

**FOUNDATION INVESTIGATION &
DESIGN REPORTS
PROPOSED WIDENING OF HIGHWAY 17 FROM
STATION 26+295 TO 26+575
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM
MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-125**

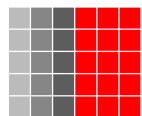
Prepared For:

D. M. WILLS ASSOCIATES

Prepared by:

**SHAHEEN & PEAKER
A Division of Coffey Geotechnics Inc.**

**Project: SPT1211B
November 28, 2008**



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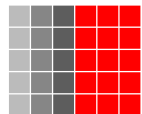
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Table of Contents

1. INTRODUCTION	1
2. PHYSIOGRAPHY AND GEOLOGY	1
3. INVESTIGATION PROCEDURES	2
4. SUBSURFACE CONDITIONS	4
4.1 Embankment Fill	4
4.2 Organic Soils	5
4.3 Upper Sand	6
4.4 Layered Silty Fine Sand/Silt/Clayey Silt	6
4.5 Silty Clay	7
4.6 Lower Sand	7
4.7 Refusal Depths	8
4.8 Bedrock	9
4.9 Groundwater Conditions.....	9

DRAWING	DRAWING No.
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BOREHOLE LOCATIONS & SOIL STRATA	1
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APPENDIX A: RECORD OF BOREHOLE SHEETS

APPENDIX B: LABORATORY TEST RESULTS

APPENDIX C: ROCK CORE PHOTOGRAPHS

APPENDIX D: SITE PHOTOGRAPHS

APPENDIX E: EXPLANATION OF TERMS USED IN REPORT

**FOUNDATION INVESTIGATION REPORT
PROPOSED WIDENING OF HIGHWAY 17
FROM STATION 26+295 TO 26+575, HIGHWAY 17,
FROM 9.5 KM EAST OF HIGHWAY 533, EASTERLY 14.9 KM
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1. INTRODUCTION

It is planned to rehabilitate Highway 17, from 9.5 km east of Highway 533 (Mattawa) easterly for 14.9 km. The project includes an eastbound truck passing lane. This report deals with the proposed truck passing lane between Stations 26+295 and 26+575.

Shaheen & Peaker (S&P) was retained by D. M. Wills Associates Limited to carry out a foundation investigation at the site of the proposed widening between Stations 26+295 and 26+575.

The purpose of the investigation was to obtain information on the subsurface conditions at the site of the proposed widening by means of boreholes.

The findings of the investigation are presented in this report.

2. PHYSIOGRAPHY AND GEOLOGY

The project site is located on Highway 17, between Mattawa and Deux-Rivieres, some 70 km east of North Bay, Ontario.

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

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

The site is located in between two rock knobs, with a grade sharply falling towards a creek valley (Resmer Creek) in between knobs. Starting from the west side, the grade at the o.g. level falls sharply towards the east (i.e. towards the project site), to about El. 208 m at about Station 26+050. The grade continues to fall to about El. 200 m at Station 26+250. From thereon, the grade falls more gradually towards the Resmer Creek at about Station 26+500 to about El. 192 m. Beyond the creek, the grade rises easterly to about 204 m at about Station 26+800, and finally, to a rock outcrop (i.e. top of the second knob) at about Station 27+100 m.

3. INVESTIGATION PROCEDURES

The fieldwork at this project site was carried out during the period from April 23 to April 26, 2008. The field investigation consisted of drilling and sampling a total of nine boreholes to depths ranging between 0.8 and 13.7 m below the ground surface, as follows:

Borehole 26+295,	15.2 m Rt of CL	-	2.7 m deep
Borehole 26+350,	13.7 m Rt of CL	-	5.3 m deep
Borehole 26+407,	10.4 m Rt of CL	-	2.1 m deep
Borehole 26+442,	17.3 m Rt of CL	-	0.8 m deep
Borehole 26+475,	5.4 m Rt of C L	-	7.8 m deep
Borehole 26+509,	5.0 m Rt of CL	-	13.7 m deep
Borehole 26+510	18.5 m Rt of CL	-	11.0 m deep
Borehole 26+555,	14.2 m Rt of CL	-	7.2 m deep
Borehole 26+575,	5.3 m Rt of CL	-	2.4 m deep

In addition, a Dynamic Cone Penetration test was performed at Stations 26+490, 21.5 m Rt and 26+530, 17.4 m Rt of CL as well as from the bottom of Borehole 26+350.

Landcore Drilling Inc. of Chelmsford, Ontario, drilling contractor, carried out the drilling, testing and sampling under the supervision and direction of a Professional Engineer from S&P.

The locations of the boreholes in the field are given on the Borehole Location Plan, Drawing No. 1.

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

Six of the nine boreholes were terminated upon encountering refusal to further sampling (i.e. split-spoon sampler bouncing without any penetration) and refusal to further augering. Borehole 26+350 was terminated after encountering refusal to further driving of the cone in the Dynamic Cone Penetration Test. In the remaining two boreholes (i.e. Boreholes 26+509 and 26+510), after encountering practical refusal on the augers, the bedrock was proven by diamond drilling methods whereby NQ size rock cores were obtained. The length of coring by diamond drilling was 3.8 m and 3.5 m in Boreholes 26+509 and 26+510, respectively.

As mentioned before, Dynamic Cone Penetration Tests were performed at Stations 26+490, 26+530 and 26+350. In Dynamic Cone Penetration Test (DCPT), a 51 mm diameter, 60 deg. apex cone point, screw-attached to the tip of A-size rods, is driven into the ground using the same driving energy as in the SPT method. By recording the number of blows to drive the cone/rod assembly into the soil every 0.3 m, a qualitative record of relative density/consistency is obtained. Although the interpretation of the test results is difficult because no samples are obtained by the DCPT method and the penetration resistances are not necessarily equal to the N-values, useful information is gained by the continuity of the results and by the elimination of unbalanced hydrostatic effects which in many cases affect the SPT values, especially in the fine-grained granular soils. In the present case, the DCPT was generally terminated when the number of blows to drive the cone/rod assembly by 0.1 m exceeded 200. At Station 26+490, due to drill inaccessibility, the DCPT was conducted manually using a 31.8 kg hammer (instead of 63.5 kg) similar to SPT method. In this case, the recorded number of blows were divided by two to obtain an approximate equivalent resistance value.

Groundwater conditions in the boreholes were observed during the drilling in the open boreholes. Upon their completion, the boreholes were grouted using a cement/bentonite mixture as per MTO procedures. In Borehole 26+555, a piezometer was installed to enable us to monitor the groundwater level over a prolonged period of time without interference from surface water.

The details of the drilling, sampling, field testing and soil conditions encountered are given on the Record of Borehole Sheets in Appendix A. An inferred subsurface profile is presented in Drawing No. 1.

A laboratory testing programme, consisting of natural moisture content measurements, Atterbert Limits tests and grain-size analyses was performed on selected soil samples. The results of laboratory tests are presented on the appropriate Record of Borehole Sheets and also in Appendix B.

The ground surface elevations at the borehole locations were provided to us by our client (D. M. Wills Associates Limited). We understand that the elevations are related to the Geodetic Datum.

4. SUBSURFACE CONDITIONS

This investigation consisted of drilling and sampling nine boreholes. As well, a dynamic cone penetration test was performed at Stations 26+490 and 26+530 and also from the bottom of Borehole 26+350. The boreholes were put down between Stations 26+295 and 26+575, all on the right (south) side of the highway, some from the top and some from the bottom of the highway embankment.

At Station 26+295 the top of road elevation is about 201.0 m, dropping to about El. 198.3 m at Station 26+500. From thereon, the top of road elevation rises easterly to El. 198.7 m at Station 28+575 and continues to rise more steeply in the easterly direction. The original grades (o.g.) in this stretch are between about 199.6 and 200.8 m between Stations 26+295 and 26+450 and falls sharply thereafter to Resmer Creek at Station 26+510 area to about El. 191 m and rising sharply to about El. 197.6 m at Station 26+575. The height of the embankment varies from about zero (i.e. almost at grade) to about 7 m at Resmer Creek.

Details of the subsurface conditions encountered in the boreholes are presented on the Record of Borehole Sheets in Appendix A, while a stratigraphic profile is given in Drawing No. 1. The various soil strata encountered in the boreholes and their geotechnical properties are briefly described in the following paragraphs. It should be noted that the soil and groundwater conditions may vary in between and beyond the borehole locations.

4.1 EMBANKMENT FILL

Boreholes 26+475, 26+509 and 26+575 were drilled from the top of the road embankment (shoulder area) and consequently encountered embankment fill materials to depths ranging from 1.2 m (Borehole 26+575) to 5.4 m (Borehole 26+509).

Granular pavement (i.e. gravelly sand) fill was contacted in two of the three boreholes to depths of 0.7 m (Borehole 26+509) and 0.9 m (Borehole 26+475). Standard Penetration tests yielded N-values of 19 and 26 blows/0.3 m in the granular fill indicating a compact condition.

Below this upper granular fill, the embankment fill was found to consist typically of fine sand with traces to some silt and occasional gravel, which is basically granular (i.e. non-cohesive) material. The grain-size distribution of samples from this fill was determined on three samples and these show the following grain-size distribution, as presented in Figure B-1 in Appendix B.

Gravel:	0-3%
Sand:	87-92%
Silt & Clay:	8-10%

Standard Penetration tests performed in the embankment fill yielded N-values of between 7 and 20 blows/0.3 m, indicating a loose to compact relative density.

Fill was also contacted in Boreholes 26+510 and 26+555, which were drilled near the toe of the embankment.

In Borehole 26+510, which was located near the existing Resmer Creek culvert, fill was found to extend to a depth of 2.3 m or to El. 191.5 m. The upper 0.3 m of this fill was found to consist of topsoil underlain by sand fill with traces of gravel. The sand fill is in turn underlain by a rather random type fill consisting of silty fine sand with traces of gravel, asphalt pieces and organics. This is a basically granular (i.e. non-cohesive) soil. N-values of 2, 8 and 2 blows/0.3 m were recorded in this fill indicating that it has not received compaction.

In Borehole 26+555 the fill was also found to consist of a rather random nature granular (i.e. non-cohesive) soil comprised of silty fine sand mixed with topsoil. This deposit was found to extend to 0.9 m (El. 195.3 m) and based on a recorded N-value of 2 blows/0.3 m it is considered to be in a very loose state of compaction.

It should be pointed out that in our experience the thickness of fill deposits can vary in between and beyond borehole locations.

4.2 ORGANIC SOILS

In Boreholes 26+295, 26+350, 26+407 and 26+442, which were drilled from the bottom of the embankment a 0.15 to 0.25 m thick layer of topsoil was contacted at the ground surface level.

A 0.2 m thick sandy organic silt was contacted in Borehole 26+575, underlying the embankment fill at a depth of 1.2 m (i.e. it appears that organic materials were not totally stripped when the embankment was first constructed). Borehole 26+510, which was located near the bottom of the valley, contacted a 2.3 m thick random, loosely placed mixed fill at El. 191.5 m. This fill was found to be underlain by a 0.3 m thick sandy organic silt layer. It appears that at this location organic soil was not stripped before the random fill was dumped.

In Boreholes 26+475, 26+509 and 26+555, which were drilled from the shoulder of the highway, no organic soils were encountered. It appears that at these borehole locations, the topsoil and other organic soils were properly stripped before constructing the existing embankment.

We would like to point out that in our experience the thickness and extent of organic soils can vary in between and beyond borehole locations. The presence of thicker organic soils

can be expected in depressed areas and in particular near water courses (e.g. Resmer Creek valley).

4.3 UPPER SAND

Underlying the topsoil, Boreholes 26+407 and 26+442 contacted at 0.2 to 0.3 m below the ground surface, a granular deposit consisting of gravelly sand. This deposit was found to extend to refusal on the augers at 2.1 and 0.8 m, respectively. Gravelly sand was also encountered in Borehole 26+509 immediately underlying the embankment fill. The grain-size distribution of a sample of the gravelly sand from this borehole is given in Figure B-2 in Appendix B. The curve indicates the following grain-size distribution:

Gravel:	28%
Sand:	60%
Silt & Clay:	12%

N-values recorded in the three boreholes in this deposit range from 6 to 22 blows/0.3 m. These results indicate a loose to compact denseness condition.

A finer grained upper granular deposit was contacted in Boreholes 26+510 and 26+575. This material, which consists of silty sand with traces to some gravel, was contacted below the sandy organic silt layer (underlying fill) at depths of 2.6 and 1.4 m, respectively and extended to 3.7 and 2.1 m (i.e. 0.7 to 1.1 m thick layer). In Borehole 26+510, N-values of 2 and 4 blows/0.3 m were recorded indicating a very loose condition while in Borehole 26+575 an N-value of 30 blows/0.3 m was recorded, which indicates a compact condition.

In Borehole 26+575, the silty sand is underlain at 2.1 m below the ground surface by a coarse granular, sandy gravel deposit. The borehole was terminated upon encountering refusal at 2.4 m below the ground surface or 1.2 m below the fill deposit.

4.4 LAYERED SILTY FINE SAND/SILT/CLAYEY SILT

In Boreholes 26+295, 26+350, 26+475, 26+509, 26+510 and 26+555, underlying the topsoil, embankment fill or the upper sand deposits, a thinly interbedded fine sand to silt and clayey silt deposit was contacted. The interbeds typically consist of silty very fine sand, sandy silt, silt to clayey silt with occasional very thin clay seams.

The deposit was encountered at depths ranging from 0.3 to 8.4 m below the ground surface, typically underlying the embankment fill or the upper sand deposits. Its thickness at the borehole locations was found to range from 0.9 to 2.2 m.

In general, the deposit is a basically fine-grained granular (i.e. non-cohesive) type of soil with some cohesive interbeds/zones, except at Borehole 26+350 where it is primarily cohesive. The grain-size distribution of three samples from the relatively coarser (i.e. non-

cohesive) and less distinctly layered zones of the deposit is given in Figure B-3 in Appendix B. These indicate the following grain-size distribution:

Gravel:	0-1%
Sand:	35-58%
Silt & Clay:	42-65%

Standard Penetration tests performed in the deposit yielded N-values which range from 2 to 25 blows/0.3 m. From the recorded N-values the relative density of the basically granular deposit is described as very loose to compact (typically loose to compact in boreholes 26+295, 26+475 and 26+555, while very loose to loose in boreholes 26+509 and 26+510) and the consistency of the cohesive soil in Borehole 26+350 as firm.

4.5 SILTY CLAY

In Borehole 26+350, a 0.6 m thick silty clay layer was contacted from 0.7 to 1.3 m below the ground surface.

Atterberg Limits tests performed in the laboratory showed the following index values:

Liquid Limit:	34%
Plasticity Limit:	23%
Plasticity Index:	11%
Natural Moisture Content:	36%

As shown on the Plasticity Chart presented in Figure B-4, Appendix B, these results are characteristic of clayey soils of low plasticity. The fact that the measured natural moisture content is in excess of the measured liquid limit indicates the material is likely to be relatively weak and compressible.

A Standard Penetration test performed yielded an N-value of 6 blows/0.3 m and a pocket penetrometer test performed on the sample recovered from the deposit showed an undrained shear strength of about 50 kPa. Based on these values, together with a tactile and visual examination of the soil sample, the deposit is considered to have a firm consistency and a practically impervious permeability.

4.6 LOWER SAND

A lower granular deposit was contacted in Boreholes 26+295, 26+350, 26+475, 26+509, 26+510 and 26+555 at depths ranging from 1.4 m (Borehole 26+295) to 9.7 m (Borehole 26+509) below the ground surface or at depths of about 1.4 to 4.3 m below original ground (o.g.) levels.

The composition of these deposits ranged from silty fine sand with traces of gravel (Boreholes 26+475 and 26+555) to gravelly sand (Boreholes 26+350 and 26+555).

The grain-size distribution of two samples from the relatively finer grained silty fine sand with traces of gravel is given in Figure B-5 in Appendix B. These show the following grain-size distribution:

Gravel:	2-4%
Sand:	68-70%
Silt & Clay:	28%

Figure B-6 shows the grain-size distribution of the relatively coarser lower granular deposit from a combination of SS4 and SS5 samples from Borehole 26+350, as follows:

Gravel:	35%
Sand:	51%
Silt & Clay:	14%

In general, N-values recorded in the deposit range from 4 to 26 blows/0.3 m, indicating a very loose to compact but typically loose to compact material.

In Borehole 26+509, the deposit was found to be a very coarse grained material, consisting of cobbles/boulders and rock fragments with some sand and gravel infill. The presence of a similar bouldery zone with shattered rock was also found in the lower zone of the deposit in Borehole 26+510. These are believed to represent a zone immediately above the bedrock. A somewhat similar zone was found in Borehole 26+555 near the bottom, immediately above the refusal depth.

4.7 REFUSAL DEPTHS

All the boreholes except for Boreholes 26+350, 26+509 and 26+510 were terminated upon encountering refusal on the spoon-sampler and/or on the augers at depths of 0.8 to 6.3 m below the approximate o.g. levels. Some of the refusal depths are likely to be close to the bedrock surface.

In Borehole 26+350, which was extended to 4.4 m below the ground surface, a Dynamic Cone Penetration test was put down from the bottom of the borehole. This test encountered practical refusal at 5.3 m.

In Boreholes 26+509 and 26+510 auger refusal was encountered at about 10.0 m and 7.5 m below the ground surface or at El. 186.5 m, 188.2 m, respectively. Below these depths, the boreholes were advanced by diamond drilling and rock coring through a bouldery layer with shattered rock pieces and the surface of the bedrock was found at

depths of 10.7 and 7.9 m below the ground surface or at El. 187.5 m and 185.9 m, respectively.

4.8 BEDROCK

As mentioned in the preceding section, the presence of bedrock was proven by rock coring in Boreholes 26+509 and 26+510. The surface of the rock was found at about 5.3 m and 5.6 m below the o.g. levels or at El. 187.5 m and 185.9 m, respectively.

These two boreholes are located about 1 m apart in the longitudinal direction along the highway and about 13.5 m apart in the transverse direction (i.e. perpendicular to the highway). It appears therefore that the bedrock surface dips by 1.6 m in the transverse direction within a distance of 13.5 m or at a gradient of about 12%, from about 5 m from the centerline of the highway to 18.5 m from the centerline, in the southerly direction near the Resmer Creek Culvert location at about Station 26+510.

The rock was cored for a vertical distance of about 3.0 m. Visual examination of the rock cores showed that the bedrock consists of a granitic rock formation with some metamorphosed magmatic rock intrusions. It is mainly pinkish grey. In addition to horizontal and nearly horizontal fractures, the presence of vertical and subvertical fractures with some slight sediment inclusions and oxidization was noted. The percentage of recovery was 100% while the RQD values ranged from zero to typically 50 to 65%. These results show that within the depths cored, the upper 1.5 m± of the rock in Borehole 26+509 is highly fractured while the rest of rock is considered to be relatively sound.

The photographs of the rock cores are given in Appendix C.

4.9 GROUNDWATER CONDITIONS

Groundwater conditions in the open boreholes were observed during the drilling and upon completion of each borehole. In addition, a piezometer was installed in Borehole 26+555 to enable us to monitor the groundwater level over a prolonged period of time without interference from surface water.

The details of the observations are shown on the Record of Borehole Sheets. These indicate that at most borehole locations the groundwater levels at the time of our investigation were close to the o.g. levels.

It should, however, be pointed out that the groundwater table can be expected to undergo seasonal fluctuations as well as fluctuations in response to major weather events.

SHAHEEN & PEAKER LIMITED


Ramon Miranda, P.Eng.



Z.S. Ozden, P.Eng.

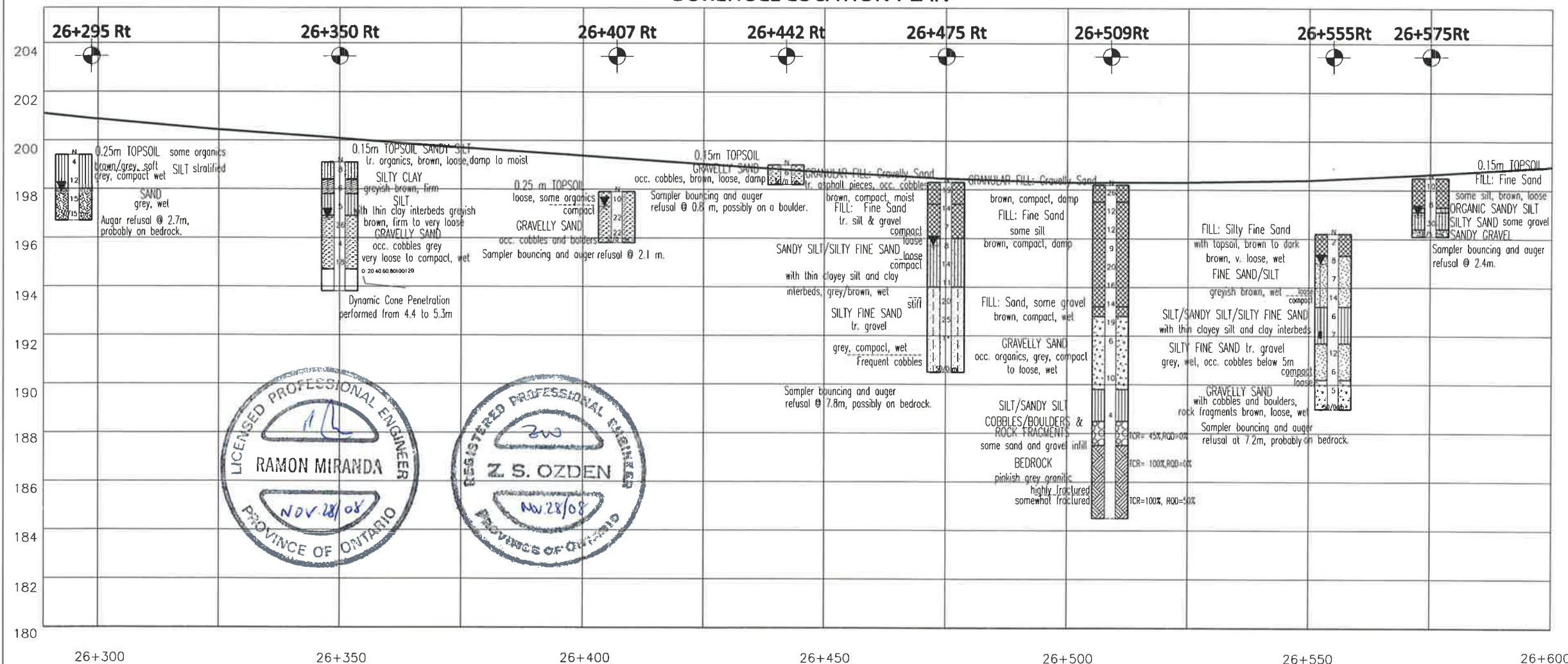
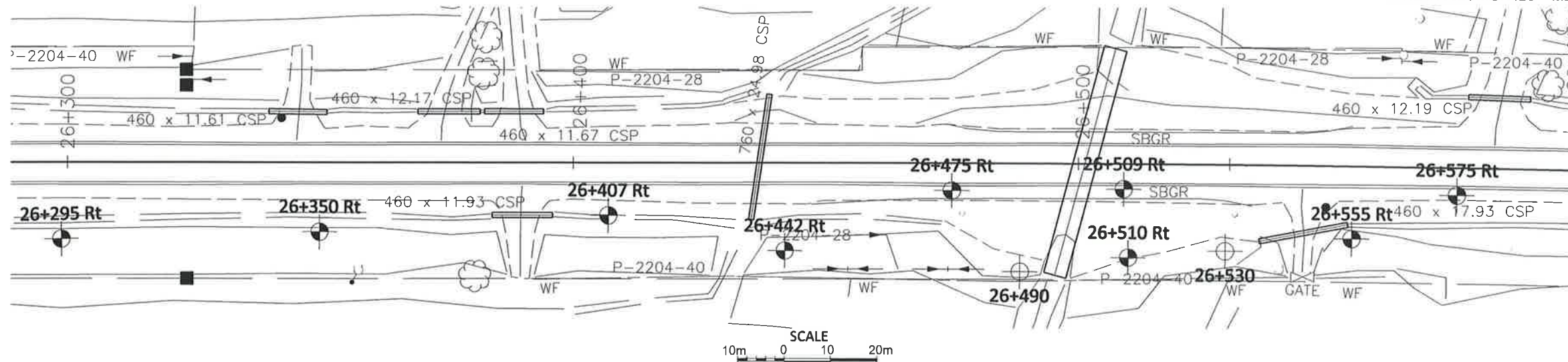
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Drawing

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

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



Appendix A

Record of Borehole Sheets

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+295Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta 26+295 15.2m Rt of C/L of Hwy 17 (D-1.5m) ORIGINATED BY GI
 DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
 DATUM Geodetic DATE 4/23/2008 CHECKED BY ZO

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa)					WATER CONTENT (%)							
199.4 0.0	GROUND SURFACE																	
	0.25 m TOPSOIL		1	SS	4	199												
	some organics brown/grey, soft																	
	grey, compact wet		2	SS	12													
198.0 1.4	SILT stratified						198											
	SAND grey, wet		3	SS	15	197												
	compact		4	SS	50/15 cm													
196.7 2.7	End of Borehole																	
	Auger refusal @ 2.7 m, probably on bedrock. Water level @ 1.5 m on completion (not stabilized)*, and hole caved @ 1.8 m																Auger Refusal	

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+350Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+350 : 13.7m Rt C/L of Hwy 17 (Ditch) (D-1 0m) ORIGINATED BY GI
 DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
 DATUM Geodetic DATE 4/23/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)								
199.1	GROUND SURFACE						20	40	60	80	100					
0.0	0.15 m TOPSOIL		1	SS	8	199 198 197 196 195 194	○ UNCONFINED	+ FIELD VANE								
198.4	Tr organics, brown, loose, damp to moist						● POCKET PENETR.	x LAB VANE								
0.7																
197.8	SILTY CLAY		2	SS	6											
1.3	greyish brown, firm															
196.9	SILT		3	SS	5											Atterberg Limits
2.2	with thin clay interbeds														Test: non-plastic	
	greyish brown, firm to very loose															
			4	SS	26										35 51 (14)	
	GRAVELLY SAND		5	SS	4											
	occ. cobbles															
	grey, very loose to compact, wet															
194.7			6	SS	18										soil back-up in	
4.4	End of Borehole														auger @ 4.3 m	
193.8	Dynamic Cone Penetration performed from 4.4 to 5.3 m															
5.3	End of Dynamic Cone Penetration Test @ 5.3 m															
	Water level @ 2.3 m (not stabilized)* and hole caved @ 2.3 m upon completion															

+ 3 X 3 : Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

METRIC

+³, ×³. Numbers refer to Sensitivity

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+442Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+442 17 3m Rt of C/L of Hwy 17 ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 4/23/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)					
199.0	GROUND SURFACE												

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+475Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+475 - 5.4m Rt C/L of Hwy 17 (Shoulder) (D-0 1m) ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 4/25/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
								20 40 60 80 100						
198.3 0.0	GROUND SURFACE													
	GRANULAR FILL: Gravelly Sand tr asphalt pieces, occ. cobbles brown, compact, moist		1	SS	19									
197.4 0.9	FILL: Fine Sand tr silt & gravel		2	SS	14									
	compact													
	loose		3	SS	7									
196.0 2.3	SANDY SILT/SILTY FINE SAND		4	SS	8									
	loose													
	compact		5	SS	14									
	with thin clayey silt and clay interbeds, grey/brown, wet													
	stiff		6	SS	11									
194.0 4.3			7	SS	20									
	SILTY FINE SAND tr gravel grey, compact, wet		8	SS	25									
	Frequent cobbles		9	SS	1*									
190.5 7.8	End of Borehole. Sampler bouncing and auger refusal @ 7.8 m, possibly on bedrock. Water level in open hole @ 2.6 m upon completion.		10	SS	50/0 cm									

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+490 (DCPT)

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+490 21.5m Rt of C/L of Hwy 17 ORIGINATED BY GH
DIST HWY 17 BOREHOLE TYPE DCPT COMPILED BY SS
DATUM Geodetic DATE 5/13/2008 CHECKED BY ZO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH (kPa)					
191.5	GROUND SURFACE						20 40 60 80 100	10 20 30					
0.0													
	Dynamic Cone Penetration Test (DCPT) performed from 0 to 1.4 m						191						DCPT hole moved 3 times due to shallow refusal.
190.1	End of DCPT						100/5cm						
1.4													

+³. X³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

SPT1211B : Highway 17 (Maitawa)

RECORD OF BOREHOLE No 26+509Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+509 5m Rt C/L of Hwy 17 (Shoulder) (D-0.2m) ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS
DATUM Geodetic DATE 4/24/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
198.2	GROUND SURFACE													
0.0	GRANULAR FILL: Gravelly Sand brown, compact, damp		1	SS	26		198							
197.5														
0.7	FILL: Fine Sand some silt brown, compact, damp		2	SS	12		197							0 92 (8)
			3	SS	12									
			4	SS	9		196							
			5	SS	20		195							
			6	SS	16		194							0 90 (10)
193.2			7	SS	14		193							
5.0	FILL: Sand, some gravel brown, compact, wet		8	SS	19									
192.8														
5.4	GRAVELLY SAND occ organics, grey, compact to loose, wet		9	SS	6		192							28 60 (12)
							191							
			10	SS	10		190							
189.8														
8.4	SILT/SANDY SILT with some thin silty fine sand and occ clayey silt and clay interbeds grey brown, very loose wet		11	SS	4		189							
188.5														
9.7	COBBLES/BOULDERS & ROCK FRAGMENTS some sand and gravel infill		12	RC	TCR= 43% RQD=0%		188							
187.5														
10.7	BEDROCK pinkish grey granitic		13	RC	TCR= 100% RQD=0%		187							
							186							
			14	RC	TCR=100% RQD=50%		185							
184.6														
13.7	End of Borehole													

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+510Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+510 : 18.5m R/C/L of Hwy 17 (Toe of Embankment) (D-4.5m) ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger & NQ Coring COMPILED BY SS
DATUM Geodetic DATE 4/27/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
193.8 0.0	GROUND SURFACE													
193.1 0.7	0.3 m TOPSOIL FILL: Sand tr gravel, brown, v. loose, wet		1	SS	2									
191.5 2.3	FILL: Silty Fine Sand tr gravel, asphalt pieces, organics grey to dark grey, loose to v. loose, damp to moist		2	SS	8									
191.2 2.6	SANDY ORGANIC SILT dark grey/blackish, v. loose, wet		3	SS	2									
190.1 3.7	SILTY FINE SAND tr gravel, tr organics dark grey, v. loose, wet		4	SS	4									
187.9 5.9	SILT/SANDY SILT/ SILTY FINE SAND/CLAYEY SILT reddish clay interbeds, grey, v. loose to v. soft, wet		5	SS	2									
186.5 7.3	SAND some gravel greyish brown, compact, wet		6	SS	3									
185.9 7.9	BOULDERS / SHATTERED ROCK		7	SS	4									
182.8 11.0	BEDROCK pinkish grey granitic		8	SS	4									
			9	SS	10									
			10	SS	50/0cm									
			11	RC										
			12	RC	TCR=100% RQD=58%									
			13	RC	TCR=100% RQD=65%									
	End of Borehole													
	Water level @ ground surface upon completion of coring (not stabilized)*													

+ 3 . X 3 : Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

Refusal to further
augering @ 7.5 m

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+530 (DCPT)

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta 26+530 17.4m Rt of C/L of Hwy 17 ORIGINATED BY Gi
DIST HWY 17 BOREHOLE TYPE DCPT COMPILED BY SS
DATUM Geodetic DATE 4/25/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH (kPa)						
194.8 0.0	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	10 20 30				
	Dynamic Cone Penetration Test (DCPT) performed from 0 to 8.3 m.							○ UNCONFINED + FIELD VANE ● POCKET PENETR. X LAB VANE						
							194							
							193							
							192							
							191							
							190							
							189							
							188							
							187							
186.5 8.3	End of DCPT.													

+³ × 3³ Numbers refer to
Sensitivity

20
15-5
10 (%) STRAIN AT FAILURE

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+555Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta 26+555 : 14.2m RI C/L of Hwy 17 (Ditch) (D-2.0m) ORIGINATED BY GI
DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
DATUM Geodetic DATE 4/26/2008 CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
196.2	GROUND SURFACE													
0.0	FILL: Silty Fine Sand with topsoil, brown to dark brown, v. loose, wet		1	SS	2		196							
195.3			2	SS	8		195							0 58 (42)
0.9	FINE SAND/SILT greyish brown, wet		3	SS	7		194							
	loose		4	SS	14		193							
	compact		5	SS	6		192							
193.2	SILT/SANDY SILT/SILTY FINE SAND with thin clayey silt and clay interbeds grey, loose, wet		6	SS	7		191							
3.0			7	SS	12		190							
191.7	SILTY FINE SAND tr. gravel grey, wet, occ. cobbles below 5 m		8	SS	6		189							
4.5	compact		9	SS	5									
	loose		10	SS	60/60									
190.2	GRAVELLY SAND with cobbles and boulders, rock fragments brown, loose, wet													
6.0														
189.0	End of Borehole Sampler bouncing and auger refusal at 7.2 m, probably on bedrock Water level in open hole and hole caved in @ 3.3 m. Piezometer installed to 4.3 m. Water level in Piezometer @ 0.5 m (El. 195.7 m) on April 27/08.													Auger Refusal, No Recovery
7.2														

+³, X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

SPT1211B : Highway 17 (Mattawa)

RECORD OF BOREHOLE No 26+575Rt

1 OF 1

METRIC

GWP 173-98-00 LOCATION Sta. 26+575 : 5.3m Rt of C/L of Hwy 17 ORIGINATED BY GI
 DIST HWY 17 BOREHOLE TYPE Hollow Stem Auger COMPILED BY SS
 DATUM Geodetic DATE CHECKED BY ZO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
198.5	GROUND SURFACE													
0.0	0.15 m TOPSOIL FILL: Fine Sand some silt, brown, loose, damp		1	SS	10		198							
197.3			2	SS	8									
1.2	ORGANIC SANDY SILT, damp													
197.1							197							
1.4	SILTY SAND some gravel grey/brown, compact, moist		3	SS	30									
196.4														
2.1	SANDY GRAVEL, brown, wet		4	SS	60/5.00									
196.1														
2.4	End of Borehole. Sampler bouncing and auger refusal @ 2.4 m. Water level @ 1.5 m (not stabilized)* and hole caved in @ 2.0 m upon completion.													

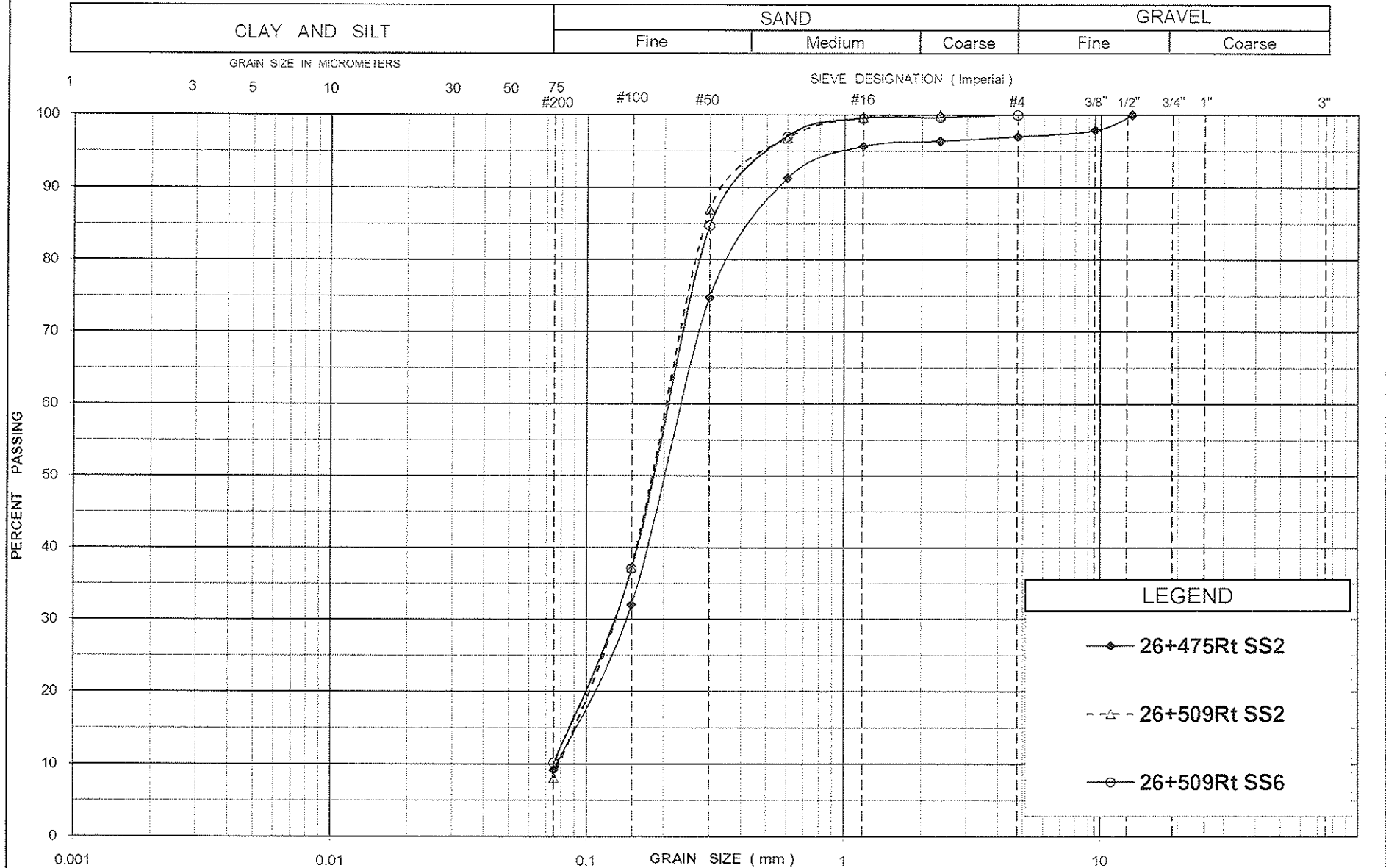
+ 3, X 3. Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

Appendix B

Laboratory Test Results

UNIFIED SOIL CLASSIFICATION SYSTEM

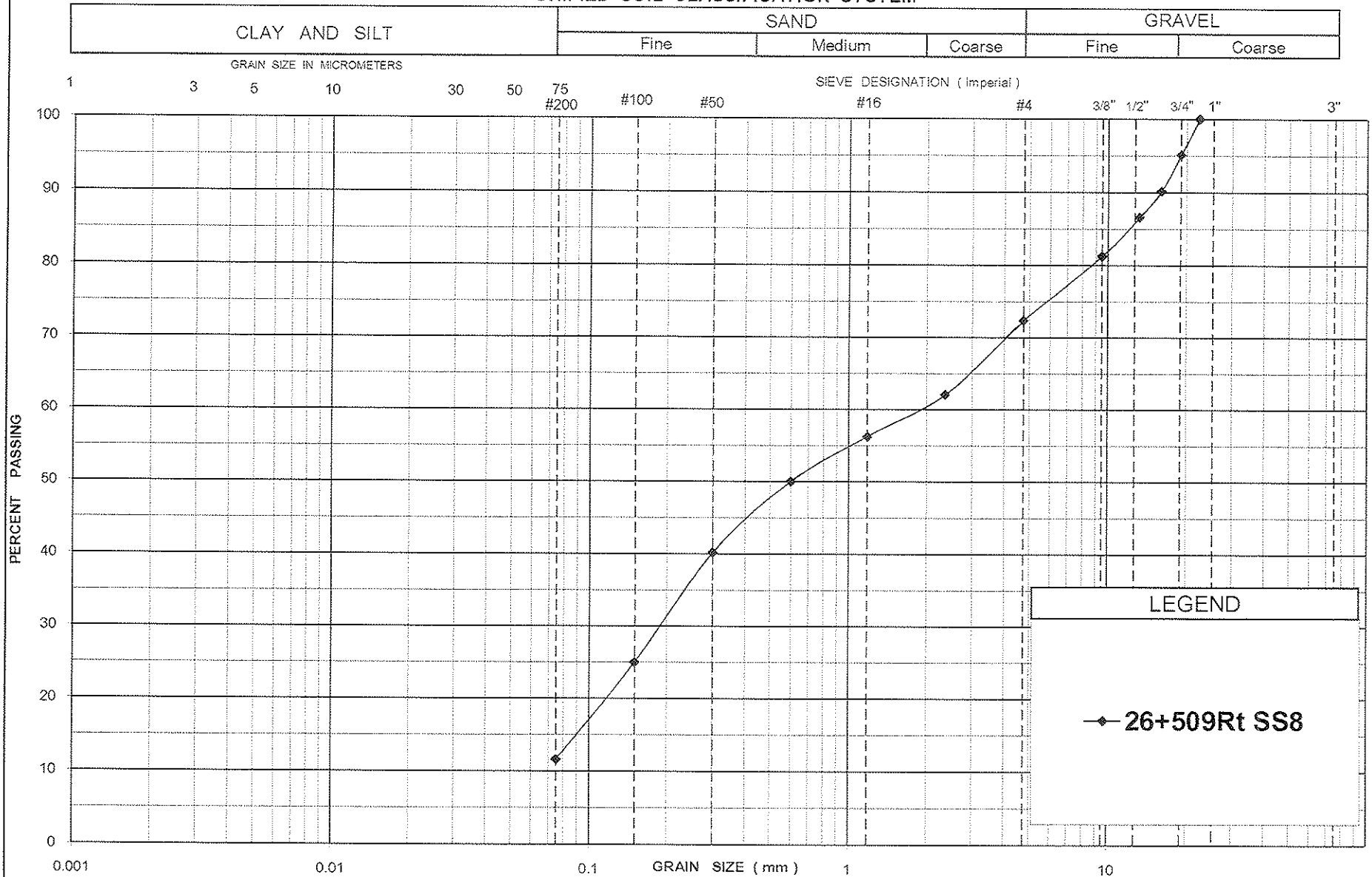


SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
EMBANKMENT FILL: Fine Sand, trace silt

FIGURE No. B-1
REF. No. SPT 1211B
DATE AUGUST 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

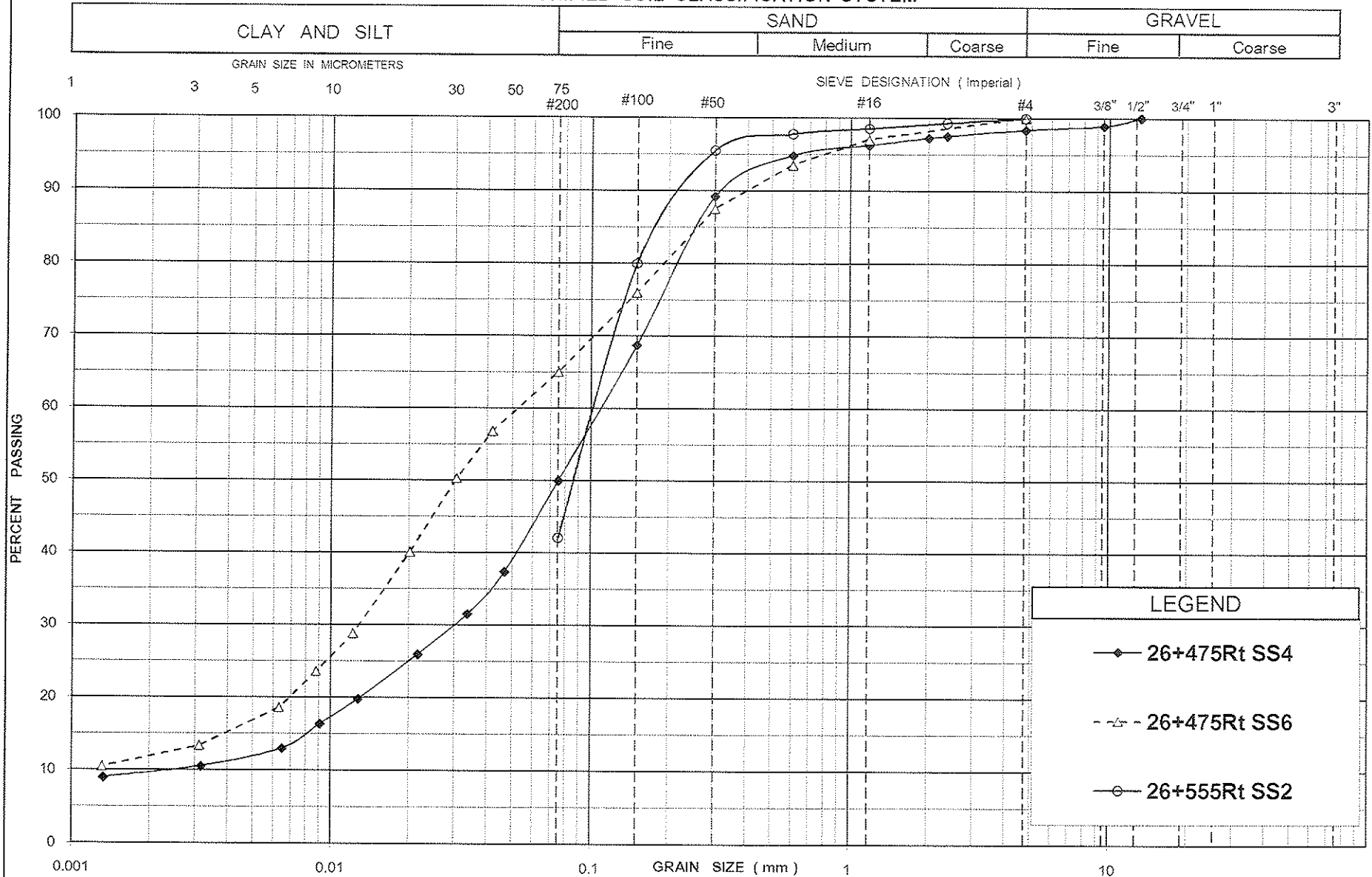
GRAIN SIZE DISTRIBUTION
GRAVELLY SAND, trace silt

FIGURE No. B-2

REF. No. SPT 1211B

DATE AUGUST 2008

UNIFIED SOIL CLASSIFICATION SYSTEM



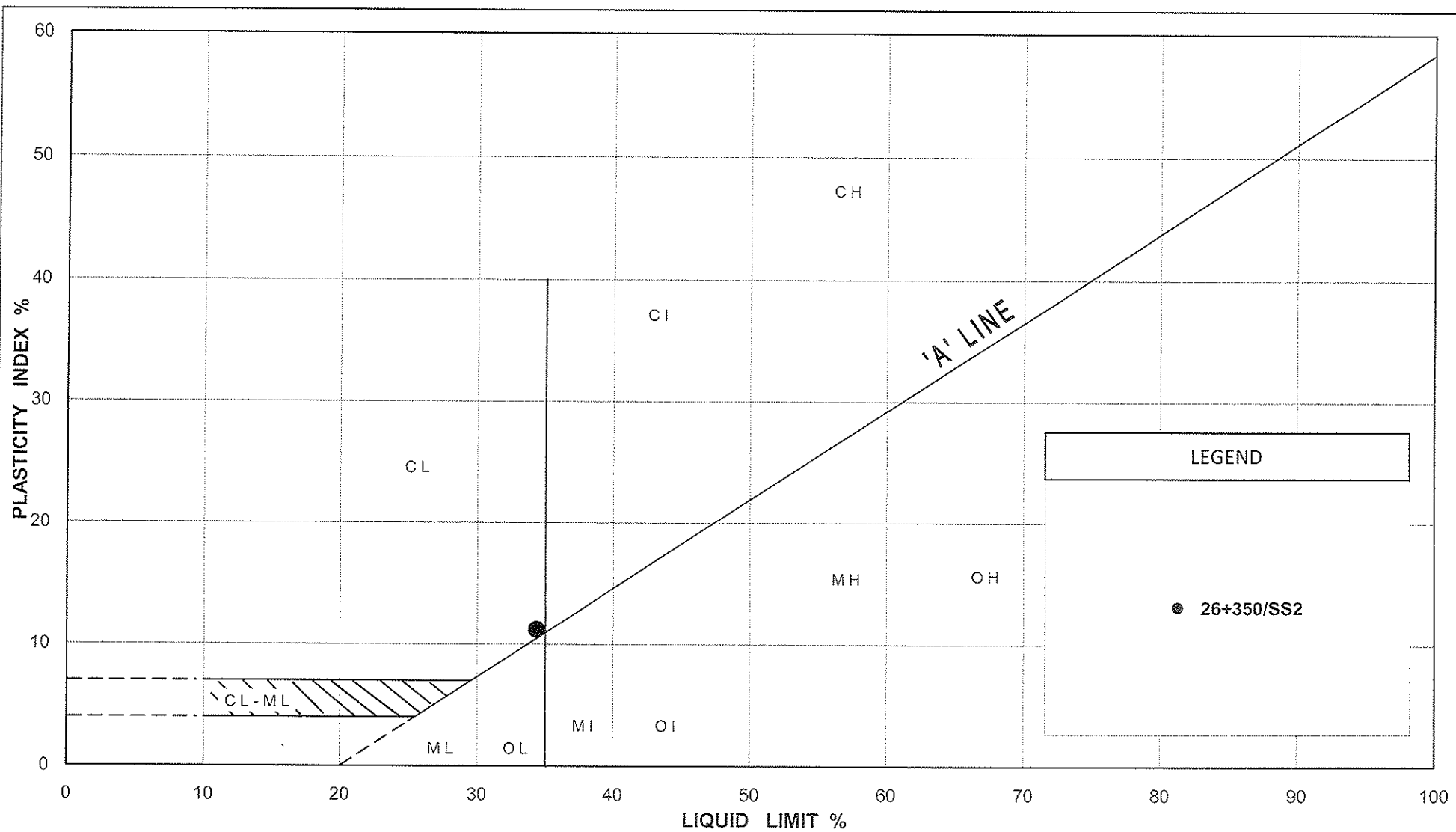
SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
LAYERED SILTY FINE SAND / SILT / CLAYEY SILT

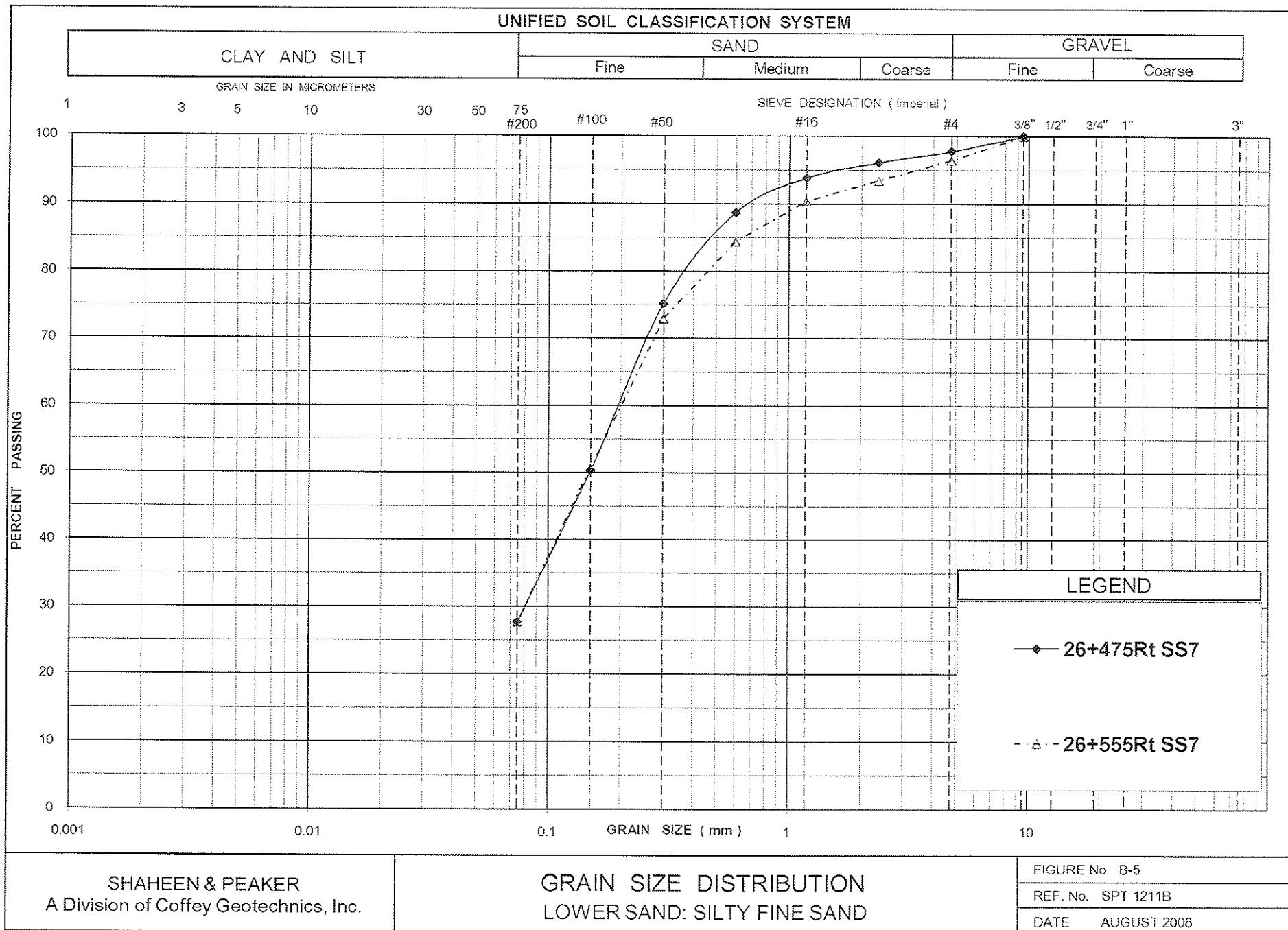
FIGURE No. B-3

REF. No. SPT 1211B

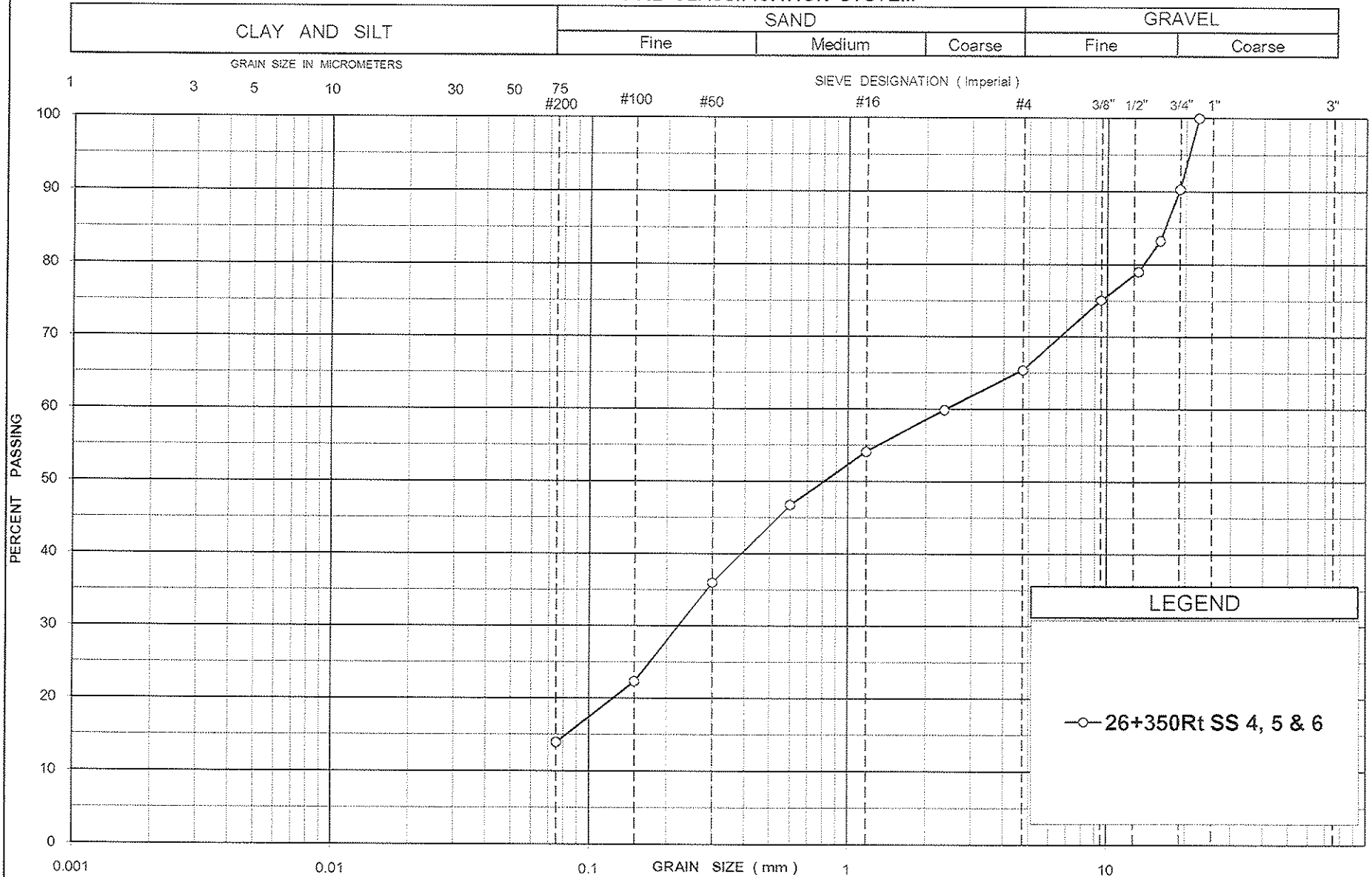
DATE AUGUST 2008



SHAHEEN & PEAKER	PLASTICITY CHART SILTY CLAY	FIGURE No. B-4
A Division of Coffey Geotechnics, Inc.		REF. No. SPT 1211B
		DATE AUGUST 2008



UNIFIED SOIL CLASSIFICATION SYSTEM



SHAHEEN & PEAKER
A Division of Coffey Geotechnics, Inc.

GRAIN SIZE DISTRIBUTION
LOWER GRAVELLY SAND

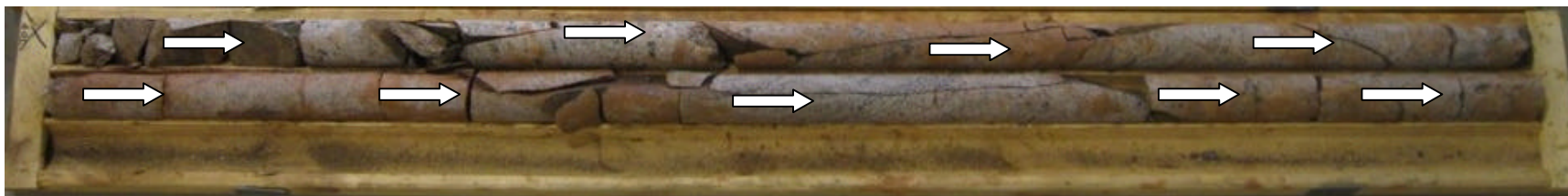
FIGURE No. B-6

REF. No. SPT 1211B

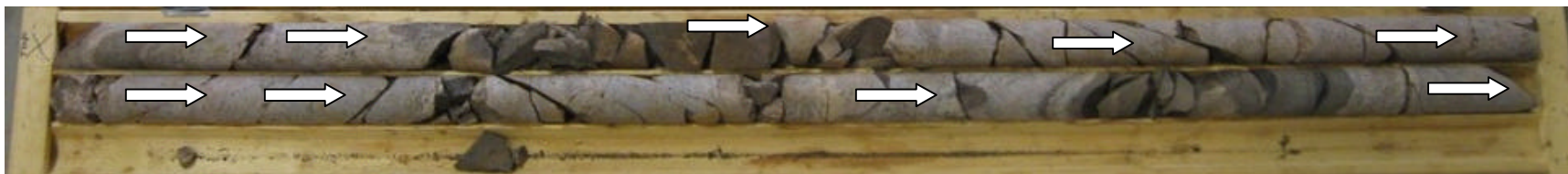
DATE AUGUST 2008

Appendix C

Rock Core Photographs



STA 26+509 Rt



STA 26+510 Rt

Appendix D

Site Photographs



Photograph 1. South side of Highway 17 at east side of Resmer Creek (looking east)



Photograph 2. North side of Highway 17 at STA.26+500 (looking West)



Photograph 3. South side of embankment at Resmer Creek (STA.26+505)



Photograph 4. South side of Highway 17 at Resmer Creek (Looking east)



Photograph 5. Surficial erosion at Resmer Creek Culvert location

Appendix E

Explanation of Terms Used in Report

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINT AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

**FOUNDATION DESIGN REPORT
PROPOSED WIDENING OF HIGHWAY 17 FROM
STATION 26+295 TO 26+575
HIGHWAY 17, FROM 9.5 KM EAST OF HIGHWAY 533
EASTERLY 14.9 KM
MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040
GEOCRES NO. 31L-125**

Prepared For:

D. M. WILLS ASSOCIATES

Prepared by:

**SHAHEEN & PEAKER
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**Project: SPT1211B
November 28, 2008**



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Table of Contents

5. DISCUSSION AND RECOMMENDATIONS	11
5.1 Embankment Stability	11
5.2 Settlement	13
5.3 Construction	13
6. CLOSURE	15

APPENDIX F: TYPICAL CROSS SECTIONS

APPENDIX G: SLOPE STABILITY ANALYSES RESULTS

APPENDIX H: OPSD

APPENDIX I: LIMITATIONS OF REPORT

**FOUNDATION DESIGN REPORT
PROPOSED WIDENING OF HIGHWAY 17
FROM STATION 26+295 TO 26+575, HIGHWAY 17,
FROM 9.5 KM EAST OF HIGHWAY 533, EASTERLY 14.9 KM
MATTAWA, ONTARIO
G.W.P. 173-98-00; AGREEMENT NO. 5006-E-0040**

5. DISCUSSION AND RECOMMENDATIONS

A new eastbound truck passing lane will be constructed on the right side of Highway 17 (i.e. on the south side) between Stations 25+100 and 27+500. Relatively high fill widening is proposed between about Stations 26+295 and 26+575 within this section of Highway 17.

Typical cross sections of the proposed widening, supplied to us by D. M. Wills Associates Limited, are given in Appendix F. These show that the existing side slopes are typically 2H:1V or slightly flatter and the slopes after widening will be similar. The highway grade will remain unchanged but the widening itself will impose loads of about 1.0 to 2.0 m of new fill due to widening. The embankments in this stretch are typically 1 to 6 m high.

5.1 EMBANKMENT STABILITY

As shown on the cross section drawings presented in Appendix F, the widening will typically mirror the existing embankment slopes. We also understand that there are no known failures along this stretch of the highway nor are any signs of impending instability.

We carried out slope stability analyses using the cross-sections provided to us. The stations were selected where, based on the borehole findings, the overburden is relatively thicker and where the embankment is relatively higher above the o.g. levels. These stations are as follows:

Station 26+479	based on Borehole 26+475
Station 26+509	based on Boreholes 26+509 and 26+510
Station 26+558	based on Borehole 26+555

The results of the stability analyses are given in Appendix G.

The results show that at Station 26+558, the factor of safety is about 1.33 for the existing conditions and increases to about 1.50 when widening is implemented using suitable earth fill (e.g. SSM or superior material) for the widening. These results are considered acceptable. Therefore, 2H:1V side slopes using select earth material for the widening, as planned, would be suitable at this station, provided that all unsuitable soils are properly stripped during construction under the foot-print of the embankments

At Stations 26+479 and 26+509 (i.e. the Resmer Creek culvert area), the calculated safety factors after the widening with 2H:1V side slopes, which are proposed, are marginal (i.e. about 1.30 or slightly lower), which is somewhat low. In this area the existing side slopes generally range between 2.1 - 3.0H:1V (see Appendix F). To increase the factor of safety to reasonably acceptable levels (i.e. to reduce the risk of a possible failure), the widened slopes can be flattened to 2 1/2H:1V from the presently proposed 2H:1V using suitable earth fill for the widening or alternatively, a 2H:1V slope can be maintained, as proposed, but using rock fill. The latter (i.e. rock fill use) is probably a more acceptable solution, since additional property may not be available for further flattening the slopes. As well, the existing culvert may not be long enough to accommodate 2 1/2H:1V side slopes. Here are some comments and recommendations for the use of rock fill.

It is recommended that rock fill widening be implemented between Station 26+470 and Station 26+540. The balance (i.e. widening between Station 26+295 and 26+470 and between Station 26+540 and 26+575) can consist of select earth fill (e.g. SSM). However, at each end a transition section from earth to rock fill may be necessary (e.g. a transition zone from earth to fill rock fill between Stations 26+460 and 26+470 on the west side and from Station 26+540 to Station 26+545 on the east side). All side slopes would be constructed at 2H:1V (i.e. both earth fill and rock fill).

Rock fill, when placed by end dumping, would typically stand at an angle of repose of 1 1/4H:1V. In addition, end dumping may be detrimental to the integrity of the culvert in the vicinity of the existing culvert. We therefore recommend that the contractor should be made aware that normal end dumping methods may not be applicable for this project (i.e. consideration may be given to an NSSP to point out this aspect).

The recommended maximum rock size is 600 mm (i.e. the rock used to widen the embankment fill should consist of 600 mm minus size rock fill).

Proper chinking would be required on top of the rock fill in the widened section to prevent the loss of granular soils (to be placed on top of the rock fill) into the voids in the rock fill.

It should also be pointed out that in our analyses, we assumed that all the organic and otherwise unsuitable soils will be removed (as per normal MTO practice) under the footprint of the widening, as well as near the toe of the existing embankment (if such soils are found). A further discussion of stripping is given in Section 5.3.

If some risk (i.e. factor of safety of 1.3 or slightly lower) is acceptable, then the material for embankment widening could be similar to that used in the existing embankment. It should however be ensured that there are no organic soils were left in place under the foot-print of the existing embankment near the toe of the embankment and that the material used for the widening will be properly compacted.

5.2 SETTLEMENT

Based on the borehole data and the proposed widening (shown in Appendix F), the maximum calculated settlement is about 80 mm. This settlement was calculated at Station 26+510 where the existing fill is highest (i.e. at Resmer Creek culvert location). At Station 26+559, the settlement was estimated to be about 60 mm. The anticipated settlements are less than 60 mm at other locations along this stretch. It should be pointed out that, similar to the slope stability analyses, when carrying out these calculations it was assumed that all organic soils under the footprint of the existing embankment were removed when the highway was first built and that all organic and otherwise unsuitable soils will be removed under the widened section as per normal MTO practice, before placing the new fill.

It is anticipated that about two-thirds of the calculated settlements will take place within a period of about two months. It is therefore recommended that if possible the paving of the highway after the construction be delayed for about six to eight weeks after the completion of the widened section to its full height.

Consideration can also be given, if and where feasible, a surcharge of about 0.5 m during this six to eight weeks.

5.3 CONSTRUCTION

All organic and otherwise unsuitable soils must be removed and replaced with properly compacted suitable soils. This must be carried out as per MTO practice within the footprint of the embankment. At the time of our investigation, the groundwater table was very close to the o.g. levels. Depending on the conditions at the time of the construction, the stripping will likely extend to below the groundwater level. It is therefore recommended that a suitable granular fill be used for backfilling below the o.g. levels and to about 0.4 m above the o.g. level.

The following stripping depths can be used for preliminary estimating purposes, at the borehole locations. It should however be pointed out that the thickness of unsuitable soils frequently varies in between and beyond borehole locations, especially in depressed areas and near water courses (e.g. Resmer Creek valley).

Table 5.2.1
Anticipated Stripping Depths for Preliminary Estimating Purposes

Borehole No.	Estimated Stripping Depth (m)	Type of Soil
26+295 Rt	0.25	Topsoil
26+350 Rt	0.15	Topsoil
26+407 Rt	0.25	Topsoil
26+442 Rt	0.15	Topsoil

Borehole No.	Estimated Stripping Depth (m)	Type of Soil
26+475 Rt	n/a*	n/a*
26+509 Rt	n/a*	n/a*
26+510 Rt	2.6	0-0.3 m Topsoil 0.3-0.7 m Sand Fill 0.7-2.3 m Random Fill 2.3-2.6 m Sandy Organic Silt
26+555 Rt	0.9	0.9 m Silty Sand Fill with topsoil
26+575 Rt	1.4	0-1.2 m Fill 1.2-1.4 m Sandy Organic Silt

*Borehole drilled from the top of embankment. Organic soils appear to have been properly stripped from beneath the embankment prior to its construction but will likely be present beyond the toe of the embankments. As well, they may also be present under the embankment near the toe.

Excavations and backfilling will need to be carried out in short sections to prevent instability of the existing embankments. The process of excavation and backfilling of each sufficiently narrow section (e.g. 3 to 4 m wide sections) should be carried out concurrently under geotechnical supervision. After stripping, the exposed subgrade should be inspected and approved by an experienced geotechnical engineer appointed by QEV. The first lift of the backfill may be thicker than standard backfill thickness of 0.3 m (e.g. possibly up to about 0.6 m thick), depending on the site conditions at the time of construction. For rockfill the first lift can be up to 1.0 m thick. As well, it is anticipated that some dewatering consisting of gravity drainage by means of ditches and pumping from filtered sumps will likely be required to facilitate the construction. It is also recommended that during the construction test pits be dug to determine if organic soils are present under the toe portion of the existing embankment, which need to be removed and replaced. All of this work should be carried out under the direction and supervision of the QEV. We recommend that a NSSP be issued for this purpose.

The face of the existing slope should be properly prepared for the widening, including benching as per MTO procedures in accordance with OPSD 208.010, as shown in Appendix H.

The fill used for the widening should consist of suitable materials which should be placed and compacted as per MTO standards. In as much as possible, within the upper 2 m, the fill should match the existing materials for the purpose of minimizing differential frost heave. The fill must be placed in sufficiently thin lifts and compacted as per MTO procedures. Typically, fill lift thicknesses (before compaction) are 0.3 m for earth fill and 1.0 m for rock fill.


Proper erosion control measures should be implemented on the face of the newly constructed earth slopes, both during the construction and permanently. This can be achieved by prompt seed and cover (OPSS 572) or sodding (OPSS 571).

6. CLOSURE

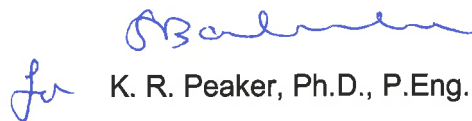
The Limitations of Report, as quoted in Appendix I, are an integral part of this report.

SHAHEEN & PEAKER LIMITED


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Zuhtu S. Ozden, P.Eng.

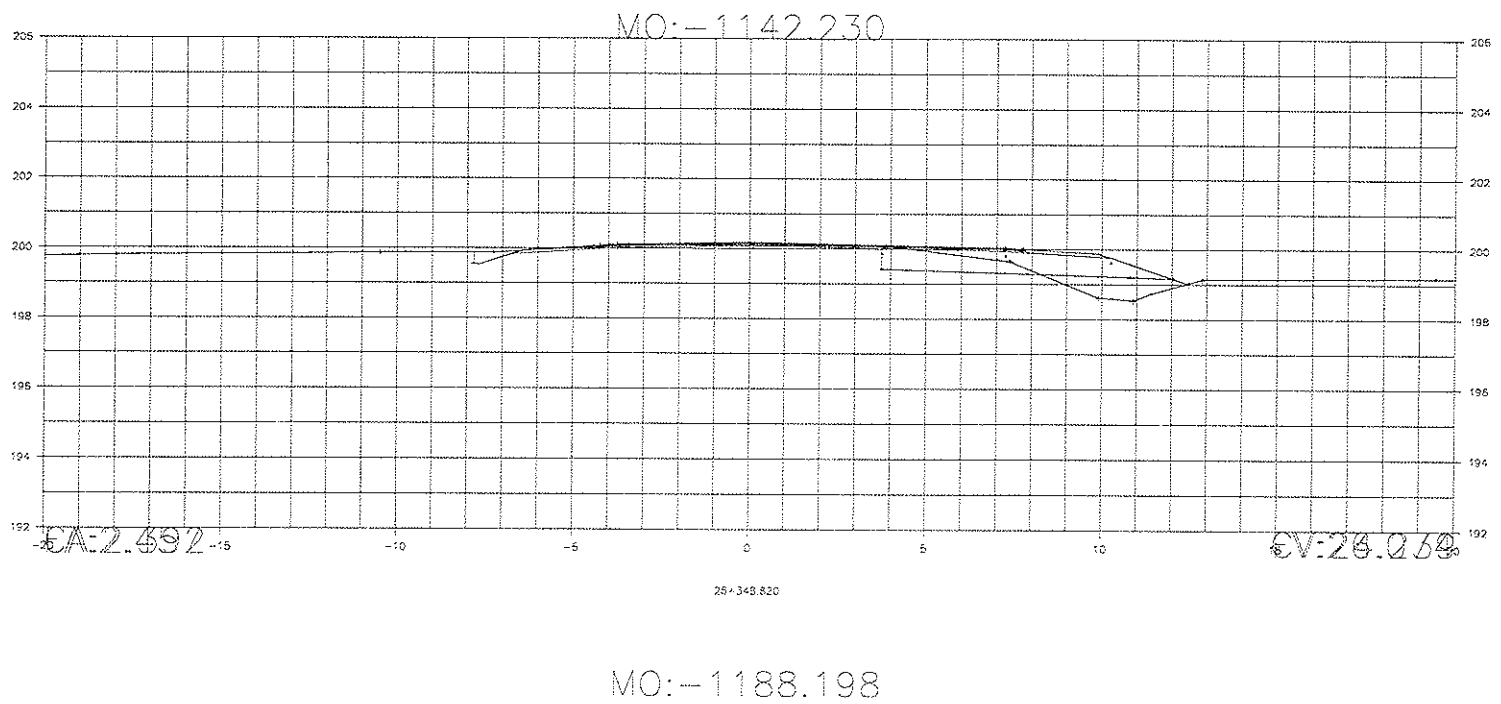
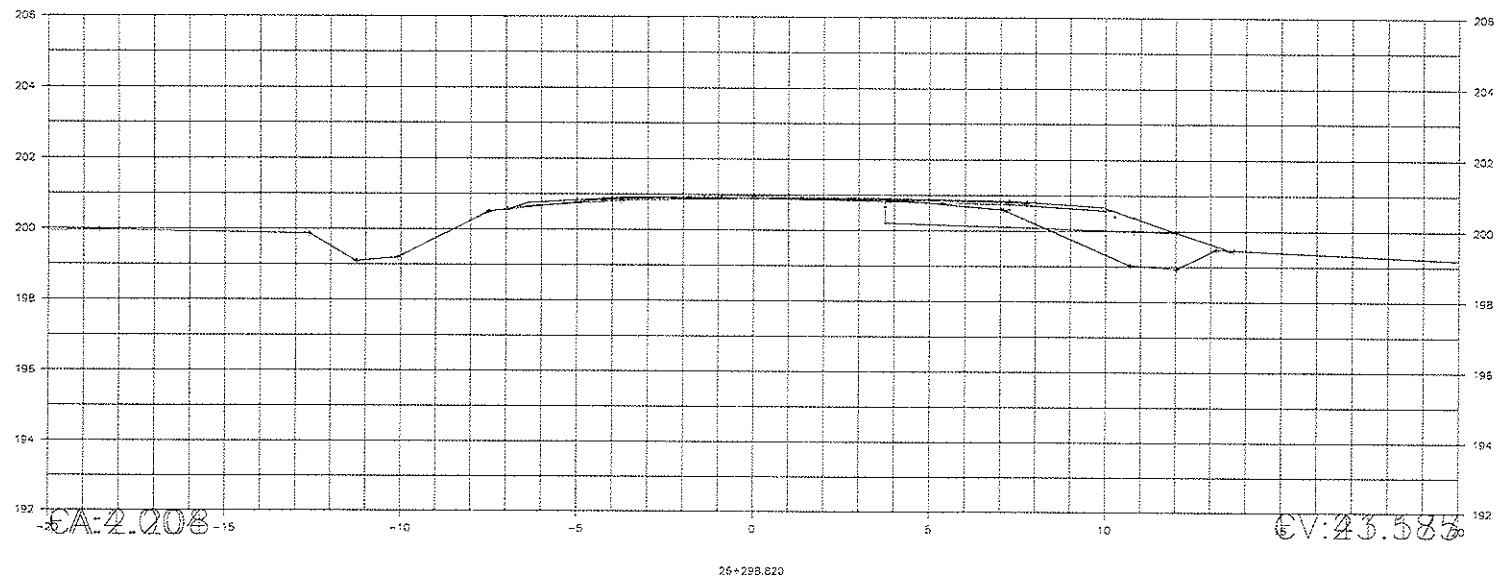
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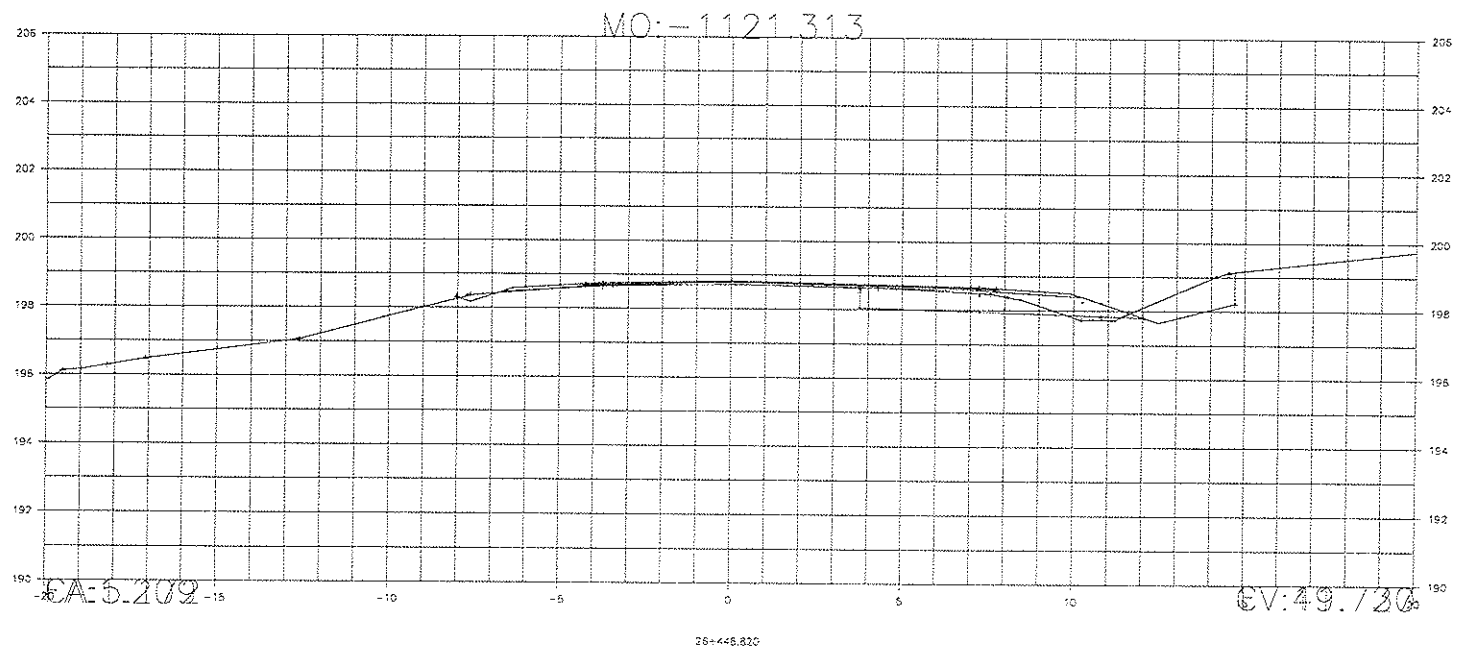
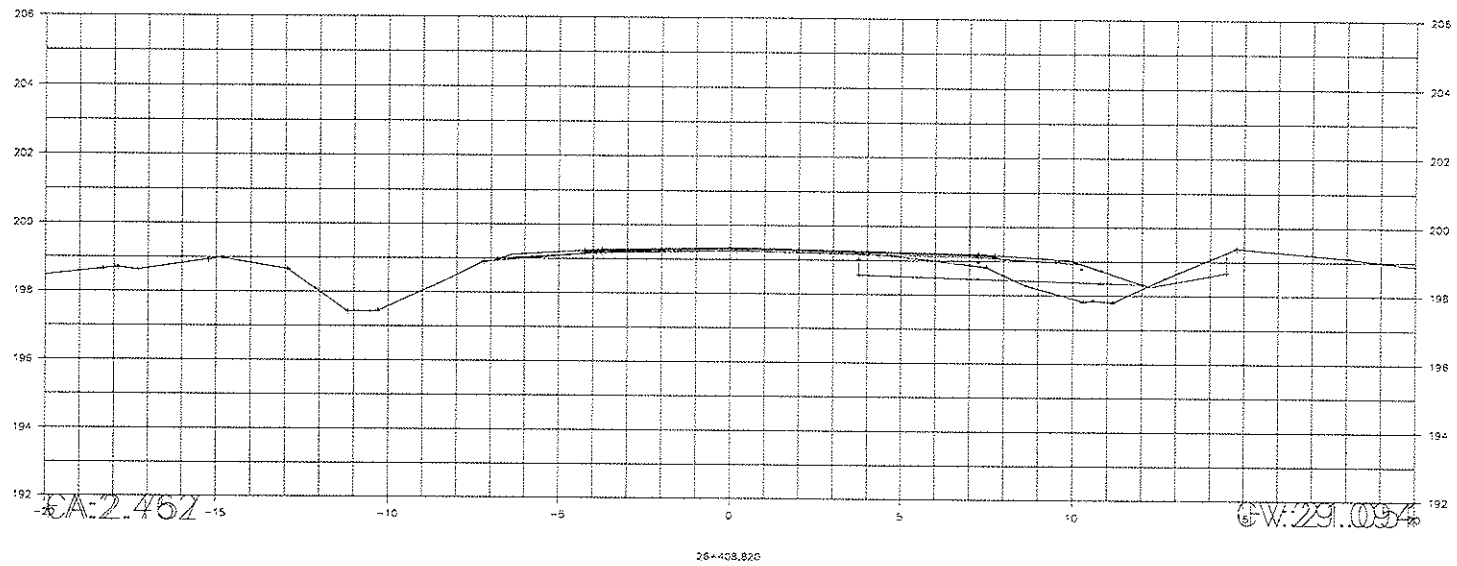

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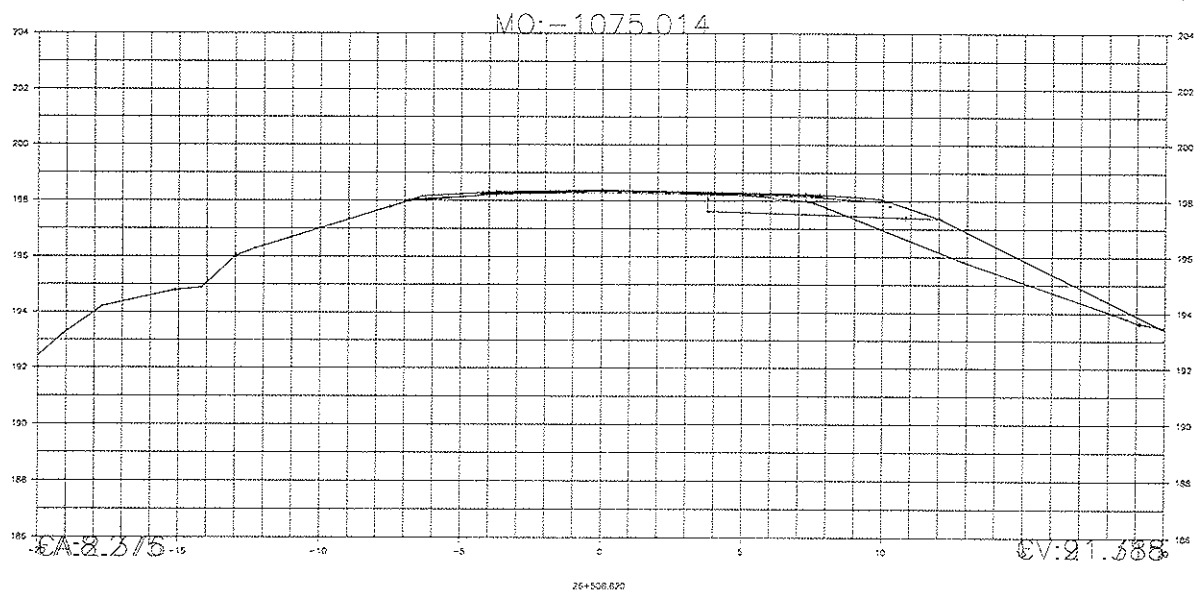
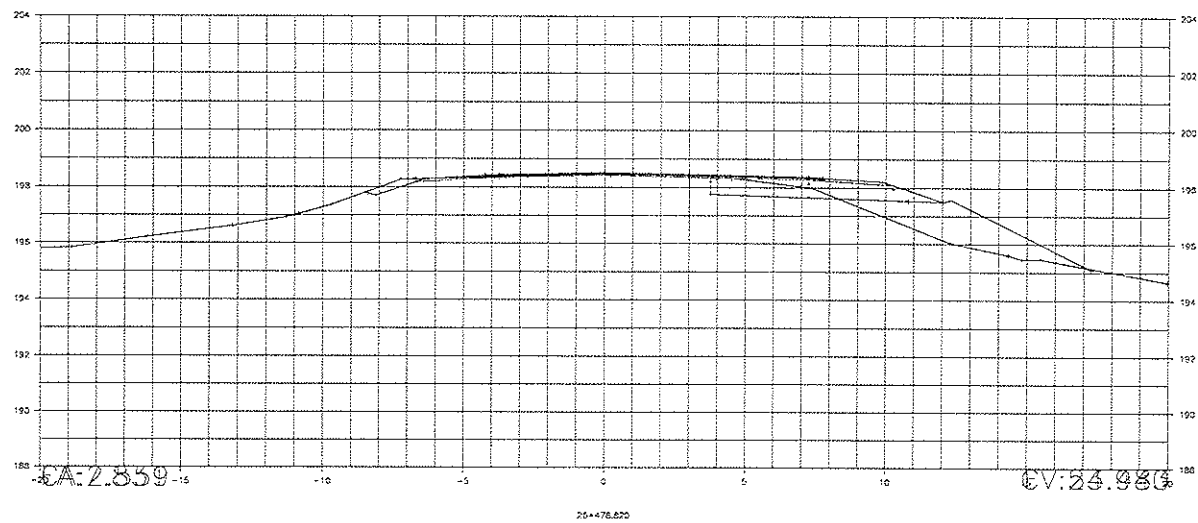
Appendix F

Typical Cross Sections

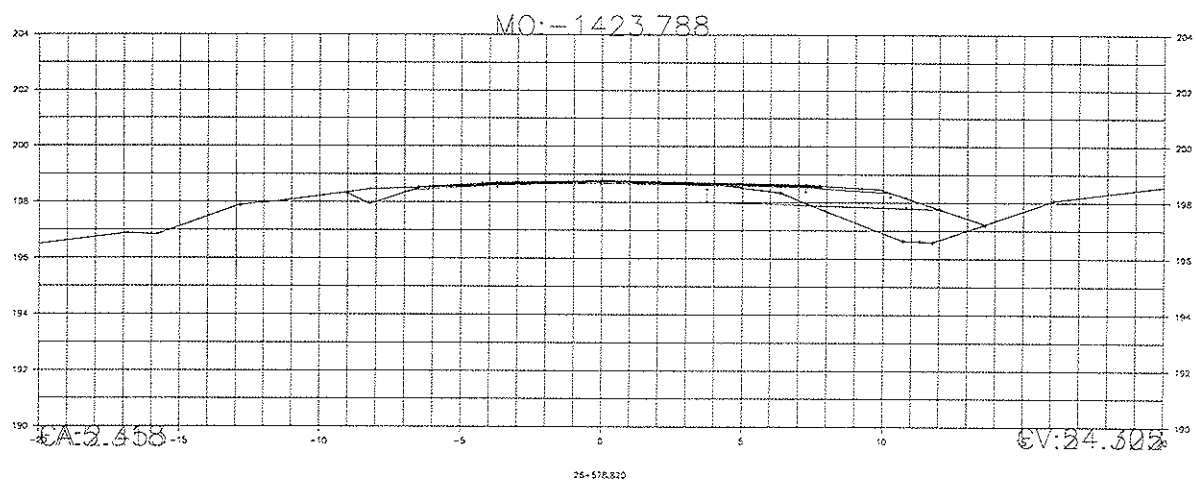
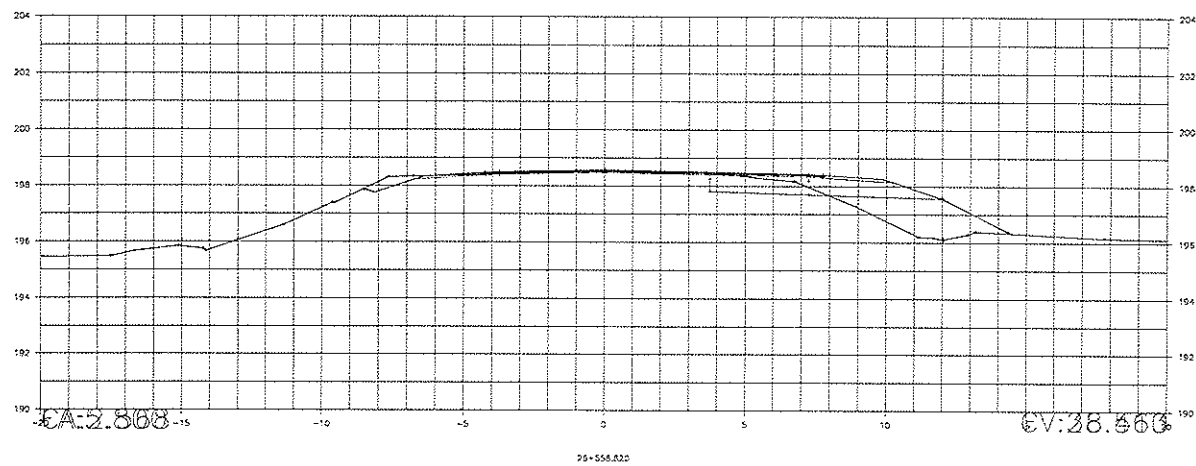




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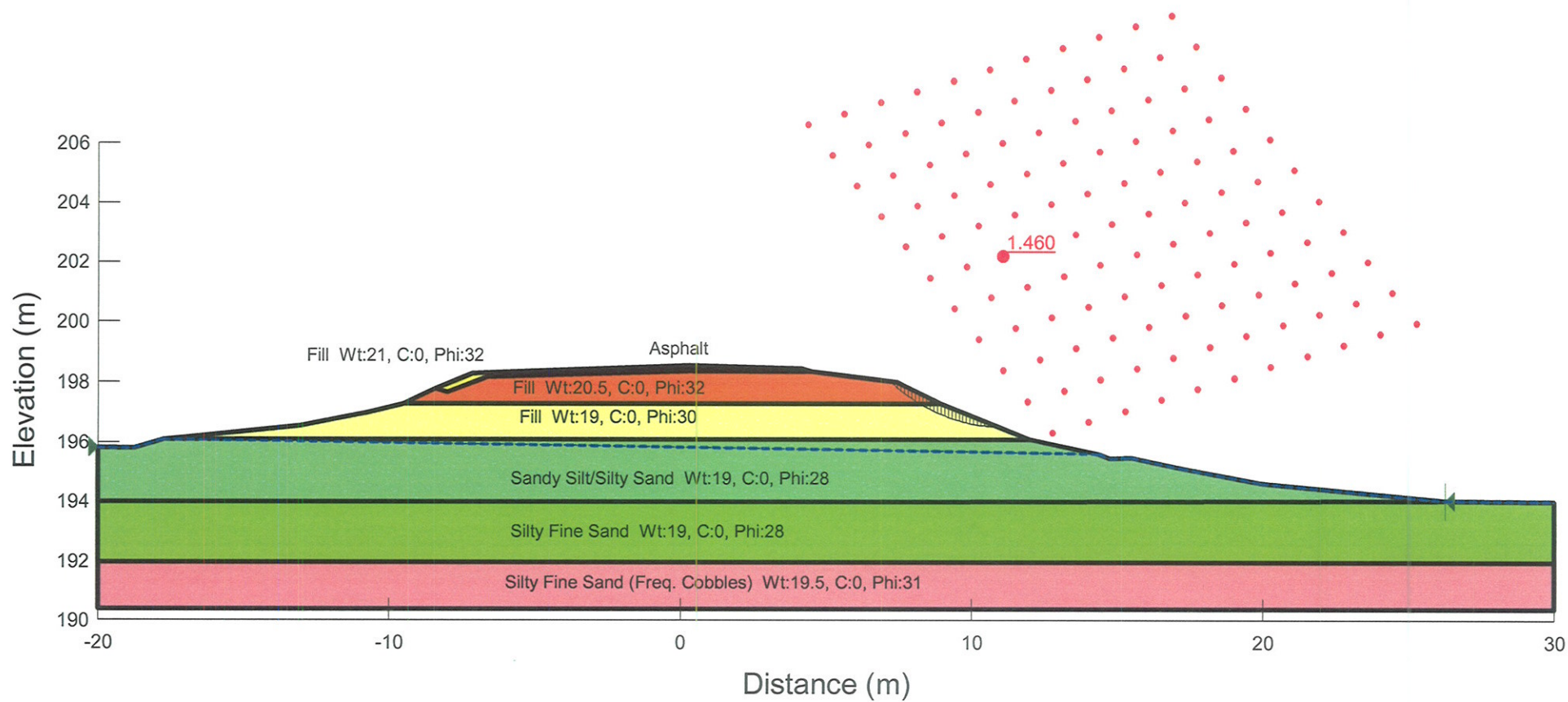
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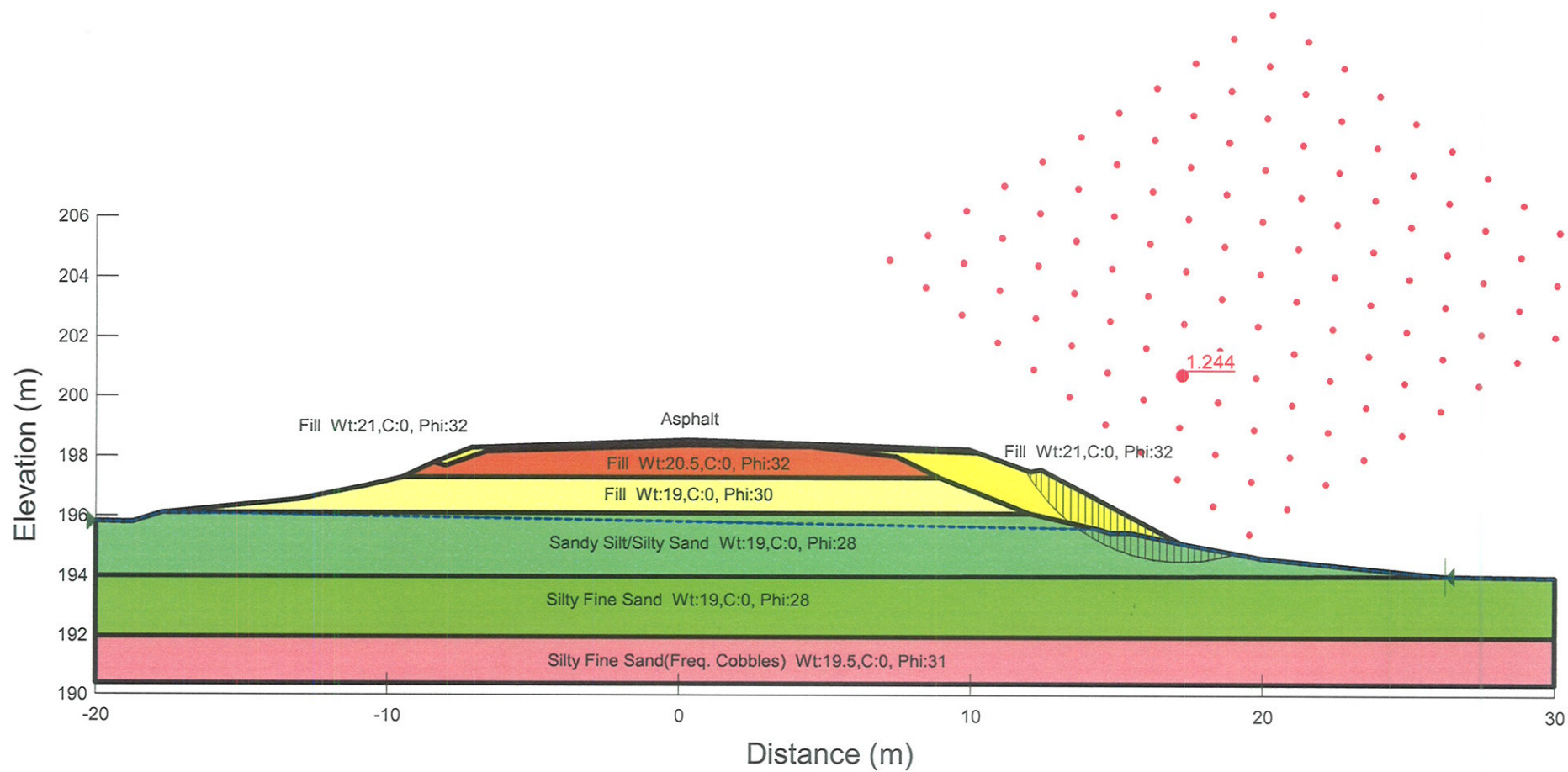
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Appendix G

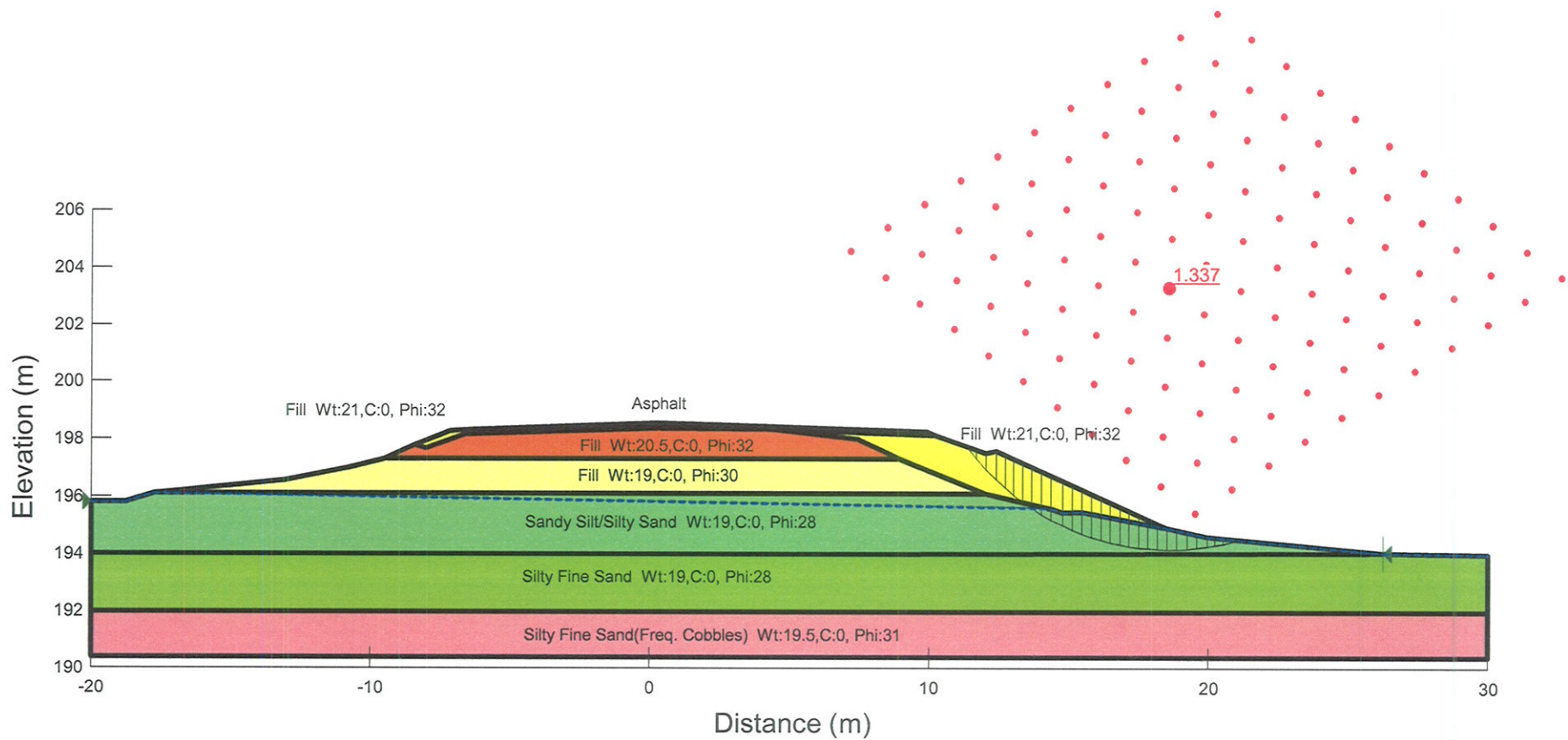
Slope Stability Analyses Results



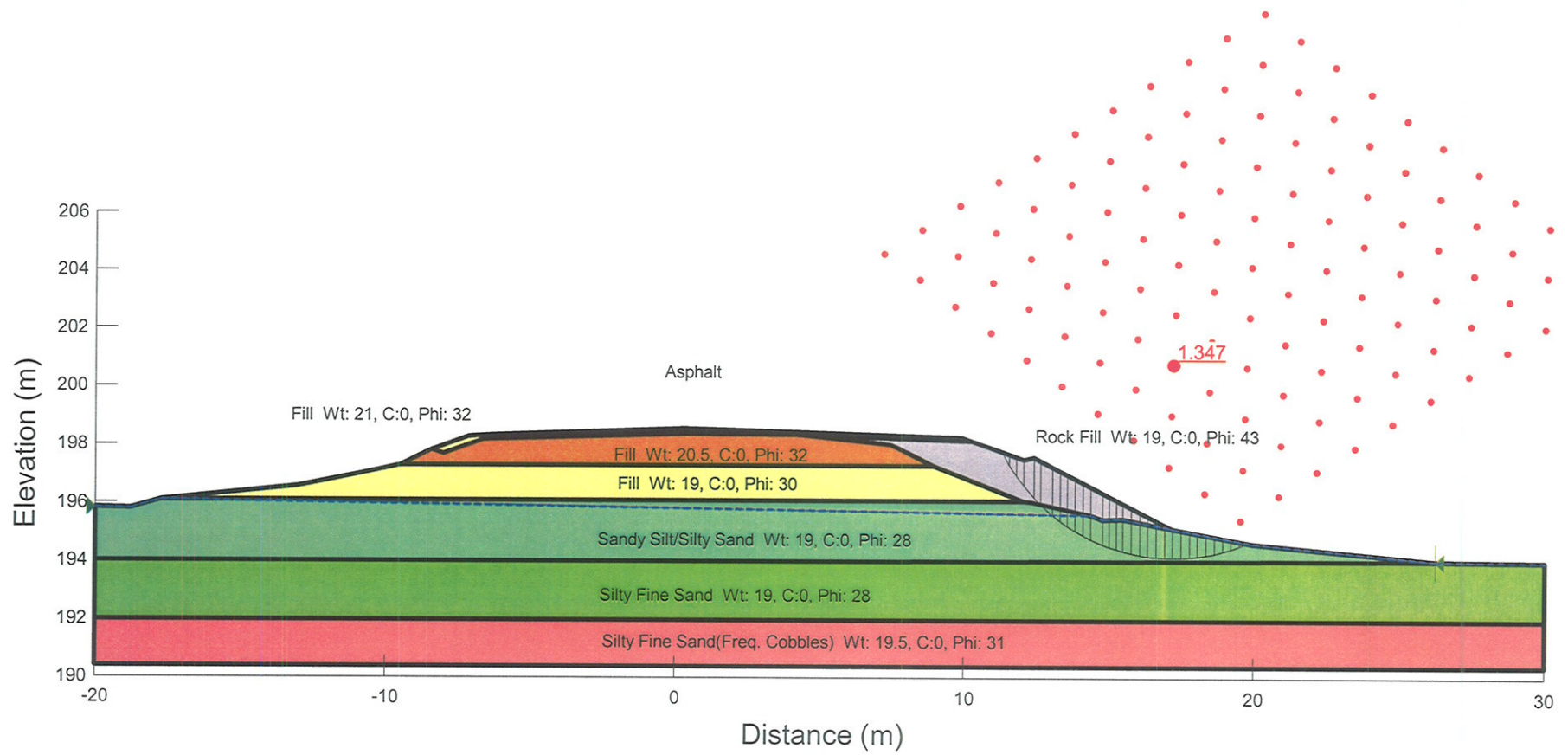
STA 26+479 Existing Slope



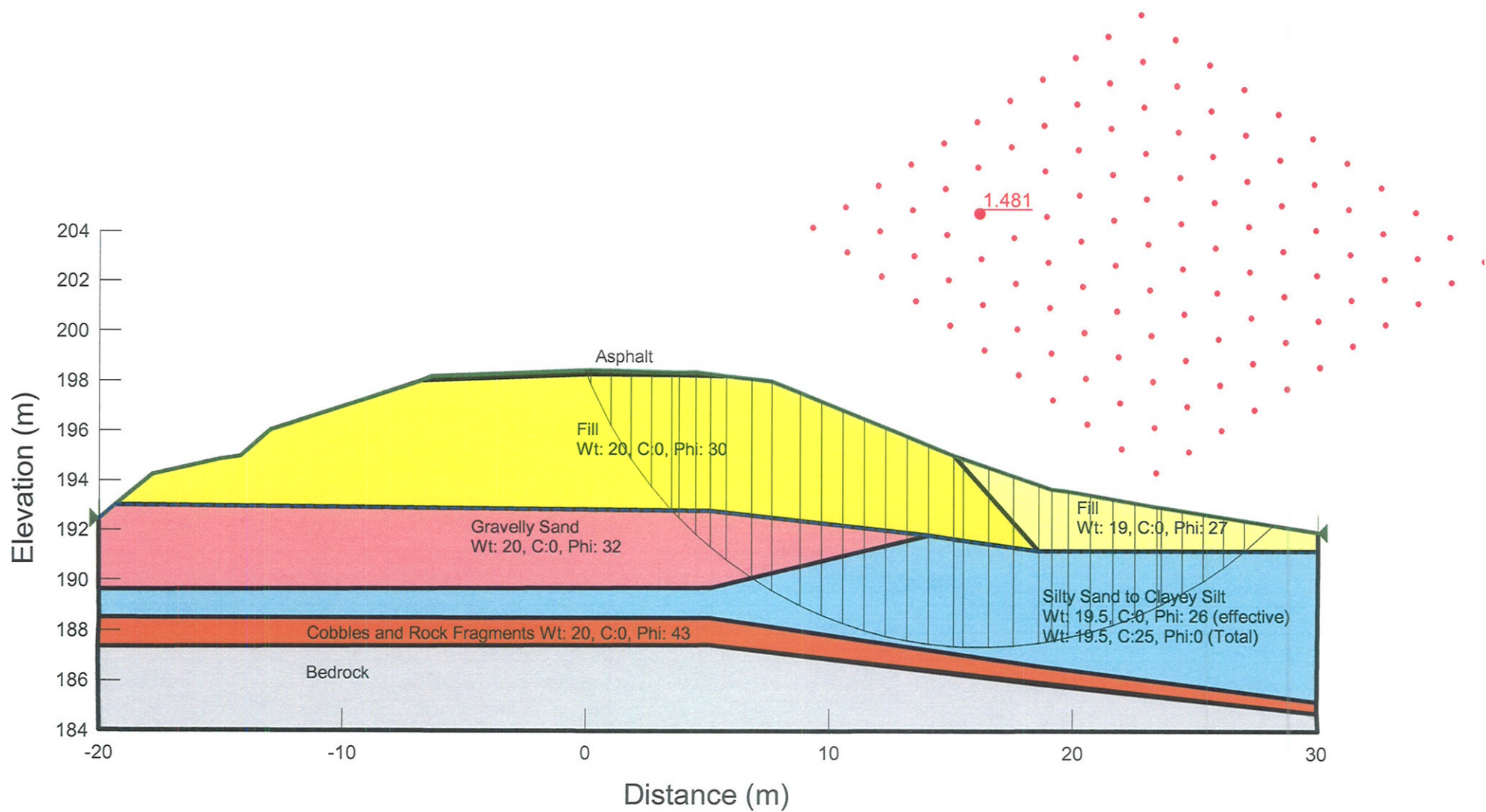
STA 26+479 After widening (with selected fill)



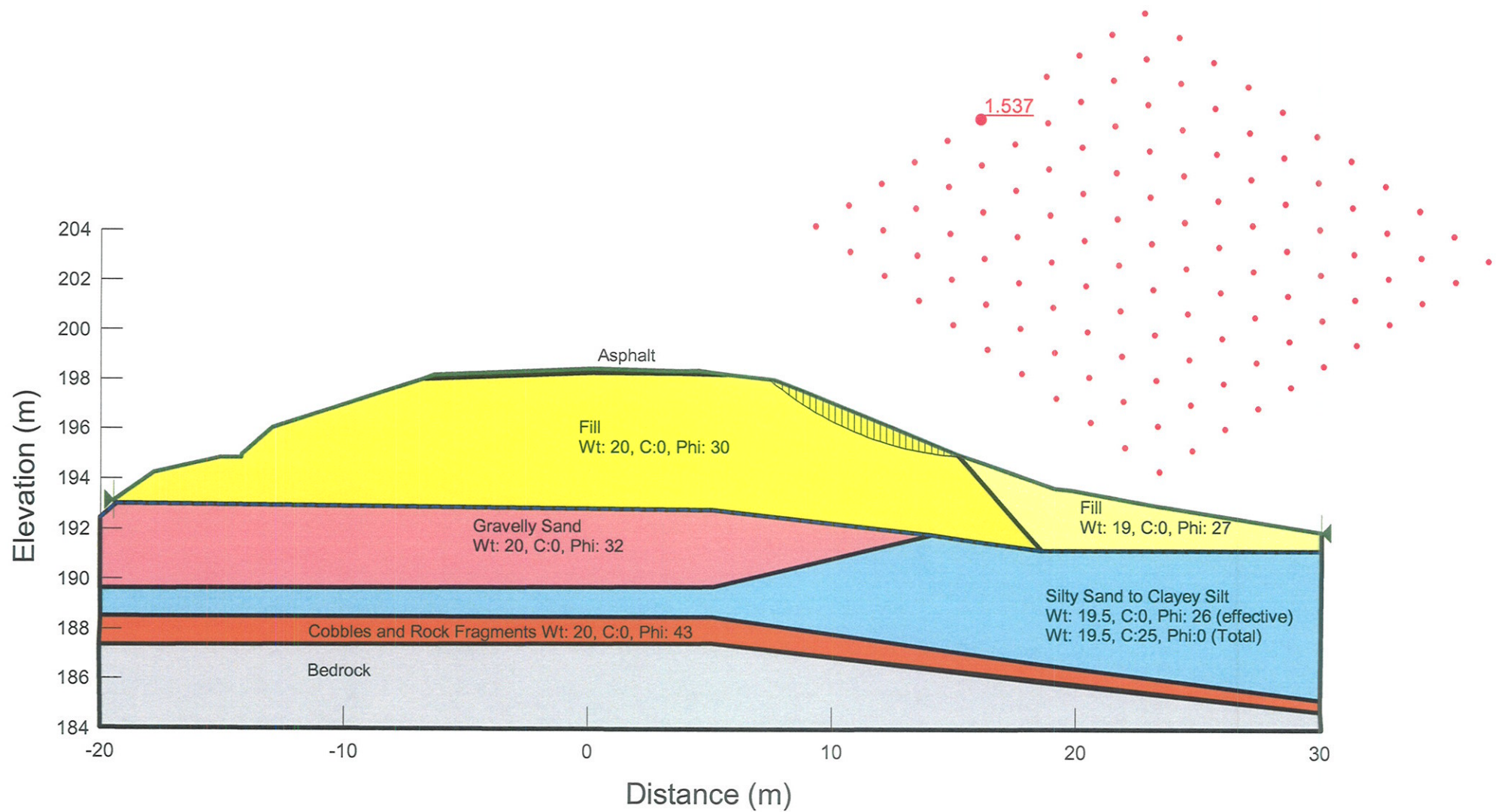
STA 26+479 After widening (with selected fill) 2.5H:1V



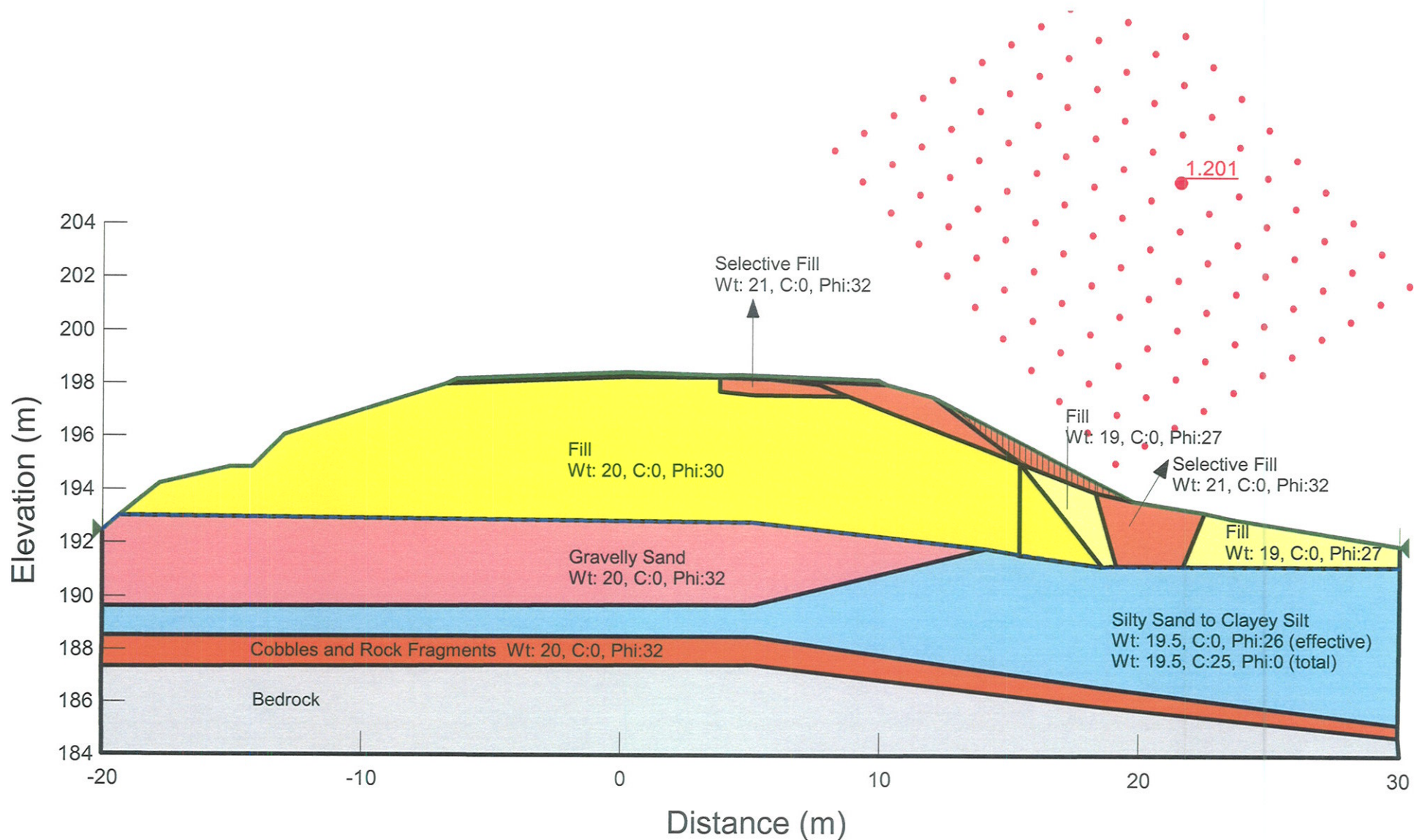
STA 26+479 After widening (with rock fill)



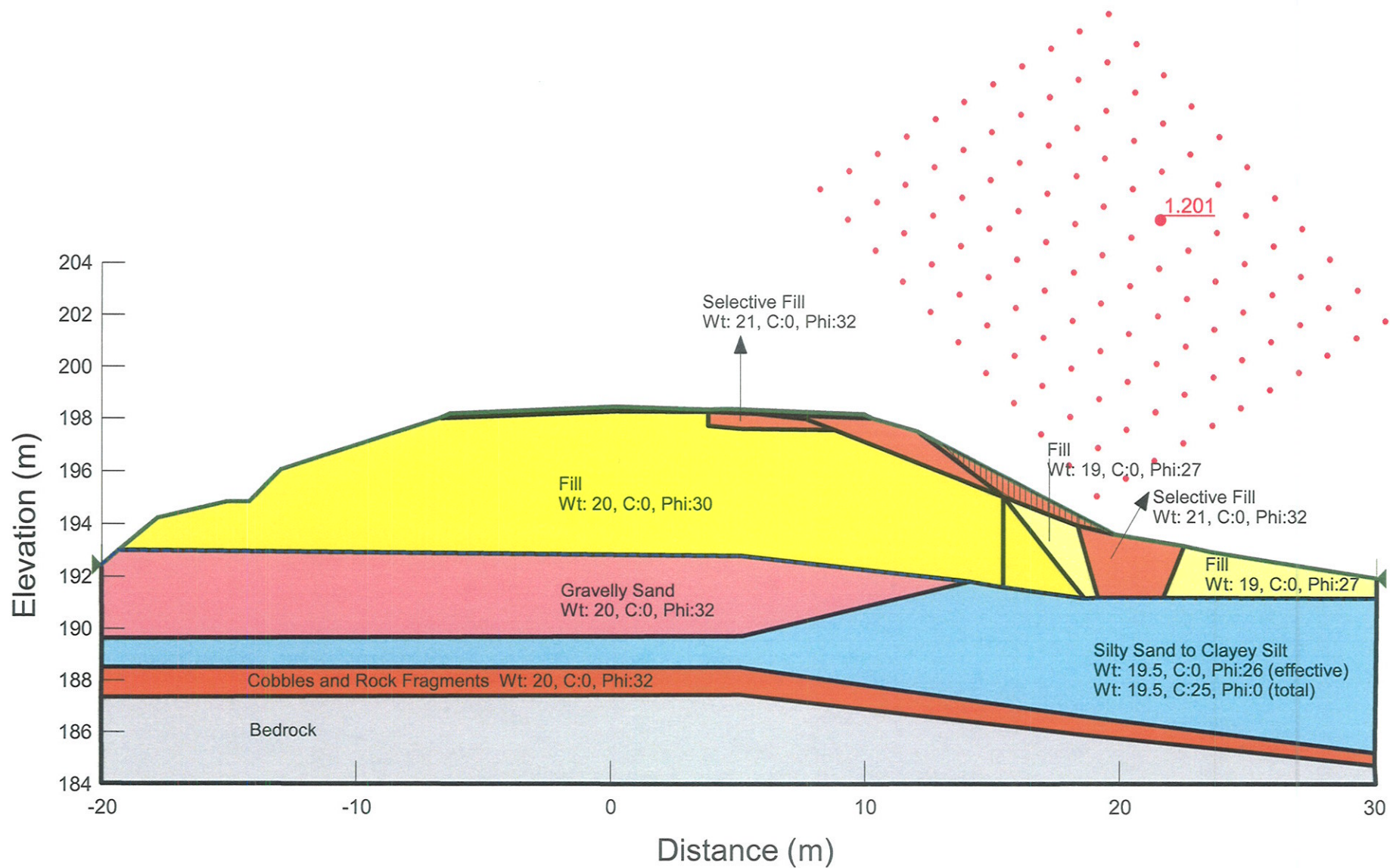
STA 26+509 Existing slope (total stress analysis)



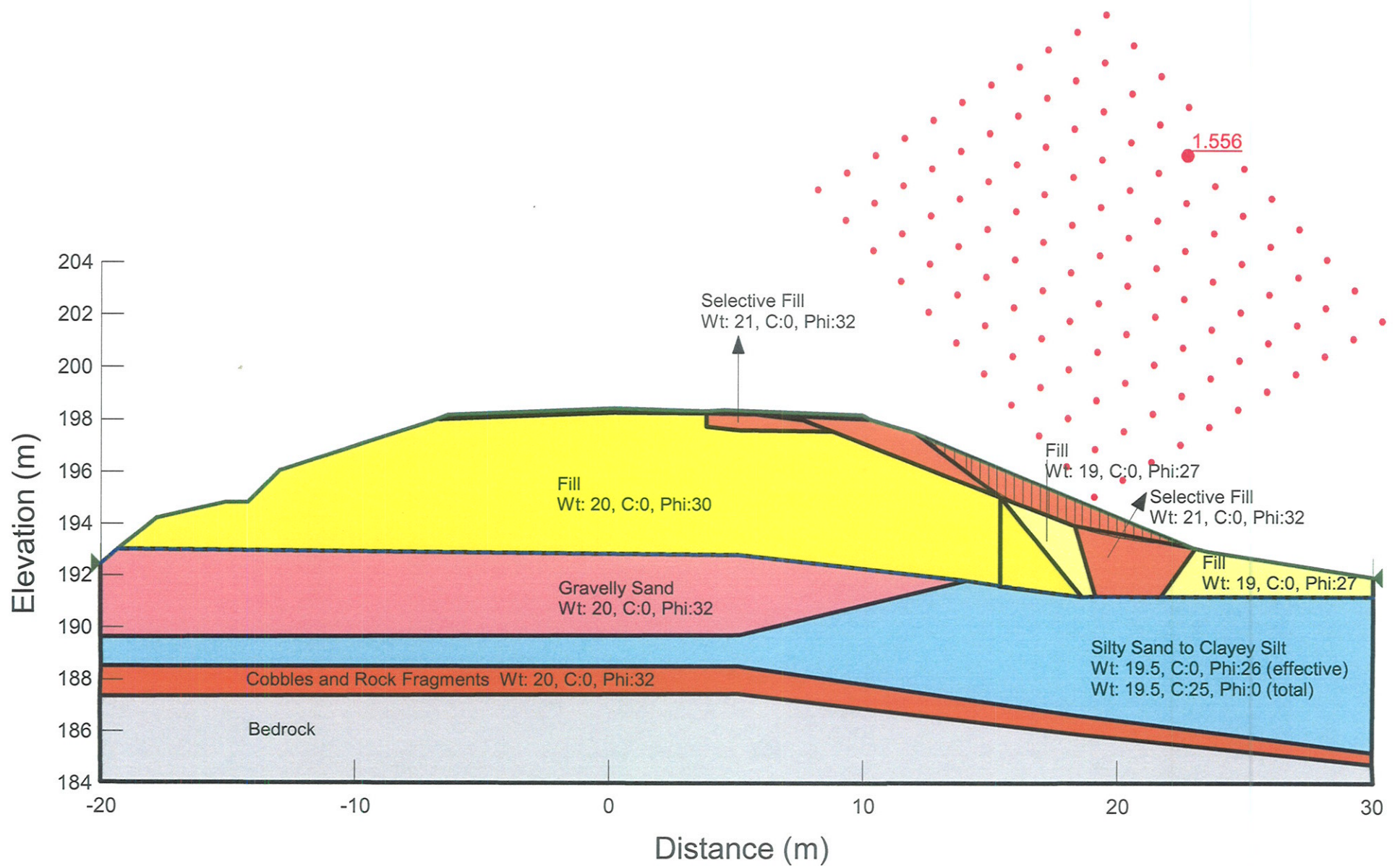
STA 26+509 Existing Slope (effective stress analysis)



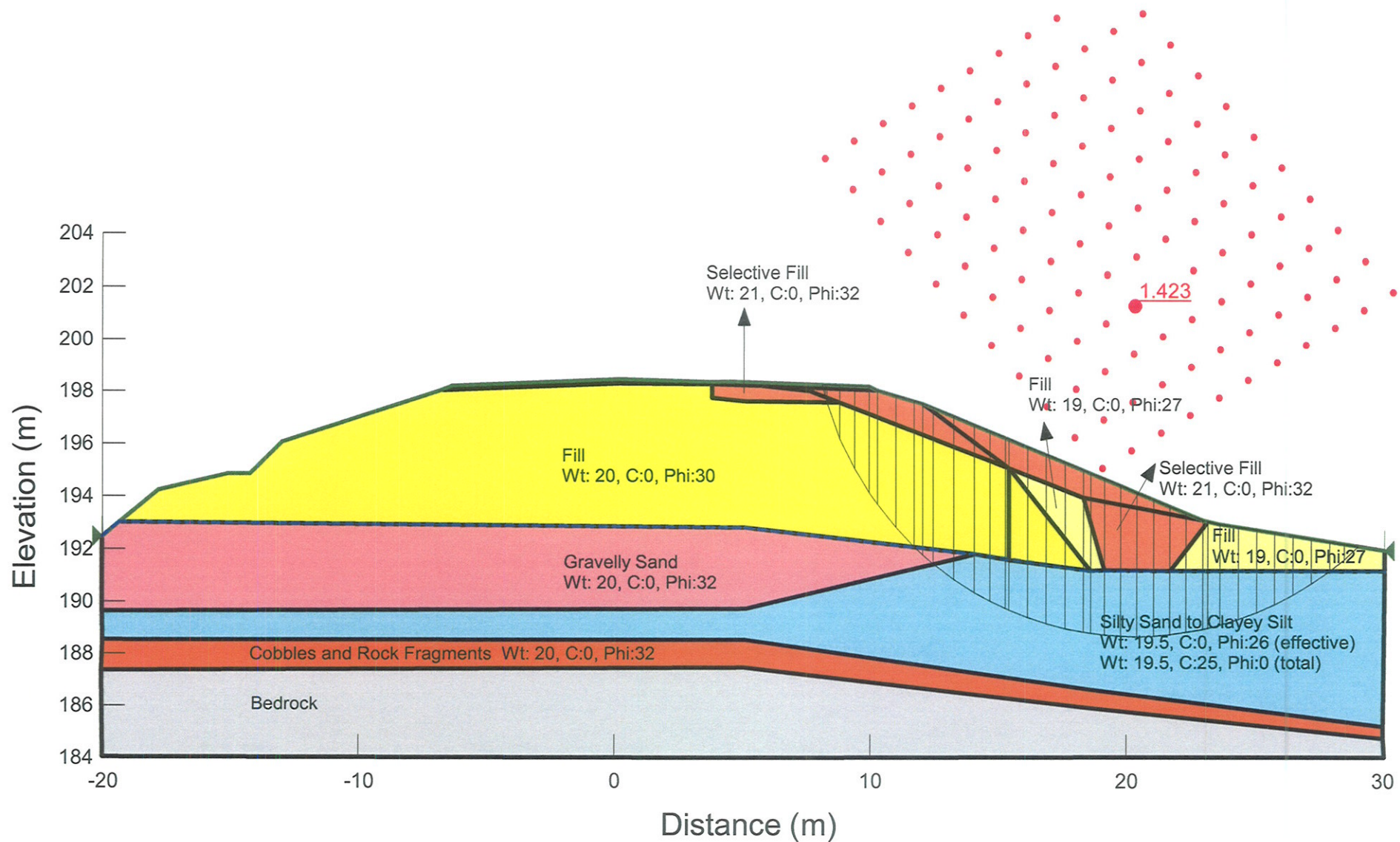
STA 26+509 Selected fill with sub-excavation (total stress analysis)



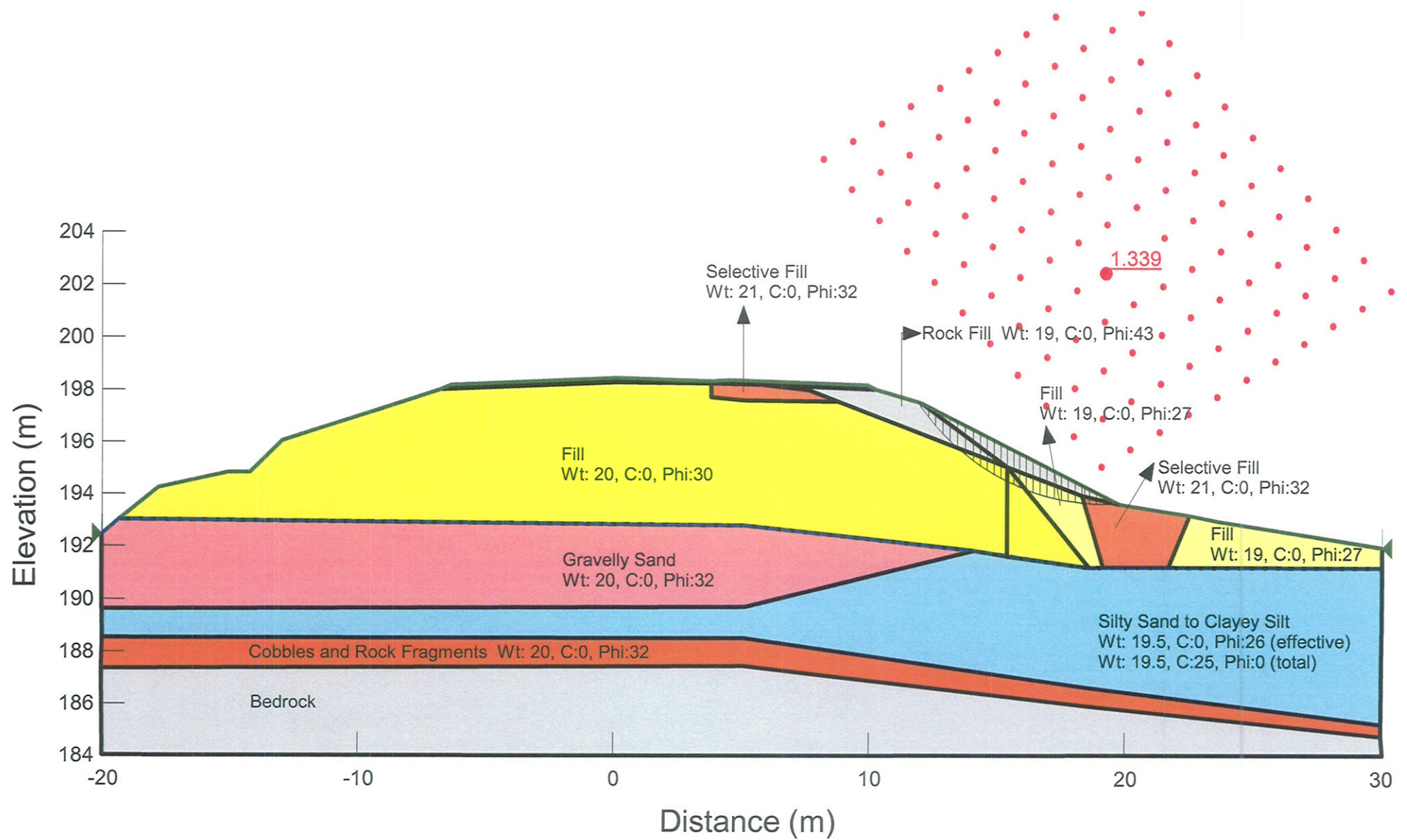
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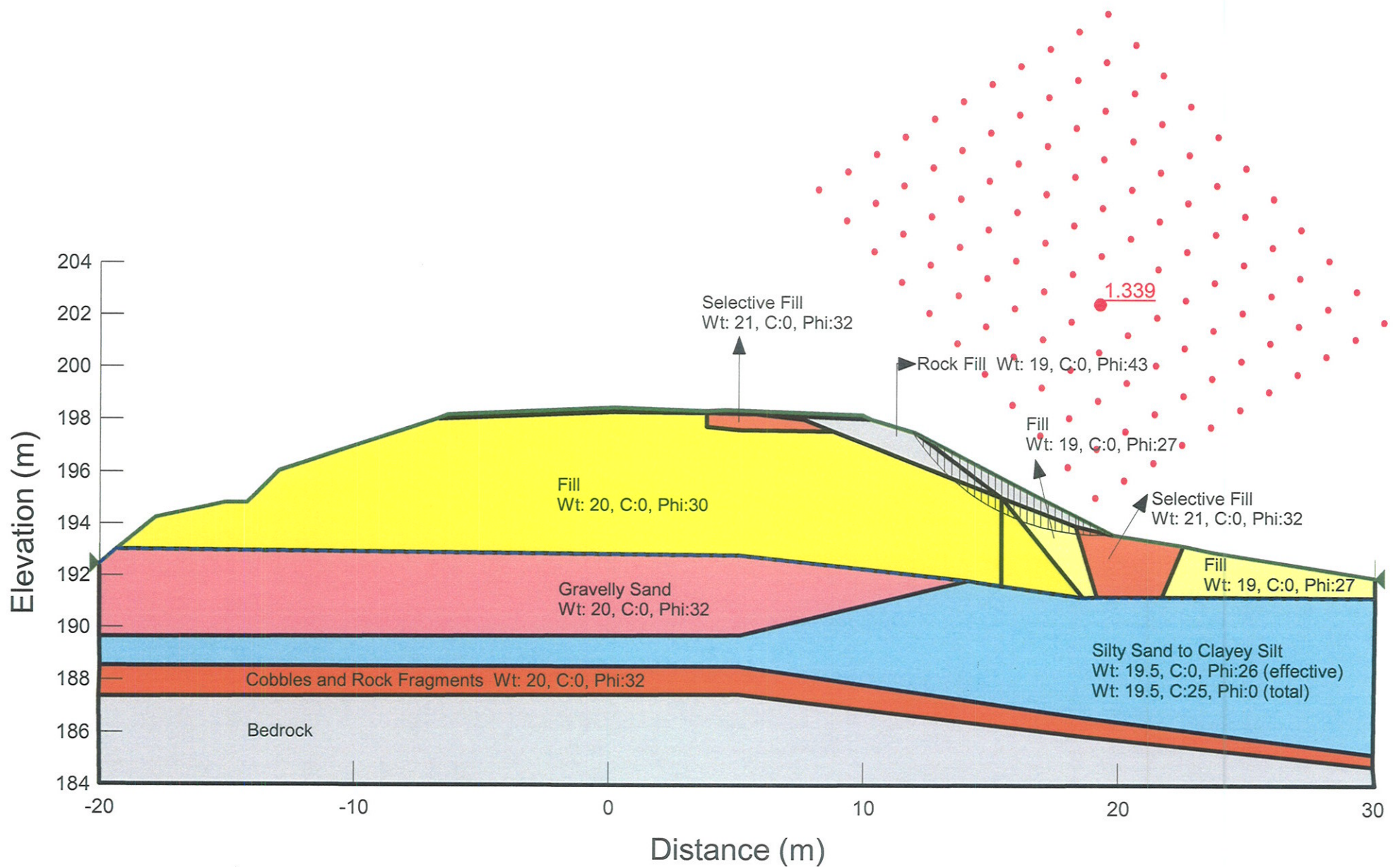
STA 26+509 Selected fill (2.5H:1V) with sub-excavation (total stress analysis)



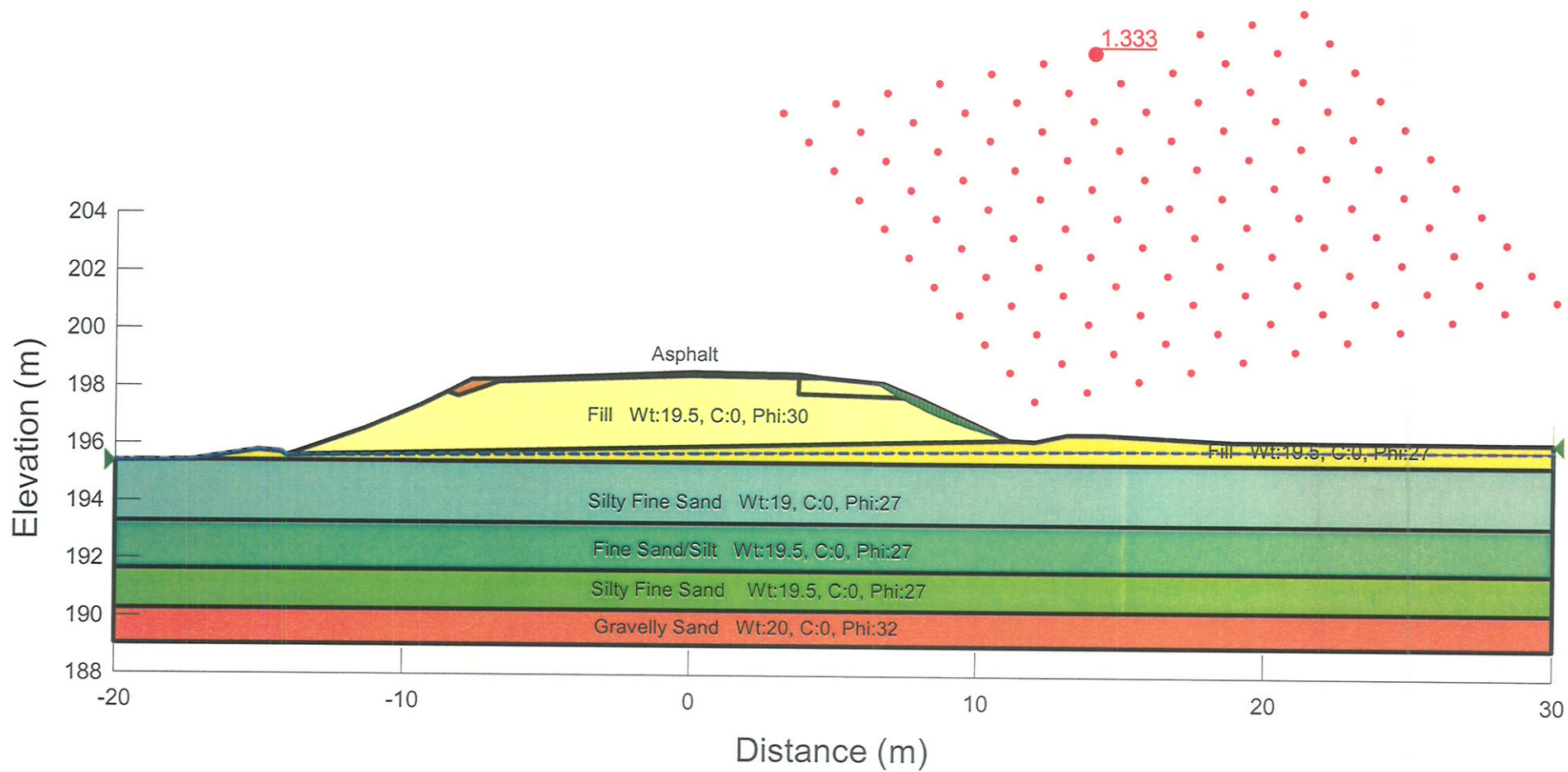
STA 26+509 Selected fill (2.5H:1V) with sub-excavation (effective stress analysis)



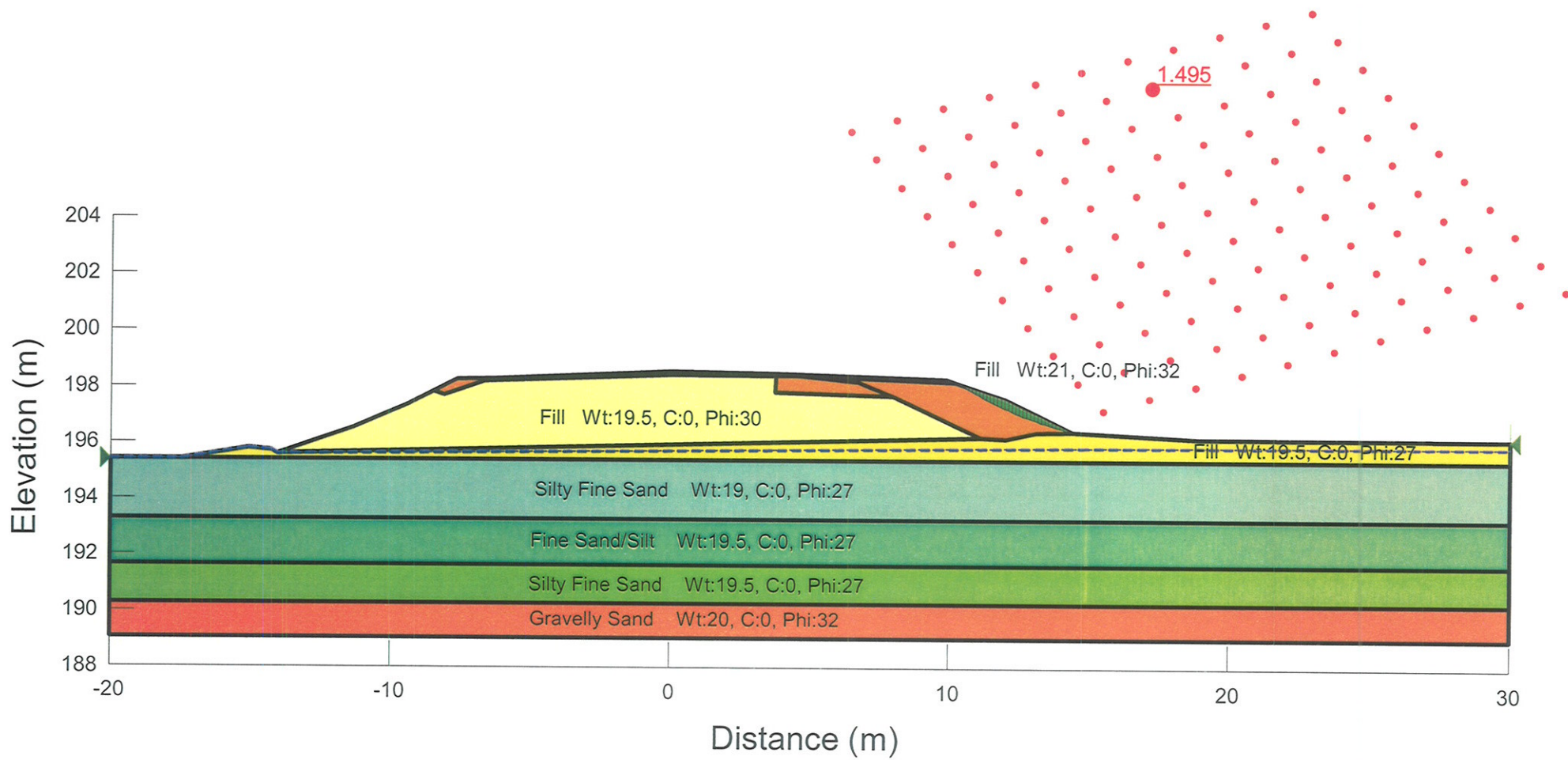
STA 26+509 Rock fill with sub-excavation (total stress analysis)



STA 26+509 Rock fill with sub excavation (effective stress analysis)



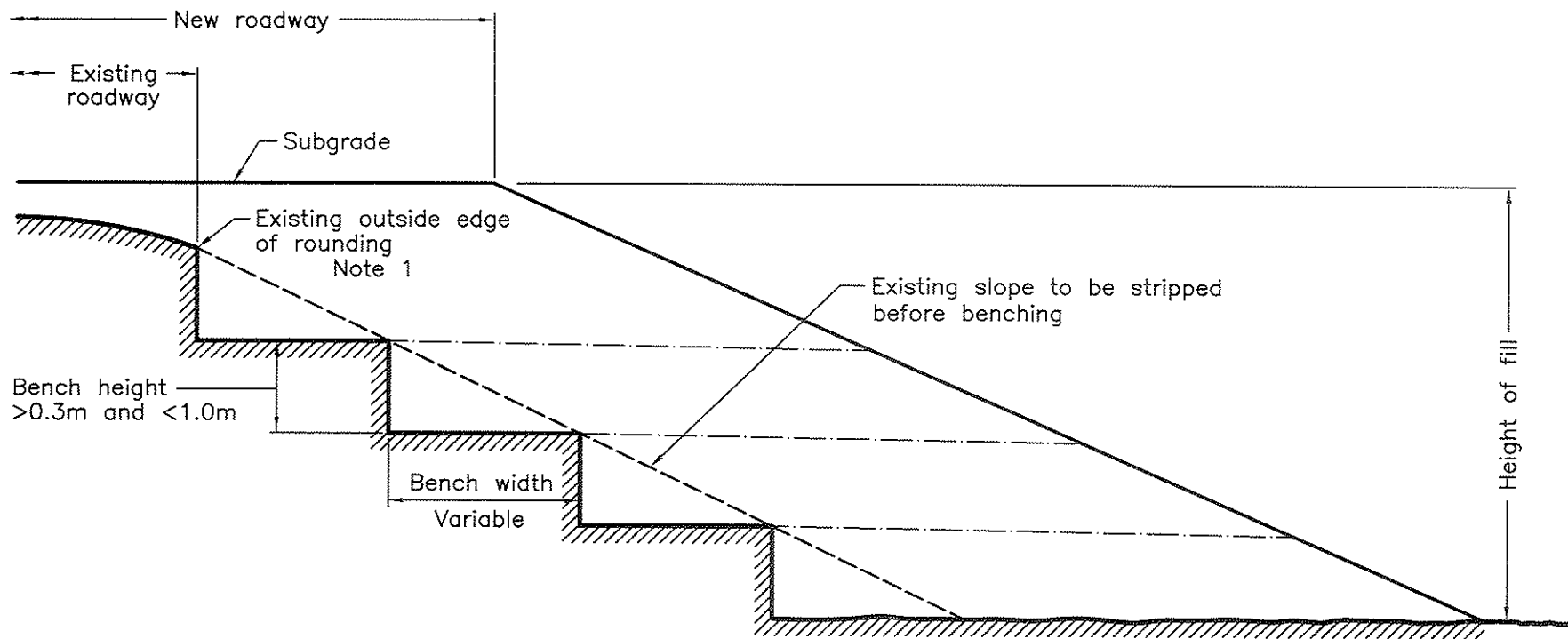
STA 26+558 Existing slope



STA 26+558 After widening (with selected fill)

Appendix H

OPSD



NOTES:

- 1 When the subgrade is below the existing outside edge of rounding, benching shall be carried out below the point where the subgrade intersects the existing slope.
- A Benching is not required on existing slopes flatter than 3H:1V.
- B Benches are to be excavated one level at a time and the compacted fill brought up before the next benching level is excavated.

ONTARIO PROVINCIAL STANDARD DRAWING

BENCHING OF EARTH SLOPES

Nov 2003 Rev 1



OPSD - 208.010

Appendix I

Limitations of Report

LIMITATIONS OF REPORT

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

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

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

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

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