL FEATURES AND/OR STRENGTH.

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

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (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
Ω	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
c_c	1	COMPRESSION INDEX
c_s	1	SWELLING INDEX
c_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
ζ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
ζ'_p	kPa	PRECONSOLIDATION PRESSURE
Ω	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
Ω_R	kPa	RESIDUAL SHEAR STRENGTH
Ω	kPa	REMOULDED SHEAR STRENGTH
S_r	1	SENSITIVITY = c_u / Ω

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{\min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
P_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	s_r	%	DEGREE OF SATURATION	D_n	mm	N PERCENT – DIAMETER
P	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
P_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	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	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $(w_L - w) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
P'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{\max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

**PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT OF HWY 577 BRIDGE
OVER MEADOW CREEK
IROQUOIS FALLS, ONTARIO
G.W.P. 181-92-00; SITE 39E-077**

GEOCRES NO. 42A-66

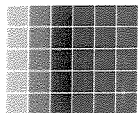
Prepared For:

MCCORMICK RANKIN CORPORATION

Prepared by:

SHAHEEN & PEAKER LIMITED

**Project: SPT1167
October 16, 2007**



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DRAFT
PRELIMINARY FOUNDATION DESIGN REPORT
REPLACEMENT OF THE HIGHWAY 577 BRIDGE
OVER THE MEADOW CREEK
IROQUOIS FALLS, ONTARIO
G.W.P. 181-92-00
SITE: 39E-077

6. DISCUSSION AND RECOMMENDATIONS

The existing Meadow Creek structure (Site 39E-077) on Highway 577 at Iroquois Falls consists of a single lane 74.2 m long, twelve-span steel girder bridge with a concrete deck and a timber substructure. The structure is supported on 50 ft (15.2 m) long (measured from the bottom of the bridge deck) timber piles. As shown in Appendix M, the timber piles supporting the 20 ft (6.1 m) long spans of the bridge are cross braced, except for a clear centre span to maintain traffic on water.

The existing single lane structure will be replaced with a two-lane bridge, as shown in Appendix N on the preliminary general arrangement drawing by McCormick Rankin Corporation. As well, Highway 577 in the near vicinity of the bridge will be re-aligned to accommodate 80 km/hr speed limit.

After evaluating various alternatives (which included the construction of the new bridge to the east and west of the existing structure – while maintaining the traffic along the existing bridge), the preliminary design team has decided that the optimum design was to construct the new bridge at the existing bridge location. To maintain traffic during the construction, a single lane (half) of the new structure will be built to immediately east of the existing bridge (while maintaining the traffic through the existing bridge). After construction of the east half, the traffic will be diverted to the new bridge, while the existing bridge is being demolished. The other half of the new bridge will be built at the location of the existing bridge (see Appendix H).

With the selected configuration, the new bridge will be a two-span, 90 m long structure, with a central pier near the center of the Creek. The south abutment of the new bridge will be about 10 m south of the south abutment of the existing bridge, while the north abutment would be about 5 m north of the north abutment of the existing bridge. As shown on the profile drawing in Drawing No. 1, the existing grade at the new abutment locations will be raised by about 1.4 m at the north abutment location and 1.6 m at the south abutment location, gradually decreasing to zero (i.e. no grade raise above existing grades) some 40 m beyond the abutment locations.

Boreholes 1, 2, 3, 5 and 7, which were drilled near the existing and proposed bridge abutments contacted silty clay fill (mixed with some organic soils) to depths ranging from

2.9 to 7.5 m below the ground or water surface. In general, the clayey fill is underlain by organic soils, ranging in thickness from 0.3 to 2.5 m, which, in turn, are underlain by a major silty clay deposit. It appears that when the existing bridge was built the valley was filled near the abutment locations, without stripping the existing surficial organic soils and that the fill was placed without systematic compaction. Photographs included in Appendix F suggest considerable differential settlements in the approach fill areas, while the bridge structure appears to have undergone little or no visible differential settlements. Patching of the asphalt is evident immediately adjacent to the existing abutments, in the photographs. As well, in Borehole 3, which was drilled from the top of the paved highway surface, very close to the south abutment, the presence of two layers of asphaltic concrete was noted with a granular fill layer in between. This is likely to be due to the settlement of the fill and the underlying organic soils, as well as the natural silty clay deposit.

The silty clay extends to depths ranging between about 17 and 26 m below the ground surface or the water level in the creek. It is a low (CL) to high (CH) but generally an intermediate plasticity (CI) clay. The measured undrained in-situ shear strengths of the silty clay in the middle portion of the deposit typically range from 40 to 80 kPa with somewhat higher values (i.e. up to 120 kPa), above and below this middle zone.

At the depths of about 19 to 30 m below the ground or the water surface (in the creek), the boreholes contacted granular soils which range from sandy silt/silty sand to sand and gravel with frequent cobbles. These deposits are water bearing and some contain boulders.

The groundwater table at the toe of the valley slopes can be expected near the creek water level which we understand is regulated at about El. 248.0 m by means of dam(s) along the creek. Beyond the toe of the valley, the water level can be expected to rise with the ground surface elevation. In the deep piezometers installed in Boreholes 3 and 5, which were drilled near the existing bridge abutments, the groundwater level in the basal granular soils was recorded at El. 246.2 and 246.4 m, respectively. The water level in the silty clay and in the fill can, however, be higher and should be determined in the detail investigation phase by installing shallow piezometers, in addition to deep piezometers. Further north and south in Boreholes 8 and 9, the piezometric levels were recorded at Elevations 250.5 and 250.4 m respectively, rising to about El. 259 m still further north and south in Boreholes 10 and 11. In several of the boreholes, the recorded groundwater levels in the silty clay did not appear to have stabilized, in spite of several weeks of monitoring.

The groundwater levels can be expected to fluctuate seasonally and also in response to major weather events.

6.1 FOUNDATIONS

The soils to a depth of between 19 and 24 m consist of compressible clayey fills with some organic soil admixture, a layer of organic soil of variable thickness, underlain by a relatively

weak and compressible silty clay deposit. These soils are considered unsuitable for the use of normal spread footing foundations including the use of engineered fill (i.e. abutment footings on compacted Granular 'A' pad). For these reasons, we recommend that the structure be supported on deep foundations. Various foundation alternatives are given in the following table.

Table 6.1.1
Summary of Foundation Alternatives

Foundation Type	Comments	Recommendations
<ul style="list-style-type: none"> ○ Normal spread footings ○ Spread footings on compacted Granular 'A' pad 	Insufficient bearing resistance, excessive settlements	Not recommended based on reliability
<ul style="list-style-type: none"> ○ Expanded Base (Franki-Type) Concrete piles 	These types of piles are not well-suited for the prevailing soil conditions at the site, as well as being an expensive alternative.	Not recommended based on practicality of construction and economics.
<ul style="list-style-type: none"> ○ Drilled caissons 	Impractical and costly due to the presence of water bearing cohesionless soils under upward hydrostatic pressure	Not recommended based on cost and reliability.
<ul style="list-style-type: none"> ○ Driven Concrete Piles 	Considered uneconomical, as well as causing relatively higher intensity of vibrations during construction since they are high displacement piles (e.g. vibrations may adversely affect the existing bridge). They are also vulnerable against obstructions, such as boulders.	Not recommended based on cost and reliability.
<ul style="list-style-type: none"> ○ Auger Press Concrete Piles 	May not provide adequate lateral support and are costly	Not recommended based on cost and reliability.
<ul style="list-style-type: none"> ○ Timber Piles 	Not suitable for a long-span structure since too many piles would be required, due to low resistance provided by individual piles.	Not recommended based on reliability.

Foundation Type	Comments	Recommendations
o Steel H-Piles	Being low displacement piles, represent the best option for the prevailing subsurface conditions.	Considered best choice based on reliability and suitability.
o Steel Tube Piles	As they are relatively high displacement piles in comparison with steel H-piles, they represent a less reliable option in comparison with steel H-piles, especially adjacent to the existing structure (i.e. may have an adverse effect on the existing bridge).	Can be considered as an alternative to steel H-piles but for this project they are less desirable than steel H-piles, as well as being unsuitable for integral abutment type structures.

6.1.1 DEEP FOUNDATIONS

Driven Steel H-piles and steel pipe piles are available options; however, some of the other deep foundation types are briefly described below, as well.

6.1.1.1 DRILLED AND CAST-IN-PLACE CONCRETE (CAISSON), CONCRETE AUGER PRESS AND EXPANDED BASE (FRANKI) CONCRETE PILE FOUNDATIONS

Owing to the presence of uncompacted fills, organic soils and relatively weak clays, drilled and cast-in-place concrete (caisson) foundations must be extended below these soils and into the underlying very dense granular deposits. However, the granular deposits are water-bearing and under upward hydrostatic pressure. These conditions render the use of caisson foundations impractical. The use of auger-press piles can be considered but these are not well suited for the prevailing subsurface conditions and may not provide sufficient lateral resistance. In addition, they will unlikely be economical. Furthermore, it will be impractical to install these piles in the Creek. Expanded base concrete foundations are not well suited for the soil types encountered at the site as well as generating excessive vibrations. The use of caissons, auger press and expanded base concrete piles is, therefore, not recommended based on reliability and cost.

6.1.1.2 TIMBER PILES

Due to their limited lengths (i.e. generally only 15 m and possibly 18 m long) timber piles will have to derive their resistance from adhesion in the silty clay deposit, rather than end-bearing. As such, they will not provide adequate support for the bridge structure, especially at the centre-pier location. In addition, at the present time, timber piles are normally not used (because of their possible lack of durability) for MTO bridges unless there is a compelling reason for their use. Their use of this project is therefore not recommended.

6.1.1.3 DRIVEN CONCRETE PILES

Concrete piles will need to be driven into the very dense granular deposits underlying the silty clay and lower clayey silt deposits. Since these piles are large displacement piles they will generate more vibrations during driving in comparison with low displacement piles such as steel H-piles. This may create problems with the existing bridge. As well, because of the presence of variable denseness condition of the basal granular soils, their lengths across the site may be unpredictable. Owing to the unpredictable pile lengths (which may require splicing) and possible damage to the piles driven into soils with cobbles and boulders, the use of driven concrete piles is not recommended, based on reliability and cost.

6.1.1.4 DRIVEN STEEL PILES

Both driven steel H-piles and steel tube piles are available options, but since the structure will be of 'integral abutment' type, the use of H-piles is preferred. In addition, low displacement piles are better suited for the prevailing subsurface conditions in comparison with tube piles, particularly since any damage to the existing structure during the driving of the piles for the new structure would be a major concern for this project. As such, the use of steel-H piles is preferred.

Relatively heavier sections of steel piles should be used due to the presence of coarse particles in the basal granular deposits, into which the piles would be driven. In our opinion, with the information available, utilizing normal MTO axial pile resistances on competent overburden (i.e. ULS = 1700 kN/pile and SLS=1200 kN/pile) would be available for HP 310 x 110 steel H-piles. However, we recommend that in order to minimize vibrations to avoid damage to the existing bridge during pile driving, lower resistances should be used. With the available information we recommend the following values for HP310 x 110 steel H-piles.

Factored Axial Resistance at ULS = 1500 kN/pile
Axial Resistance at SLS = 1000 kN/pile

It should be pointed out that the resistances given above do not include an allowance for downdrag forces. If, as discussed later on in this report, settlements are minimized then downdrag allowance need not be made.

In order to adequately penetrate the very dense granular soils, with cobbles and boulders, a sufficiently heavy section such as HP 310 x 110, equipped with reinforced flanges as per OPSD-3000.100, is recommended.

Based on the results of the boreholes, the following Table 6.1.1.4.1 summarizes the estimated average pile tip elevations that may be assumed for preliminary design purposes.

Table 6.1.1.4.1
Anticipated Pile Lengths

Support Element/Borehole No.	Existing Ground/Water Surface Elevation (m)	Estimated Approximate Pile Depth Below Existing Ground/Water Surface (m)	Estimated Approximate Pile Tip Elevation (m)	Founding Stratum
North Abutment/ Borehole 5	251.6	32.6	219.0	sand with gravel, some silt, some cobbles
North Abutment/ Borehole 6	248.1	26.0	222.1	sandy silt to silty sand till
North Abutment/ Borehole 7	253.6	33.6	220.0	sand with some silt and gravel, occasional cobbles and boulders
Central Pier/ Borehole 4	247.8	23.3	224.5	sand with gravel, cobbles and boulders
South Abutment/ Borehole 3	251.5	28.5	223.0	silty sand with possible till zones, occasional cobbles
South Abutment/ Borehole 2	247.8	24.5	223.3	cobbles and boulders in a matrix of sand and gravel
South Abutment/ Borehole 1	253.3	29.5	223.8	sand with boulders*

*assumed stratum (below borehole bottom elevation)

The above estimated pile tip elevations are based on the assumption that the piles would penetrate through the very dense soil with cobbles and boulders within the overburden. However, some piles could encounter refusal on boulders before reaching the tabulated tip elevations, while others may extend somewhat deeper.

Depending on the final configuration of the new bridge in relation to the existing bridge, somewhat lower or higher pile resistances than recommended may be used at the final design stage.

If by means of light-weight fill (e.g. EPS) use the settlements would be limited to less than 20 mm immediately adjacent to the abutment locations, then there would be no need to make an allowance for downdrag forces on the piles supporting the abutments.

For frost protection, pile caps should have a permanent earth cover of at least 2.4 m, or equivalent artificial insulation.

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

Horizontal Resistance at ULS	=	120 kN/pile
Horizontal Resistance at SLS	=	40 kN/pile

Lateral resistance of the piles can be supplemented if necessary by the horizontal components of battered piles (for integral abutments this may not be feasible). In this instance, we recommend that the batter be limited to no flatter than 4:1, as in practice greater pile batter is difficult to install. As well, when driving piles from the river and close to the existing bridge even 4:1 batter maybe excessive for ease of construction.

In selecting pile locations (especially in the case of battered piles), the locations and pile tip positions of the piles supporting the existing bridge structure should be taken into consideration.

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

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). In this case, however, since the existing clayey fill is rather weak, consideration may be given to omitting the CSP design since, depending of the pile configuration, the clayey fill itself maybe sufficient to provide this 3 m flex zone.

6.1.1.5 STEEL TUBE PILES

Steel tube piles are less suited than steel H-piles for this project as they are high displacement piles and may induce more vibrations. As well, they are normally not used for the support of integral type abutments. If, however, it is necessary to use this type of pile, here are some brief comments for preliminary design purposes.

Tube piles will provide lower resistance in comparison with H-piles, as they will not drive as deep, but lower resistances may be compensated by the relatively shorter pile lengths. Closed end steel tube piles have the advantage that they can be inspected after driving and prior to pouring the concrete, for possible damage that may have incurred while driving. They should have a sufficient wall thickness and base plate thickness to minimize potential damage caused by the expected hard driving conditions when the pile penetrates the end bearing zone in the basal granular materials. The end plates should not be wider than the base area of the piles (i.e. should not project beyond the circumference of the pile) so that adhesion/friction is not adversely affected. Tube piles will need to be filled with concrete after their installation and inspection for possible damage.

Steel tube piles of 300 mm nominal diameter (e.g. 324 mm x 12.7 mm) driven to at least 1 to 2 m into the very dense soil can be expected to provide a Factored Axial Resistance at ULS of 1000 kN/pile and an Axial Resistance at SLS equal to 700 kN/pile about 1 to 1.5 m above the tip elevations quoted for steel H-piles given in Table 6.1.1.4.1.

6.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls 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.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') 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:

$K_a = 0.27$

$K_b = 0.35$

$K_o = 0.43$

$K^* = 0.45$

Compacted Granular 'B' Type I

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

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressure:

$K_a = 0.31$

$K_b = 0.41$

$K_o = 0.47$

$K^* = 0.57$

Rock Fill

Angle of Internal Friction, $\phi = 40^\circ$ (unfactored)

Unit Weight = 18 kN/m³

Coefficient of Lateral Earth Pressure:

$K_a = 0.27$

$K_b = 0.32$

$K_o = 0.36$

$K^* = 0.42$

Where K_b is the 'intermediate' earth pressure coefficient for a partially restrained structure.

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

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

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

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

If rock fill is used for backfill, special care is required to prevent damage to the retaining structures. In such a case, a cushion of Granular 'A' material or finely-graded rock fill (e.g. less than 200 mm nominal diameter) should be placed between the structure and the rock fill. This cushion should be at least 0.45 m wide and if Granular 'A' is used, proper filtering should be provided to prevent the loss of finer particles from the Granular 'A' cushion into the coarse rock fill. As was mentioned, however, coarse materials (e.g. rock fill) should not be used in areas through which piles will be driven.

6.3 APPROACH EMBANKMENTS

At present, it is expected that the grade along the centerline at the proposed abutment locations will be raised by about 1.6 m at the south abutment and about 1.4 m at the north abutment location. As the original grade (o.g.) rises away from the bridge location, the grade raise above o.g. will gradually decrease to zero about some 40 m from the abutment locations or at about Stations 19+805 and 19+980.

Typical proposed embankment cross-sections provided to us by MRC are given in Appendix I. As shown on the cross-sectional plots, up to about 3.5 m of fill may be placed along on the east side of the existing north approach fills, due to the proposed widening.

We carried out a preliminary slope stability analysis at two critical sections, namely at Station 19+936 (near the proposed north abutment, easterly widening) and at Station 19+846 (near the proposed south abutment, westerly widening). Based on the borehole results and the preliminary cross sections provided to us at these two locations, no foundation slope failures are anticipated for embankments with normal 2H:1V side slopes

with earth fill and 1¼H:1V side slopes for rock fill, assuming that all organic or otherwise unsuitable materials will be removed as per MTO standard procedures, prior to placing the new embankment fills.

The use of rock fill is recommended for embankments to be placed below the high water line (El. 248.0-248.5 m) for ease of construction, erosion resistance and for environmental considerations (e.g. favourable fish habitat). However, rock fill should not be used in areas through which piles will be driven. In such locations, the maximum nominal particle size should be 75 mm.

The results of the slope stability analyses at these two stations are given in Appendix J. The soil parameters used for the short-term (undrained) and long-term (effective stress) analyses are given on the individual figures given in Appendix J. It should be noted that the use of Expanded Polystyrene (EPS) was assumed in our analyses. This is considered necessary to minimize settlements due to grade raises over weak underlying soils, as will be discussed later in this report.

All organic or otherwise unsuitable soils will need to be removed within an envelope area given by an imaginary line not steeper than 1:1 from the toe of the proposed embankment widening as per MTO procedures. Based on the available borehole data, for preliminary estimating purposes, the average thickness of unsuitable soils to be stripped on land can be taken as 0.4 m. However, the thickness of unsuitable soils in non-engineered fills and especially beneath watercourses can be variable. In Boreholes 2 and 6, drilled in the creek, near the shore, the thickness of unsuitable soils at Borehole 2 is 0.8 m (i.e. peat layer), while at Borehole 6, it is about 2 m. Where deep excavations are required (e.g. more than 0.4 m deep), this should be carried out in short sections and immediately backfilled in order to prevent a slip of the ground or instability. Dumping of material for embankment construction will need to be gradual, from one end of the construction to the other end, so as not to cause failures of the existing fills and the underlying weak soils.

Compacting soils near or below water table will be difficult and will require the use of granular soils or rock fill.

Fills used to build embankments should be compacted as per MTO standards. As well, proper benching of the existing slopes should be implemented when widening the existing embankments, as per MTO procedures and in accordance with OPSD 208.01.

Based on the presently available data, the anticipated settlements due to 1.4 to 1.6 m grade raise at the centerline immediately adjacent to the proposed abutments is about 150 to 210 mm. About 40% of this settlement figure is due to the consolidation settlement of the natural silty clay deposit while the remaining 60% is due to the settlement of the overlying organic soils and the existing fill soils. In our analysis, the stresses induced on the natural

silty clay deposit were assumed to be within the pre-consolidated range (i.e. C_r was used rather than C_c in determining the consolidation settlement of the silty clay deposit).

Beyond the centerline towards the east side, where the height of the fill typically reaches 2 to 3 m above o.g. due to widening of the existing embankment, the estimated settlement increases to about 280 to 400 mm (including secondary consolidation). About 90% of these settlements can be expected to be completed within about eight to ten years.

Regardless of the time rate of the settlement, it is our opinion that settlements of these magnitude are unacceptable adjacent to a newly constructed structure. For this reason, pre-loading or surcharging (with or without wick drains) or the use of light-weight fill will need to be implemented to reduce the settlements immediately adjacent to that structure to less than about 20 mm. With the presently selected alignment and construction sequence (i.e. east half of the structure will be built while the existing structure is in use as shown in Appendix H), neither preloading nor surcharging is feasible and therefore, as was mentioned before, the use of expanded polystyrene blocks (EPS) would be the recommended choice.

In principle, the EPS would be placed to reflect the fill thickness (i.e. where the proposed fill thickness is thicker, the EPS would also be thicker). As such, along the center portion of the embankment the thickness of EPS would be thickest, gradually decreasing away from the bridge, mirroring the grade raises above o.g. The use of EPS can probably be discontinued about some 30 m to 35 m beyond the abutment locations. Since the height of the fill above o.g. will vary along individual cross sections, the thickness of EPS will also vary in the transverse direction and will generally be thicker on the east side, where the primary portion of the widening of the embankments is to take place, as shown in cross sections in Appendix I.

The following design criteria with EPS 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 K. The design and construction of the EPS should be in accordance with MTO Special Provision entitled "Expanded Polystyrene Embankment."
- ❖ The bottom of EPS should not be extended below El. 248.0 m, to prevent an uplift condition near the edges of the embankment and, as well, to avoid placing the EPS below water level (assuming water level in the creek will be $248.0 \pm$ and no higher than 248.5 m).
- ❖ 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 possible uplift as well as to avoid damage due to

ultra-violet light exposure.

- ❖ Where feasible, the existing soil beneath the EPS should be removed to a depth of 0.6 m below the bottom of the EPS which will be placed under the paved and shoulder portion of the embankment and replaced with compacted granular fill. The top 0.1 m of this fill should consist of sand fill (i.e. no gravel) to prevent damage to the EPS. Beyond the paved portion and the shoulders, the soil replacement can be decreased from 0.6 to 0.3 m.

The following procedures are recommended.

The site to receive the EPS should be stripped to a depth of about 0.6 m below the bottom of EPS elevation, if possible, below the paved portion of the roadway and the shoulders (to a depth of about 0.3 m beyond the paved portion and the shoulders). After stripping, the exposed subgrade should be visually inspected and where the presence of unsuitable soils is noted (e.g. organic soils) these should be removed and replaced with compacted suitable materials. Where feasible, the approved subgrade should be compacted from the surface (i.e. the compaction effort will depend on the site conditions, such as the proximity of the groundwater level). The granular soils which will be placed beneath the EPS should be laid in layers not exceeding 300 mm in thickness and properly compacted. In wet conditions, however, a thicker lift may be necessary.

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

Suggested EPS Thickness

Grade Raise Above O.G.	Required EPS Thickness Under Pavement & Shoulders	Beyond Shoulders
0.5 m	1.3	0.75
1.0 m	1.8	1.5
1.5 m	2.3	2.1
2.0 m	3.0	2.7
2.5 m	3.7	3.3
3.0 m	4.4	4.0
3.5 m	n/a	4.6

EPS blocks will need to be protected from ultra-violet light damage during the construction. There may be a requirement for tying the EPS from Stage 1 construction to Stage 2, since the individual blocks will not be overlapping.

As was mentioned before, in some instances (especially on the east side of north embankment widening) providing sufficient EPS thickness to fully compensate for potential settlements may not be feasible, since the placement of EPS below the water line is not recommended. In such cases, settlements of up to 100 mm towards the toe of the

embankment will probably not cause significant problems. It is, however, recommended that suitably wide embankment platform(s) be provided in anticipation of such settlements.

6.4 CUT SLOPES

With the presently proposed road alignment and/or widening, cut slopes of up to 8 m in height will likely be required on the north side of the bridge within a distance of about 250 m and up to 11 m high cut slopes on the south side within a distance of some 500 m from the bridge. To investigate the subsurface conditions at this preliminary design phase, Boreholes 8 and 11 were drilled along the proposed highway alignment to the north and Boreholes 9 and 10 south of the bridge, for slope stability considerations. Piezometers were installed in each of these four boreholes to monitor the groundwater levels over prolonged periods of time without interference from surface water. These boreholes show below some surficial fill or topsoil, the presence of silty clay to full borehole depths (which range between 12.8 and 19.5 m below the ground surface). The groundwater levels in the piezometers installed in these four boreholes were recorded at depths ranging from 2.7 m (Borehole 11) to 13.2 m (Borehole 9) below the ground surface. However, water levels may not have stabilized in Boreholes 8, 9 and 10, at the time of our investigation (i.e. recorded water levels between 6.8 and 13.2 m below ground surface).

Slope stability analyses were carried out to determine safe slopes for preliminary design purposes. The following soil parameters and groundwater conditions were used for the purposes of analyses (also see Appendix L).

Table 6.4.1
Typical Soil Parameters Used for Slope Stability Analyses

Soil Type	Bulk Unit Weight kN/m ³	Total Shear Strength Parameters(Short-term)		Effective Shear Strength Parameters (Long-Term)	
		Undrained Shear Strength C (kPa)	Angle of Internal Friction (ϕ) Degrees	Cohesion Intercept C (kPa)	Angle of Internal Friction (ϕ) Degrees
Riprap	20	0	43	0	43
Granular 'A' filter materials placed against slope face	21	0	34	0	34
Surficial sand	18	0	29	0	29
Surficial clayey silt/silty clay	18	80-100	0	3	27
silty clay	17	40	0	3	26

Selected sections of the proposed roadway cut slopes (provided to us by MRC) were analyzed by the limit equilibrium approach. The analyses were 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) analyses calculations. The program calculated a factor of safety based on the limit equilibrium of

forces and moments, assuming circular slip surfaces. The factor of safety is defined as the ratio of available shear strength to the shear strength that must be mobilized to maintain a condition of limiting equilibrium. Long-term stability of the slopes was examined using effective shear strength parameters, which tends to be more critical than the short-term condition. In this analysis, the pore pressure distribution along the slip surface was based on presumed groundwater levels as shown in the figures. The required minimum safety factors against failure were assumed to be 1.3, as per MTO procedures.

Based on the data available, parameters used and the analyses, the following steepest side slopes are recommended.

Table 6.4.2
Recommended Side Slopes

North Side of Bridge
Boreholes 8 and 11
Assumed Water Level = 2.5 m below top of cut slope

Height of Cut	Recommended Steepest Slope
0 – 5.9 m	2½H:1V
6.0 – 7.5 m	2½H:1V plus 2 m wide mid-height berm
7.6 – 8.0 m	2¾H:1V plus 2 m wide mid-height berm

South Side of Bridge
Boreholes 9 and 10
Assumed Water Level = 5.0 m below top of cut slope

Height of Cut	Recommended Steepest Slope
0 – 5.9 m	2½H:1V
6.0 – 7.5 m	2½H:1V plus 2 m wide mid-height berm
7.6 – 9.0 m	2¾H:1V plus 2 m wide mid-height berm
9.1 – 10.0 m	3H:1V plus 2 m wide mid-height berm
10.1 – 10.9 m	3¼H:1V plus 2 m wide mid-height berm
11.0 m	3½H:1V plus 2 m wide mid-height berm

With the parameters, method of analysis and assumed ground and groundwater conditions, 2H:1V side slopes are theoretically feasible for cut slope heights of up to 6 m. Based on local experience, however, no steeper than 2½H:1V side slopes are recommended for relatively shallow cuts as well (i.e. less than 6 m).

In our analyses, we assumed a 200 mm thick Granular 'A' drainage layer will be placed on the natural cut slope surface overlain by a 300 mm thick riprap layer.

As with most clay slopes, the long-term (effective stress) analysis is the critical case, which is very sensitive to the position of the groundwater. To provide an adequate safety factor (i.e. not less than 1.3) with these cut slopes it must be ensured that water levels will not rise above the water levels assumed in the analysis (i.e. 5.0 m at Boreholes 9 and 10 and 2.5 m at Boreholes 8 and 11, on the south and north sides of the bridge, below existing grades, respectively). For this reason, the detailed investigation should include long-term monitoring of the groundwater levels in the new boreholes as well as in the existing boreholes (i.e. Boreholes 3, 5, 8, 9, 10 and 11 put down during the present investigation). With the present information, we recommend that French drains be incorporated in the design along the top of the slopes, to ensure that water levels will not rise above the levels assumed. Furthermore, as can be seen from Table 6.4.2, we have assumed the provision of 2 m wide mid-height berm for slope heights of 6.0 m or higher, in accordance with MTO Northeastern Region procedures. We recommend that an at least 1.5 m deep French drain be also provided along this mid-height berm. All French drains must be free to drain through a frost-free outlet. We also suggest that consideration be given in the final design to increase the width of the mid-height berm from 2.0 m to 2.5 m or even 3.0 m. In this way, the French drains will be easier to construct (i.e. construction equipment can be accommodated) and as well, any future maintenance would be easier, based on our experience.

6.5 CONSTRUCTION

The groundwater level near the Creek would be controlled by the water level in the Creek and since the water level in the Creek is controlled at about $248.0 \pm$ m, the groundwater level during the construction near the abutment locations can be expected at about El. 248 m. This level may possibly rise up the valley slopes, mirroring the rise in the ground surface elevation. Near the Creek, if and where excavations extend near or below about El. 248.5 m, dewatering will likely be required to facilitate the construction and to effect adequate compaction of the newly placed fills. If the existing fill soils are clayey, as was the case in the boreholes drilled, then dewatering by means of gravity drainage and pumping from open sumps should suffice for excavations extending about El. 248.0 ± 0.5 m. If pervious soils are encountered then pumping from closely spaced, strategically placed filtered sumps would be required, depending on the depth of excavation below the prevailing groundwater level. Beyond the filled areas up the valley, the soils will likely consist of natural silty clays and in this case, gravity drainage and pumping from open sumps should suffice.

The side slopes of the embankments should be protected against erosion during the construction and permanently, including riprap protection placed to the high water level. The riprap should be separated from the embankment fills or native soils using a suitable filter material. As was discussed before, however, it is our recommendation that rock fill be used below the water level, for ease of construction and environment purposes. The

placement of suitable filter may also be required between any existing earth fill and the coarse rock fill.

Rock fill or coarse materials (i.e. particle sizes exceeding 75 mm) should not be used in areas through which piles would be driven. We understand that a previous bridge may have existed at this site prior to the existing bridge which was built in the 1940's. Depending on the location of that previous (i.e. pre-1940's) bridge, its foundations may cause obstructions for the proposed new bridge. This aspect should be looked into and if so, the existing (now buried) foundations, such as piles may need to be extracted. Furthermore, any interference of the presently existing bridge piles with the proposed bridge piles will need to be carefully looked into, especially if battered piles are to be constructed. From the present information, the piles of the existing bridge appear to be vertical. The use of batter in the westerly direction on the outside of the central pier of the new bridge may need to be avoided.

The construction of the central pier foundations will require a suitable scheme for constructability. For example, a partially submerged floating steel form with CSP inserts (for guiding the piles to be driven) has been proposed by MRC, as such a scheme was successfully used on another project. In this respect, MTO experience include the use of large diameter steel tube piles which are driven a short (but sufficient) distance into the creek bottom to provide proper guide in driving the battered piles. At present time 10V:1H batter is proposed except for the piles driven in Stage 1, immediately adjacent to the existing bridge, which will be driven vertically to increase the offset to the existing piles.

The two-phase construction sequence will involve providing proper support of the westerly portion of the existing approach fills, while excavating to the required subgrade elevations and thereafter to allow for pile driving, EPS placement, etc. This temporary support system to facilitate the proposed staged construction can consist of soldier pile and timber lagging which would be utilized to support the excavations near the existing and proposed bridge abutments and possibly further, some distance beyond the proposed bridge abutment locations. Interlocking steel sheet piling is unlikely to be suitable for this purpose near the existing bridge, since vibrations caused during driving may cause damage to the existing bridge, as well as cost implications. As was mentioned before, non-engineered fill underlain by organic soils were encountered to some 7 to 8 m depths. Because of this, soldier piles may need to be extended to considerable depths in order to provide adequate free-standing support above the proposed excavation elevations. If this is the case, then consideration may be given to the use of bulk-head (i.e. deadman) support and/or raker footings for the support of the upper (free standing) portion of the temporary support scheme. This will reduce the vertical length of shoring below the ground surface (e.g. the length of the soldier piles). For the design of the raker footings to be supported in the existing inorganic fill materials, the following tentative geotechnical resistances can be assumed for preliminary design (for raker footings to be placed at 45 degree to the vertical), based on the presently available data.

Factored Geotechnical Resistance at ULS = 60 kPa

Factored Geotechnical Resistance at SLS = 40 kPa

These values will need to be reviewed and revised depending on the findings of the detailed investigation, details of the proposed shoring and finally the inspection of the fill supporting the rakers, during the construction.

Special consideration will have to be given to the impact of the pile driving on the existing bridge during Phase 1 construction, as well as its impact on the foundations of the newly built easterly (Phase 1) half of the bridge when constructing the westerly (Phase 2) half. As shown in the drawings given in Appendix M the existing bridge appears to be supported on timber pile foundations, which are 50 ft (15 m) long, as measured from below the bridge deck level. This means that, based on the available subsurface data, at the abutment locations the piles were driven through the clayey fills which were placed for the construction of the bridge(s) and end (i.e. pile tip elevation) in the stiff silty clay. Beyond the filled areas, the piles would be driven through surficial creek deposits into the underlying firm to very stiff silty clay with tips within the stiff silty clay. This means the piles are frictional piles, with a small theoretical geotechnical resistance due to end bearing in the stiff silty clay.

We recommend that for the detail design, typical loads carried by the existing piles be calculated and compared to theoretical available resistances, since pile driving would affect the closest pile(s) more adversely. If necessary, arrangements may be made to transfer some of the loads from the existing piles which would be more affected to other piles which would be less affected by pile driving.

We understand that with the present arrangement, the closest piles to be driven will be at a horizontal distance of about 10 m at the south abutment location and 5.0 to 5.5 m at the north abutment location. At the central pier location, the minimum horizontal distance to the closest pile would be 2.5 m. These distances are likely to be acceptable. However, as a minimum measure, continuous monitoring of the vibrations should be implemented during pile driving as well as settlement monitoring. This would be required during the first phase for the existing bridge and the east half of the new bridge during the second phase of construction.

At the abutment locations, the new piles would be driven through existing un-engineered fills, and if vibrations are found to be excessive, pre-augering may be necessary to the surface of the natural silty clay soils.

The intensity of the vibrations (both at abutment locations and at the pier location) are expected to increase once the piles penetrate through the natural silty clays into the clayey silts (where they occur) and further intensify when the piles are driven into the underlying dense to very dense granular deposits. For this reason, reduced pile resistances were

recommended to reduce vibrations. These recommendations will need to be reviewed in more depth at detail design.

6.6 FROST PROTECTION

Design frost protection depth for the general area is 2.4 m. Therefore, a permanent soil cover of 2.4 m or its thermal equivalent of insulation is required for frost protection of foundations, including any pile caps. In case of riprap (or rock fill) only one half of the riprap or rock fill thickness should be taken as being effective in providing frost penetration.

6.7 SCOPE OF ADDITIONAL INVESTIGATION

After the details of the bridge design concept are finished an additional investigation will need to be conducted for the final design. The scope of the investigation will depend on the details of the proposed structure and proposed ground elevations at the bridge location, as well as the details of cuts and fills along the new alignment of Hwy 577, near the structure. Tentatively, the following programme is proposed, based on our present knowledge of the design and the subsurface conditions.

One deep borehole for the north abutment*

One deep borehole for the central pier*

One deep borehole for the south abutment*

* These three boreholes will probably need to be drilled from the creek.

One borehole for the north approach

One borehole for the south approach

Six to seven boreholes for the proposed cuts and fills, bringing the total number of boreholes to between eleven to twelve.

We recommend that piezometers be installed to monitor the groundwater table. These should include deep piezometers as well as shallow piezometers, the latter being for the determination of the perched water table.

Laboratory testing should include, in addition to normal routine MTO testing requirements, one-dimensional consolidation testing (i.e. oedometer tests).

7.0 CLOSURE

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

SHAHEEN & PEAKER LIMITED



Ramon Miranda, P.Eng



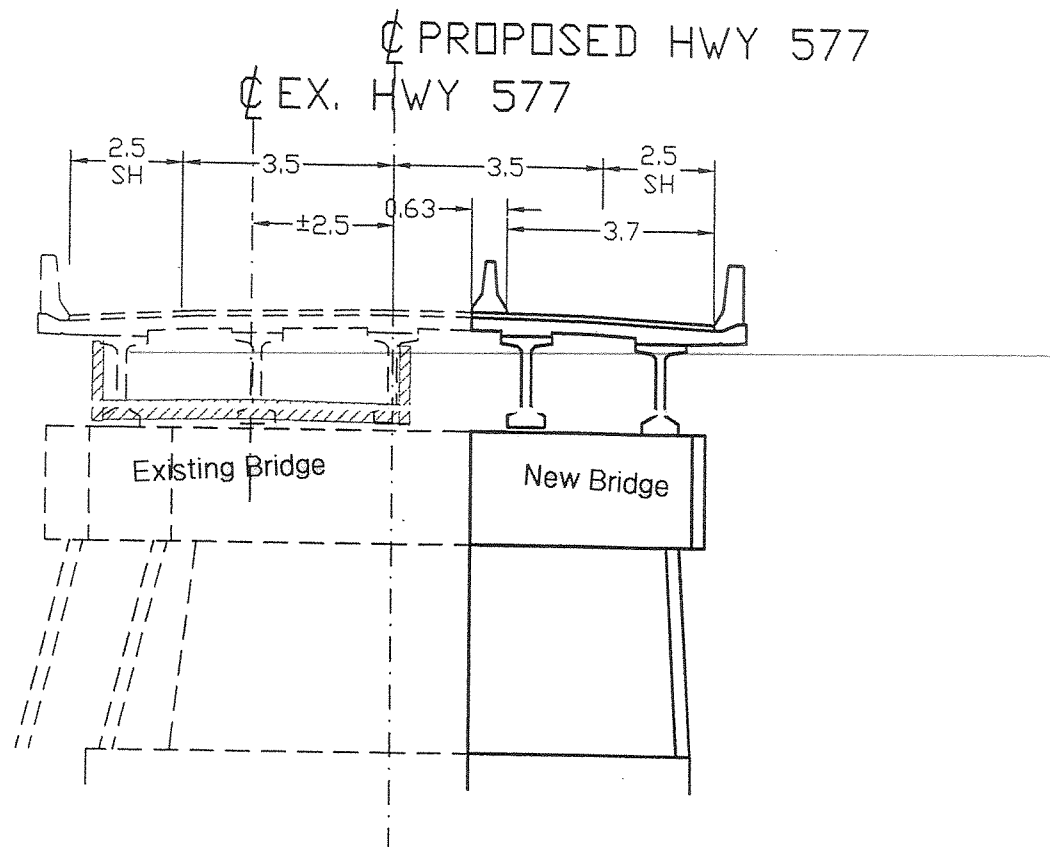
Z.S. Ozden, P.Eng.

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Appendix H

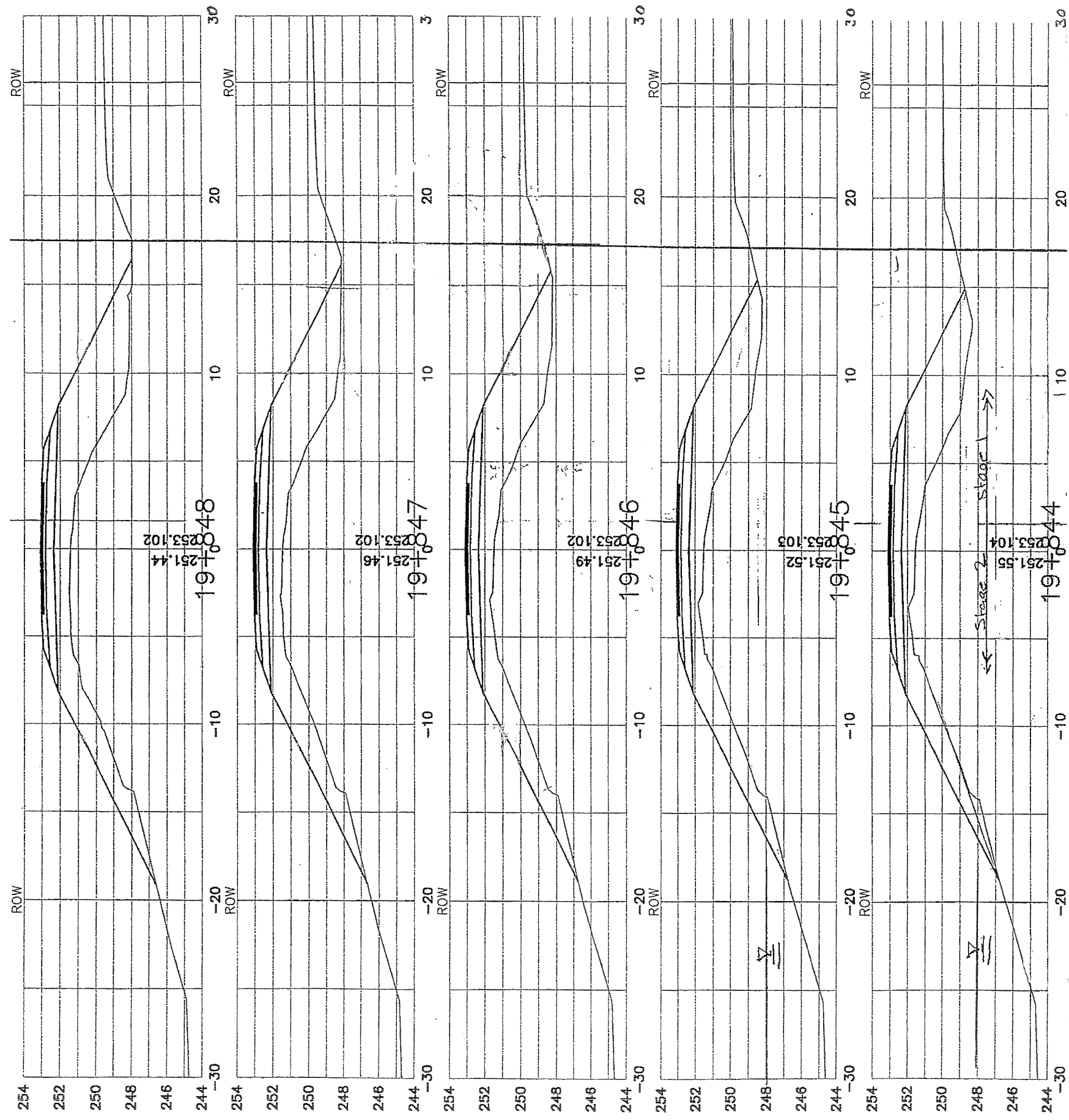
Proposed Construction Staging



SHAHEEN & PEAKER LIMITED		
Scale: NTS	Replacement of the Meadow Creek Bridge Highway 577 G.W.P. 181-92-00; Site 39E-077	Reviewed By:
Date:Feb 2007		
PROPOSED CONSTRUCTION STAGING		
Project: SPT1167	IROQUOIS FALLS, ONTARIO	Appendix H

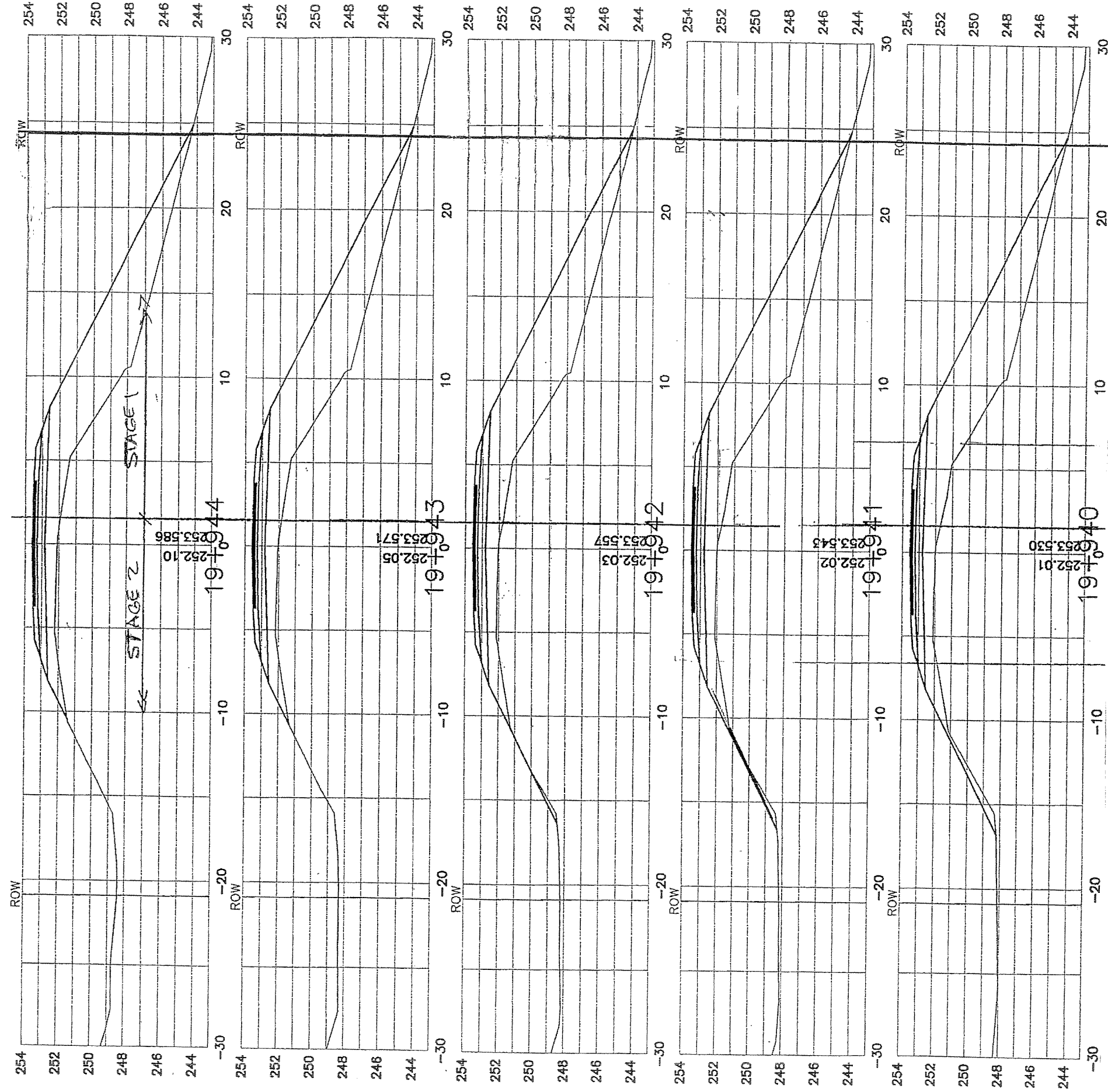
Appendix I

Typical Proposed Embankment Cross-Sections



Appendix I

Typical Proposed Embankment Cross-Sections Near South Abutment



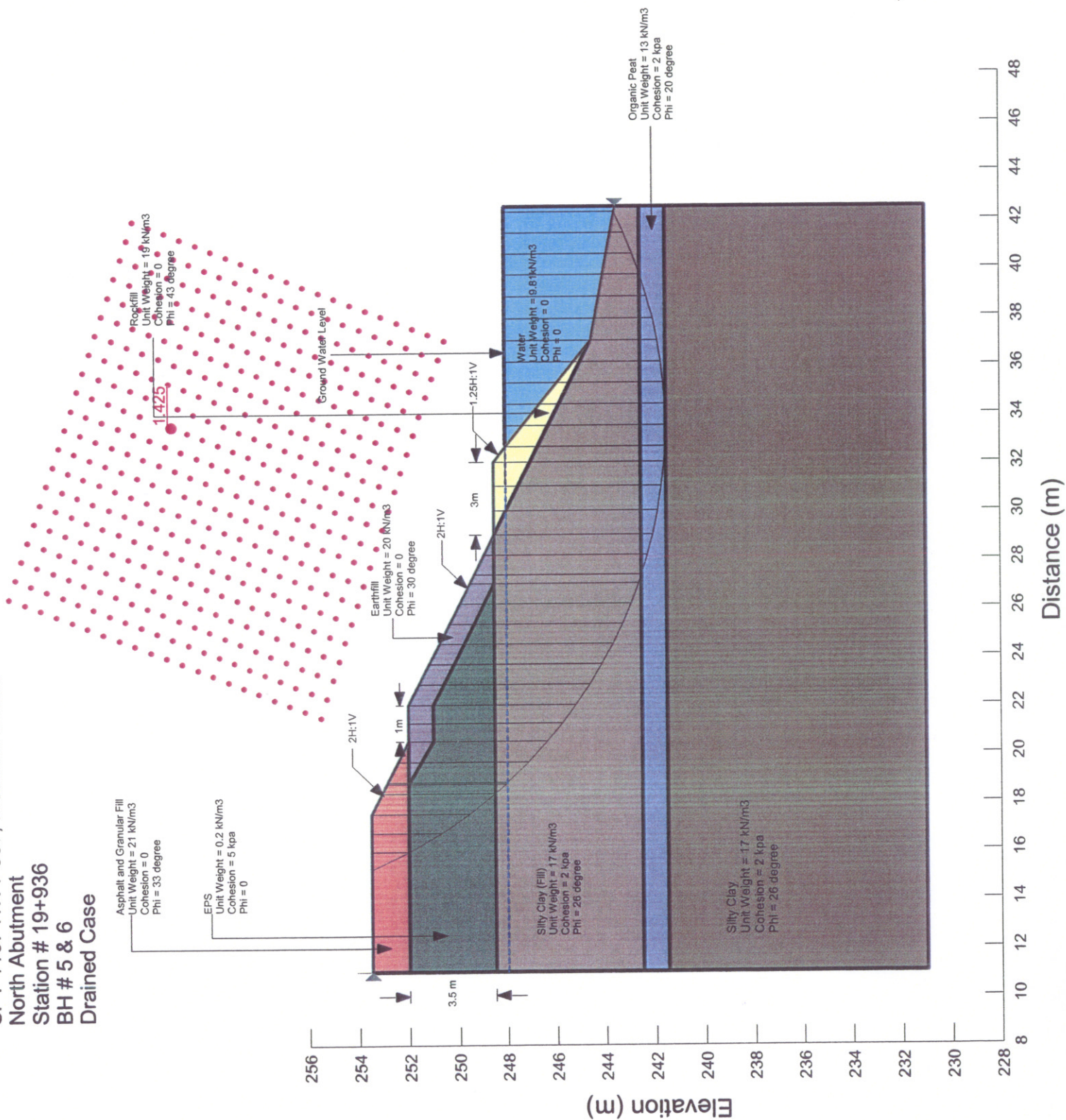
Appendix I

Typical Proposed Embankment Cross-Sections
Near North Abutment

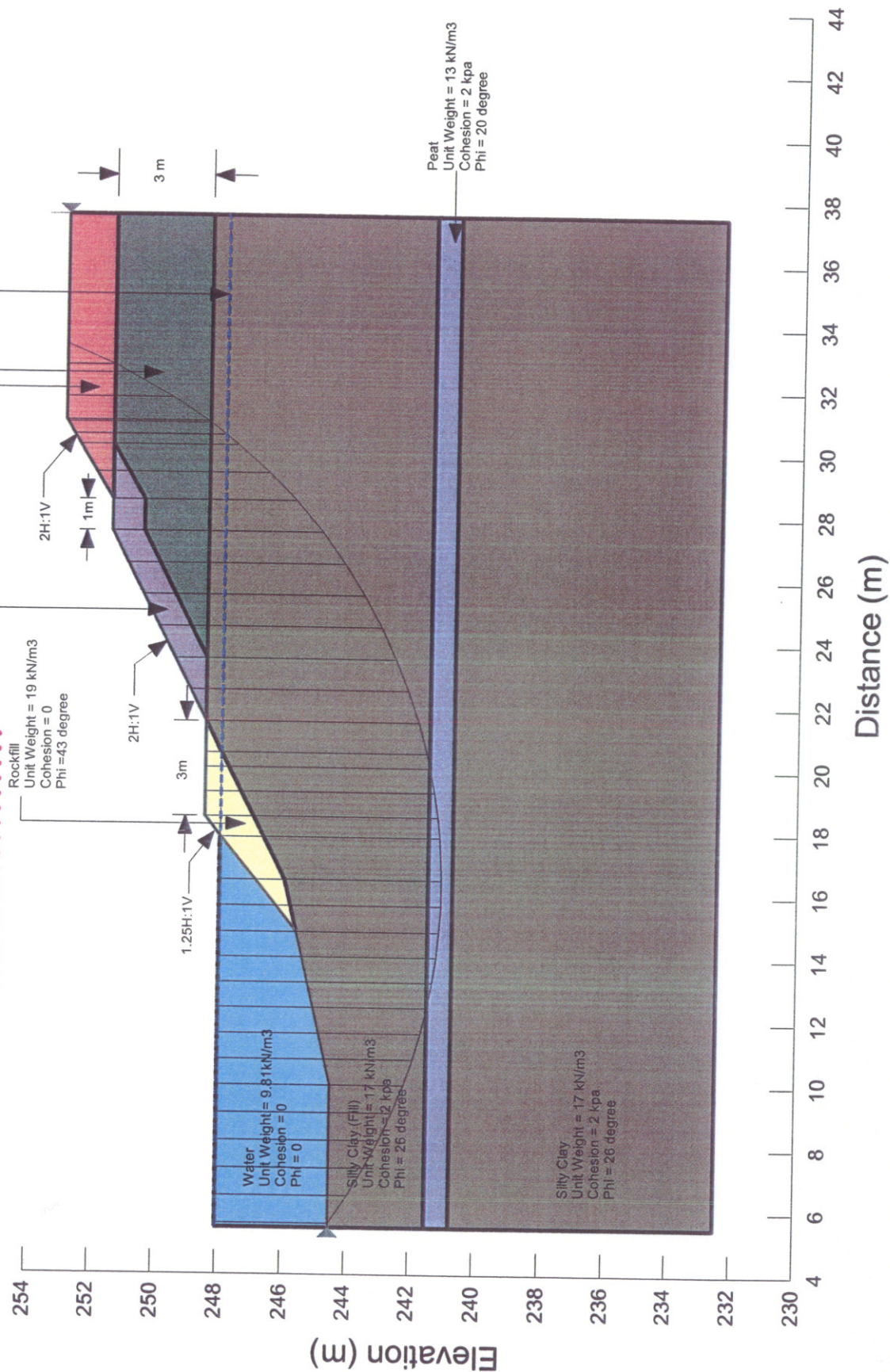
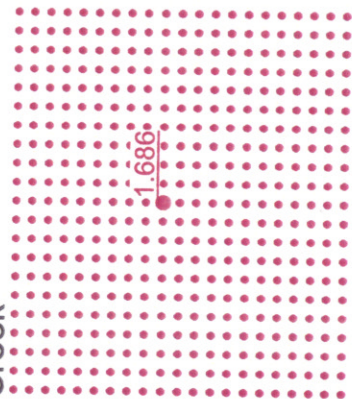
Appendix J

Embankment Slope Stability at Stations 19+846 and 19+936

SPT 1167 HWY 557, Meadow Creek
 North Abutment
 Station # 19+936
 BH # 5 & 6
 Drained Case



SPT 1167 HWY 557, Meadow Creek
 South Abutment
 Station # 19+846
 BH # 2 & 3
 Drained Case



Appendix K

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.

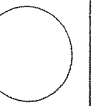
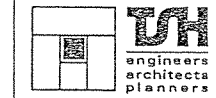
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METRIC

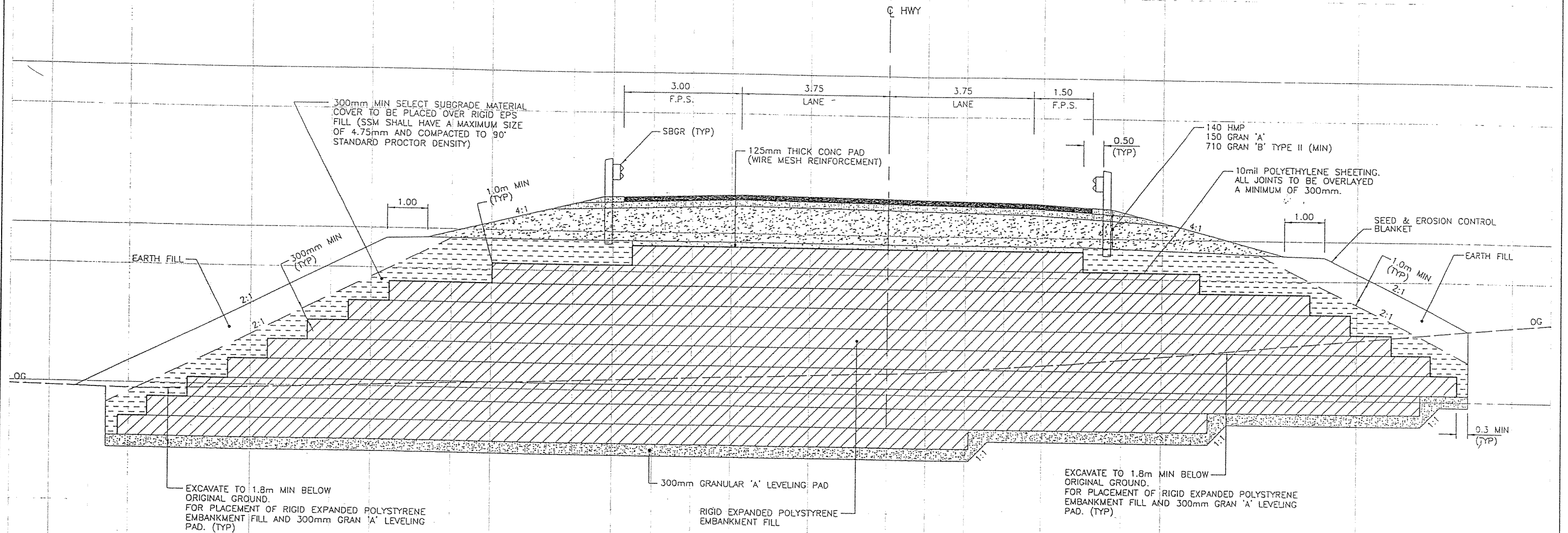
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT
WP

TYPICAL SECTIONS



SHEET
1



RIGID EXPANDED POLYSTYRENE EMBANKMENT
TYPICAL SECTION

N.T.S.

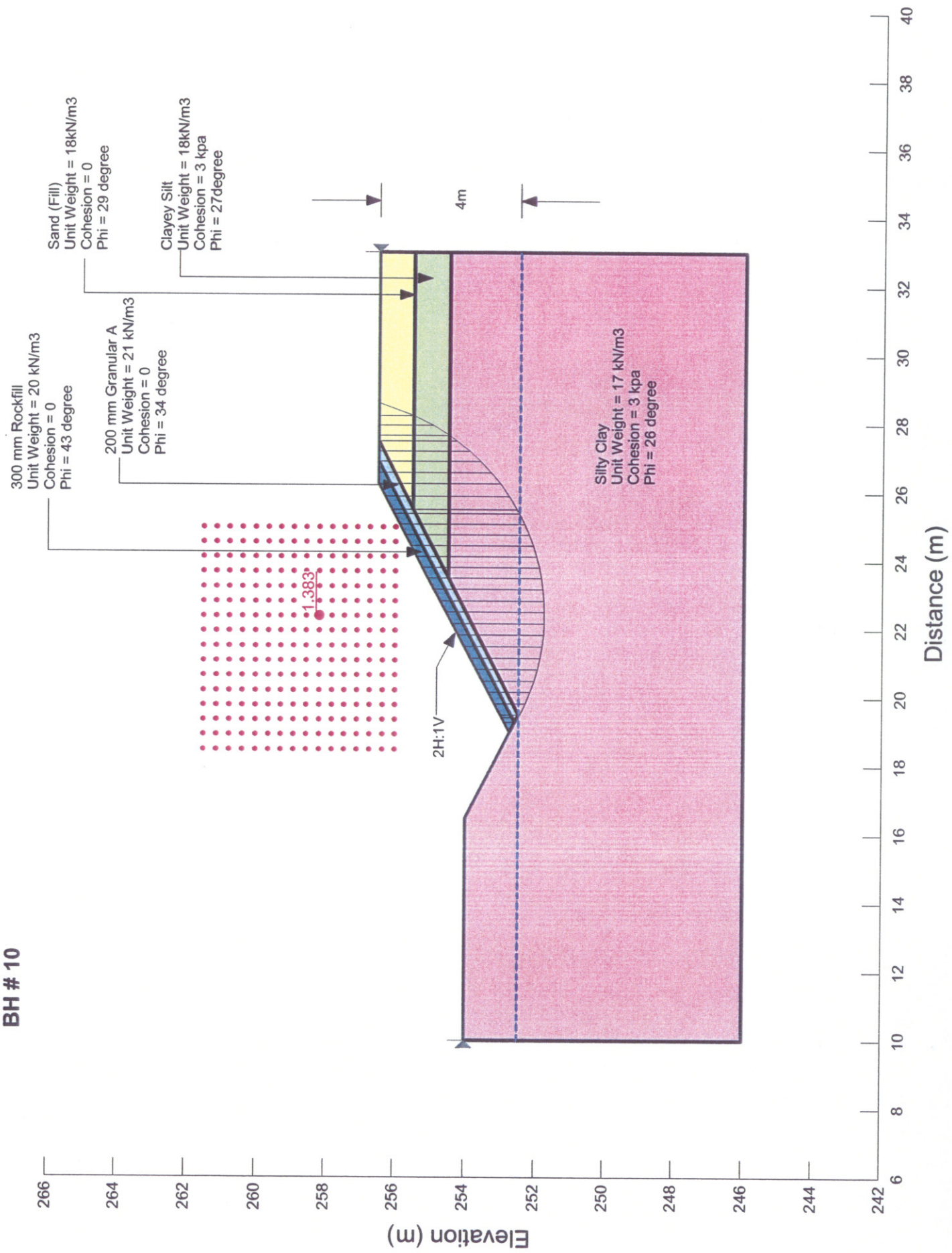
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MINISTRY OF TRANSPORTATION, ONTARIO
PH-9-707 88-05

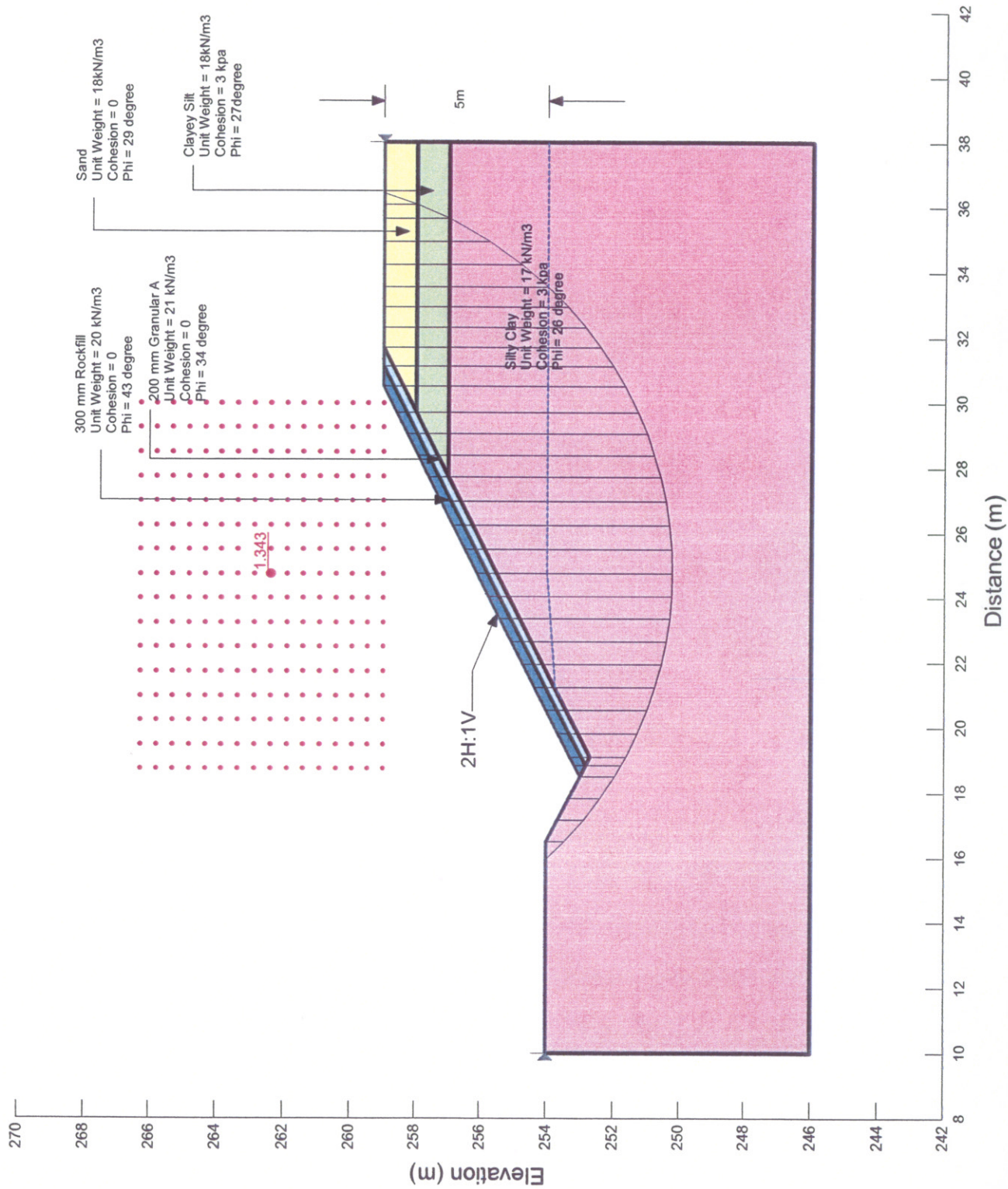
Appendix L

Typical Cut Slope Stability Analyses Results

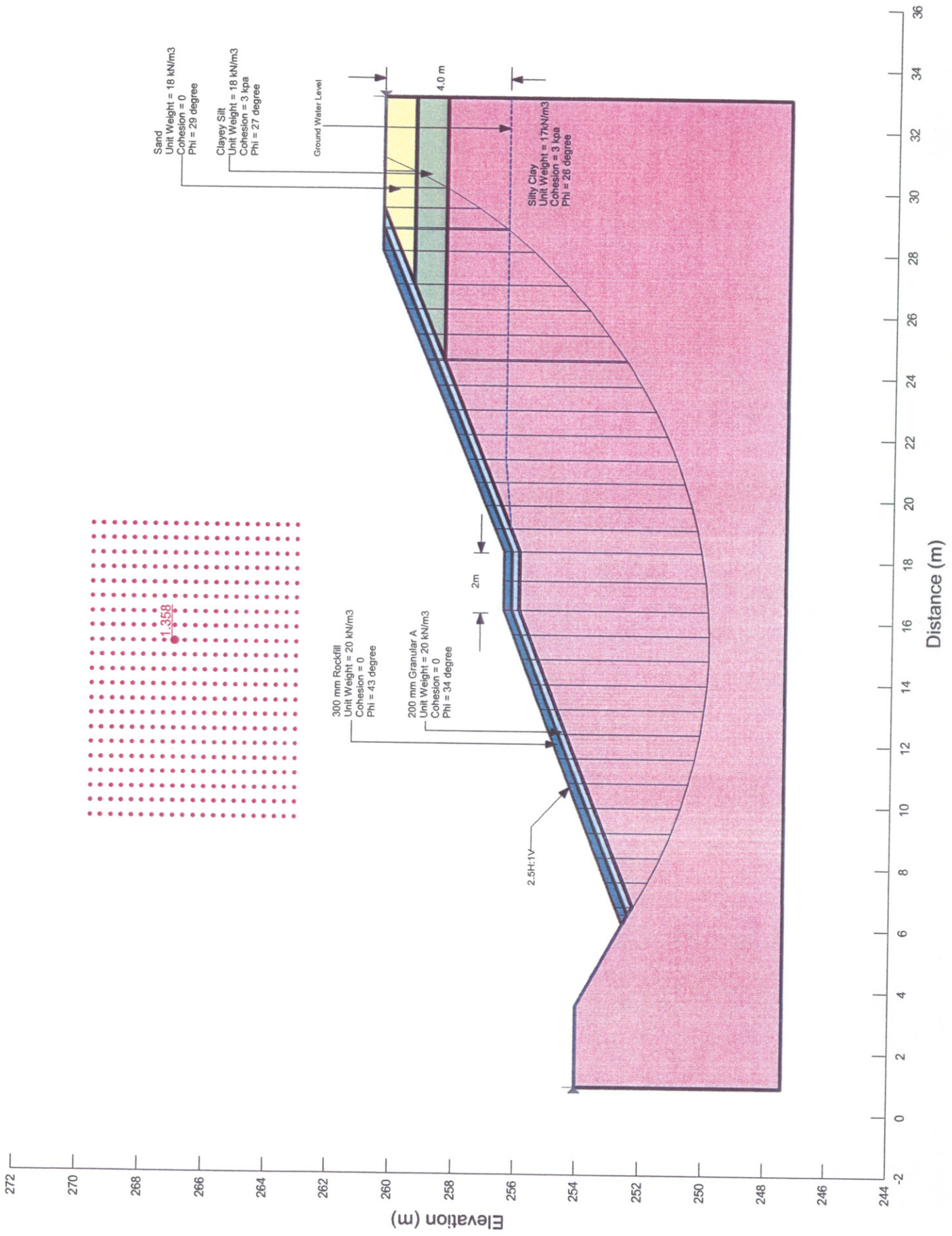
HWY 557, SPT 1167
Height of Cut = 4 m
BH # 10



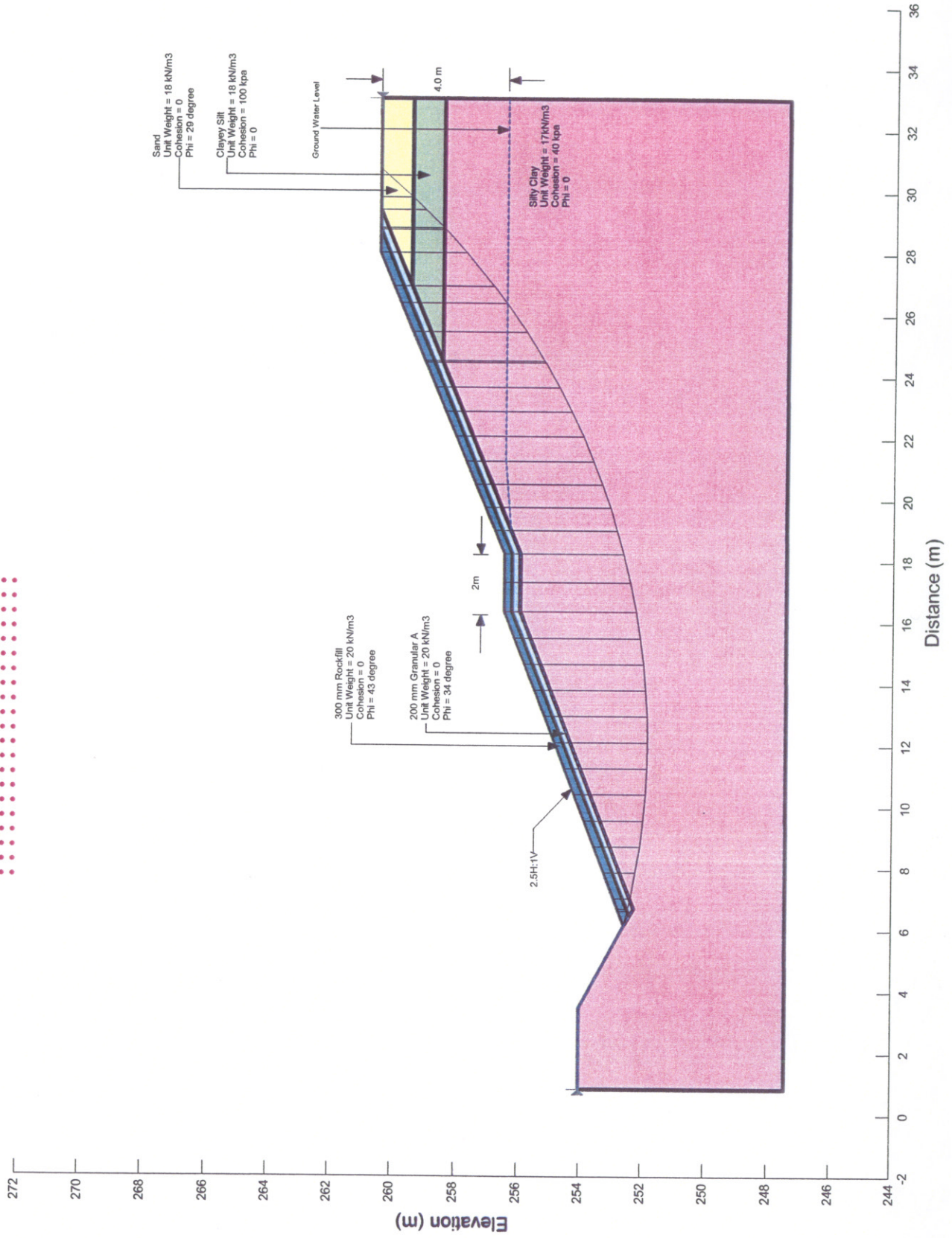
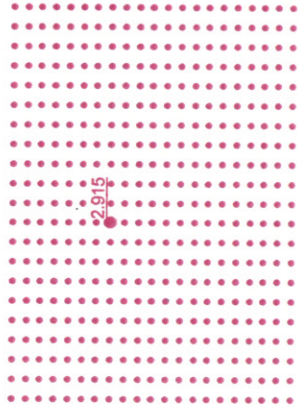
HWY 557, SPT 1167
 Height of Cut = 6 m
 Drained Case
 BH # 10



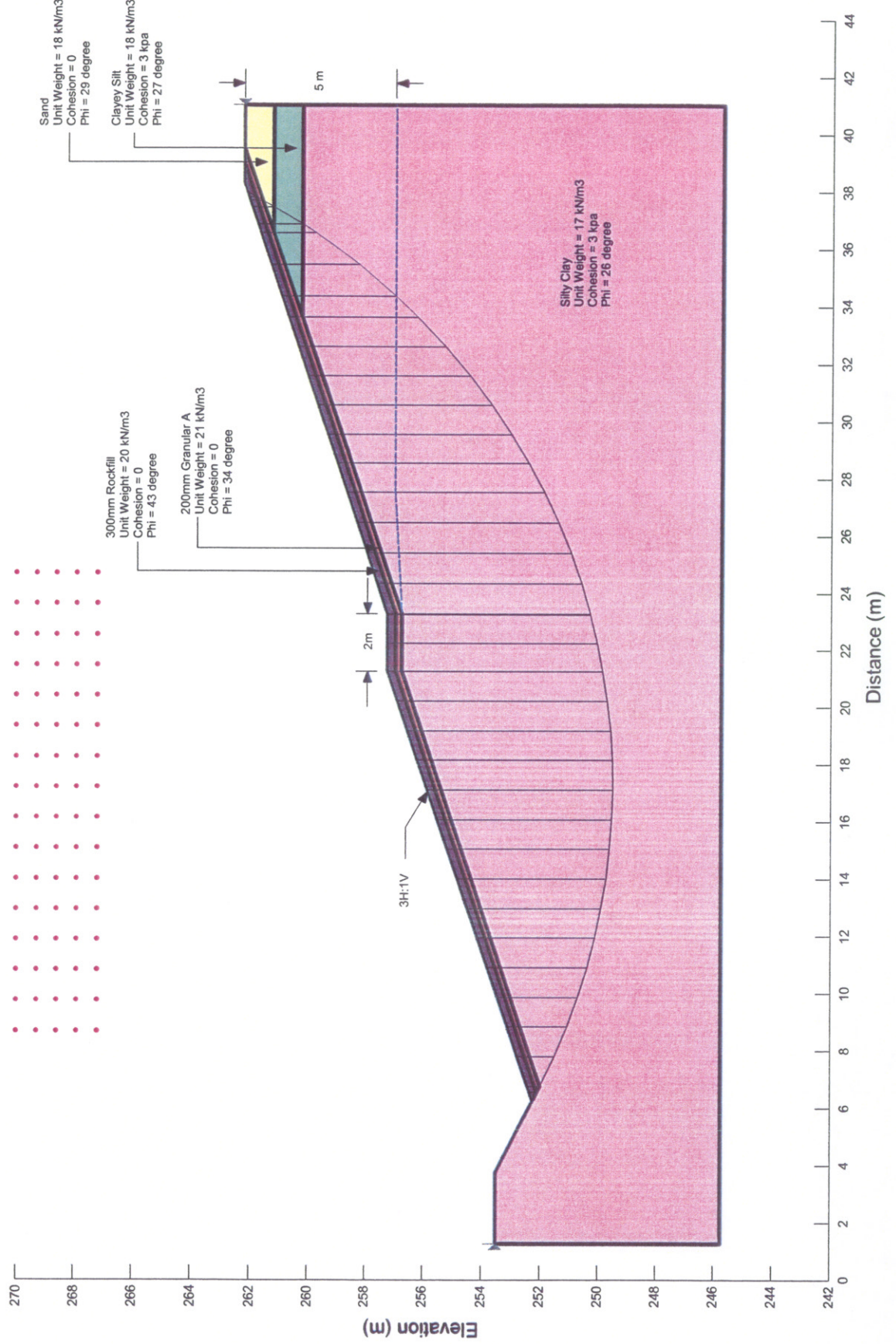
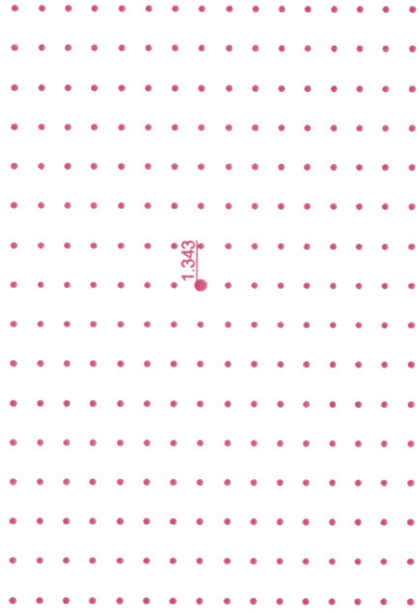
HWY 557, SPT 1157
Height of Cut = 8m
Drained Case
BH # 10



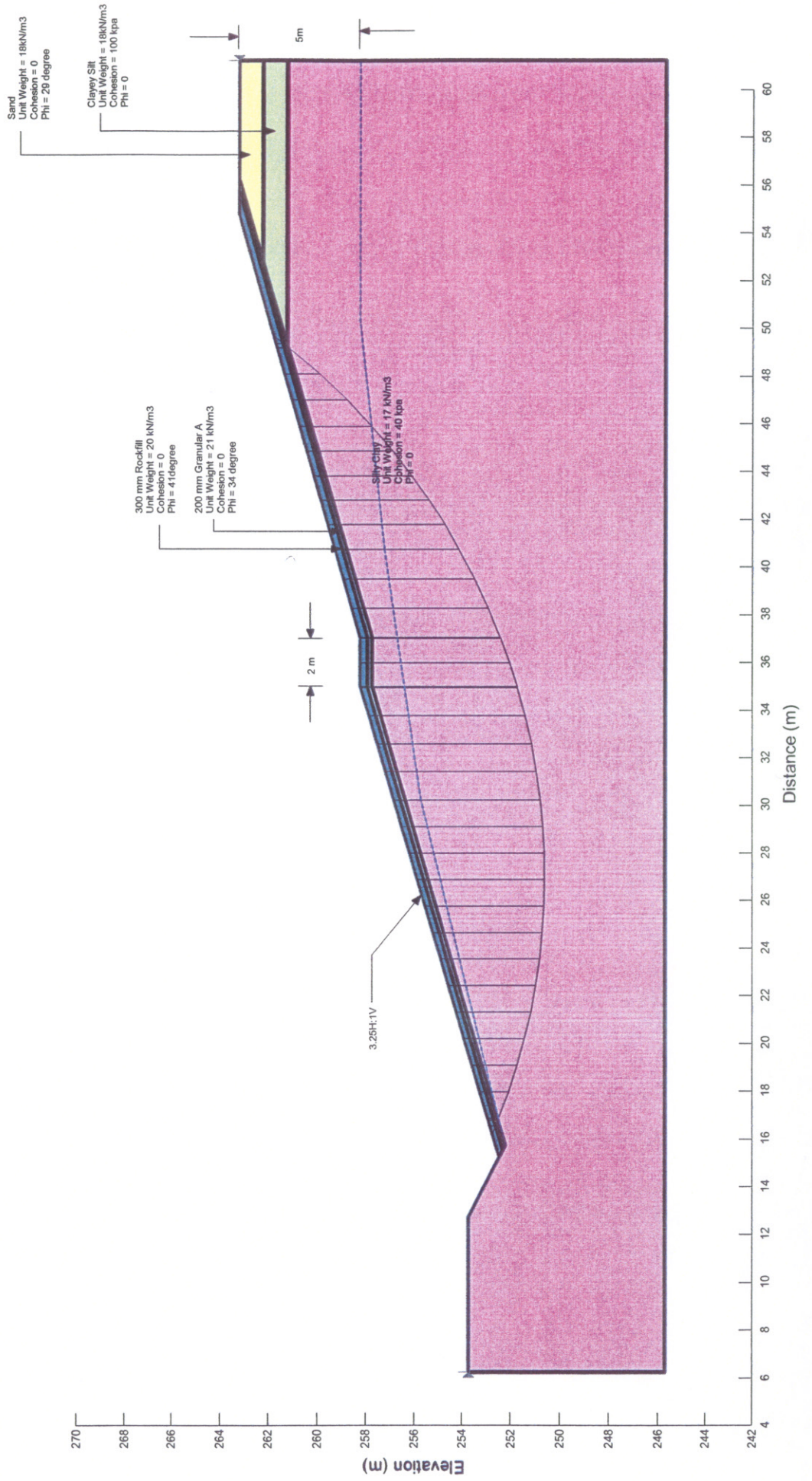
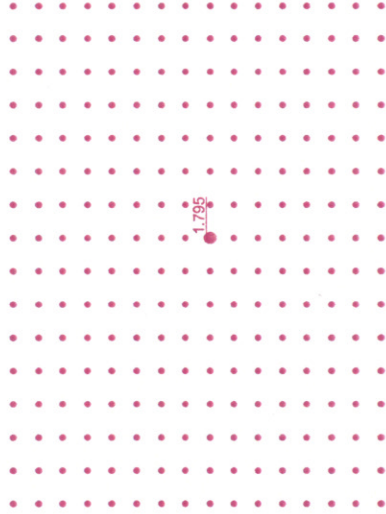
HWY 557, SPT 1157
 Height of Cut = 8m
 Undrained Case
 BH # 10



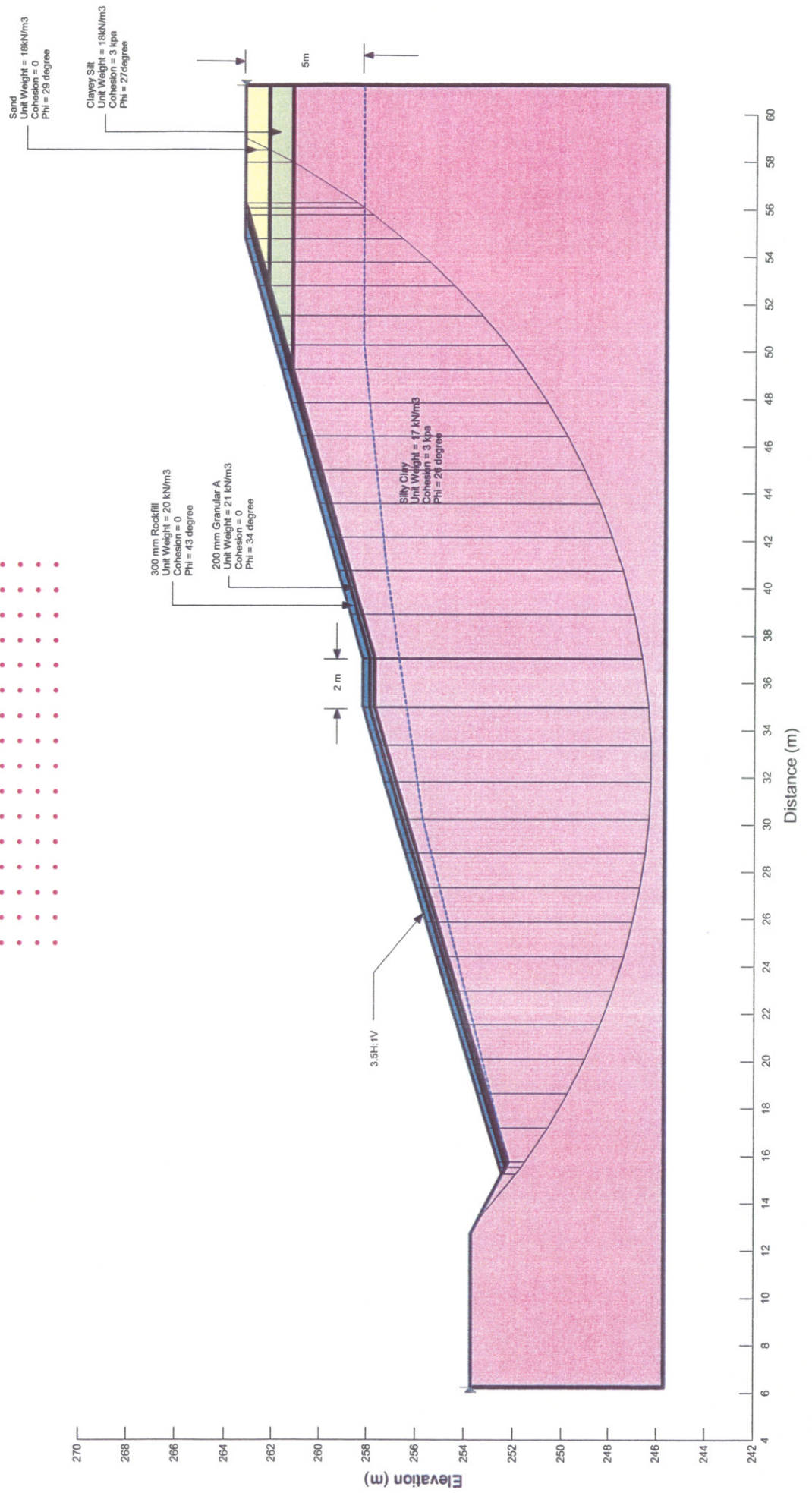
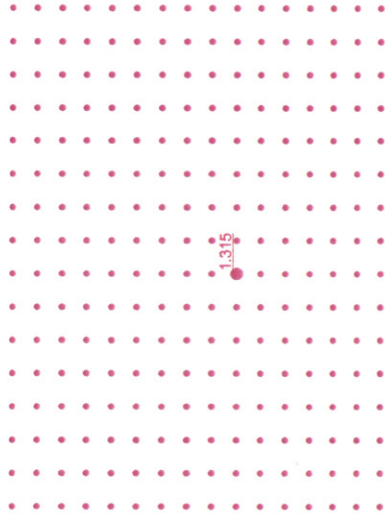
HWY 557, SPT 1167
 Height of Cut = 10 m
 Drained Case
 BH # 10



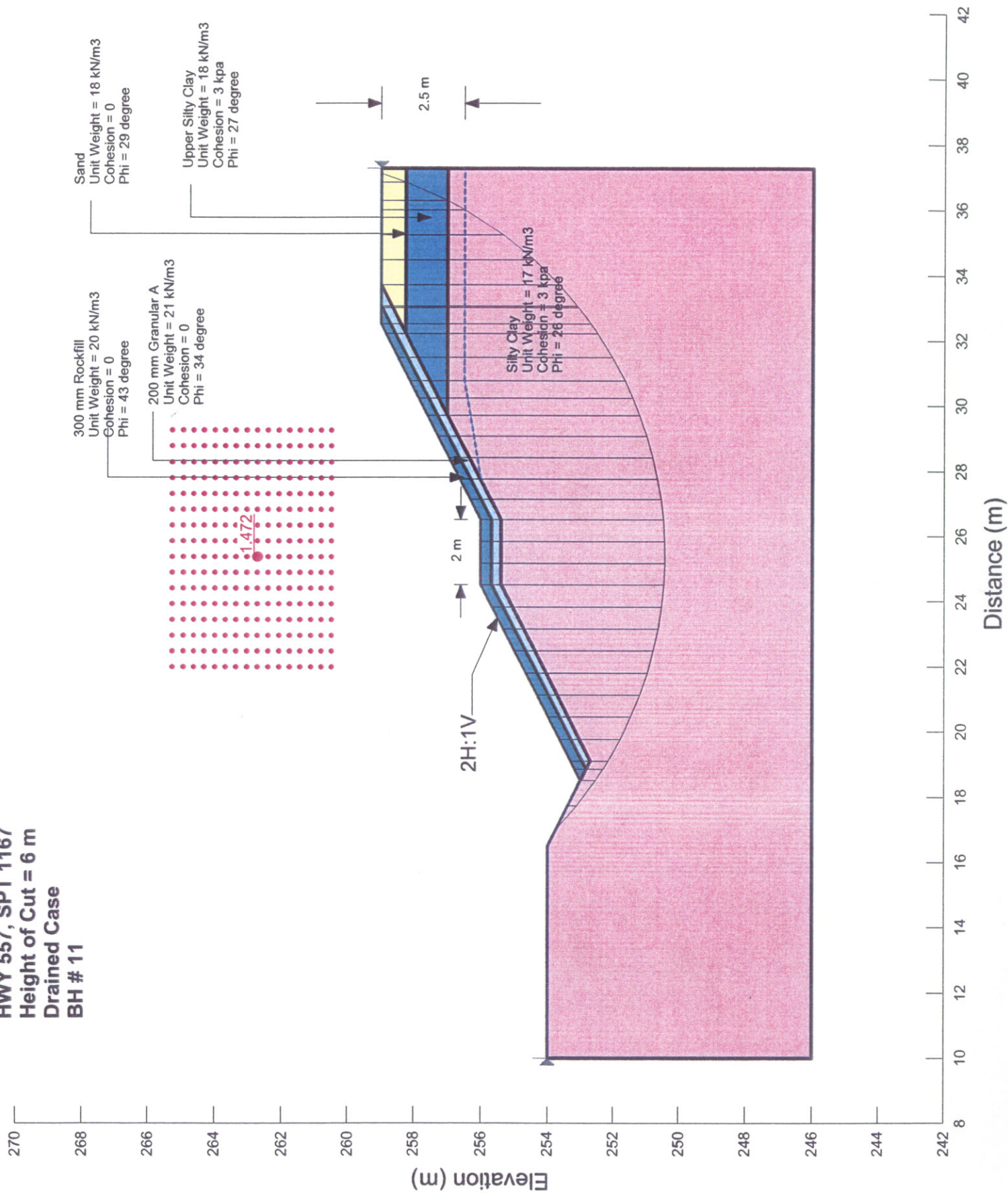
HWY 557, SPT 1167
 Height of Cut = 11 m
 Undrained Case
 BH # 10



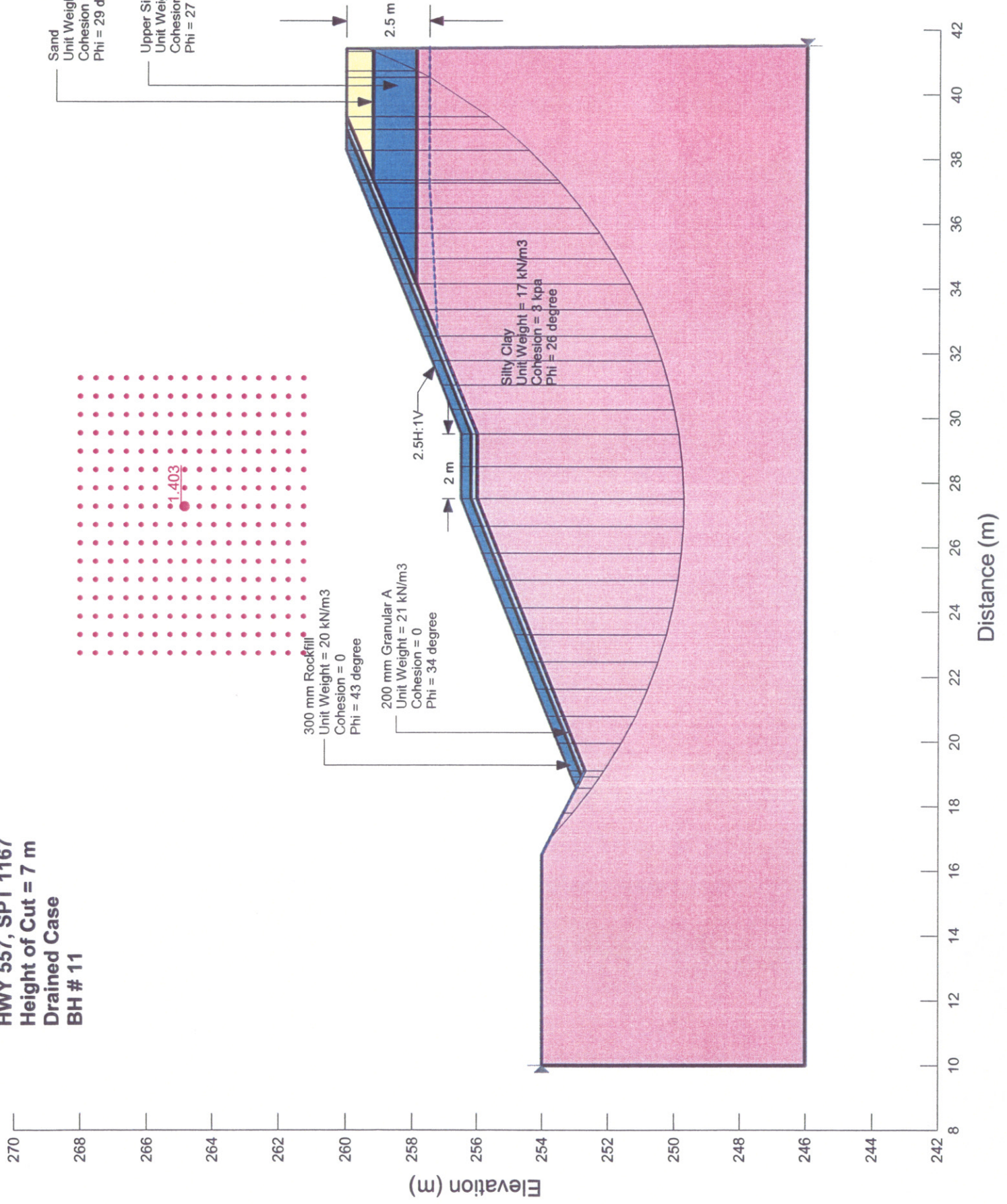
HWY 557, SPT 1167
 Height of Cut = 11 m
 Drained Case
 BH # 10

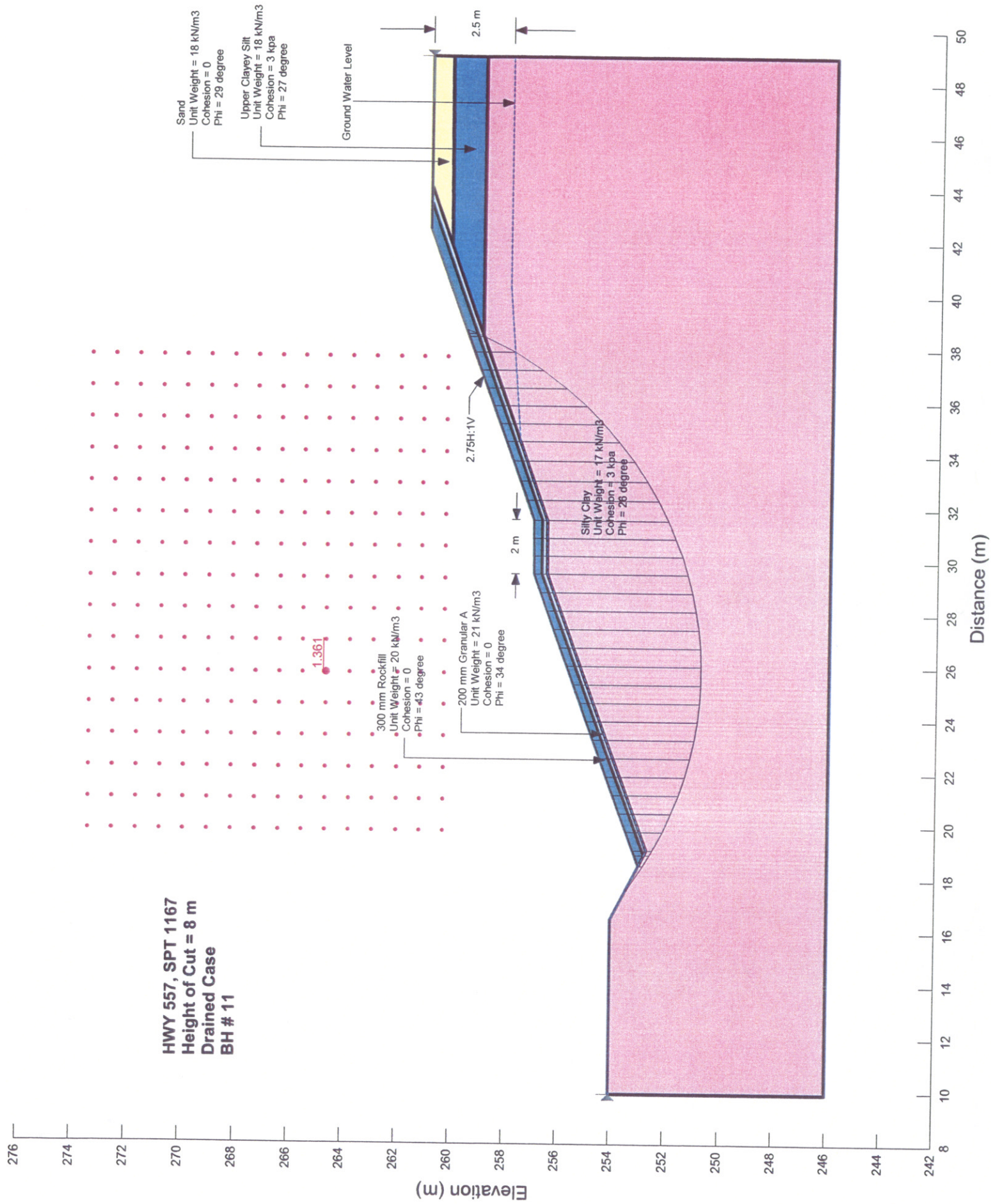


**HWY 557, SPT 1167
Height of Cut = 6 m
Drained Case
BH # 11**



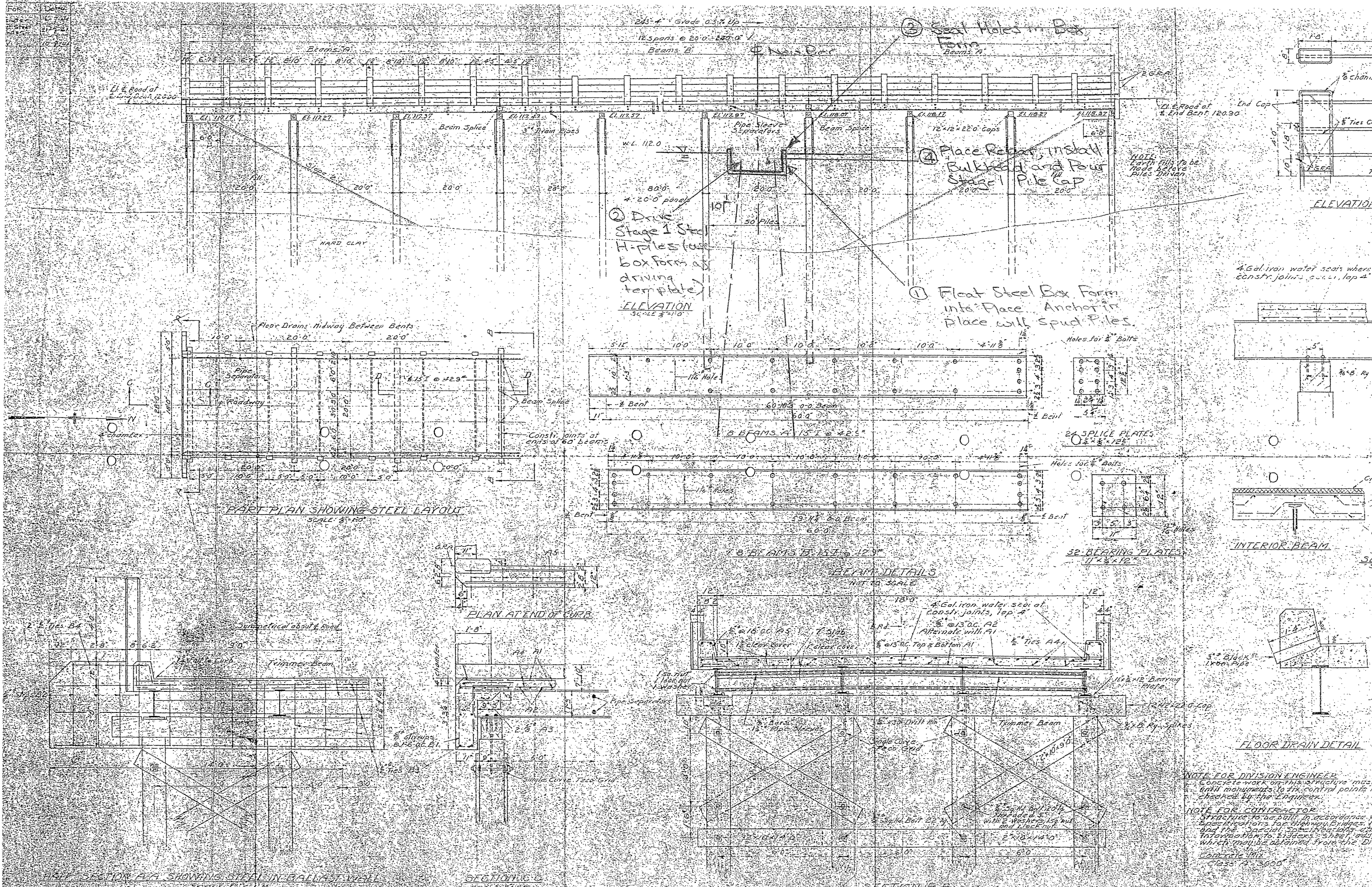
**HWY 557, SPT 1167
Height of Cut = 7 m
Drained Case
BH # 11**





Appendix M

Existing Bridge Configuration



② Drive Stage 1 Steel H-piles (use box form as driving template)
ELEVATION
SCALE 3/4" = 1'-0"

① Float Steel Box Form into Place. Anchor in place with Spud Piles.

④ Place Rebar, install Bulkhead and Pour Stage 1 Pile Cap

③ Seal Holes in Box Form Beams A

NOTE: Earth Fills to be made before piles driven

4 Gal. iron water seals where constr. joints occur, lap 4"

NOTE FOR DIVISION ENGINEER
Concrete work on this structure must until monuments to fix control points be checked by the Engineer.

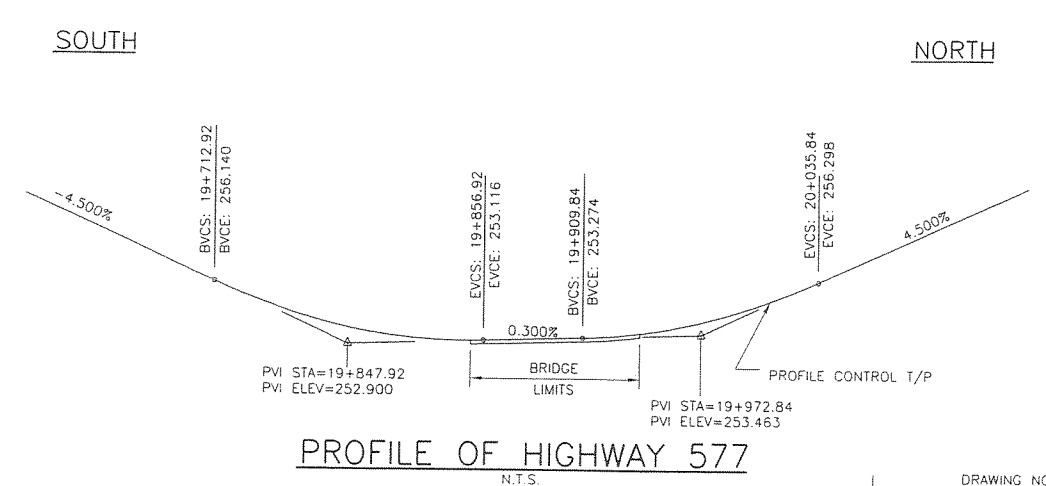
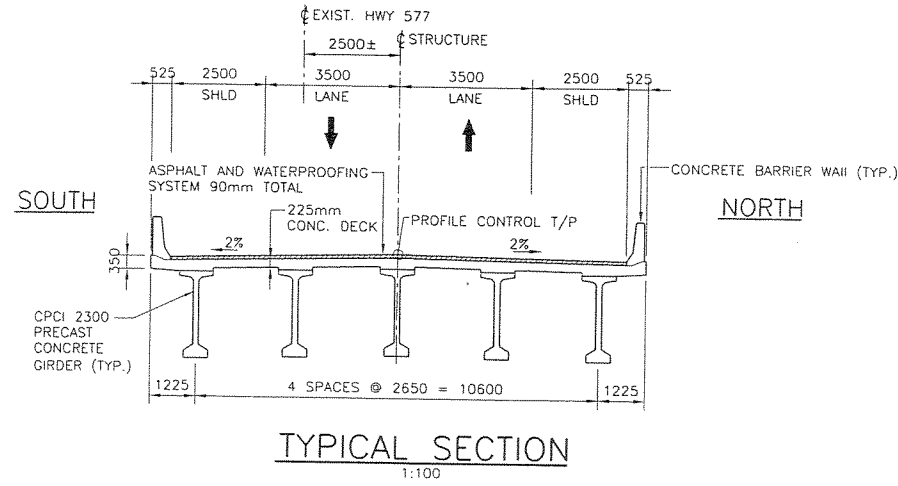
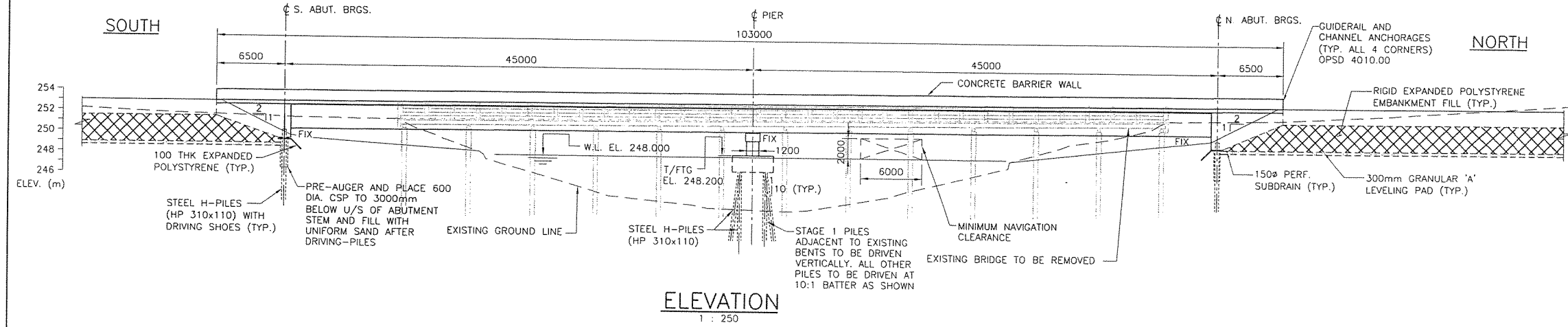
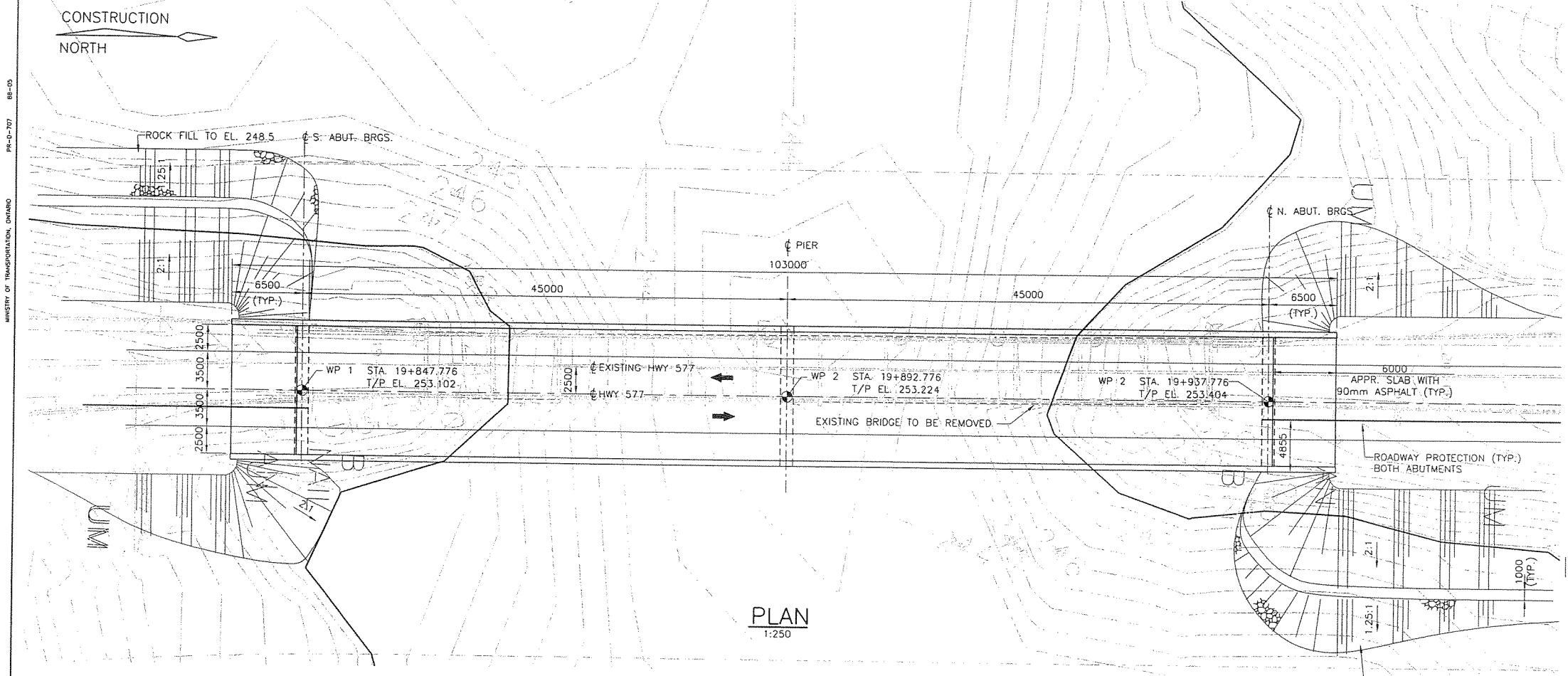
NOTE FOR CONTRACTOR
Structure to be built in accordance with Specifications for Highway Bridges 19 and the Special Specifications and Information to Bidders sheet which may be obtained from the Div.

Concrete Mix
Class A-3000

Appendix N

Preliminary General Arrangement by MCR

FILE LOCATION: S:\6336\ DRAWING NAME: S6336-300-001.CADWG DRAWN BY: SALLY Z MODIFIED: 2007/01/24 DATE PLOTTED: 2007/01/24 15:26:11 15:26:4



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

DISTRICT
CONT. No.
WP No.

HWY 577
MEADOW CREEK BRIDGE

PRELIMINARY GENERAL ARRANGEMENT

MRC

McCORMICK RANKIN
CORPORATION

GENERAL NOTES:

CLASS OF CONCRETE

PRECAST GIRDERS 50 MPa
REMAINDER UNLESS OTHERWISE NOTED 30 MPa

CLEAR COVER TO REINFORCING STEEL

FOOTINGS 100 ± 25
DECK
TOP 70 ± 20
BOTTOM 40 ± 10
REMAINDER UNLESS OTHERWISE NOTED 70 ± 20

REINFORCING STEEL

1. REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
2. BAR MARKS WITH PREFIXED 'C' DENOTE EPOXY COATED BARS.
3. UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
4. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES

1. THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESSES FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESSES ARE DIFFERENT FROM THOSE GIVEN IN THE DRAWING, THE CONTRACTOR SHALL ADJUST THE REINFORCING STEEL TO SUIT.
2. NO BACKFILL SHALL BE PLACED BEHIND ABUTMENTS UNTIL DECK CONCRETE HAS REACHED 25 MPa STRENGTH.
3. BACKFILL SHALL BE PLACED SUMULTANEOUSLY BEHIND BOTH ABUTMENTS KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME. AT NO TIME SHALL THE DIFFERENCE IN ELEVATION BE GREATER THAN 0.5m.
4. ALL ELEVATIONS ARE GEODETIC DATUM.

LIST OF ABBREVIATIONS

W.P. - DENOTES WORKING POINT
T/P - DENOTES TOP OF PAVEMENT
T/FTG - DENOTES TOP OF FOOTING
NTS - DENOTES NOT TO SCALE
SHLD - DENOTES SHOULDER

LIST OF APPLICABLE STANDARD DRAWINGS

OPSD 4010.00 GUIDE RAIL AND CHANNEL ANCHORAGE

REVISIONS

NO.	DESCRIPTION	DATE

DESIGN
DRAWN

CHK
CHK

CODE
SITE

LOAD
STRUCT

SCHEME
SCHEME

DATE
DWG

Appendix O

Statement of Limitations

LIMITATIONS OF REPORT

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

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

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

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

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

Appendix G

Explanation of Terms Used in Report