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REPORT ON

ROSE CREEK TAILINGS IMPOUNDMENT SITE CHARACTERIZATION AND SEISMIC STABILITY ASSESMENT REPORT ANVIL RANGE MINING COMPLEX, YUKON

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EXECUTIVE SUMMARY

This report presents the site characterization and seismic stability assessment of the Rose Creek Tailings Impoundment which is located at the Anvil Range Mining complex in the south central Yukon.

The characterization of this tailings facility is based on based CPT (cone penetration test) soundings during October 2003, geotechnical laboratory testing carried out in January 2004, and on a review of previous geotechnical investigations, dam design and construction records dating back to 1972.

The tailings deposition patterns have been reviewed to provide the context in which to assess the variable soil types and insitu condition encountered during the CPT soundings.

Laboratory testing has been carried out on two different gradations of the tailings (a fine and coarse) for estimates of the critical state properties of each gradation for use in the CPT interpretations.

The CPT investigation results show that the tailings are generally layered in patterns consistent with historical spigotting arrangements. Very soft silts or fine tailings slimes are found in the backwater and pond areas. Tailings sands can be found close to the spigot points. Between the near and far zones, the gradation of the tailings changes and interlayering due to the tailings stream meandering is the more prominent effect.

Two liquefaction assessment methodologies based on CPT data were used. The methods and equations for each approach are presented and discussed. Both approaches predict that the tailings sands are predicted to be prone to initial liquefaction under the seismic exposure considered, although the sands appear sufficiently dense for initial liquefaction to not lead to a flowslide.

The central issue for the tailings impoundment is the behaviour of the fine tailings slimes during an earthquake. In the analysis of the fine tailings slimes, one of the liquefaction methods appears inappropriate for these soils; the other method predicts widespread liquefaction of the slimes, and with a high potential for subsequent flowslides. The residual strengths of the reconstituted slimes were also low in the laboratory testing, consistent with this evaluation of the insitu data.

Stability and deformation analyses were carried out for the Intermediate Dam. The seismic stability under the MCE is below unity and the displacement analysis indicates minor horizontal and vertical displacement using a Newmark analysis.

Seismically induced flowslide in the fine tailings in the Intermediate area tailings are estimated to be contained by the Intermediate dam, if the Secondary Dam retains the Secondary area tailings under the seismic loading. However, a flowslide in the Intermediate tailings could displace the Intermediate pond over the crest.

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1.0 INTRODUCTION

1.1 Purpose

This report presents the site characterization of the Rose Creek Tailings Impoundment that is located at the Anvil Range Mining complex in the south central Yukon. Figure 1.1 shows the site location.

The characterization of this tailings facility is based on CPT (cone penetration test) soundings during October 2003, a geotechnical laboratory testing program carried out in January 2004, and on a review of previous geotechnical investigations, dam design and construction records which date back to 1973.

This report is intended to solely provide a basis for the seismic stability assessment of the tailings impoundment as part of the current closure studies for this mine site. As such, the report is technical in nature and is directed at a readership with the appropriate technical background.

1.2 Background

Tailings were deposited into the Rose Creek impoundment over the period of 1969 to 1992, the tailings being produced from adjacent lead-zinc mining and which was carried out by:

- Cyprus Anvil Mine 1969 to 1982 (8,000 tons per day production);
- Curragh Resources 1986 to 1992 (13,000 tons per day production); and
- Anvil Range Mining Corporation 1995 to 1998 (13,000 tons per day production).

The mine was not always in operation during the above ownership history, with no production for the four years from June 1982 to June 1986. There was a further break in production between 1992 and 1995. Mining operations ended at the site in 1998.

The tailings impoundment fills a section of the previous Rose Creek Valley and the flow of Rose Creek has been diverted upstream of the impoundment area into a Diversion Canal that runs along the south side of the impoundment. A recent (2003) aerial photograph of the impoundment is shown on Figure 1.2 with the four main dams used to develop the impoundment identified. Three of the dams retain tailings, with the furthest downstream retaining water (the polishing pond). The diversion canal is also indicated on this figure.

Starting at the downstream end of the impoundment, the first dam structure is the *Cross Valley Dam*. This dam, which is a maximum of about 19 m high, impounds water and is used as a polishing pond. Upstream of this pond is the *Intermediate Dam*. The Intermediate Dam is a maximum of about 34 m high and forms the limit of the *intermediate tailings impoundment*. Further upstream is the *Secondary Dam*, which retains the tailings in the *secondary impoundment* and is itself about a maximum of 27 m high on the west section. The east section of the secondary dam runs parallel to the rose creek diversion and is typically about a maximum height of 4 m to 5 m. Upstream of this is the *Original Dam* impoundment dam, which impounds tailings in the *original tailings impoundment*. The original dam is about a maximum of 26 m high (from original foundation to current crest) however, almost the entire downstream slope is buried with tailings in the Secondary tailings area.

Overall there is an elevation change of about 50 m, from the toe of the Cross Valley dam to the tailings at the back of the original impoundment, over a horizontal distance of about 3,700 m. Over the current tailings surface there is a change of about 20 m from the tailings surface at the highest upstream end of original area to the exposed beach upstream of the intermediate dam.

1.3 Scope and Authorization

The original scope of work for this project was presented in our proposal P32-1410 dated October 9, 2003 in response to a request for proposals prepared by SRK Consulting Inc. (SRK), acting on behalf of the Interim Receiver for Anvil Range Mining Corporation and the Type II Mines Project Office. The scope was to study the tailings physical properties and seismic stability at the Rose Creek tailings Impoundment at the Anvil Range Mining Complex in the south central Yukon.

Golder received written authorization from SRK to proceed with the initial part of the work and comprising data review, CPT investigation, CPT data reduction, and preparation of the sections for stability analyses. The remaining scope items were to be defined based on the input from SRK and DIAND while the fieldwork was progressed.

Golder presented SRK with a revised scope document in our November 7, 2003 letter, Task 1 of the letter being characterization of the tailings. This task was in part based on a review of available documents (a summary of which is listed in Appendix I) and in part on an insitu characterization of the Rose Creek tailings impoundment through a cone penetration test (CPT) program. Task 2 described our proposed laboratory testing program and Task 3 the engineering analyses for seismic stability from the CPT interpretations.

SRK, Golder and DIAND held a telephone conference call on December 19, 2003 and discussed the December 12, 2003 draft Site Characterization Report. During this call it was agreed that Golder would proceed with a limited laboratory testing program on the tailings and prepare a revised draft characterization report which presented the basis of the CPT interpretation, laboratory testing results and the seismic stability assessment.

Golder issued a draft "Rose Creek Tailings Impoundment Site Characterization and Seismic Stability Assessment Report" Dated February 9, 2004 for review and comment.

SRK, Golder and DIAND held a second telephone conference call on February 25, 2004 and discussed the February 9, 2004 draft Site Characterization Report. During this call it was agreed that Golder would proceed with a stability analyses and Newmark deformation analyses for the intermediate dam and that comments would be included on the viability of placing a cover on the tailings based on geotechnical data obtained from the CPT investigation, all of which would be reported in the final version of the Site Characterization and Seismic Stability Assessment Report.

2.0 IMPOUNDMENT DEVELOPMENT

2.1 Development Stages

Tailings were impounded in Rose Creek over the twenty three year period of September 1969 to July 1992. The impoundment was developed in three stages.

- The *original tailings impoundment* was used from September 1969 to June 1982.
- The *secondary tailings impoundment* was used from mid 1975 to June 1982, and June 1986 to Oct 1986.
- The *intermediate tailings impoundment* was used from Oct 1986 to July 1992, after which, tailings disposal was in-pit.

The three tailings impoundments are shown on Figure 1.2, and all were developed by using starter dams to close the low end of each impoundment. Dam crest were raised by various methods over the years of mining. The final impoundment is made up of a total of four dams which were constructed to provide storage volume, deposition points and water retention. Dam construction history was as follows:

- Original Dam-starter dam 1969, crest raises 1970 to 1974;
- Second Dam starter dam 1974 to 1975, crest raises in 1978, 1970 and 1980;
- Intermediate Dam starter dam in 1981, crest raises in 1988, 1989, and 1991; and
- Cross Valley Dam constructed in 1980 to 1981.

The Original Dam parallels the approximate original alignment of the Rose Creek channel, giving it a pear-shaped configuration, to make use of a natural esker along its southeastern portion. Initially constructed with a waste rock starter dyke of about 1000 m long with a height of about then raised upstream with tailings. The current maximum height taken from the dam crest to original ground would be about 26 m. Only the later part of its construction was engineered. Tailings were deposited from both sides of this structure and at present only a few meters of the dam exist above the elevation of the secondary impoundment.

The construction of the Secondary Dam was carried out in 1974-1975, and was in effect built as two abutting structures. One section is referred to as the *West Dam* and the other the *East Dam*. The complete Secondary Dam crosses the valley west of the Original Impoundment Dam (refer to Figure 1.2) and turns to follow the Rose Creek Diversion Canal. The initial realignment of Rose Creek was started during the construction of this dam. This structure was an engineered structure and was constructed using standard dam construction practices available at the time of construction. The dam is about 1,900 m long with approximate maximum structure heights of 27 m and 4.5 m for the West Dam and East Dam portions, respectively. Tailings were deposited from both sides of the West Dam portion and from the upstream side of the East Dam portion.

The Down Valley Scheme was developed in 1981 to the west of the Second Impoundment. This scheme involved the construction of two new dams: the Intermediate Dam and the Cross Valley Dam. The Intermediate Dam, as shown on Figure 1.2 divides the Polishing Pond from the Intermediate Impoundment. The dam is about 700 m long and has a maximum height of about 34 m. The Cross Valley Dam forms the western limit of the Tailings Facility (refer to Figure 1.2) and contains the Polishing Pond. The Cross Valley dam is about 500 m long and has a maximum section height of about 19 m. The Down Valley scheme also involved extending the Rose Creek diversion to where it is today.

2.2 Tailings Deposition Patterns

The tailings deposition patterns have been reviewed to provide the context in which to assess the variable soil types and insitu condition encountered during the CPT soundings. Broadly, hydraulic deposition of tailings separates the tailings from the mixture produced at the mill to a range of soils. The coarser fraction is deposited close to the spigot point, with the very fine (and soft) slime fraction traveling to backwater ponds and so forth. Meandering of the tailings stream on the surface produces an interlayering of coarser and finer soils.

Tailings deposition patterns have been estimated from five aerial photos of the facility dating from 1972, 1975, 1979, 1997 and 2003.

Initially, between about 1969 to about 1974, tailings were mainly discharged into the original impoundment. It appears that a main single discharge point was used located at about the Faro creek inlet. A tailings delta can be seen in the 1972 photo, Figure 2.1, with the tailings flowing to the south and then to the east. It is possible a pond existed at eastern end of impoundment and it appears the finer tailings were being deposited here. Two or three additional minor tailings delta fans are observed along a road cut under the mine access road to the east of this main discharge.

Also noted is that along the original dyke are a number of spigot fans which was understood to be made from the end of pipe discharges directed to the north to build up coarse tailings sand zones and raise the upstream side of the original dyke. This appears to have taken place along the full length of this dyke.

Figure 2.2 shows the 1975 photograph and at this time the starter dam for the secondary dam was complete including both the west and east sections with tailings deposition and tailings pond being impounded. It is understood that tailings deposition began into the secondary impoundment after the initial 1974 construction season and before the secondary dam was complete. A temporary dam was constructed upstream of the secondary dam alignment at the Rose Creek channel and winter tailings discharge into a temporary impoundment. In March 1975, a failure of this structure resulted in the flow of tailings water and tailings directly into the Rose Creek.

After about mid 1975, it appears that tailings were discharged from the crest of secondary dam, which is consistent with the practice used in raising the original dam with the coarse tailings immediately upstream of the dam crest being used to raise its crest.

During development of the second impoundment, tailings continued to be sent to the original impoundment so that deposition happened in both more or less concurrently. In the case of deposition into the original impoundment, it appears to have continued to be primarily from the Faro Creek fan. The fan of tailings seen in 1975 extended along almost the full northern limit of this area.

By 1979, as observed in Figure 2.3, it appears that the surface of the original tailings impoundment had become disused and dry. Tailings deposition continued into the secondary impoundment, with what looks like coarse tailings spigotted off the crest of the secondary dam to create coarse tailings beach for upstream construction. In addition to crest discharge, a number of large tailings fans are observed downstream of the toe of the original dam. The effect of this discharge pattern was to push fines (slimes) into the central area of the secondary impoundment.

The photograph taken in 1997, Figure 2.4, shows the Rose Creek tailings impoundment structures in what was known as the Down Valley project. Neither the original nor the secondary tailings areas were in use. Tailings were being deposited in the Intermediate area. During review of the Down Valley project design reports, it was determined that it was originally planned for the tailings discharge lines to be piped to the crest of the Intermediate dam and to create an upstream beach of tailings off the crest. This method would have been consistent with previous tailings discharge practices at this impoundment. However, the facility was not operated in this manner, possibly due to a change of ownership at the mine. Consequently, tailings discharged into the intermediate impoundment were generally from the north east corner, just below the secondary dam (closer to the mill and eliminating pipes required to get tailings to the dam crest). This single point discharge resulted in a long beach in the intermediate impoundment, with generally finest tailings (slimes) being deposited against the Intermediate Dam.

Finally, a 2003 air photograph (Figure 1.2) shows the impoundment in its present state. Ponded water is evident against both the intermediate and cross-valley dam. It would appear that the elevation and size of these ponds vary with time of year and with volume of water pumped to the water treatment facility. Upstream of the Intermediate Pond, the tailings surface appears to generally be dry; however, some possible signs of surface water flow are visible.

2.3 Containment Structures

The foundation conditions, a description of the cross-section and the construction sequence for each of the four dams is discussed in the following sections. This discussion is based on our review of the reports listed in Appendix I and in particular on the foundation boreholes put down over the years. A summary of the borehole data form our data review are presented in Appendix I, Table AI.1.

2.3.1 Original Impoundment Dam

The Original Impoundment Dam was initially developed with the construction of a waste rock starter dyke (at the northern end of the dam) which was tied-in to a 4.5 m to 6.0 m high natural esker at the eastern end of the dam. Logs for ten boreholes were discovered and which show that the waste rock starter dyke was founded on dense to very dense native sand and gravel with cobbles, over compact to dense sand, over loose to compact fine sand to silty sand, over soft to stiff non-plastic to low plastic silt (foundation silts), over dense to very dense sand and gravel (basement materials). The natural esker is a dense to very dense sand and gravel deposit with cobbles and boulders. The esker is underlain by compact to dense sand and gravel with cobbles (basement materials).

Typical cross-sections through the Original Impoundment Dam are presented on Figure 2.5. Cross-section A-A presents details where the dam was initially constructed with a 7.5 m to 9.0 m high waste rock dyke. Cross-sections B-B and C-C present details where the natural esker was used to provide initial tailings containment. While construction details for the waste rock dyke are not available, it is likely that nominal compaction using dozers and haul trucks was employed. The starter dyke consists of compact to dense waste rock with a fine silty sand.

Shortly after the commencement of tailings deposition, seepage from the toe of the dam was noted. A wide berm of waste rock was placed along the downstream toe of the dam to increase the stability of the dam.

Subsequent raisings of the Original Impoundment Dam were required to provide storage volume for the tailings. Prior to the 1973 geotechnical drilling investigation and following work, the Original Impoundment Dam was not an engineered structure. The structure was raised by an upstream method of dredging and compacting tailings. A berm of tailings of about a 3.7 m width was compacted using dozers and haul trucks. The outermost shell is an oxidized dense fine sand. At present, as shown on Figure 2.5, the

Original Impoundment Dam is almost buried by tailings with an approximate 1.5 m tailings surface differential between the original and secondary tailings areas.

A spillway was provided along the eastern end of the Original Dam, the location of this spillway shifting further eastward as the dam was raised.

2.3.2 Secondary Impoundment Dam

A geotechnical drilling program was performed in 1973 for the west portion of the Secondary Impoundment Dam, and logs for 22 boreholes were found. In general, the stratigraphy at the site consists of compact to very dense layered terrace sands, gravels and boulders, with substantially less dense and less desirably graded materials occurring beneath the present sub-valley stream channel cut. The latter materials, considered to be normally consolidated with respect to present stress levels, consist of layered alluvial silts and fine sands of generally loose to marginally compact relative density. Although no piezometers were installed, drilling observations and surface examinations suggested that the groundwater level was generally hydrostatic with that of the creek bed level and that the water table generally increased away from the creek toward the bedrock outcrop in the vicinity of the terrace roots. In the east of the Second Impoundment Dam foundation conditions comprised boulder nests, silty sand to sand and gravel and discontinuous permafrost over shallow bedrock.

A typical cross section of the West Secondary Dam is shown on Figure 2.6, which was constructed as a zoned earthfill embankment. The initial phase of construction included the construction of a low hydraulic conductivity core and drainage blanket. The dam was raised using an upstream method with an internally zoned downstream shell and a compacted tailings upstream shell. There are tailings on both sides with an approximate tailings surface differential of 13.7 m. The downstream slope of the West Dam is 2H:1V.

The East Dam was constructed using compacted tailings sand with a granular outer shell on the downstream side, as shown on Figure 2.7. It has a typical height of about 4.5 m with a downstream slope of 2H:1V.

2.3.3 Intermediate Dam

The foundation conditions along the alignment of the Intermediate Dam are based on the logs of ten boreholes. Much of the dam alignment traverses the upper level terrace gravels. The unconsolidated foundation did not contain extensive occurrences of permafrost in the valley bottom area. The foundation is a compact fine grained material in the proximity of the original Rose Creek channel. Some colluvial action is apparent in the south abutment area where organic seams occur within a till-like mass. It appeared that this stratum became coarser and more competent with in the downstream (i.e., western) direction. The majority of the foundation of the Intermediate Dam interfaced with sand and gravel alluvium. The north abutment consisted of relatively coarse sand and gravel till and some alluvium. The south abutment consisted of frozen till with some colluvium.

The Intermediate Dam was constructed as a zoned earthfill dam as shown on Figure 2.8. A compacted clay core was provided as a seepage barrier through the structure. A cut-off trench beneath the core, which keyed into the foundation soils, was provided to limit seepage along the core/foundation soils interface. Sand and gravel internal drainage systems were provided to convey seepage out the downstream shell of the dam. The sideslopes are at 2H:1V. The dam has a maximum height of about 34 m.

The initial phase of the dam was constructed in 1981. The dam was subsequently raised using downstream construction methods.

A spillway was constructed in the valley wall to the north of the dam with a present-day invert at Elev. 1047.7 m a.m.s.l.

2.3.4 Cross Valley Dam

The foundation conditions of the Cross Valley Dam are based on the large number of borehole logs found (48). The valley floor in the area of the Cross Valley Dam contains mainly sand and gravel with extensive lenses of frozen, inorganic fine sand and silts. The marginally compact alluvium which forms the unconsolidated foundation of the Cross Valley Dam is highly variable, ranging from fine grained materials in the vicinity of the original Rose Creek channel, to gap graded silty gravels, to schist-rich till in the north abutment area.

A typical cross-section of the Cross Valley Dam in presented on Figure 2.9. The dam was constructed in one phase to its current configuration during the construction seasons of 1980 to 1981. The dam is a zoned earthfill structure with a compacted till core as its seepage barrier. An upstream till blanket and a cut-off trench beneath the core were constructed to limit the seepage through the foundation materials. Internal granular drainage systems were provided to reduce seepage pressures within the downstream shell. The sideslopes are 2H:1V. The dam has a maximum height of about 19 m.

Following construction of the dam in about 1991, a toe drain was added at the downstream toe of the dam to assist with toe seepage control.

An emergency spillway was provided on the north abutment of the dam and has an invert at approximately Elev. 1031.2 m a.m.s.l.

3.0 LABORATORY TESTING

3.1 Prior Work

Some laboratory tests were carried out during our first involvement with the site in 1973. This included testing of the tailings. The testing was used to establish basic index properties (density, specific gravity, and gradation) as well as strength parameters. Both triaxial and direct shear tests were carried out on reconstituted and "undisturbed" shelby tube samples. However, details on the test procedures/data are absent with only summary results being reported.

The specific gravity of the tailings was found to vary substantially from one sample to another. A range of 2.9 to 4.5 was reported for the specific gravity of retained tailings in the impoundment, with a specific gravity of 3.7 for the tailings sands used in the dam shell zone.

Triaxial testing comprised what is now known as the consolidated isotropic undrained (CIU) procedure with excess pore water measurement. Void ratio change during consolidation was not recorded. Only peak strength data was recorded, and there are no stress-strain curves. Procedures taken (if any) to ensure full saturation of the sample were not documented.

Triaxial tests were carried out on two undisturbed samples, one on a sandy silt (54% sand, 46% silt sized or finer) and one on a slimes sample (0% sand, 88% silt, 12% clay). Both samples came from within the original impoundment area. Three tests were reported for each sample, using different confining stress prior to shear. However, there was no testing to discern the effect of density on the soil behaviour and there are no reported stress paths from which this might be inferred. The reported peak strengths are ϕ' = 37 degrees for the sandier sample and 29 degrees $<\phi'<$38.5 degrees for the finer slimes sample (the low friction angle was for the highest confining stress used). The void ratio of each test is unknown. Although the plots in the report show three results for each undisturbed sample, it appears possible that these were in fact three test stages on one sample. Such "multistage" tests were a common procedure at the time, and involved reconsolidating a sample at a higher confining stress after it was loaded to failure. This was done twice so that the effect of stress level on the sample could be readily seen, although the effect of density change was not usually ever measured and the effect of disturbance on the soils behaviour in the two higher stress stages was not addressed in data reduction.

There were three triaxial tests and three direct shear test on a remoulded samples of a sandy silt tailings (40% sand, 60% silt sized and finer). Remoulding procedures are not documented and the void ratio of the tested specimens is unknown. The test data supported a single $\phi' = 33.5$ degrees.

Overall, it can be concluded from the above testing that an estimate of the critical state friction angle is about $\phi'_c = 30$ degrees. This angle is independent of density and stress level, and is the lower bound of the various tests. Greater densities, of both sands and silts, will dilate and show higher friction angles. However, there is insufficient data in these tests to determine the relationship between increased density and increased friction angle.

3.2 Reconstituted Tailings Sample Testing 2004

A limited geotechnical laboratory testing program was carried out in January 2004 directed at determining index properties and the critical state line (CSL) for each of two gradation samples of the tailings, with one sample a representative coarse sample and the other a representative fine sample. The aim of testing these two gradations was to provide data on the range of soil properties to be expected with the variable insitu fines content of the tailings deposit. These soil properties were required to reduce uncertainties in the evaluation of liquefaction potential from the CPT data.

3.2.1 Origin of Samples

Two tailings samples were prepared at the Golder Burnaby laboratory by combining some of the samples which Gartner Lee Ltd. (GLL) obtained during their 2003 Sonic drill investigation program at the Rose Creek tailings impoundment. Fifteen of the sonic core samples were selected from four different drillhole locations based on a review of both the draft Sonic drillhole logs and the CPT data located near the sonic holes. Table 3.1 presents a summary of the GLL sonic samples provided in January 2004. The samples were dried and then sorted by fine and coarse gradation to create the two bulk samples for the testing program. In addition to these Sonic drilling samples, fine tailings material was available from the 14 moisture content samples gathered from an auger hole drilled next to SCPT03-21 to a maximum depth of 15 m. These moisture contents were used for an estimate of the insitu void ratio with depth next to this SCPT location.

The combined samples resulted in a 3.8 kg fine sample A and a 4 kg coarse sample B. Figure 3.1 presents the grain size distribution representative of each of the two tailings bulk samples. The fine gradation, Soil A, is a sandy silt with some 66.2% silt sized or smaller ("fines"). The coarser gradation, Soil B, was approaching a silty sand, with some 30.1% of the sample silt sized or smaller.

Table 3.1 Summary of Samples Combined for the January 2004 Laboratory Strength Testing Program

Gartner Lee Ltd. Sonic Drill Program					
Well #	**Tailings Sample Name**	**Tailings description**	**Nearest CPT03-**	**Estimated CPT F**	**Golder Laboratory Sample**
P03-05	DS3	Very fine sand	17	0.7	COARSE
P03-05	DS4	Very fine sand	17	1.7	
P03-05	FS4	Very fine sand	17	0.8	
P03-05	GS2	Find sand	17	2.0	
P03-06	DS3	Fine sand, traces of silt	30	1.8	
P03-06	DS1	Med to fine sand, trace silt	30	1.7	
				RANGE 0.7 to 2	
P03-04	DS1	Fine sand with silt	21	1.3	FINE
P03-04	DS2	Fine sand with silt	21	2.4	
P03-08	BS2	Clayey silt	32	2.4	
P03-08	CS1	Very silty and clayey fine sand	32	1.4	
P03-08	FS2	Fine sand, some silt, trace clay	32	1.4	
	SCPT 03-21	fine sandy silt	21		
				Range 1.3 to 2.4	
P03-06	CS3	Fine sand, traces of silt	30		Only fine portion used in the FINE sample
P03-06	ES2	Med to fine sand, trace silt	30		
P03-04	CS1	Fine sand with silt	21		
P03-04	DS4	Fine sand, traces of silt	21		

Specific gravity (Gs) of both tailings gradations was determined using procedures following ASTM standard D854, with the results shown on Table 3.2. Gs was larger for the coarse sample than the fine, and both were higher than expected for silicious soils, but explained by the heavy nature of lead-zinc ore tailings.

Minimum and maximum void ratio were determined for the two tailings gradations using procedures following ASTM standards D4253 and D4254, respectively.

Table 3.2: Index Properties of Combined Samples of Rose Creek Tailings

	G_s	D_{50} (mm)	D_{60} (mm)	D_{10} (mm)	Cu	Fines Content (%)	Maximum void ratio, e_{max}	Minimum void ratio, e_{min}
Fine Sample A	3.97	0.05	0.065	0.005	13	66.2	0.837	2.017
Coarse Sample B	4.48	0.1	0.12	0.05	2.4	30.1	0.556	0.990

Where: D50, D60 and D10 are the equivalent grain diameter in mm corresponding to 50%, 60% and 10% passing. Fines content here is defined as % passing the No. 200 sieve size.

3.2.2 Overview of Testing

Details of the procedures for the critical state triaxial testing program are presented in Appendix II. A limited number of both undrained and drained triaxial tests were carried out on both loose and dense reconstituted samples of each gradation of tailings to determine the CSLs.

The critical state is the condition in which soil shears at constant volume and with no tendency to change volume; this second condition is especially important and inattention to it has been the reason for virtually all the reported non-uniqueness of CSL. The principal interest of the present work is in the CSL in void ratio (e) versus mean effective stress (p') space, and it is usual to represent the CSL with the equation:

$$e_c = \Gamma - \lambda_{10} \log(p') \qquad [3.1]$$

where e_c is the void ratio at the critical state for the mean effective stress $p' = (\sigma'_1 + \sigma'_2 + \sigma'_3)/3$. Γ is the void ratio of the critical state at a reference stress that is by convention taken as p'= 1 kPa. λ_{10} is the slope of the CSL in a plot of e vs. $log(p')$ and the subscript denotes that base 10 logarithms are used. This use of base 10 is again a convention for convenient plotting of the data. When used in mathematical models it is normal that natural logarithms are used, which simply involves dividing λ_{10} values by 2.3.

A total of eight triaxial tests on reconstituted samples prepared by moist tamping method were carried out as summarized in Table 3.3.

Table 3.3: Triaxial Tests Performed

Tailings	# Tests	Triaxial Test Type
Coarse gradation (30.1% fines)	4	2 CIU, 2 CID
Fine gradation (66.2% fines)	4	2 CIU, 2 CID

Where: CIU= isotropically consolidated undrained and CID= isotropically consolidated drained.

Table 3.4 summarizes the void ratio and stress conditions at different stages for the tests carried out. Void ratio at saturation (e_{sat}) for each test in at an effective confining pressure (p'_{sat}) of between 20 and 24 kPa. Void ratio at the end of consolidation pressure is presented as e_o and p'_0, respectively. All samples were isotropically consolidated.

The consolidation of the samples from their initial full saturation void ratio to that at which they were tested in shear was isotropic and was applied in increments. Figure 3.2 plots the void ratio at preparation and at the start and end points of each stage of consolidation for each test sample. An average compression index (C_c) values for each sample has been calculated and included on Table 3.4.

Table 3.4: Void Ratio and Consolidation Pressure Data for Rose Creek Tailings

Tailings Sample	Triaxial Test Type	At end of saturation	After consolidation and prior to shear		Average Consolidation Compression Index
		e_{sat}	p_0' (kPa)	e_0	C_c
66.2% Fines	A_CIU-1	0.929	400	0.756	0.14
66.2% Fines	A_CIU-2	0.996	995	0.697	0.18
66.2% Fines	A_CID-3	0.950	799	0.695	0.17
66.2% Fines	A_CID-4	0.864	204	0.744	0.13
30.1% Fines	B_CIU-1	0.877	404	0.792	0.07
30.1% Fines	B_CIU-2	0.899	794	0.776	0.08
30.1% Fines	B_CID-3	0.883	803	0.769	0.07
30.1% Fines	B_CID-4	0.865	205	0.816	0.05

The testing summarized in Table 3.4 is all on loose reconstituted samples. There were no tests on dense samples of either tailings gradation.

3.2.3 Results for Fine Sample A -Sandy Silt (66.2% Fines)

Figure 3.3 presents a summary of the results of the four triaxial tests on the fine sample "A" and shows the estimated CSL. The summary data is presented in a deviator stress vs. strain plot, a stress path plot, a void ratio vs. log p' plot, and (depending on the test conditions) either an excess pore pressure with strain or volumetric strain vs. axial strain. Appendix II includes the same series of plots for each individual test.

Accurate tracking of void ratio by dimensions and cell and pore space volume changes resulted in void ratio calculations very close to the void ratio determined at the end of the test by the sample freezing method. No membrane penetration corrections have been applied to the calculated void ratio data.

The CSL has been estimated by fitting a trend line through the end points of the void ratio vs. mean effective and on the mean effective stress vs. deviator stress as shown on Figure 3.3. Based on this fitting, the critical state parameters determined were:

- $\Gamma = 1.076$
- $\lambda_{10} = 0.159$
- $M_{tc} = 1.2$

It is noteworthy that it was not possible to test the fines samples at anywhere near their reconstituted void ratio. As can be seen on Figure 3.3, there was substantial contraction of the samples during sample saturation and samples were tested in shear starting from void ratios in the range $0.7 < e < 0.8$. These void ratios can be compared with void ratios frequently exceeding 1.0 insitu which are documented below in Table 4.2. The measured strengths in these reconstituted tests cannot be used at face value and must be adjusted to the insitu void ratio.

3.2.4 Results for Coarse Sample B – Silty Sand (30.1% fines)

Figure 3.4 presents a summary of the results of the four triaxial tests on the coarse sample "B" and the estimated CSL. The summary data is presented in a deviator stress vs. strain plot, a stress path plot, a void ratio vs. log p' plot, and (depending on the test conditions) either an excess pore pressure with strain or volumetric strain vs. axial strain. Appendix II includes the same series of plots for each individual test.

Accurate tracking of void ratio by dimensions and cell and pore space volume changes resulted in void ratio calculations very close to the void ratio determined at the end of the test by the sample freezing method. No membrane penetration corrections have been applied to the calculated void ratio data.

The CSL has been estimated by fitting a trend line through the end points of the void ratio vs. mean effective and on the mean effective stress vs. deviator stress as shown on Figure 3.4. Based on this fitting, the critical state parameters determined were:

- $\Gamma = 0.921$
- $\lambda_{10} = 0.082$
- $M_{tc} = 1.19$

Similar to the fine tailings samples, the undrained strengths measured in testing the reconstituted coarse samples cannot be used for insitu conditions at face value and must be adjusted to the insitu void ratio.

3.3 Summary of Tailings Properties

From the results of the limited testing program carried out on the Rose Creek tailings Table 3.5 summarized the critical state measurements as soil behaviour parameters for use in the CPT data interpretation. These properties are independent of void ratio and stress level.

Table 3.5: Critical State Parameters for Rose Creek Tailings

Tailings Sample	Γ	λ_{10}	M_{tc}
A - 66.2% Fines	1.076	0.159	1.2
B - 30.1% Fines	0.921	0.082	1.19

The critical friction ratio M_{tc} is smaller than expected for common quartz sands, is small in comparison to what is often found with silts, and is rather small for tailings which often have high frictional strengths. It is, however, consistent with the estimate of $\phi'_c = 30$ degrees from the testing three decades ago and which was discussed in Section 3.1 above.

The slope of the CSL as represented by λ_{10} can be viewed as a 'compressibility' of the tailings. The value for the coarser sample is about an expected value, but the finer gradation is markedly more compressible than might have been expected based on its gradation.

The altitude parameter Γ has no special significance in itself, as what determines the soils behaviour is the void ratio difference between the soil and its CSL at the same stress and which is defined as the state parameter $\psi = e - e_c$ (see Figure 5.7).

Figure 3.5 shows the ratio of undrained strength, s_u to the initial confining pressure p'_o in terms of the samples initial state parameter ψ_0. Also shown on this plot are the bounds on expected behaviour based on testing several hundred samples. The Rose Creek tailings data plots at the low end of the expected undrained strength, and there is minimal difference between the coarse and the fine sample. This pattern is consistent with both the very similar M_{tc} measured in the two gradations and the rather low value of M_{tc} compared to other soils/tailings.

Also shown on Figure 3.5 is the residual strength ratio s_r/p'_0 in the undrained tests at the end of shearing. In the case of the coarser tailings, there is a substantial strength drop post-peak as can be seen in the stress strain curves on Figure 3.4. The fines (slimes) gradation showed a different behaviour in the testing with minimal strength reduction between the peak and residual condition. However, as noted above, the void ratio of the samples as tested were markedly denser than the void ratio of the insitu slimes. The expected behaviour of the slimes insitu is much closer to that of the sands tested, and at a void ratio of 1.0 the residual strength will be less than a tenth of that measured in the present laboratory testing at an average void ratio of about 0.75.

4.0 TAILINGS INVESTIGATION 2003

4.1 Description of Investigation

A cone penetration testing (CPT) program was carried out at the Rose Creek Tailings Impoundment site between October 15th and 18th 2003 under the direction of John Cunning, P.Eng. The CPT work was carried out with a Midnight Sun Drilling Ltd. CME 750 tire mounted drill rig for pushing and the Contec Investigation Ltd. CPT equipment.

Two different cones were used, a conventional 10 cm^2 and a 15 cm^2 cone. Both were right cylindrical units with 60 deg points conforming to the ASTM D5778 standard. The smaller cone comprised a standard three channel piezocone (with inclinometer measurements to check deviation during sounding); measured data from each sounding includes the uncorrected cone tip resistance (q_c), the sleeve friction (f_s) and the dynamic pore pressure (u_2) reading (with filter element at the cone "shoulder" location). The larger cone had the same suite of measurements during penetration but also included a geophone to allow measurements of the shear wave velocity of the soil when the cone was stationary (and which was when rods were added). Both cones were pushed into the ground at the standard 20 mm/sec rate and using industry standard 1 m rods.

A total of 36 holes were completed for a total length of 482 m of cone soundings. Table 4.1 lists the details of the soundings, while Figure 4.1 shows the location of each sounding in the plan of the tailings impoundment. Broadly, the investigation was arranged to systematically investigate several section lines through the impoundment.

Pushing the CPT into the ground causes excess pore pressures in soil that are less pervious than clean sands. The dissipation of these excess pore pressures was measured at 41 locations. Typically, about once per hole, during pushing through the tailings, if a high dynamic pore pressure was observed, pushing would be stopped and the pore pressure recorded with time until an equilibrium value was achieved. At the end of each hole, which was typically in the natural ground materials below tailings, the final pore pressure was dissipated.

Five CPT soundings were used to measure seismic velocity profiles. At these soundings, seismic shear wave velocity (V_s) was measured after each 1 m push profile the shear wave velocity with depth. These holes are marked SCPT (05, 17, 21, 25, 33).

One auger hole for sampling was drilled next to the SCPT 21. A total of 15 m was drilled to obtain samples for moisture content analysis and estimation of the insitu void ratio.

Table 4.1: Summary of CPT Soundings

CPT Sounding	Date	Type of Cone	Final Depth (m)	Estimated Water Table depth (m)	Comments
CPT03-01	15-Oct-03	10 cm^2 013	11.30	10	
CPT03-02	15-Oct-03	10 cm^2 013	13.35	9	
CPT03-03	15-Oct-03	10 cm^2 013	19.08	8	
CPT03-04	15-Oct-03	10 cm^2 013	22.30	9	
SCPT03-05	15-Oct-03	10 cm^2 013	14.98	5	Seismic Hole
CPT03-06	15-Oct-03	10 cm^2 013	17.10	13	
CPT03-07	15-Oct-03	10 cm^2 013	16.88	12	
CPT03-08	15-Oct-03	15 cm^2 147	9.98	9	
CPT03-09	16-Oct-03	15 cm^2 147	16.27	9	
CPT03-10	16-Oct-03	15 cm^2 147	18.33	7	
CPT03-11	16-Oct-03	15 cm^2 147	1.38	6	Shallow refusal, moved hole.
CPT03-11B	16-Oct-03	15 cm^2 147	8.15	6	
CPT03-12	16-Oct-03	15 cm^2 147	1.70	6	Shallow refusal, moved hole.
CPT03-12B	16-Oct-03	15 cm^2 147	14.23	6	
CPT03-13	16-Oct-03	15 cm^2 147	11.95	5	
CPT03-14	16-Oct-03	15 cm^2 147	5.82	4	
CPT03-15	16-Oct-03	15 cm^2 147	9.00	5	
CPT03-16	16-Oct-03	15 cm^2 147	16.92	6	
SCPT03-17	16-Oct-03	15 cm^2 147	17.20	7	Seismic Hole
CPT03-18	16-Oct-03	15 cm^2 147	17.90	11	
CPT03-19	17-Oct-03	15 cm^2 147	21.52	10	
CPT03-20	17-Oct-03	15 cm^2 147	4.40	N/A	Drilled out to 10 and 15 feet.
CPT03-20X	17-Oct-03	15 cm^2 147	3.78	4	
SCPT03-21	18-Oct-03	15 cm^2 147	26.17	10	Seismic Hole
CPT03-22	17-Oct-03	15 cm^2 147	20.17	12	
CPT03-23	17-Oct-03	15 cm^2 147	7.35	4	Drilled out to 5 feet.
CPT03-24	17-Oct-03	15 cm^2 147	17.90	3	Drilled out to 5 feet.
SCPT03-25	17-Oct-03	15 cm^2 147	11.32	3	Seismic Hole
CPT03-26	17-Oct-03	15 cm^2 147	10.95	3	
CPT03-27	17-Oct-03	15 cm^2 147	11.07	2	
CPT03-28	17-Oct-03	15 cm^2 147	10.23	3	
CPT03-29	17-Oct-03	15 cm^2 147	9.62	4	
CPT03-30	18-Oct-03	15 cm^2 147	14.45	11	
CPT03-31	18-Oct-03	15 cm^2 147	4.10	4	
SCPT03-32	18-Oct-03	15 cm^2 147	23.80	2	Seismic Hole
CPT03-33	18-Oct-03	15 cm^2 147	21.73	9	

4.2 CPT Sounding Results

Logs providing a record of the measured CPT data for all 36 holes are included as Appendix III of this report.

The estimated water table depth as presented in Table 4.2 has been determined through a review of the pore pressure dissipations data (Contec Investigations Ltd. Report 03-210) and based on the measured cone pore pressure profile for each CPT.

A summary of the seismic shear wave velocity profiles and tables of V_s with depth data are included in Appendix IV.

4.3 Shear Modulus (Shear Wave Velocity)

The shear wave velocity of the tailings was measured using vertical seismic profiling in which the travel time is measured for a seismic wave to cross a known distance. These measurements used an adapted CPT for the seismic receiver, and were carried out by Conetec as part of the testing. The measured data is contained in the Conetec report and is summarized here as it can be used to infer tailings state directly and is also necessary to properly evaluate the CPT data.

The shear modulus (G) is directly related to the shear wave velocity through the relationship $G = \rho V_s^2$ where ρ is the soil density. Figure 4.2 shows the measured V_s data from the site converted to G using the estimated soil density and plotted against depth.

The shear modulus of soil is usually strongly affected by the soils stress level. As a first step in assessing the trend in the tailings stiffness, the G values were also plotted against the estimated vertical effective stress in the middle of each respective test zone as shown on Figure 4.3. Results from all five soundings are shown on this plot. There is a trend for modulus to increase linearly with stress, which is a clay-like behaviour and can be represented as the soil has constant rigidity I_r where rigidity is modulus divided by stress level. A trend line for constant rigidity is shown, and certainly captures much of the effect for increasing modulus with depth. However, there remains substantial scatter in the data.

Apart from stress level, shear modulus is influenced by soil type and soil density (sands and silts can have a range of densities at any given stress level). Because deposition conditions were similar, as a first approximation the effect of density variation can be neglected. The data shown on Figure 4.3 was therefore sorted by soil type. For each test, a range of soil type index (I_c, see Section 5.1.2 below) values were estimated from the CPT data and corresponding to the best estimate and the credible upper and lower limits for the zone in which shear wave velocity was measured. These I_c values were then used to sieve the G data by soil type. Three distinct groupings of data were found as shown on Figure 4.4. These plots support the following average relationships:

- Tailings sands: $G = 109\left(\frac{\bar{\sigma}_v}{100kPa}\right)^{0.53} : MPa$ [4.1a]

- Tailings silts: $G = 90\left(\frac{\bar{\sigma}_v}{100kPa}\right)^{0.90} : MPa$ [4.1b]

- Very loose slimes: $G = 56\left(\frac{\bar{\sigma}_v}{100kPa}\right)^{1.0} : MPa$ [4.1c]

The tailings sands comprises data for CPT behaviour Zone 5 (sands, typically less than 10% silt) and Zone 6 (sandy silts). The exponent in the power law relation for G is very typical for these types of soils. The scatter in the data at higher stress levels is usual even in very tightly controlled laboratory testing; at lower stress levels it suggests that there may be a neglected density effect.

The tailings silts of the soil behaviour Zone 4 differ primarily from the sands in the exponent on the power law relationship. They are also about 20% softer for the range of stress levels in the Rose Creek impoundment.

The very loose slimes plot in the clays to organic soils region of the soil behaviour chart (Zones 2 and 3). These soils are a further step softer than the tailings silts and show an exponent of unity. This is exactly the behaviour expected of idealized soft clays.

The performance of the above equations was checked by comparing the trend line predicted using them with the measured insitu data. The results are presented on Figure 4.5, and show generally good matches throughout the profiles.

4.4 Shear Wave Velocity and Void Ratio

During the CPT investigation, one auger hole was drilled to a depth of 15 m next to SCPT03-21 and samples of the tailings obtained at 1 m intervals. The samples were collected directly off the augers and sealed in plastic bags. The samples were taken to the Golder Burnaby laboratory where the moisture contents were determined. Following moisture content determination, all samples were combined and this was used to measure the specific gravity. If the tailings samples were saturated and remained saturated during sampling, the void ratio could be determined by $e = w * G_s$, where w is moisture content and G_s is specific gravity of soils. The laboratory measured a $G_s = 3.61$ and this value was used to calculate the void ratio in Table 4.2, the table .

Table 4.2 presenting a summary of V_s, void ratio from moisture content and the estimated mean stress with depth.

Table 4.2: Void Ratio, Shear Wave Velocity and Effective Stress at SCPT03-21

Depth (m)	Sample Moisture Content, (%)	Void ratio, e [1]	Estimated mean effective stress, p'(kPa)	SCPT03-21 Measured Vs, (m/s)
1.52	23.7	0.86	33	181
2.51	10.6	0.38	41	168
4.50	25.9	0.93	58	154
5.49	34.0	1.23	67	169
6.48	30.2	1.09	75	177
7.47	26.2	0.95	84	181
8.46	32.0	1.16	92	191
9.45	30.6	1.11	101	204
10.44	28.4	1.02	110	216
11.43	35.8	1.29	118	242
12.42	27.8	1.00	127	283
13.41	18.5	0.67	135	250
14.40	29.2	1.05	144	258
15.24	26.8	0.97	151	228

NOTE: 1. Assuming a constant $G_s = 3.61$

5.0 TAILINGS CHARACTERIZATION

5.1 Methodology

5.1.1 General

The CPT measures the soil response to the displacement imposed by the penetrometer. This soil response is not a direct measure of the soils properties or state, which are the parameters of interest. To recover information on the soil, the CPT data must be processed (often called “interpretation”, although the correct mathematical description is “inversion”). There are a variety of algorithms available for this, all of which include various assumptions. The assumptions can significantly affect the results obtained.

A feature of all penetration tests is that soil resistance increases with depth even for constant soil properties. This makes the insitu stress one of the key considerations in processing CPT data. Two alternative approaches to allowing for the stress level have developed. Within North American practice, it has become common to “correct” the measured data to a reference stress level. This reference stress level approach maps the measured data to what would have been measured at the reference stress level if nothing else were changed. The reference stress approach is usually denoted by the subscript “1”, so that q_{t1} is the result obtained from mapping measured q_t data to the reference stress level. By convention, the reference stress level is a vertical effective stress of 100 kPa.

The alternative approach to the reference stress level is to work in a conventional framework of applied mechanics and use dimensionless parameters. This is predominantly a European approach. A dimensionless approach allows scaling through the laws of mechanics and avoids dubious “corrections.” In this situation, the q_t data is reduced to the dimensionless number Q where $Q = (q_t - \sigma_v)/\bar{\sigma}_v$. The corresponding dimensionless form for the CPT sleeve friction is $F = f_s/(q_t - \sigma_v)$ and the piezometric data is $B_q = (u_c - u_0)/(q_t - \sigma_v)$.

Algorithms to process CPT data to recover engineering parameters of interest have been based on both the reference stress and the dimensionless approaches. Generally, algorithms based on the reference stress approach have relied on correlations to a limited experimental or experience data base while those based on a dimensionless approach have often been based on idealized theory. But, in either case, getting the correct vertical effective stress profile at the sounding location is an essential starting point. This raises a difficulty in the case of tailings as: (1) tailings are often very compressible, which makes the density change with depth because of their compression under self-weight; and, (2) tailings may contain metallic ore particles giving a greater or variable specific gravity for the soil than more common natural soils. This assessment has relied on some site specific measured specific gravity values and measured compressibility values, and which have been assumed to apply everywhere in the impoundment as average values.

Some algorithms directly relate anticipated behaviour, for example resistance to earthquake induced liquefaction, to the CPT resistance. However, a number of adjustment factors are involved and these are related to the soil type. There is also substantial uncertainty about the adjustment for stress level effects.

The approach followed in the present work was to identify a range of candidate algorithms. These algorithms are used directly, but we have also considered the effect on them of changing from natural soils (for which most algorithms were developed or calibrated to) to the tailings material at the Rose Creek Tailings impoundment. Primarily we have been concerned with three factors: the constant volume friction angle of tailings; the high compressibility of tailings; and the high density of the lead-zinc tailings.

5.1.2 Alternative Methodologies for Soil Type

The CPT was first developed as a stratigraphic profiling tool, and over the years experience has accumulated on how soil type is related to the CPT measurements. Figure 5.1 shows a chart that relates soil type to the dimensionless tip resistance Q and the dimensionless friction on the side of the CPT, F. This chart was first proposed some 20 years ago (Robertson 1990) and is now widely used; our own checks of soil type versus CPT data have found this chart to be a sound basis for estimating soil type in natural soils.

One approach in using Figure 5.1 is to classify the soil by the zone type of the chart. This is the approach used by Conetec in their plots of the CPT soundings from the site (Conetec Investigations Ltd. 03-210).

The piezocone also provides data on the piezometric response of the soil, and this data can assist in classifying the soil since overconsolidated clays commonly have strongly negative pressures during CPT sounding whilst soft normally consolidated clays show positive excess pore pressures. Robertson (1990) suggested that classification zones could be also inferred form the CPT's B_q data as also illustrated on Figure 5.1 using a chart of Q vs B_q.

A limitation of soil type classification is that soil type varies continuously. For example a silty sand as described on the Q-F or Q-B_q chart can have anything from 10% to 30% silt content. An index of soil behaviour type I_c was introduced to combine the CPT data into a single measure. This index does not necessarily exactly correspond to a geological classification based on particle sizes, as I_c is primarily a measure of the soil's drainage, strength and compressibility. Rather, the soil type index is a standard measure derived from trends in many thousands of CPT soundings.

Two measures of soil type index have been proposed to represent the smooth relationship between soil type and CPT data. Both output the same descriptor, denoted by I_c, but the meaning of a particular numerical value depends on which algorithm is used. The two algorithms are:

- Been & Jefferies (1992); and
- Robertson & Wride (1998).

Broadly, the algorithm of Been & Jefferies uses all three CPT measurements and transforms the two graphs of Figure 5.1 to a single plot of *Q(1-Bq)+1 vs F.* This helps distinguish between clays and silts. Using the parameter group *Q(1-Bq)* to normalize drained and undrained CPT data was independently suggested by Been et al (1988) and Houlsby (1988). Houlsby (1988) noted that the parameter group *Q(1-Bq)+1* corresponded to a simplification of the methodology for measuring overconsolidation of clay developed by Konrad & Law (1987), and that it is equivalent to normalizing CPT resistance by the vertical effective stress established during CPT penetration as a consequence of CPT induced excess pore pressure. This observation lead to a preference for *Q(1-Bq)+1 vs F* as the way to unifying the soil type evaluation plots, and the unified framework is shown on Figure 5.2 (after Been & Jefferies, 1992). The corresponding algorithm for I_c is:

$$I_c = \sqrt{\{3 - \log(Q(1-B_q)+1)\}^2 + \{1.5 + 1.3\log(F)\}^2} \qquad [5.1a]$$

The algorithm of Robertson & Wride (1998) reverts to only using *Q-F* data. The algorithm simply provides an approximation to the zone boundaries shown on the *Q vs F* soil classification chart on Figure 5.1. The algorithm is:

$$I_c = \sqrt{(3.47 - \log(Q))^2 + (1.22 + \log(F))^2} \qquad [5.1b]$$

The algorithm of Robertson & Wride is more complex in that it uses a variable exponent to allow for the effect of stress level and which depends on soil type; the proposed role of this last algorithm is for estimating the cyclic strength of soil ("CRR") from penetration data rather than soil type evaluation itself.

Table 5.1 below summarizes the indicated soil type in terms of I_c values. In the case of the Been & Jefferies algorithm, these I_c values are compared with the soil zone boundaries of the original classification chart on Figure 5.2.

Table 5.1: Values of Ic Corresponding to Soil Type Zones Shown on Figure 5.1

Soil Classification	Chart Zone (Figure 5.1)	Been & Jefferies (equation 5.1a)	Robertson & Wride (equation 5.1b)
Gravelly sands	7	$I_C < 1.25$	$I_C < 1.31$
Sands: clean to silty	6	$1.25 < I_C < 1.90$	$1.31 < I_C < 2.05$
Silty sand to sandy silt	5	$1.90 < I_C < 2.54$	$2.05 < I_C < 2.60$
Clayey silt to silty clay	4	$2.54 < I_C < 2.82$	$2.60 < I_C < 2.95$
Clays	3	$2.82 < I_C < 3.22$	$2.95 < I_C < 3.60$
Organic soils (peats)	2	$3.22 < I_C$	

The reliability of the relationship between soil type was investigated by examining two CPT profiles in detail and comparing the results of I_c calculated by both equations [5.1a] and [5.1b]. The chosen CPTs were SCP03-21 and SCPT03-32, these being selected as having representative sections of each the very soft slimes and of the denser sand deposits. For each CPT sounding the computed values of I_c from the two methods have been plotted, in each case showing the soil type boundaries as gridlines. The I_c calculation method proposed by Robertson and Wride (1998) was not included in this comparison since this algorithm is used for estimating the cyclic resistance ratio. Figure 5.3 shows the results for SCPT03-21 and Figure 5.4 show the results for SCPT03-32. For both CPTs, for low values of I_c, the two methods produce nearly identical classifications of soil type. However, the inclusion of B_q in Been & Jefferies algorithm produces different results for strata in which pore pressure is significant.

Looking at the tailings slimes zone on SCPT03-21 between 3.6 m and 11.7 m depths, the Robertson & Fear algorithm identifies the soil as consistently and uniformly slightly softer than the upper limit of the clay range (Zone 3). Figure 5.5 shows the data from this same zone but now plotted on Robertson's Q versus B_q chart; the majority of the data lies in clay zone, but very near the border with organic soils. This mismatch in the classification is not unusual when relying on two separate charts and has been commented on in the literature. Interestingly, the Been & Jefferies algorithm results in the soil being classified as uniformly right at the lower expected boundary for clay. Given that the tailings slimes are hydraulically deposited and not geologically aged, this is entirely consistent with the geological idealization for the softest of clays. This view is reinforced by the data from 23.6 m to 25.7 m on the same sounding.

The second sounding considered, SCPT03-32, shows a similar result. Between 2.6 m and 4.2 m depth, the Robertson & Fear algorithm suggest a slightly more silty behaviour than the Been & Jefferies algorithm. But, the B_q data reveals a rather soft clay with the data clustering around $B_q \approx 0.7$. Further down the profile, between 19.3 m and 23.7 m, both algorithms indicate soft clay behaviour but with the Been & Jefferies algorithm suggesting that the behaviour was in the region of compressible organic-like soils. The B_q data reveals a soil behaviour with extremely high values.

Conformation of the view that the tailings slimes are comparable to very soft clays comes from the B_q values reported in the literature for the soft and sensitive Champlain clays of Eastern Canada. Figure 5.6 shows results on CPT soundings in five such clays as reported by Konrad & Law; the B_q values or these near-normally consolidated sensitive clays are uniform with depth the values are B_q<0.8. The B_q data alone for Rose Creek slimes indicates that the slimes are softer and more contractive than these sensitive clays.

Based on the above results, the Been & Jefferies algorithm for I_c (equation [5.1a]) is adopted as an appropriate index for the soil type classification of the Rose Creek tailings.

5.1.3 Alternative Methodologies for Soil State from CPT

Because any soil can exist over a range of density and because loose soil behaves differently to dense soil, as found in particular with the Rose Creek tailings and discussed in Section 2 above, a measure of soil state is needed. The state parameter ψ is adopted here for this purpose. The state parameter is the difference between the soils current and critical void ratio at the same pressure, as illustrated on Figure 5.7.

The state parameter offers three key advantages over relative density for silt tailings: 1) the strength (and dilatancy) behaviour of soil is a near unique function of ψ and is not affected by stress level; 2) similarly, the soil's behaviour is not affected by variations in gradation when the behaviour is characterized by ψ; 3) ψ avoids using the maximum and minimum void ratios, which are difficult to measure with silty soils and in all likelihood misleading (and which makes estimates of relative density unreliable).

Of the other alternatives to ψ, the relative dilatancy index I_B (Bolton, 1986) avoids error (2) in the previous paragraph but remains vulnerable to errors in estimating the maximum and minimum reference values. There is also no established methodology with which to measure I_B insitu. A more interesting alternative, or more accurately complement, to ψ is the soil's elastic shear modulus. This was discussed in the previous section.

The methodology for determining ψ was developed first for sands, in which the CPT is *drained*. However, much of the penetration soundings at Rose Creek are *undrained*. The effect of drainage conditions on the evaluation of the penetration test data will be presented after describing the background on the drained methodology, as the drained conditions comprise a coherent methodology that is used in part.

The drained methodology was initially based on normalized data from various calibration chamber testing programs. The calibration chamber is essentially a large cell, often about

1.2 m in diameter. Chamber studies involve carefully placing sand in the test chamber so that it is, to the maximum practical extent, of constant and known density. Then, a desired stress regime is applied with vertical and radial stresses independently controlled so that the effect of geostatic stress ratio can be evaluated. The CPT is pushed into the sand in the calibration chamber, just as in the field, with CPT data recorded in the usual way. Testing over a range of densities and applying a range of confining stress levels allows development a mapping between the CPT penetration resistance q_c, initial confining stress, geostatic stress ratio, and state (or relative density). However, because setting up a sample of sand in the calibration chamber is not a trivial undertaking – over 2 tonnes of sand is involved – the number of such programs is small.

Fifteen years ago, Golder Associates undertook a systematic evaluation of the then available data from calibration chamber tests on the CPT. A sample of sand was obtained from each of the reported chamber programs, and was tested to determine the respective CSLs. This then allowed the CPT data to be expressed in terms of ψ. The results were reported in two papers (Been et al, 1986, 1987) and showed that the CPT responded to soil state according to the simple equation:

$$Q = \frac{3}{1+2K_0}\, k \, \exp(-m\psi) \qquad [5.2]$$

where k,m are two soil specific coefficients and K_0 is the geostatic stress ratio (ratio of horizontal to vertical insitu effective stresses).

Equation [5.2] is used to determine ψ from CPT data by inverting the equation to the form:

$$\psi = -\frac{1}{m}\ln\left(\frac{1+2K_0}{3}\,\frac{Q}{k}\right) \qquad [5.3]$$

It is emphasized that [5.3] must be used for drained penetration. This is checked by looking at the B_q value. At least part of the penetration test profiles at Rose Creek are drained, so that [5.3] may be used for this part of the site. The key issue then becomes what are the appropriate k,m values.

Detailed finite element analysis (Shuttle & Jefferies, 1998) were undertaken to investigate the relationship between the coefficients k,m and the fundamental mechanical properties of the soil. The numerical simulations have shown that the CPT parameters k,m sensibly controlled by four dimensionless soil properties: the shear rigidity, I_r; the critical friction ratio, M; a dilatancy parameter N, and the plastic hardening modulus, H. Other factors have only small effect and can be neglected. The general form of the relationship takes the form:

$$k = (f_1(I_r)\, f_2(M)\, f_3(N)\, f_4(H)\, f_5(\lambda)\, f_6(\nu))^{1.45} \qquad [5.4a]$$

$$m = 1.45\, f_7(I_r)\, f_8(M)\, f_9(N)\, f_{10}(H)\, f_{11}(\lambda)\, f_{12}(\nu) \qquad [5.4b]$$

where the functions $f_1 - f_{12}$ are simple algebraic expressions as presented in Shuttle and Jefferies (1998).

The laboratory testing (Section 3 above) has provided estimates of the soil properties to be used in equation [5.4]. As discussed the estimated properties are as: $M = 1.2$ and $\lambda = 0.08$ for the coarser sample and which is representative of the tailings showing drained CPT penetration. The properties N and H were not measured in the limited testing program undertaken. N is estimated as 0.3, this being an average value for sandy soils. H is taken as ranging from $H = 70$ at $\psi = 0$ to $H = 200$ at $\psi = -0.2$; these are typical sand values scaled down in by the ratio of measured λ to typical λ of sands. There are theoretical grounds for treating H and λ as correlated parameters, although it is preferable to measure H directly.

Based on this testing, use of equations [5.4] gives the plot of k, m values presented as Figure 5.8, and which have been used for processing the CPT data when drained conditions prevailed. The measured I_r insitu based on the trend equation [4.1] is used with Figure 5.8.

When *undrained* conditions prevail, the approach must be modified to allow for the excess pore water pressures around the CPT (and these were very large in the softer tailings). To date, for silts, there is nothing comparable to the sand calibration chamber tests of the CPT nor have any numerical simulations comparable to those of Jefferies & Shuttle been done. However, two approximate approaches have been put forward: Plewes et al., (1992) and Been & Jefferies (1992). Both are based on the same methodology, and they differ only in one of the assumptions.

The starting point is the observation that CPT behaviour from sands through to clays can account for drained through to undrained conditions by changing the normalized CPT resistance from Q to $Q(1-Bq)+1$. This is exactly the same group of dimensionless CPT parameters as was adopted for the unification of the classification charts. It is equivalent to normalizing CPT resistance by the vertical effective stress using the pore pressure regime established during CPT penetration, rather than that existing prior to testing. During drained penetration with $B_q = 0$ the approach becomes identical to the framework presented above. Been et al (1988) suggested that Equation [5.3] would continues to be valid, even for undrained penetration of clays, provided that the coefficients k,m were redefined to effective values k^*, m^*:

$$Q(1-B_q)=\frac{3}{1+2K_0}\,k^*\exp(-m^*\psi) \qquad [5.5]$$

Note that the "+1" has been dropped in going from simplified theory to the proposed equation [5.5]. Figure 5.9(a) shows the data on which the Been et al suggestion was based. In the case of sands, ψ is defined as presented earlier. In the case of clays, it was assumed that ψ was related to the logarithm of overconsolidation ratio (OCR). This is a little simplified, but is not unreasonable to get a first estimate of a unified framework. The well known Cam Clay and Modified Cam Clay have explicit relationships between ψ and ln(OCR), and that of Modified Cam Clay was adopted in developing Figure 5.9(a).

It was then further assumed that the principle difference between the various sands and clays tested was the two critical state parameters λ, M. This allowed the data shown on Figure 5.9(a) to be presented in terms of the inversion coefficients k^*, m^* and soil type as represented by the slope of the CSL λ. Figure 5.9(b) shows data illustrating this point and which support the relationships:

$$k^*/M = 3 + 0.85/\lambda_{10} \qquad [5.6a]$$

$$m^* = 11.9 - 13.3\,\lambda_{10} \qquad [5.6b]$$

Soil compressibility λ is used as an index because there is presently insufficient experience or data to use H directly (which would be preferable) – hence reliance is placed on H being related to λ, which is correct for the well known Cam Clay constitutive model.

Equation [5.5] has no allowance for the effect of soil rigidity I_r – the equation was developed before the effect of I_r on the CPT was understood. As a consequence, [5.5] may suffer from bias with stress level. It should be most accurate for a vertical effective stress of about 100 kPa.

The next step is to estimate λ from the CPT data, and here the approach of Plewes et al., (1992) differs from Been & Jefferies (1992). In the case of Plewes et al., an inverse scaling between λ and F was suggested as a first approximation, and Figure 5.10 shows the data to support this. The relationship suggested was:

$$\lambda = F/10 \quad (\textit{for F as \%}) \qquad [5.7]$$

Been & Jefferies (1992) related λ to the soil type index I_c, through the equation:

$$1/\lambda = 34 - 10\,I_c \qquad [5.8]$$

Experience suggests that the Plewes et al approach provides the closer correspondence to independent estimates of λ and it is used here. This view is corroborated by the measured data from Rose Creek tailings which fits the trend of Equation [5.7] and is presented on Figure 5.10.

For the silt tailings, the penetration is undrained and therefore the k* value from [5.5] cannot be compared with that determined using [5.4]. However, the CPT sounding in the sandier part of the tailings were drained and this allows a basis for comparison between the two methods. A site specific calibration was developed as follows, and is required because the Plewes method is intended for "screening" level assessments.

- Laboratory testing of the coarse tailings sample resulted in $\lambda_{10} = 0.08$ and $M_{tc} = 1.2$ from which $k = 27$, $m = 9.5$ are estimated from the drained finite element simulation at $I_r = 400$ (which is typical of the implicit stiffness in the Plewes et al data);
- Similarly, the same properties for the coarse tailings sample used in equations [5.6a,b] give $k^*=16$ and $m^* =10.8$;
- During drained penetration, $B_q=0$, and thus $k^*=k$ and $m^*=m$ in this situation, an equivalence requiring site-specific calibration factors for k^*, m^* of respectively $\alpha=27/16= 1.7$ and $\beta=9.5/10.8= 0.88$;
- For $B_q\neq0$, which is partially to undrained penetration, these site specific calibration factors α, β established under drained conditions were assumed to apply.

The estimated site calibration for k^* m^* are plotted on Figure 5.11 against F, respectively, and with two relationships indicated. The effect of the site specific calibration is to infer more contractive behaviour for the fine tailings than would otherwise have been the case. As a further check on the calibration, discussed later, shows that the undrained strength brittleness from peak to residual based on water contents insitu and the laboratory CSL closely matches the brittleness estimated from the calibrated CPT methodology.

5.2 Summary of Methodology Adopted

Based on the considerations presented above, the data processing protocol adopted is as follows:

- use $Q(1-Bq)+1$ to calculate I_c;
- Estimate G using I_c and $\bar{\sigma}_v$;
- Estimate λ_{10} from F;
- Use k^*, m^* from Figure 5.11;
- Estimate ψ from

$$\psi = -\ln[(Q(1-B_q)+1)/k^*]/m^* \qquad [5.9]$$

This applies to both drained and undrained CPT penetration.

5.3 Tailings Type

CPT data has been plotted with regard to location in the tailings impoundment using a series of cross sections. Six sections denoted A-A' through to F-F' have been selected to present the CPT data. Figure 4.1 presents the locations of these sections. Figures 5.12 to 5.17 presents the cone penetration resistance data, measured cone pore pressure and soil behaviour index I_c with depth for each CPT in the respective section. Also shown on the section is an estimated outlines of the impoundment structures.

The soil type index has been reduced to a coloured bar on Figure 5.12 to 5.17 to illustrate the interlayering of the coarse and slimes fractions in the tailings. The estimated location of the water table at each CPT sounding at the date of the sounding is also shown.

5.4 Comparison of Tailings State/Type with Deposition History

Section 2.2 discusses the tailings deposition history through a review of the air photos of facility over time. This section reviews the CPT characterizations in light of the tailings deposition. Figure 4.1 presents the locations of the CPT soundings.

5.4.1 Original Tailings Area

At total of seven CPT soundings were carried out in the Original Tailings Area, five located just upstream of the existing original dam crest, and two located some distance upstream of the dam crest.

CPT03-01 was located upstream of the original dam and east from the main tailings deposition fan located near the old Faro Creek Channel. After a thin relatively coarse and compact tailing crest, the CPT profile indicated a relatively loose and fine tailings profile down to the base of the tailings. This is consistent with this sounding being located far from the deposition point.

CPT03-07 was located upstream of original dam and generally closer to the main tailings deposition fan located near the old Faro Creek channel. The CPT profile indicates a relatively compact and interbedded coarser and fine tailings over the full depth, which is consistent with being located nearer to the discharge point.

CPT03- 02, 03,04, and 05 were all located immediately upstream of the existing original dam crest. Generally, all these CPT profiles showed more compact and coarse tailings in the initial depths, grading to looser zones of layered finer and coarse tailings towards the base depths. These profiles are consistent with the upstream construction method used for this dam raising.

CPT03-06 was located near the original dam crest and closest to the main tailings deposition fan located near the old Faro Creek Channel. The initial profile indicated mix fine and coarse tailings over compact coarse tailings, consistent with this area being the main fan over most of the early life of the original area.

5.4.2 Secondary Tailings Area

CPT03-08, 09, 10, 11 11B, 12 and 12B were all located in the secondary tailings area, immediately downstream toe of the above tailing portion of the original dam.

All these CPT profiles indicated generally, compact relatively coarser tailings, with some finer tailings towards the base of the CPT 12B which is located farthest to the east in this area. Again this is consistent with tailings deposition being carried out from the toe of the original dam into the secondary impoundment.

CPT03-13, 14, 15, 16 and 17 are all located in the east secondary tailings area away from the original dam. CPT's 13 and 16 are both located more in the center of this area and their profiles indicate mixed zones of loose coarse and fine tailings, both being very loose and fine at the base. CPT's 14, 15 and 17 are each located just upstream of the east dam, and these profiles indicated a mixed interbedded tailings with both loose coarse and fine zones. This is consistent with the upstream construction used for the east dam. In the east area of the secondary impoundment, the CPT profiles indicated a looser fine zone at the base of these consistent with this east area being a potential backwater and far from deposition point.

CPT03- 18 and 19 are located at southern side of the west secondary area. A range of loose to compact, coarser tailings were indicated over the full depth of 18 and upper depths in 19, indicating alignment with a possible deposition fan.

CPT03- 21, 22 and 30 all located upstream of the west Secondary dam each indicated loose and fine to very fine tailings for full depth. The lower depth of CPT03-19 also indicated these loose fine tailings. Of note, this area contains some of the greatest depths of tailings in the impoundment.

CPT-03-20, 31, 20X were all located very close to the upstream crest of the Secondary dam. The upper raises of this dam are compacted tailings. Each of these there profiles exceeded the pushing capacity of the drill rig, even with anchoring, and after drillout of various thicknesses, produced CPT profiles indicated a dense tailings.

5.4.3 Intermediate Tailings Area

CPT03- 23 to 29 area all located in the area downstream of the west Secondary Dam, and generally indicated coarse tailings zones with limited interbedded zones of loose tailings.

CPT-03-32 and SCPT03-33 are located as close to the Intermediate Pond on the beach as access would allow. Both indicated highly interbedded tailings of coarse and very loose finer tailings.

5.5 Summary of State with Depositional Environment

Using equation [5.9], Ψ with depth was calculated for each CPT location. Positive Ψ zones are of interest as positive ψ identifies areas of potentially contractive tailings. Figure 5.18 shows a plan of the tailings impoundment, all the CPT locations, and at each CPT location the total percent of the CPT trace which was calculated to have a + Ψ value. The higher the value on this figure, the more likely the tailings in the area of the CPT could be susceptible to flowslide.

This + Ψ value at each CPT location was compared to the expected tailings deposition patterns and confirms what was expected. For example, immediately upstream of the Original and Secondary Dams the CPT holes show a low fraction of + Ψ tailings. Only at CPT 03-01 located well back form the Original Dam crest was a zone of high $+\Psi$ noted. On the downstream side of the original dam, which is the most upstream area of the secondary impoundment, a low number of low $+\Psi$ CPT holes were noted. In the central area upstream of the west Secondary Dam and in most all the upstream of the east Secondary Dam were large zones of high $+\Psi$ noted.

Into the Intermediate impoundment, a highly mixed tailings deposition environment was noted on the basis of $+\Psi$ values. As expected, the two CPT closest to the intermediate dam (generally furthest from the deposition point) were found to be high $+\Psi$ CPT's. At the foot of the secondary dam, a zone of high $+\Psi$ was noted. Variable $+\Psi$ CPT soundings were noted in the area downstream of the Secondary Dam.

6.0 LIQUEFACTION ASSESSMENT METHODOLOGY

Liquefaction assessments are conventionally divided into two stages: (1) assessment of whether initial liquefaction will arise during the design earthquake scenario(s); and, (2) assessment of what might happen should the soils liquefy. The first type of assessment is often referred to as a "triggering" analysis with the second looking into the potential for flowslides versus limited deformation. This protocol has been followed in the present study.

6.1 Seismic Exposure

Seismic exposure for the site was obtained from Atkinson (2003). This "Draft Report: Seismic Hazard Assessment for Faro, YK" presents a site specific seismic hazards assessment for the Faro mine site. The report indicates a maximum credible earthquake (MCE) for Faro of approximately M7 at 10 to 20 km depth corresponding to the 0.0001 p.a. spectrum which would result in peak ground acceleration (PGA) in the range from a median of 0.3 g to a mean of 0.5 g.

The seismic stability assessment of the CPT data for the Rose Creek tailings has been carried out using a M7 event with a PGA of 0.5 g following procedures described by Seed and Idriss (1971). The Cyclic Stress Ratio (CSR) with depth was determined as follows.

$$CSR = \frac{\tau_{av}}{\sigma'_{vo}} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d$$

The resistance to initial liquefaction, termed the Cyclic Resistance Ratio (CRR), was calculated by either the NCEER method or using the state parameter approach. Both approaches were all scaled from the reference M7.5 event to an M7.0.

6.2 NCEER Method for Initial Liquefaction

6.3 Description

The National Center for Earthquake Engineering Research (NCEER) liquefaction assessment method has developed over the past thirty years or so. Because of difficulties in cyclic testing of soils under relevant conditions to earthquake loading, and the absence of appropriate constitutive (stress-strain) models thirty years ago, the concept was to observe where liquefaction occurred in an earthquake and where it did not. These observation were then related to the estimated cyclic shear stress experienced by that ground and the prior state of that ground. Field evidence of liquefaction generally consisted of surficial sand boils, ground fissures or lateral spreads. Data were collected mostly from flat to no more than gently sloping sites and which where underlain by Hollocene alluvial or fluvial sediments no deeper than 15 m. The earthquake resisting strength measure adopted was the dimensionless ratio $\tau_{cyc}/\bar{\sigma}_{v0}$, which is now commonly called the cyclic resistance ratio CRR, and where τ_{cyc} is the equivalent uniform cyclic shear stress causing liquefaction and $\bar{\sigma}_{v0}$ is the initial vertical effective stress before the earthquake. The initial insitu state measure adopted was the SPT resistance, primarily because of its prevalence worldwide in the then available case history record. It is common now to replace the SPT with the much more accurate and repeatable CPT.

Initial work developing this framework was by Prof. Seed and his colleagues at Berkeley, but this has been extended by other workers over the years. The approach was subject to a workshop arranged by the NCEER in 1996 to develop a consensus on the methodology amongst the workers contributing to this form of liquefaction assessment, in particular on the various coefficients and adjustment factors. In the present context, the CPT-based methodology suggested by Robertson & Wride (1998) is relevant and is the basis of what follows. Updates and commentary on the NCEER consensus have been suggested by Seed et al (2001), and these are also recognized. In some respects the NCEER method is the standard of practice today for liquefaction assessment. However, workers with opposing views/methods were not invited to the workshop so that the consensus falls short of a universally accepted framework. In particular, the NCEER methodology includes physically inconsistent relationships.

As noted, the basis of the method is to classify liquefaction case histories into whether or not liquefaction occurred. To produce a unified chart, data is adjusted from the actual earthquake ground motion that occurred at the site to that equivalent to a M7.5 earthquake. Similarly, the penetration resistance is adjusted to that equivalent to what might have been measured at an initial vertical effective stress of 100 kPa. The adjusted data then gives a chart such as that illustrated on Figure 6.1, and which presents a line demarking *liquefaction* from *no liquefaction* in terms of the resistance at M7.5 earthquake ($CRR_{7.5}$) to the CPT penetration resistance adjusted to standard conditions and called q_{c1N}.

The available CRR for the insitu conditions and the design (or actual) earthquake is given by:

$$CRR = CRR_{7.5}\, K_M\, K_\sigma\, K_\alpha \qquad [6.1]$$

Where $CRR_{7.5}$ is taken from Figure 6.1 using the adjusted measured CPT at the site as the input. The various *K* terms in equation [6.1] are to adjust from the standard chart back to the insitu conditions and are discussed below after first considering the methodology for getting q_{c1N} from measured q_c values.

Penetration resistance increases with depth with constant soil properties because of the increased stress in the soil. This makes penetration resistance on its own a poor indicator of soil state, and it is essential to include the effect of stress level on the measured resistance. The concept behind q_{c1N} is to allow for this stress level effect by multiplying the measured values by a factor, C_N that is itself a function of stress level.

The origin behind C_N was the premise that relative density was the fundamental parameter controlling sand behaviour. The first work recognizing the effect of stress level on the determination of relative density was a study by Gibbs & Holtz (1957). Subsequently Marcuson & Bieganousky (1977) reported on a reasonably comprehensive set of tests using the SPT in a relatively crude calibration chamber like arrangement (there was no provision for control or measurement of horizontal stress), and their results are illustrated in Figure 6.2. This figure shows that the C_N function is different for a relative density of 40% compared to 60% or 80% for the same sand, and it also differs between sands. Various approximations for C_N were proposed from curve-fitting the test data. However, the substantive step was a comparative study by Liao & Whitman (1986) who suggested that there was not a great deal of difference between the various functions proposed for C_N, and that a reasonable average relationship was the simple (and now widely used) equation:

$$C_N = \sqrt{\frac{\bar{\sigma}_v}{\sigma_{ref}}} \qquad [6.2]$$

Conventionally, the reference stress level is now taken as σ_{ref}= 100 kPa (which is approximately 1 tsf, and hence the "1" subscript).

Robertson & Wride (1998) suggested a variation on [6.2]. Their suggestion was based on the recognition that penetration tests in clay-like soils do not scale with the square root of stress as suggested by [6.2] but instead scale directly. This could be thought of as scaling by an exponent of 1.0 rather than 0.5 in [6.2]. Robertson & Wride then suggested that perhaps C_N should vary with soil type, an approach following Olson (1994). Their suggested algorithm was for C_N to use an exponent of either 0.5, 0.75, or 1.0 and depending on the value of I_c according to their revision of that formula. The flowchart for their algorithm is shown on Figure 6.3.

The Robertson & Wride algorithm also includes a further adjustment factor that they refer to as K_c. Penetration resistance shows a dependence on soil type – very high resistances can arise in dense sand but are an order of magnitude less in dense very overconsolidated clay. In the case of liquefiable soils, this has been recognized by relating the actual insitu penetration resistance to an "equivalent clean sand" value using an adjustment based on silt content. Robertson & Wride suggested that a factor K_c be used so that the penetration resistance to be used in relating Figure 6.1 to the site conditions is:

$$q_{c1N,CS} = K_c \ q_{c1N} \quad [6.3]$$

Robertson & Wride suggested a K_c relationship based on their revision of the soil type index I_c (discussed earlier), illustrated on Figure 6.4. As can be seen, K_c becomes asymptotically large as $I_c \Rightarrow 2.6$. Robertson & Wride took this as evidence that liquefaction would not arise for silts and clays, which is consistent with the NCEER method.

K_M is the earthquake magnitude adjustment factor. Youd & Noble (1997) have indicated a range of relationships determined by various workers, summarized on Figure 6.5. Magnitude scaling compensates for the differing number of significant cycles in an earthquake, as larger earthquakes tend to have longer duration of shaking. This is important because soil behaviour depends on the number of cycles as much as the cyclic stress ratio. However, as can be seen from Figure 6.5, there is a rather wide divergence of suggestions on how K_M relates to the design earthquake. This does not affect the present analysis much because the design earthquake for Rose Creek is a M7 event, which is very close to the reference M7.5 conditions and only requires small adjustments.

K_σ is the stress level adjustment factor and is the opposite in principle of the factor C_N used to adjust the measured CPT data. The data base of the liquefaction case histories is dominated by shallow sites (liquefaction at <15 m depth), and early cyclic triaxial testing had shown an effect of initial effective confining stress on liquefaction resistance. Hynes & Olsen (1999) provide the most recent summary on K_σ, which is presented in Figure 6.6. A wide range of behaviour is apparent. In some soils an order of magnitude increase in initial vertical effective stress has almost no effect on the CRR (clean sands) while in others (sandy silts) the CRR might reduce by 70%. Most curiously, Hynes & Olsen suggest that the different behaviours are related to initial density of the soil with dense soils having proportionately greater reduction in CRR for a given stress increase, despite the data indicating that soil type has a strong influence.

K_α is a factor introduced to capture the perceived effect of sloping ground, as the case history record is dominated by near level ground sites. The idea behind K_α comes from cyclic triaxial tests. If cyclic triaxial tests start from an anisotropic stress condition, then a larger CRR is obtained for any chosen number of cycles to liquefaction compared to starting from isotropic conditions. Seed (1983) extrapolated from this laboratory result to slope stability by noting that the anisotropic stress conditions in the triaxial sample could be expressed in terms of the dimensionless stress ratio $\alpha = \tau_{st} / \bar{\sigma}_{v0}$ and that the same ratio could be defined for a layer beneath sloping ground. Various workers have developed relationships between K_α and α, but conflicting trends are apparent. These conflicts have not been reconciled and present practice is that K_α=1 should be used in all situations (Youd et al, 2001).

6.3.1 Example of Predicted Liquefaction Resistance

Sounding SCPT03-21 was selected to evaluate the Robertson & Wride methodology for the Rose Creek tailings. The methodology described above was applied with the following choices:

- K_M = 1.2, based on the central trend through the NCEER workshop best estimate as presented on Figure 6.5.
- K_σ = 1.8352 – 0.1834Ln($\overline{\sigma}_v$), which is a close approximation to the Harder & Boulanger (1997) recommendation for sands at stress levels of less than 1000 ka.
- K_α = 1, as discussed earlier.

The results are shown on Figure 6.7. This figure shows the base q_t data versus depth together with profiles of other derived parameters with depth and which lead to the final profile of CRR versus depth.

An important aspect of the Robertson & Wride method in the context of tailings is the effect of silt content. The Robertson & Wride method assesses this affect using both the CPT friction ratio F and their version of the I_c index. In both cases, a criterion is applied as to whether either of these parameters indicates the potential for no liquefaction under any circumstances. The criterion for no liquefaction is if $I_c>2.6$ *and* $F>1$ then the soil is likely non-liquefiable. As can be seen from Figure 6.7 this criterion classifies about half of the CPT profile as non-liquefiable. In essence, only the tailings sands might be liquefiable according to the Robertson & Wride criterion.

For the sands encountered in the sounding, which lie predominantly between 12 m and 23 m depth, the measured q_t data typically gives $q_{c1N} \approx 50$. The subsequent effect of the "silt content correction" K_c is to nearly double this value to an "equivalent sand" value $q_{c1N,cs} \approx 90$. This last "normalized" resistance implies a cyclic resistance ratio of about CRR ≈ 0.18 for a M7 earthquake at the reference stress level. Applying the stress level adjustment factor K_σ gives a net of available CRR that falls in the range 0.1<CRR<0.13.

Figure 6.7 also shows the computed cyclic stress ratio. This substantially exceeds the available cyclic strength of the tailings sands at all points in the profile, and as such indicates the potential for cyclic mobility, lateral spreads and so forth.

6.3.2 Summary of Deficiencies in Method

The very high positive state parameter computed from the CPT data, and which is corroborated by the very high excess pore pressures (i.e., B_q values) during CPT penetration, are indicative of a weak soil that might normally be expected to be prone to liquefaction. Yet the opposite conclusion is reached by the Robertson & Wride method.

The conclusion that the substantial silt layers are not susceptible to triggering liquefaction is dominated by the adjustment factor for penetration resistance because of soil type, here using the Robertson & Wride K_c. However, the conclusion is one of the method, and the detail about the particular proposal for K_c does not affect the conclusion. The difficulty arises because the data base on which this liquefaction assessment method has developed is largely natural sands and because the NCEER framework is not based in mechanics.

On the first point, that Figure 6.1 and which underlies the method is based on largely sand data only, the NCEER methodology uses silt content to relate differences in behaviour by soil type. But, there is no basis for this. Even straightforward drained triaxial strength tests on various sands cannot be normalized (i.e., brought to a single unified trend) just using silt content as a behaviour index.

On the second point, the absence of a proper framework in mechanics, the difficulty is that equations are being fitted to trends but not related to fundamental soil properties (e.g., constant volume friction angle, compressibility, dilation angle). The framework used to plot trends used in the NCEER approach cannot be derived from even the simplest of idealizations for soils.

The counter argument is that the NCEER method states what is known to work based on experience. But this provides no assurances when dealing with high silt content tailings, which are outside the experience base of the method. A further approach should be considered.

6.4 State Parameter Approach

6.4.1 Description

The state parameter ψ was introduced earlier as an index expressing soil state. Its utility as an index lies in some soil behaviours being highly correlated to it, as illustrated for peak friction angle and peak dilatancy on Figure 6.8. The relationship between dilatancy and ψ is unsurprising as ψ is almost an identity of the maximum volumetric strain potential of the soil, regardless of soil type. The state parameter has become the universal choice for a material description in constitutive models that are density and stress level independent (e.g., Jefferies, 1993; Gajo and Wood, 1999; Li and Dafalias 2002, 1999; Papadimitriou *et al*, 2001).

The utility of ψ in the present context is that it offers an alternative to the silt content correction procedure of the NCEER method. With the state parameter approach, existing experience is cast in terms of ψ and this can then be used directly as the behaviours expected for the silt. The determination of ψ insitu has been presented above, so the task is to express the experience chart of Figure 6.1 in terms of ψ.

The post-liquefaction strength of the soil is also directly related to ψ, since post-liquefaction movements involve the soil shearing to its critical state.

This approach is now presented.

6.4.2 Resistance to Initial Liquefaction

The liquefaction/no-liquefaction experience chart shown as Figure 6.1 is based on reducing the case histories to a single characteristic normalized penetration resistance, $q_{c1N,cs}$. This penetration resistance can be regarded as directly equivalent to the dimensionless CPT resistance Q for a "typical" clean sand at a stress level of 100 kPa.

Reasonable soil properties for a "typical" clean sand at are M = 1.25, λ_{10} = 0.05, and K_0 = 0.7. Similarly, at 100 kPa vertical effective stress it is reasonable to take I_r = 600. Using these soil properties, CPT coefficients k = 31.5 and m = 9.3 were calculated. These coefficients were then used to calculate ψ for each of the case histories of Figure 6.1 and relying on the reported characteristic q_{c1Ncs} value reported by Robertson & Wride. The results are shown on Figure 6.9 (a).

A line has been drawn on Figure 6.9(a) to distinguish between the liquefaction and no-liquefaction cases. Because all other properties investigated to date are simply related to ψ this line has been drawn using an exponential function as a simple best-fit to the data. It is actually slightly conservative compared to the line of the NCEER plot, giving a lower CRR at both near critical and very dense states as illustrated on Figure 6.9(b). The liquefaction/no-liquefaction line on the state parameter plot is given by the equation:

$$CRR_{M\,7.5} = 0.03\ \exp(-12\psi) \qquad [6.1]$$

No K_σ is applied in using [6.1] nor are there any soil type (i.e., K_c) adjustments. Rather, the effect of these conditions is directly dealt with in computing ψ from the CPT data as described earlier.

Theoretically, the critical state friction ratio M and the ratio of elastic to plastic modulus should appear in the equation [6.1]. However, the Rose creek tailings have a critical state friction angle comparable to normal sands (actually slightly less) and the undrained stress paths found in the laboratory testing (which are controlled by the elastic/plastic ratio) are similar to other sands. Therefore the coefficients in [6.1] appear reasonable for the Rose Creek tailings.

6.4.3 Post-Liquefaction Strengths

If liquefaction is undrained (as commonly assumed), the soil must move to the CSL at constant void ratio. In this situation and for a semi-log idealization of the CSL (the usual approximation), the shear strength at the critical state after liquefaction s_r is related to the initial insitu conditions by the theoretical equation:

$$\frac{s_r}{\overline{\sigma}_v} = \frac{1+2K_0}{3}\,\frac{M}{2}\,\exp\left(\frac{\psi}{\lambda}\right) \qquad [6.2]$$

Experience with [6.2] is that while it is an adequate description for very loose soils, it over-estimates the strength with lightly dilatant soils. This issue of the critical state strength being greater than the strength calculated from back-analysis of failures has been known for more than two decades, having first arisen in the investigation of the Lower San Fernando failure. Seed (1987) in particular brought the issue to some prominence. However, Seed's case-history based approach was expressed in a patently incorrect framework in that it treated residual strength (which is in units of stress) as being controlled by normalized penetration resistance (which is dimensionless). A factor with units of stress was missed.

Several workers have adopted Seed's suggestion that back-analysis be used to formulate operating strengths, but then cast the results in a more reasonable form. Figure 6.10 shows a currently accepted form which relates the back analyses in terms of a mobilized stress ratio (Stark & Mesri, 1992). However, even this approach is relying on the stress-level normalized penetration resistance rather than a true dimensionless framework.

Figure 6.11 presents the same data as investigated by Stark & Mesri but now presented in terms of the state parameter. Error bars are shown to indicate the uncertainties in residual strength from the back analysis and uncertainties in the characteristic insitu state. Also shown on this figure is the undrained strength at the critical state for a typical clean sand based on equation [6.2]. As can be seen the operating field strengths are somewhat scattered but there is an average trend through the data than could be approximated by:

$$\frac{s_r}{\overline{\sigma}_v} = 0.1 - 0.15\,\psi \qquad [6.3]$$

The reason that the field data often shows strengths which are but a fraction of the critical state strength lies in two factors. First, many of these case histories were not undrained and what happened in those cases was much affected by pore water migration (e.g., the Lower Sand Fernando slip was some two minutes after the earthquake stopped). Second, undrained conditions do not really exist with semi dense sands as there is a local redistribution of pore water on the scale of less than a meter. Both factors can be accounted for in a full analysis, but such analyses are at the limits of current research. Practically, an equation such as [6.3] is used. We have followed that conventional approach.

7.0 SEISMIC STABILITY OF TAILINGS

7.1 Resistance to Initial Liquefaction

The results of applying both the NCEER method and state parameter approach to CPT soundings are presented in Appendix V as predictions of CRR profiles for each sounding and comparison of those profiles with the CSR profile for the MCE.

The NCEER method predicts initial liquefaction for tailings sands in all of the thirty three CPT soundings. Broadly, the sand tailings have a cyclic strength insitu that is about one third of the cyclic stress expected in the design earthquake situation. But, as noted earlier, the extensive slimes deposits are predicted to be "likely non- liquefiable".

The state parameter approach predicts initial liquefaction in most of the tailings sands in all thirty three soundings. However, the strengths predicted by this method are about 50% greater than those from the NCEER method. Where the two methods really differ is in the slimes. In the case of the slimes, the state parameter approach predicts a very large shortfall in cyclic strength and correspondingly widespread liquefaction under the MCE situation. The extent of the predicted liquefaction in the tailings is considered to be consistent with the tailings deposition history.

Table 7.1 presents a summary of the predicted percent of each CPT profile that is considered liquefiable for the M7 earthquake with a PGA of 0.5 g. In general, when the selected seismic exposure as presented in Section 6.1 is applied to the CPT data, initial liquefaction is predicted for some portion of each CPT sounding in the tailings impoundment by either of the two assessment methods. However, the NCEER method indicates liquefaction for a lower percentage of the overall data due to the "not likely liquefiable" classification of the high fines content tailings material.

Table 7.1: Comparison of Predicted Extent of Liquefaction by CPT

CPT	% of CPT profile indicating liquefaction based on CRR/CSR Ratio for MCE	
	CRR from qc_{1n} (Robertson and Wride 1998)	CRR from ψ
CPT03-01	20%	90%
CPT03-02	60%	80%
CPT03-03	70%	90%
CPT03-04	60%	90%
SCPT03-05	60%	90%
CPT03-06	90%	90%
CPT03-07	70%	90%
CPT03-08	90%	90%
CPT03-09	80%	90%
CPT03-10	60%	90%
CPT03-11B	60%	50%
CPT03-12B	70%	100%
CPT03-13	60%	90%
CPT03-14	30%	80%
CPT03-15	50%	90%
CPT03-16	60%	90%
SCPT03-17	40%	80%
CPT03-18	60%	90%
CPT03-19	40%	100%
CPT03-20X	60%	50%
SCPT03-21	30%	100%
CPT03-22	30%	90%
CPT03-23	90%	100%
CPT03-24	40%	100%
SCPT03-25	70%	80%
CPT03-26	80%	80%
CPT03-27	50%	90%
CPT03-28	70%	80%
CPT03-29	70%	80%
CPT03-30	50%	90%
CPT03-31	50%	30%
SCPT03-32	30%	100%
CPT03-33	10%	100%

7.2 Post-Earthquake Flowslide Potential

The post-liquefaction flowslide potential depends on the residual strength. There are two cases to be considered.

First, if the undrained post-liquefaction strength exceeds the drained strength then the structure or impoundment slope will be stable after an earthquake, regardless of magnitude, if it is stable now. Originally this idea was used as part of the steady state approach and was explicitly stated by Poulos (1981). Nowadays it is generally accepted that the less than steady state strengths are mobilized and as illustrated on Figure 6.11 where the field case histories plot below the theoretical line (this theoretical line is exactly the steady state strength). But as is also clear on this figure, an insitu state denser than about ψ <-0.07 will ensure that $s_r / \overline{\sigma}_v > 0.3$. Such a strength ratio is close to the mobilized friction for a drained loading, and as such ensures that if the situation is presently stable under drained conditions then it will not flowslide post-earthquake. Much of the tailings sands meet this criterion.

Second, the actual shear stress on the soil has to be greater than its residual strength. This can be assessed on a basin-wide basis by comparing average slopes with the available strength ratios. Using an infinite slope idealization, the shear stress ratio of a slope α and with the water table at ground surface can be expressed as:

$$FOS = \left(\frac{\gamma_b}{\gamma_t}\right)\left(\frac{\tan\varphi'}{\tan\alpha}\right) \qquad [7.1]$$

The available residual strengths have been computed for each of the CPT soundings and are plotted on the individual sheets for each sounding presented ion Appendix V. Strength ratios in the slimes are rarely greater than $s_r / \overline{\sigma}_v > 0.01$. Comparison of this strength ratio (equivalent to tan ϕ' in 7.1) to a basin-wide overall slope ratio of 50 m elevation change over 3700 m length (slope α of 0.8 degrees) along with ratio of buoyant and total density in equation 7.1 predicts an infinite slope FOS of about 0.4 and indicating that a flowslide could develop in a post-earthquake situation in the fine tailings.

As a further check, the present undrained strength of the slimes was estimated. This was done by computing a strength from the CPT data using the usual equation:

$$s_u = \frac{q_t - \sigma_v}{N_k} \qquad [7.2]$$

where N_k is a soil specific coefficient (often called the "cone factor"). A coefficient N_k = 12 was chosen as not unreasonable for soft silts. Using this N_k, undrained strength ratios were computed and are also shown on the plots in Appendix V. Strength ratios generally exceed about $s_u / \overline{\sigma}_v > 0.12$. These strength ratios are sufficient to provide the existing observed static stability of the impoundment.

An important aspect of s_u and s_r is the difference between these two values. This difference is the potential brittle loss of strength in a liquefaction situation, however that situation might arise. As can be seen from the computed profiles in Appendix V there is the potential for typically 90% of the undrained strength to be lost in the case of the slimes – such strength drops have been found in catastrophic flow slides following liquefaction, with the extent of soil movement depending on the topography.

The difference between s_u and s_r was considered earlier in terms of the laboratory testing (Section 3). It was noted there that the laboratory tests on the finer gradation, which did not show much brittle strength reduction, had a void ratio markedly denser than found insitu. If the average insitu void ratio is used, then the CSL determined in the laboratory indicates about a 90% strength reduction from peak to residual (sometimes more, depending on the difference between the tested and the insitu void ratio). This is consistent with the CPT-based assessment but using different inputs and a different methodology.

8.0 DAM STABILITY ANALYSES

8.1 Overview

A concurrent study by Byrne and Seid-Karbasi (2004) was carried out to investigate the liquefaction potential of the foundation of the Intermediate Dam. The results of the draft report prepared by Byrne and Seid-Karbasi (2004) indicated that based on the limited insitu SPT data, some zones of the foundation of this dam were susceptible to liquefaction under the MCE. As a result the stability and displacements of the Intermediate Dam during the MCE scenario were investigated.

The stability and deformation of the Secondary Dam were not considered in this study. At the West Secondary dam, most of the available information on this structure is related to the portion of the dam now completely buried by tailings. The CPT was unable to penetrate on the upstream side of the tailings fill used for this dam. At the East Secondary dam, there was very limited foundation data.

For the Intermediate Dam, the methodology comprised psuedostatic limit equilibrium stability analyses to define a yield ground acceleration, and the subsequent use of the Newmark method to estimate ground displacement. Some key features of this approach are presented as follows.

The psuedostatic method neglects the distribution and time history of inertial forces in the dam during an earthquake. Instead, the effect of ground motion is idealized as a uniform horizontal acceleration acting to destabilize the slope. The output of the analysis is the value of this acceleration that just brings the slope to incipient movement, and this is refereed to as the yield acceleration. The method only considers the non-liquefaction situations. A crucial assumption of the method is that the soil does not weaken during any earthquake induced displacements. Reduced strength values may be appropriate even when considering compact soils to allow for dilation during shear causing a reduction in soil density and correspondingly strength.

The Newmark (1965) method developed from the recognition that almost all earthfill structures will be overloaded by strong ground motion, but that such overloading does not necessarily mean the structure fails. Rather displacements will occur but these can be quite small and stop when the ground motion stops. The displacements in the case of well constructed earth dams are often in the order of less than 1 m and may not affect the serviceability of the structure. The Newmark method computes the horizontal displacements by assuming the earthfill above the slip surface identified in the pseudostatic analysis deforms as a rigid block on that surface. The Newmark method can be applied either using simple bounding curves based on dominant characteristics of the design ground motion or with more detailed results using actual earthquake records. The latter approach was used here, with the records being supplied to us by Atkinson (2003).

Importantly, note that the present findings are based on further investigation of the Intermediate Dam showing that its foundation is not liquefiable.

8.2 Idealization of Intermediate Dam

A single cross-section of the Intermediate Dam was selected for analysis as presented in Figure 2.8. Referring to the mean sea level elevation datum, the current crest of the dam is at elevation 1049.4 m, the upstream pond assumed to be at the level of the spillway invert of elevation 1047.5 m and the downstream shell of the dam is submerged by the cross valley pond assumed to be at elevation 1031.2 m.

Pseudostatic analyses for earthquake load conditions were carried out for the long-term condition using effective stress strength parameters for drained conditions.

A discussion of the foundation conditions for this dam was presented in Section 2.3.3. The Intermediate Dam foundation ranges from a compact fine grained material in the proximity of the original Rose Creek channel to relatively coarse sand and gravel till and some alluvium near the north abutment and some frozen till with some colluvium near the south abutment. The dam shell was constructed from compacted sand and gravel and the core from glacial till core both obtained from a local borrows. Drainage and filter sand and gravel zones were placed on both sides of the core and under the downstream rockfill shell.

In the liquefaction assessment carried out by Byrne and Seid-Karbasi (2004), it was identified that a portion of the foundation of the Intermediate Dam may be susceptible to liquefaction under the MCE. This requires further investigation and the present analysis are based on the premise that such investigation will show that the dam's foundation is actually adequate.

The limited laboratory strength testing data was reviewed for foundation soils, borrow materials used for shell construction and tailings. This data was used to estimate ranges of expected material properties for use in the analyses which is summarized in Table 8.1.

Phreatic conditions through the dam cross section have been estimated based on the geometry of the inclined till core and assuming that the drainage layer at the base of the downstream shell remains effective. The assumption of drainage layer efficiency requires confirmation in the field, and consideration to the installation of piezometers for this purpose should be given for the site works being planned for 2004.

Table 8.1: Summary of Material Properties used in Slope Stability Analysis

Material	Saturated Unit Weight (kN/m^3)	Effective Stress Parameters		Basis for Property Selection
		Cohesion, c' (kPa)	Angle of Internal Friction (°)	
Dam Core – glacial till	21	0 35	35 32.5	Consolidated drained triaxial testing
Shell – sand and gravel	12	0	32 to 36	Consolidated drained triaxial testing and previous experience with material with similar grain size distribution
Foundation Sand and Gravel	21	0	32 to 36	Previous experience with material with similar grain size distribution
Tailings	24	0	30	Consolidated drained triaxial testing – both fine and coarse gradations

Figure 8.1 illustrates an example of the idealized dam section used in the analyses. Other idealizations used the same geometry and only varied material properties in accordance with Table 8.1.

8.3 Estimated Yield Acceleration

Limit equilibrium analysis was carried out with the Slope/W software using a Morgenstern & Price non-circular slip surface method. Slope/W does not output the yield acceleration directly. It is therefore necessary to run a series of analyses with varying horizontal accelerations and plot the computed factor of safety (FOS) against the input acceleration. The yield acceleration is that corresponding to a factor of safety of unity.

In order to estimate a yield acceleration for the dam, the range of possible failure surfaces was limited to those that passing through the dam crest immediate upstream tailings, such that a seismically induced failure on this surface in combination with liquefaction flow slide in the tailings would result in release of tailings impounded by the dam.

Figure 8.2 presents the psuedostatic FOS based against the seismic accelerations calculated for a range of the shell and foundation materials friction angles that correspond to the uncertainty identified in Table 8.1.

The static FOS corresponds to zero horizontal acceleration and ranges from about 1.3 to 1.6, depending upon what the actual friction angles of the shell and foundation are.

As horizontal acceleration is increased, the factor of safety reduces. A FOS = 1 gives the yield acceleration (a_y) which lies in the range 0.11g $<a_y<$ 0.18g, see Figure 8.2.

A FOS less than unity during an earthquake does not mean “failure”, merely that the dam will deform. The extent of this deformation is computed in the next section.

8.4 Deformation Analysis

As presented in Section 6.1, the site specific seismic hazard assessment indicates a peak ground acceleration (PGA) in the range from a medium of 0.3 g to a mean of 0.5 g for an 0.0001 p.a. spectrum. Included in the seismic hazard assessment were the acceleration time history records for three earthquake ground motions which would produce a similar PGA for this site (Atkinson 2003). For the present purposes these three strong ground motion records in two directions each, as selected by others, were used for estimating the dam movement. These records are summarized on Table 8.2 (only horizontal records were used).

Table 8.2: Summary of Ground Motion Records

Earthquake	Component	a_{max} (g)	v_{max} (cm/s)	Newmark Method Maximum Calculated Displacement (cm)	
				a_{yield} = 0.11g	a_{yield} = 0.18g
Loma Prieta Earthquake - Gilroy Sewage Plant	90° 0°	0.37 0.54	43.8 35.8	38.5 42.5	34.5 36.4
Loma Prieta Earthquake - Lick Lab	90° 0°	0.41 0.44	21.9 22.0	30.6 36.5	27.2 31.5
Northridge Earthquake – Pacoima Dam	265° 175°	0.43 0.41	31.4 45.0	23.4 23.3	20.4 19.4

The recorded ground motions were used at face value and without computing the effect that dam response might have on them. Although shear modulus and damping of the dam control whether ground motions are amplified or attenuated by the dam, there is no data for these properties in the dam. The calculation is approximate and, as will be illustrated, relatively insensitive to the actual a_{max}.

The Newmark method was implemented in a visual basic program that was verified against the simplified limiting values presented by Newmark (1965). This program output the computed ground and dam displacement, with the overall deformation being taken as the vector sum of the two (the calculations are related to an inertial frame of reference). Figure 8.3 presents an example of the computed output and which shows how displacement accumulates.

Because computed motions are vector, the orientation of the dam to the ground motions matters. This was allowed for by computing with each record twice, once assuming that positive acceleration was orientated downslope and once assuming that negative acceleration was downslope. Whichever assumption gave the greatest displacement was taken as the assumption to adopt for that record. This condition was adopted for all six records provided.

Figure 8.4 shows the results from two simulations of the same record, one with a_y = 0.11 g and the other with a_y = 0.18 g. The results of which indicate that the computed yield acceleration range results in about a 5 cm range of computed displacements.

The overall estimates of maximum horizontal displacement are presented on Figure 8.5. For the range of earthquake acceleration time histories considered the dam would experience between 0.2 and 0.5 m horizontal deformation. The dam would also experience a concurrent vertical settlement which is estimated to be about half the horizontal based on the downstream slope angle of the dam.

9.0 DISCUSSION

9.1 Tailings Liquefaction and Movement

The central issue for the tailings impoundment is the behaviour of the fine tailings slimes during an earthquake. Although the tailings sands are predicted to be prone to initial liquefaction by both of the assessment methods used, they appear sufficiently dense for initial liquefaction to not lead to a flowslide.

The principal difficulty in evaluating the expected behaviour of the fine tailings slimes is that they lie outside the experience base on which the usual liquefaction assessment method has been developed. This is further compounded by the slimes showing undrained behaviour during CPT soundings and which limits the applicability of existing theoretical approaches to evaluation of CPT data. Existing theoretical approaches could be extended to cover the Rose Creek tailings and their undrained CPT behaviour, but this was outside the scope (and available time) for the present work. The approach adopted here therefore built on a general screening method proposed a decade ago, updating it in light of recent developments, and estimating a site-specific calibration for it. This adopted approach predicts widespread liquefaction of the tailings slimes, and with a high potential for subsequent flowslides under the MCE.

The state parameter approach used to estimating liquefaction potential of slimes from CPT data, although well founded in mechanics, is unusual and obviously as such open to questioning. However, when the scale of the shortfall in predicted cyclic strength is considered it is difficult to see how credible changes in coefficients within the method could lead to a conclusion that the slimes would be stable in an earthquake – the shortfall is simply too great. Further corroboration of this finding is given by the very high excess pore pressures measured during CPT sounding and which exceed those encountered in the metastable Lake Champlain clays.

Further, it is not just an issue of liquefaction under the MCE. Some of the finer tailings zones, such as at SCPT03-21 and SCPT03-32, indicate that peak ground acceleration in the range of 0.05 to 0.1 g could trigger liquefaction. This acceleration is much lower than that expected for the MCE, which is about a 1 in 10,000 year return period seismic loading event. These lower PGA have a higher likelihood of occurrence. A yield type acceleration analyses should be carried out on each CPT profile to obtain further insight into areas susceptible to flowslides under moderate seismic loading.

Given the possibility of a flowslide in the tailings, the issue for closure is that weather such movement might result in a release of the tailings from the impoundment. A tailings mass balance was made for the intermediate impoundment based on the tailings surface contour data provided for this work (which had a 2 m contour interval). It would appear that there is sufficient volume in the area below elevation 1049 upstream of the Intermediate Dam crest to contain tailings from above elevation 1049 m (the expected settled post earthquake intermediate dam crest elevation). Thus, in a flowsliding situation, the Intermediate Dam may not be overtopped by the liquefied slimes. However, this assumes that the Secondary Dam remains stable and contains the secondary impoundment tailings. There is insufficient storage at the Intermediate Dam to contain all potentially liquefied slimes.

This estimate of containment of liquefied slimes is based on an assumed tailings surface below the Intermediate Pond. A bathymetric survey should be undertaken to determine the top of tailings surface elevations in this pond to confirm the mass balance estimate.

9.2 Implications for Tailings Cover Integrity

We understand that a cover is being considered over the Rose Creek Tailings impoundment. Some of the objectives of placing a cover over the tailings could include:

- Physical isolation of tailings from human and wildlife,
- Reduction of the contact of the tailings and surface waters; and
- Reduction of the oxygen available to the tailings.

Independent of the objectives, the cover will need to be constructed over the current tailings surface and will be required to maintain the design integrity over the long term.

The following presents a number of geotechnical issues that relate to this integrity of a tailings cover. The comments are based on inferred tailings properties from the CPT investigation.

- Constructability of a cover over the entire tailings area will be an issue. Generally, the tailings have a variable thickness upper crust of drier, slightly stronger tailings which are underlain by very soft and weak tailings. Construction equipment operating on the tailings surface to place the cover may or may not be supported by this upper tailings zone.

- Repeat truck traffic on the tailings surface could increase pore pressure at finer tailings zones with depth and if not dissipated, could trigger static liquefaction.

- Any cover placed at the surface of tailings should be expected to result in a highly variable surface profile. Due to the heterogeneous nature of the tailings deposit, large differential settlements can be expected and this will result in an uneven cover surface following construction.

- Any cover placed on the tailings surface will result in an increase in the degree of consolidation of the underlying tailings. During the consolidation process, excess tailings pore water will be released from the deposit either into underlying foundation or at surface and will require the appropriate water quality considerations.

- Depending on grain size distribution of cover and type of construction traffic on the cover material, there could be substantial mixing of the cover material and the upper tailings materials.

- Substantial zones of the fine tailings are expected to be triggered into liquefaction and potentially flowslide type displacements under a moderate seismic loading.

10.0 CLOSURE

This report addresses the geotechnical issues of the seismic stability for the tailings in the context of the mine closure planning for the Rose Creek tailings impoundment.

As shown in this report, there are more than a reasonable grounds for the prediction of wide spread liquefaction of the fine tailings slimes and potential for flowslides under even a moderate seismic loading event.

This conclusion in principle leads to the following two possible remediation strategies for the closure of the Rose Creek Tailings Impoundment:

1. Containment of the potentially flowslide susceptible tailings with engineered containment dams which would include additional consideration for the proximity of tailings to the Rose Creek Diversion Channel; and

2. Densification or otherwise stabilizing the fine tailings slimes to lessen the potential for flowslide displacements.

An engineered containment would be subject to confirmation of a seismically stable foundation at the current Intermediate and Secondary Dams. Containment could be achieved by construction of additional crest elevation either with sand tailings or the locally borrowed natural soils.

Densification of the tailings could be carried out by either drainage through de-watering or densification through addition of energy. The dewatering based option may have required additional environmental considerations related to tailings water quality. For the densification option, there are a number of possible approaches. But, based on the size of this impoundment and the maximum depth of the loose tailings zones, the most likely option would be through blast densification. This could be proved up through a trial densification program to confirm cost and effectiveness.

A number of assumptions have been made in order to reduce the CPT data from the tailings to predict the liquefaction potential. While further work could be carried out to refine some of the assumptions in this approach, it is our opinion that this would not result in any substantial change to the conclusions on the extent of potential liquefaction of these tailings. Correspondingly, no further work in the tailings is recommended in this direction.

Our opinion is that in-place tailings containment by raising of the Intermediate Dam and associate works adjacent to Rose Creek, subject to the confirmation of the Intermediate Dam foundation related to seismic stability, may be a viable approach for the closure of this impoundment. We recommend that further work be carried out in this direction, to at least an outline design level, for cost comparison with the alternative remediation (ie in pit disposal). Specifically, we envisage and recommend four actions:

- Confirm the stability of both the Intermediate and Secondary Dam foundations;
- Carry out a detailed bathymetric survey of the Intermediate Pond;
- Carry out a "run out" analysis for the potentially liquefying tailings to define the heights of required containment structures;
- Develop an outline engineering design for the required containment raising works including a bill of quantities for the earthmoving.

We trust that this report meets your current needs, feel free to contact the undersigned if you have any questions.

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200 0 200 Kilometers

Scale - 1:10,000,000

LEGEND

- City
- Town/Village
- Provincial Border
- Water
- Primary Highway
- Railroad

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

SITE LOCATION PLAN

Golder Associates
Burnaby, B.C.

PROJECT No. 03-1413-078			SCALE AS SHOWN	REV. 0
DESIGN	JC	12 Dec. 2003	**FIGURE 1.1**	
GIS	CDB	12 Dec. 2003		
CHECK	JC	12 Dec. 2003		
REVIEW				

REFERENCE
Base digital data obtained from ESRI Inc.,
DATUM: WGS84 PROJECTION: Geographic

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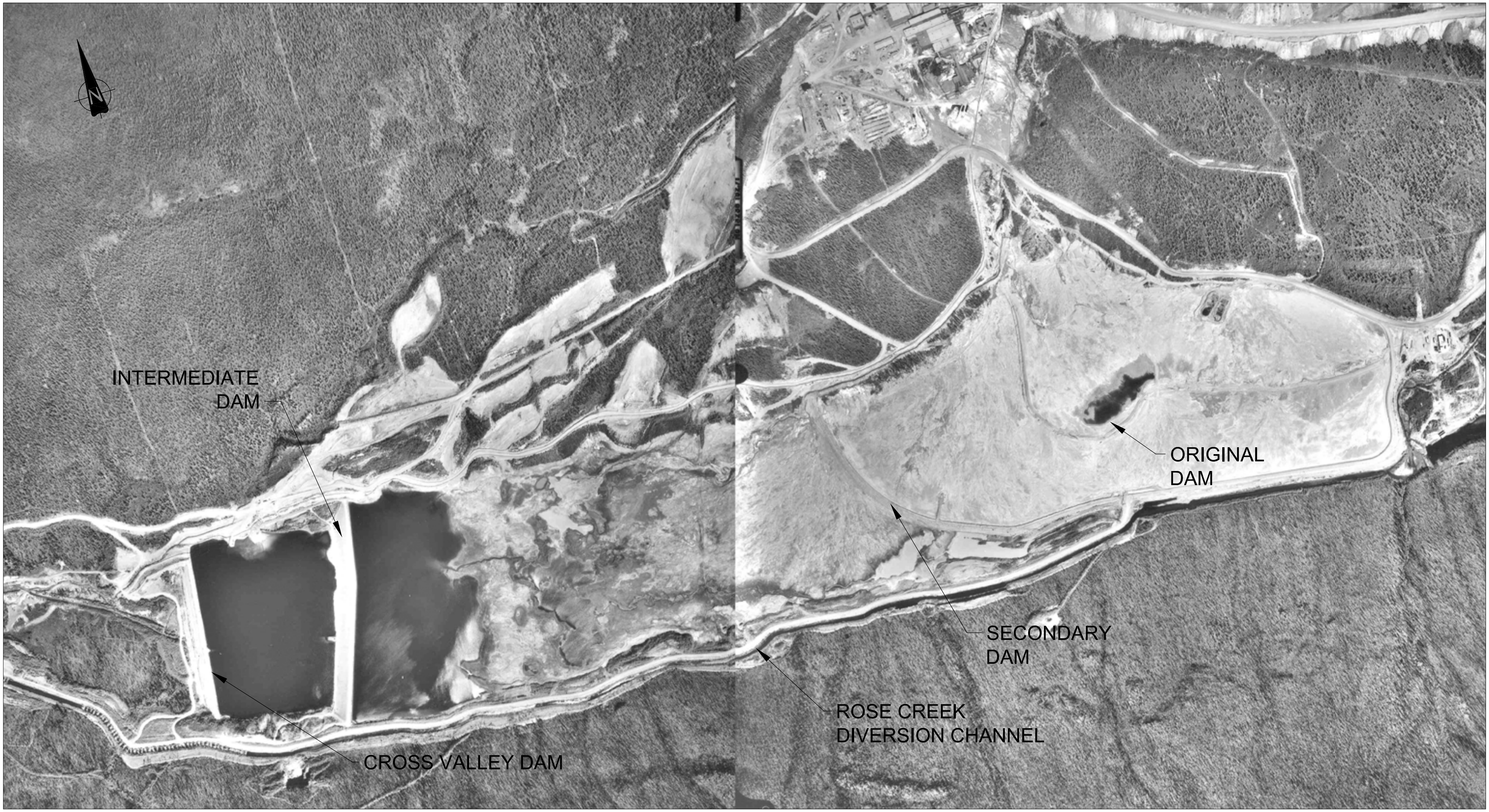
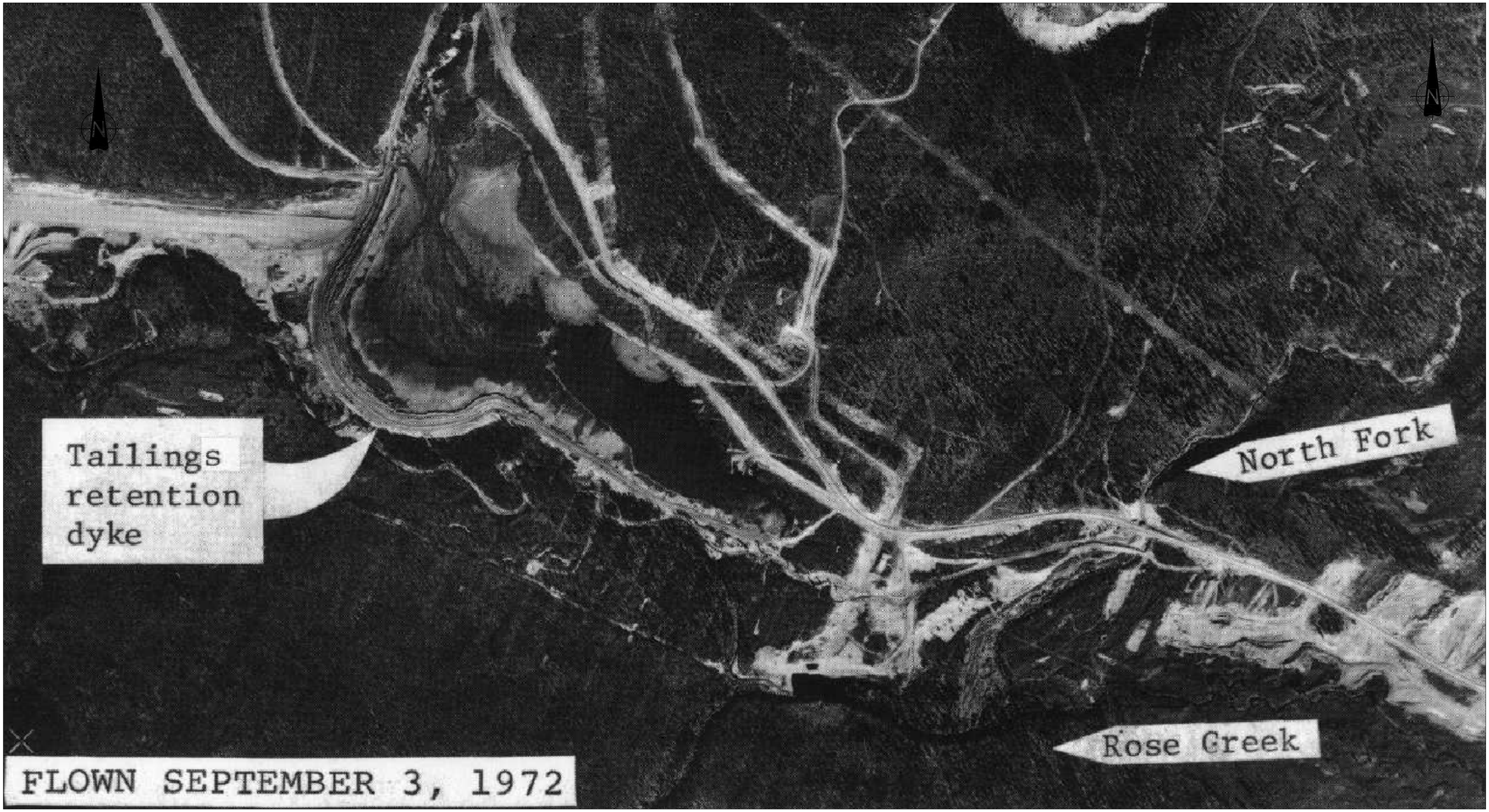


PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	**ROSE CREEK TAILINGS - 2003**

Golder Associates
BURNABY, B.C.

PROJECT No. 03-1413-080			FILE No. P1413080-02.dwg	
DESIGN	JCC	11 DEC 03	SCALE NTS	REV. 0
CADD	SRR	11 DEC 03		
CHECK	JCC	11 DEC 03	**FIGURE 1.2**	
REVIEW				

REVISION DATE: 04/06/10 9:53am By: rroque CADD FILE: N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\cad\P1413080-02.dwg



PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	**ROSE CREEK TAILINGS - 1972**

Golder Associates
BURNABY, B.C.

PROJECT No.	03-1413-080		FILE No.	P1413080-02.dwg
DESIGN	JCC	11 DEC 03	SCALE NTS	REV. 0
CADD	SRR	11 DEC 03		
CHECK	JCC	11 DEC 03	**FIGURE 2.1**	
REVIEW				

REVISION DATE: 04/06/10 9:56am By: rroque CADD FILE: N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\cad\P1413080-02.dwg

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

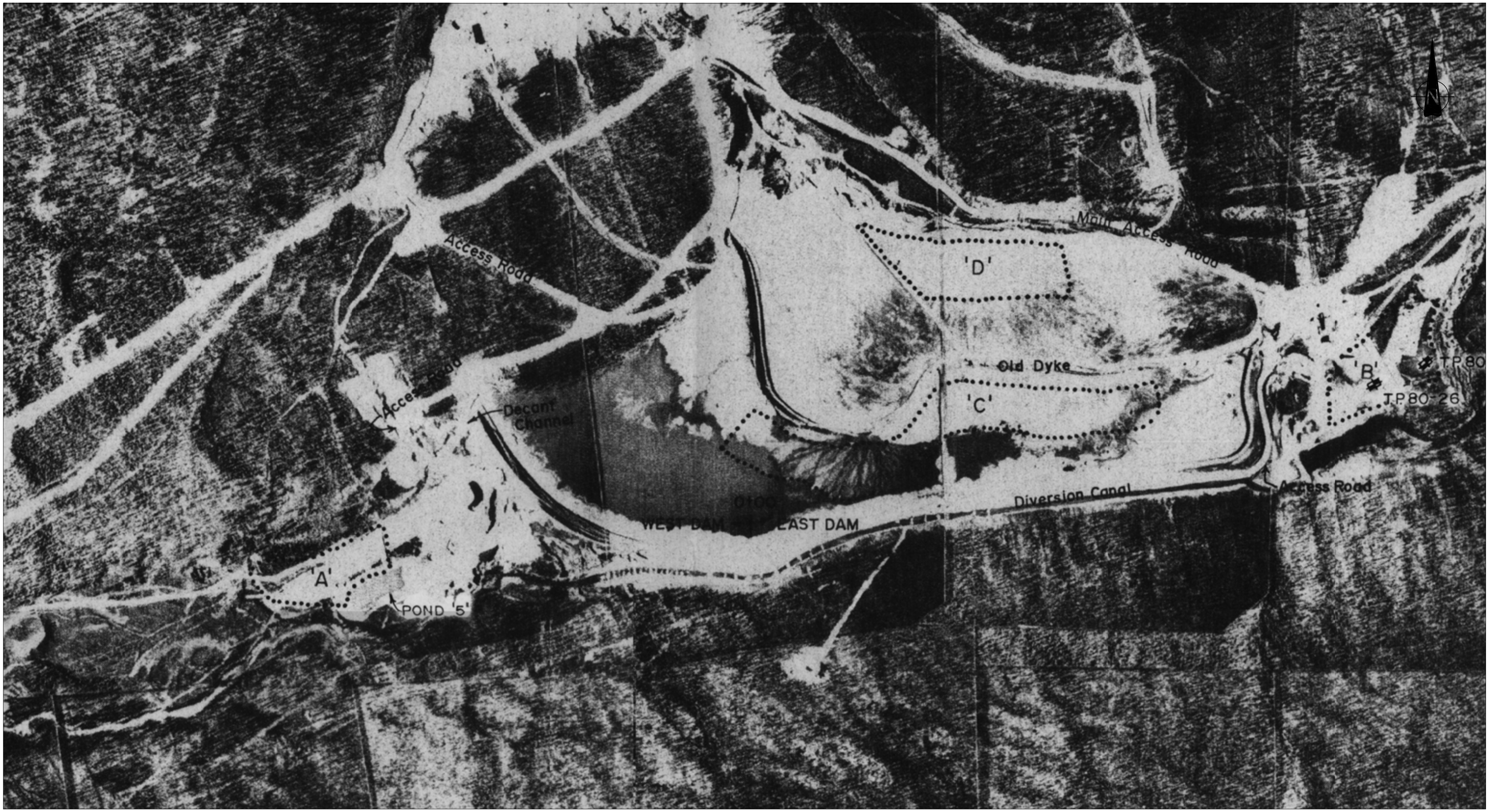
TITLE

ROSE CREEK TAILINGS - 1975

Golder Associates
BURNABY, B.C.

PROJECT No. 03-1413-080			FILE No.	P1413080-02.dwg
DESIGN	JCC	11 DEC 03	SCALE NTS	REV. 0
CADD	SRR	11 DEC 03		
CHECK	JCC	11 DEC 03	**FIGURE 2.2**	
REVIEW				

REVISION DATE: 04/06/10 9:57am By: rroque CADD FILE: N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\cad\P1413080-02.dwg



PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

ROSE CREEK TAILINGS - 1979

Golder Associates
BURNABY, B.C.

PROJECT No. 03-1413-080			FILE No. P1413080-02.dwg	
DESIGN	JCC	11 DEC 03	SCALE NTS	REV. 0
CADD	SRR	11 DEC 03	**FIGURE 2.3**	
CHECK	JCC	11 DEC 03		
REVIEW				

REVISION DATE: 04/06/10 9:59am By: rroque CADD FILE: N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\cad\P1413080-02.dwg

PROJECT

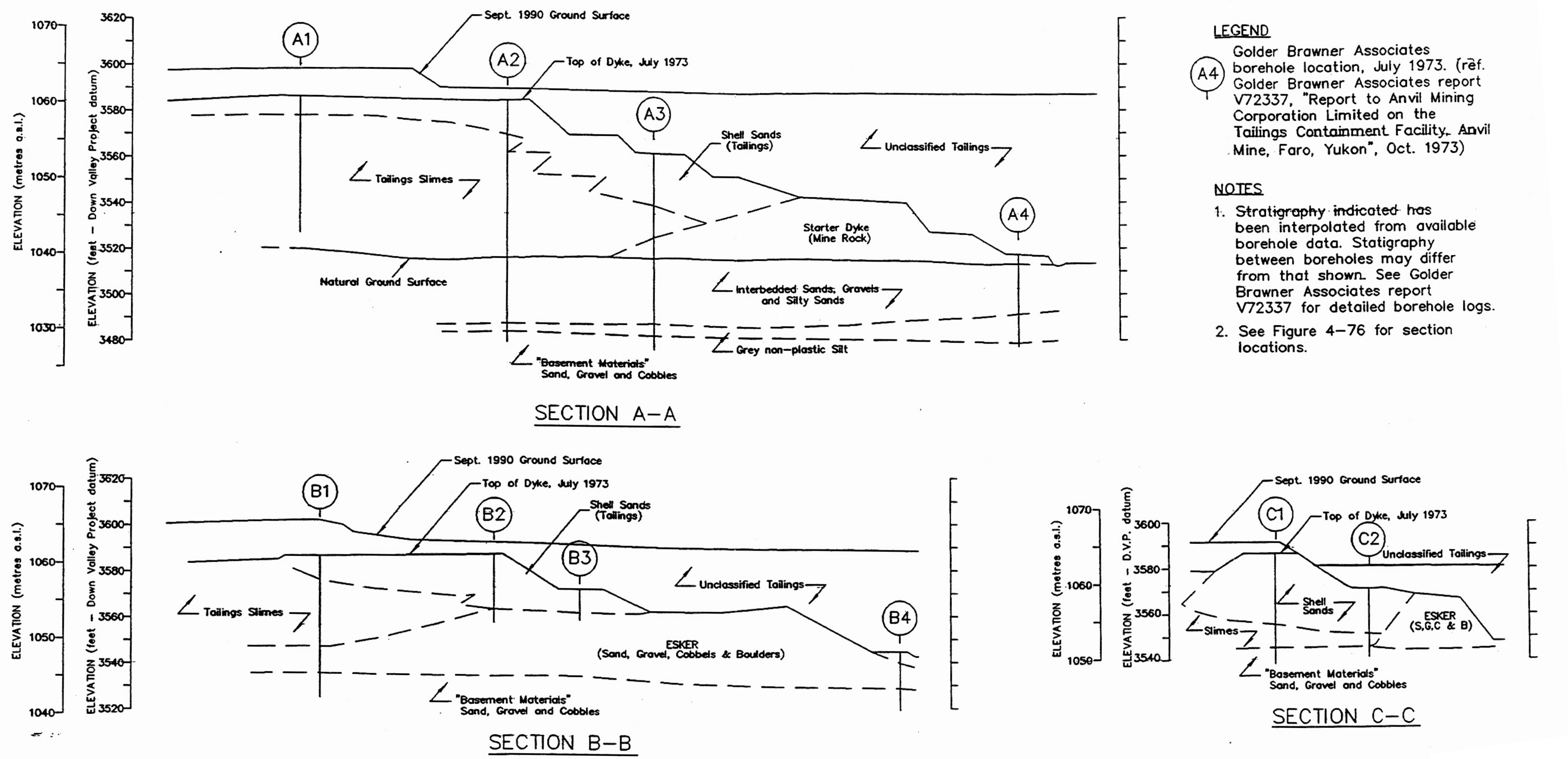
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

ROSE CREEK TAILINGS - 1997

Golder Associates
BURNABY, B.C.

PROJECT No.	03-1413-080		FILE No.	P1413080-02.dwg	
DESIGN	JCC	11 DEC 03	SCALE	NTS	REV. 0
CADD	SRR	11 DEC 03			
CHECK	JCC	11 DEC 03	**FIGURE 2.4**		
REVIEW					



LEGEND

(A4) Golder Brawner Associates borehole location, July 1973. (ref. Golder Brawner Associates report V72337, "Report to Anvil Mining Corporation Limited on the Tailings Containment Facility, Anvil Mine, Faro, Yukon", Oct. 1973)

NOTES

1. Stratigraphy indicated has been interpolated from available borehole data. Statigraphy between boreholes may differ from that shown. See Golder Brawner Associates report V72337 for detailed borehole logs.
2. See Figure 4–76 for section locations.

0 20 40 60 80 feet
Scale 1"= 50'

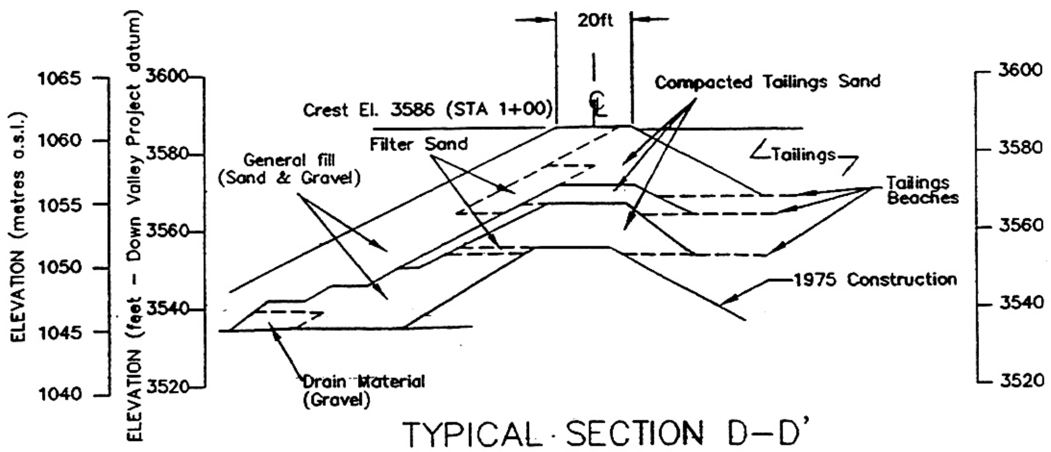
Reference: Golder Associates (1973). Report to Cyprus Anvil Mining Corporation Limited on the Tailings Containment Facility, Oct., Figures 7,8 and 9

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	ORIGINAL TAILINGS EMBANKMENT SECTIONS

PROJECT No.	03-1413-080		FILE No. FIGURES 3
DESIGN	CJ	06FEB04	SCALE NTS REV.
CADD	SS	06FEB04	FIGURE 2.5
CHECK	CJ	06FEB04	
REVIEW			

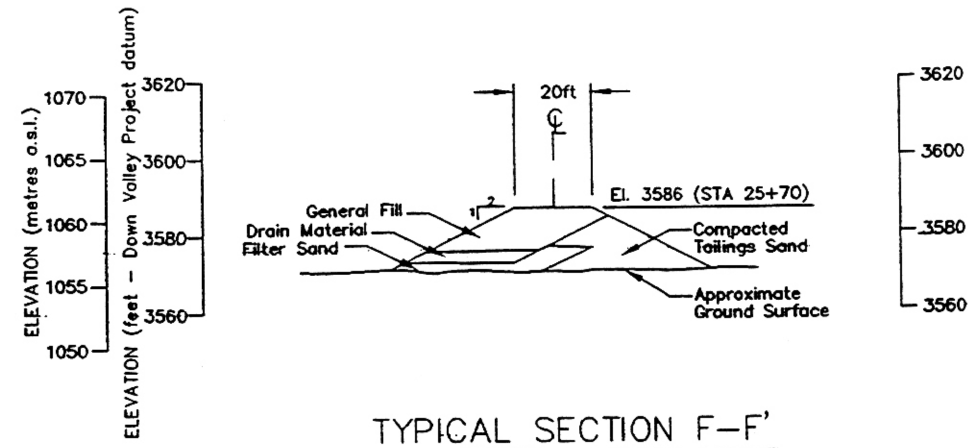
Golder Associates

REVISION DATE: BY: COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 3.cdr



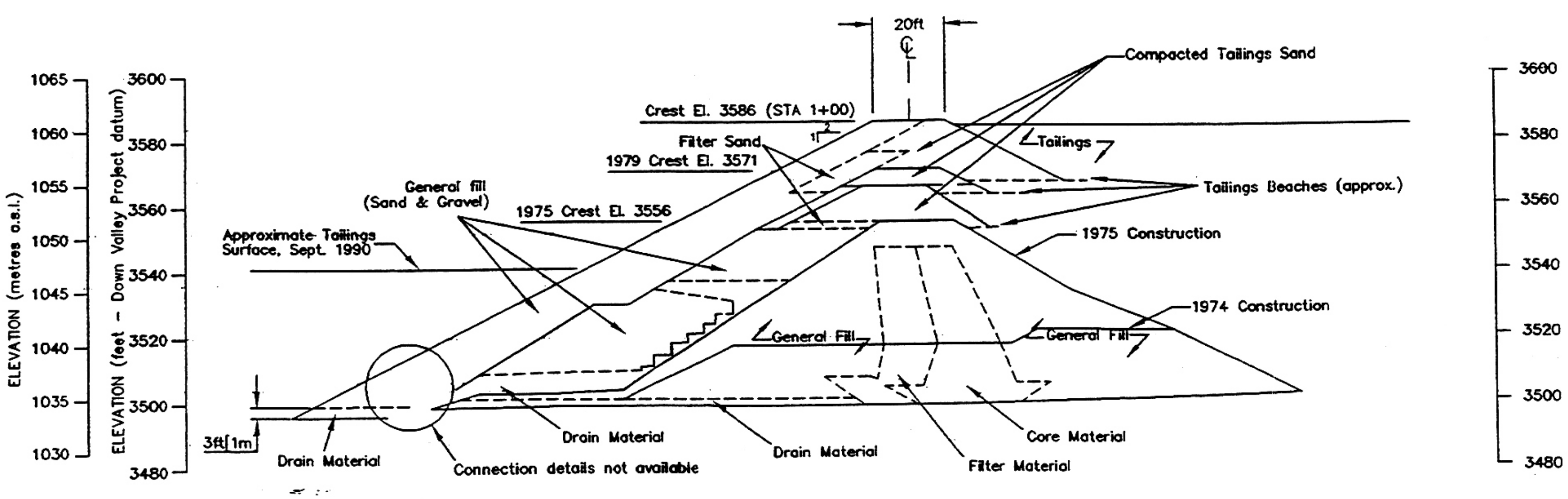
TYPICAL SECTION D–D'

(REPRESENTS STATIONS 0+00 TO 10+00, AND 19+00 TO 26+00)



TYPICAL SECTION F–F'

(REPRESENTS STATIONS 26+00 TO 27+86)



TYPICAL SECTION E–E'

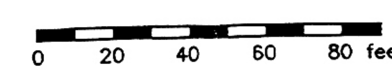
(REPRESENTS STATIONS 10+00 TO 19+00)

REFERENCES

1. Golder, Brawner & Associates Dwg. V74011, "Construction Carried Out to End of 1975"' Sept. 1975.
2. Golder Associates, Report No. 782–2736, "1978 Dyke Raising for the Tailings Retention Facility, Faro, Yukon Territory", April 1979, Figure 2 "Tailings Dam, 1978 Construction".
3. Golder Associates, Report no. 802–2024, "Construction Report, Cyprus Anvil Mining Corporation, 1980 Raising of the Tailings Dam, Faro, Yukon Territory", Appendix II, (Tender Drawings).

NOTES

1. All elevations are in feet relative to the Down Valley Project datum. To convert to sea level datum (NAD 27), subtract 106ft (32.3m). To convert from feet to metres, multiply by 0.3048m/ft.



Scale 1"= 50'

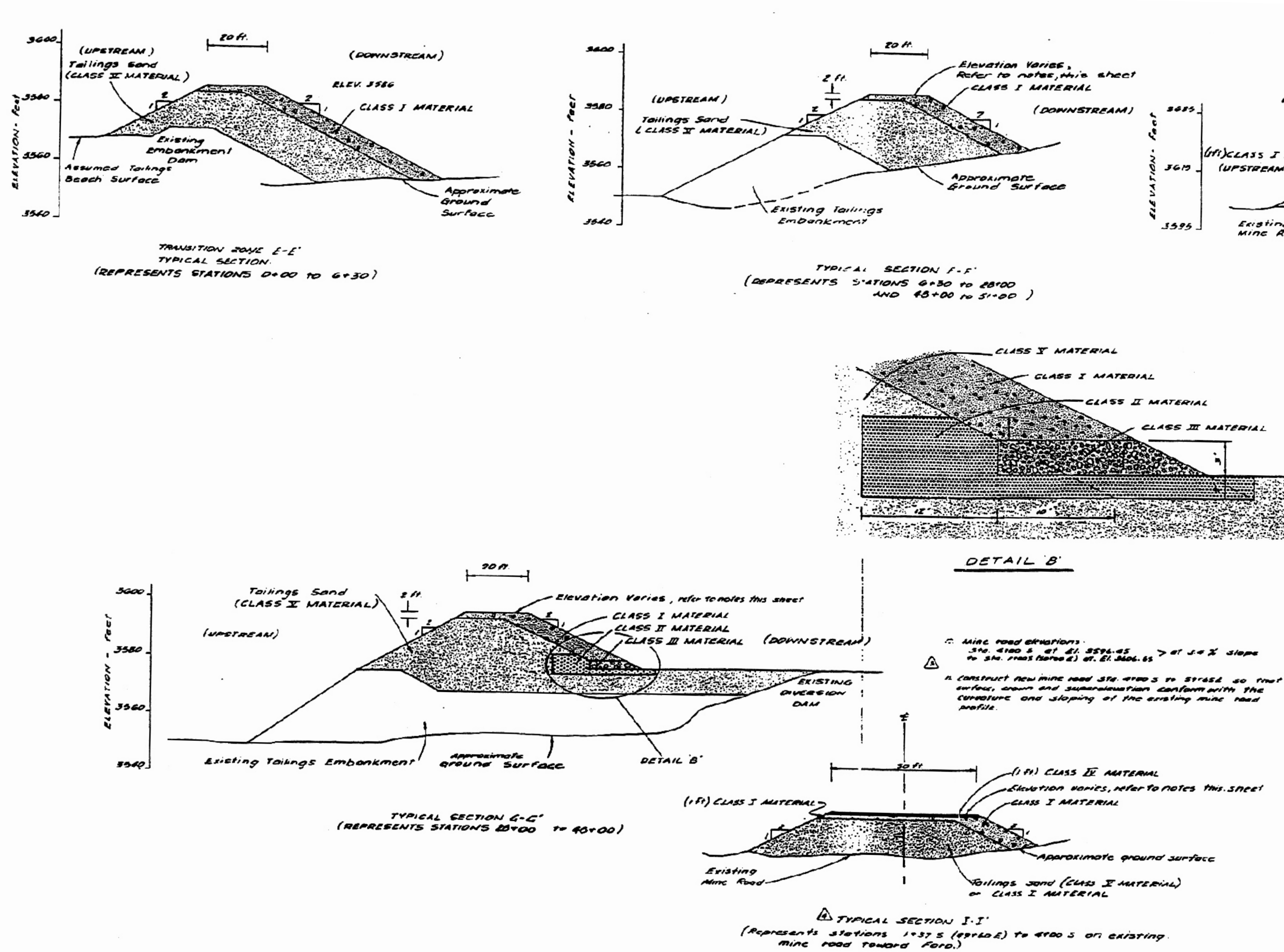
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
SECOND IMPOUNDMENT - WEST DAM
TYPICAL SECTIONS

Golder Associates

PROJECT No.	03-1413-080		FILE No. FIGURES 3
DESIGN	CJ	06FEB04	SCALE / REV.
CADD	SS	06FEB04	
CHECK	CJ	06FEB04	**FIGURE 2.6**
REVIEW			

REVISION DATE: BY: COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 3.cdr



NOTES

1. East Dam Station 0+00 to 6+30 is a transition zone which evenly shifts the centerline construction of the West Dam to downstream construction on the East Dam.

2. East Dam crest elevations:

   Station 0+00 at El. 3586 feet
   to Station 20+00 at El. 3586 feet
   to Station 30+00 at El. 3590 feet > at 0.40% slope
   to Station 45+00 at El. 3600 feet > at 0.67% slope
   to End of Construction at El. 3620 feet > at 1.33% slope

3. Area beneath proposed new construction foundation to be cleared of all deleterious materials and proof rolled with a heavy vibrating roller to the satisfaction of the engineer. Areas found to be unsuitable are to be subexcavated and replaced with Class I material compacted to 100% of the standard Proctor maximum dry density at optimum moisture content.

4. Materials used in 1980 construction to be compacted in 12" loose lift thickness to 100% of the standard Proctor maximum dry density at the optimum moisture content.

5. Class III material not to be placed in direct contact with Class V material without a minimum 1 foot thick layer of Class II filter material in between.

6. All sections looking in direction of increasing station numbers.

7. All distances and elevations are subject to final field survey.

8. Top surface of existing embankment dams to be scarified for a depth of 0.5 feet prior to placement of new fill.

9. Construct new dam top road widths as follows so as to conform with existing mine road width at 60+00 E

Station	Width (ft)	Station	Width (ft)
50+00 E	50	56+00 E	44
51+00 E	49	57+00 E	43
52+00 E	48	58+00 E	42
53+00 E	47	59+00 E	41
54+00 E	46	59+65 E	40.35
55+00 E	45		

DRAWING TAKEN FROM:

Scale: 1 inch to 20 feet Job No. 792-2041 Drawing No. 5 Drawn by R.W. Date April 1980 Reviewed by Date April 1980	CYPRUS ANVIL Faro Yukon Territory Cyprus Anvil Mining Corporation	TYPICAL EAST DAM SECTIONS 1980 EMBANKMENT DAM RAISING	Golder Associates Golder Geotechnical Consultants Ltd. Sheet 5 of 5

PROFESSIONAL ENGINEER ... GILCHRIST ... YUKON TERRITORIES

DISCLAIMER

THIS DRAWING WAS PREPARED BY GOLDER ASSOCIATES LTD. ("GOLDER") FOR THE ACCOUNT OF A CLIENT. THE MATERIAL IN IT REFLECTS SPECIFIC PROJECT PURPOSES RELATING TO THE FARO MINE AND GOLDER'S BEST JUDGEMENT IN LIGHT OF THE INFORMATION AVAILABLE TO GOLDER AT THE TIME OF PREPARATION. ANY USE WHICH ANVIL RANGE MINING CORPORATION ("ANVIL") OR ANY THIRD PARTY MAKES OF THIS DRAWING, OR ANY RELIANCE ON OR DECISIONS TO BE MADE BASED ON IT, ARE THE RESPONSIBILITY OF ANVIL AND SUCH THIRD PARTIES. GOLDER ACCEPTS NO RESPONSIBILITY FOR DAMAGES, IF ANY, SUFFERED BY ANVIL OR ANY THIRD PARTY AS A RESULT OF THE DECISIONS MADE OR ACTIONS BASED ON THIS DRAWING.

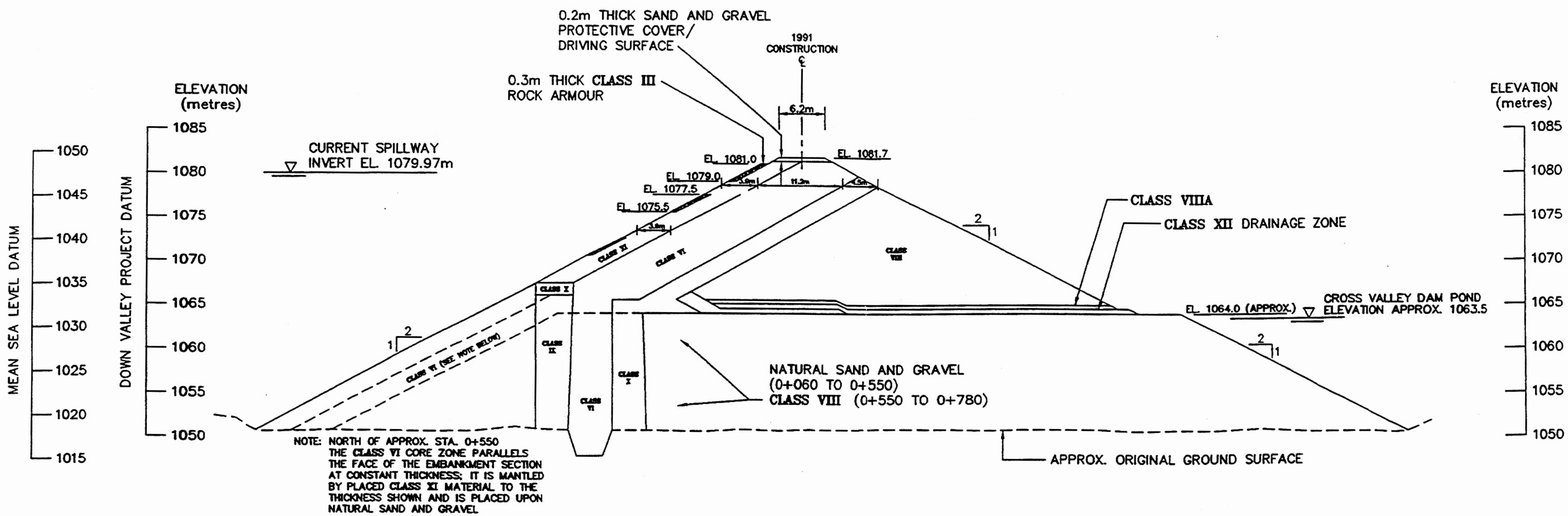
NOTES

1. ALL ELEVATIONS ARE IN FEET RELATIVE TO DOWN VALLEY PROJECT DATUM. SUBTRACT 106 FT, AND MULTIPLY BY 0.348 TO CONVERT TO METRES ABOVE MEAN SEA LEVEL.

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	SECOND TAILINGS IMPOUNDMENT EAST DAM SECTIONS

PROJECT No.	03-1413-080		FILE No.	FIGURES 3
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04		
CHECK	CJ	06FEB04	FIGURE 2.7	
REVIEW				

Golder Associates



NOTES :

1. Embankment geometry and internal zoning as shown in Golder Associates Drawing 912-2402-3, Int. Dam Raising & C.V. Dam Toe Drain, Cross Section and Detailed Plan, Rev. 1, Aug. 8, 1991.

2. All elevations are referenced to Down Valley Project Datum. Subtract 32.3m from elevations shown to convert to mean sea level (NAD27) datum.

3. Refer to Golder Associates as built reports for detailed descriptions of material classes. General descriptions as follows :

CLASS VI	Dam Core (glacial till)
CLASS VII	Upstream Shell (silty sand and gravel)
CLASS VIII	Downstream Shell (sand and gravel)
CLASS VIIIA	Drainage Filter (sand and gravel)
CLASS IX	Upstream Filter (silty sand)
CLASS X	Downstream Filter (sand and gravel)
CLASS XI	Tailings Sand (fine to medium sand)
CLASS XII	Drainage Zone (gravel)

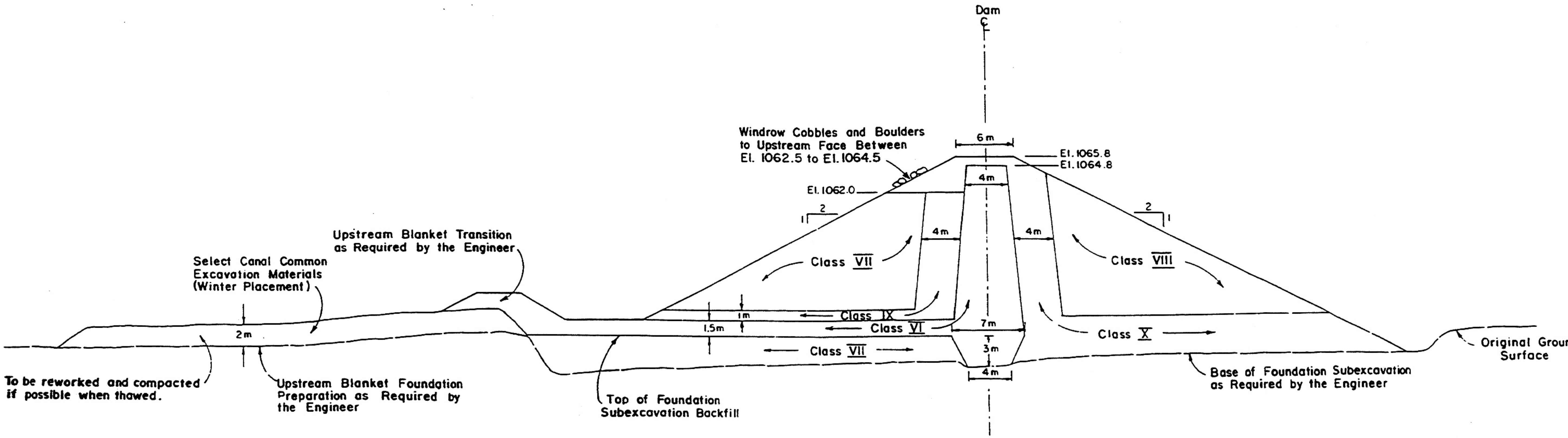
0 5 10 15 20 25 metres
1:500

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
INTERMEDIATE DAM
CURRENT CONFIGURATION

Golder Associates

PROJECT No.	03-1413-080		FILE No. FIGURES 3
DESIGN	CJ	06FEB04	SCALE / REV.
CADD	SS	06FEB04	
CHECK	CJ	06FEB04	FIGURE 2.8
REVIEW			



CROSS VALLEY DAM TYPICAL CROSS - SECTION

DISCLAIMER

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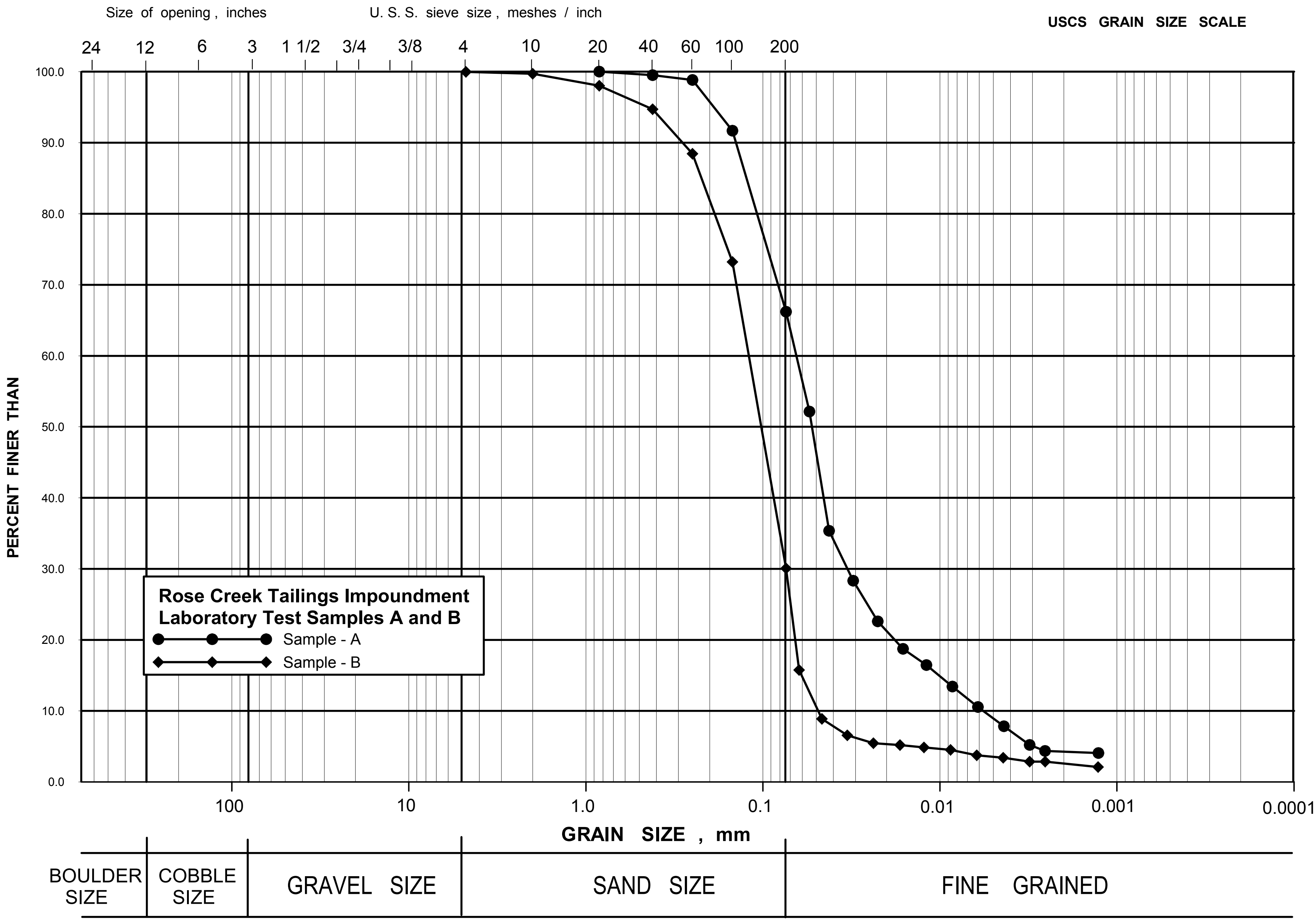
DRAWING TAKEN FROM:

	Scale 1:300 Job No 792-2025 Drawing No 7922025-9 Drawn by W.T. / Reviewed by D.G. Date August 1980 / Date August 1980	CYPRUS ANVIL Faro Yukon Territory Cyprus Anvil Mining Corporation	CROSS VALLEY DAM AND SPILLWAYS CROSS - SECTIONS	Golder Associates Golder Geotechnical Consultants Ltd. Sheet 9 of 12

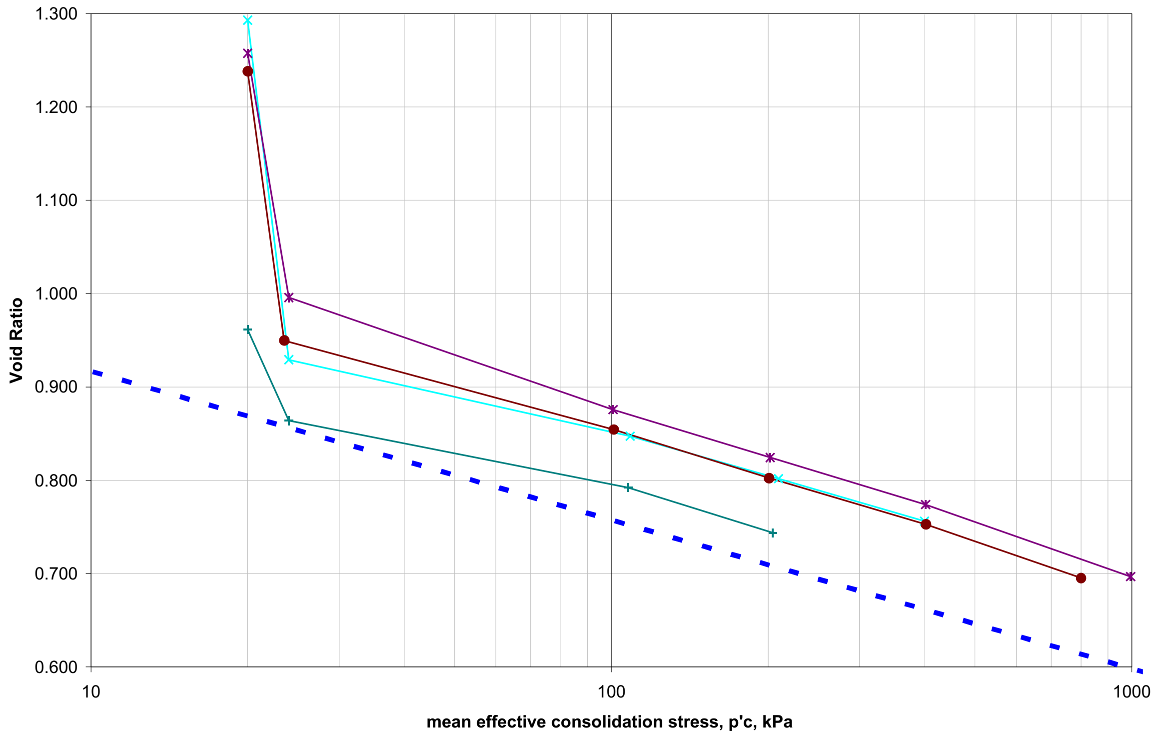
PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	CROSS VALLEY DAM TYPICAL SECTION			
PROJECT No.	03-1413-080		FILE No.	FIGURES 3
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04		
CHECK	CJ	06FEB04	FIGURE 2.9	
REVIEW				

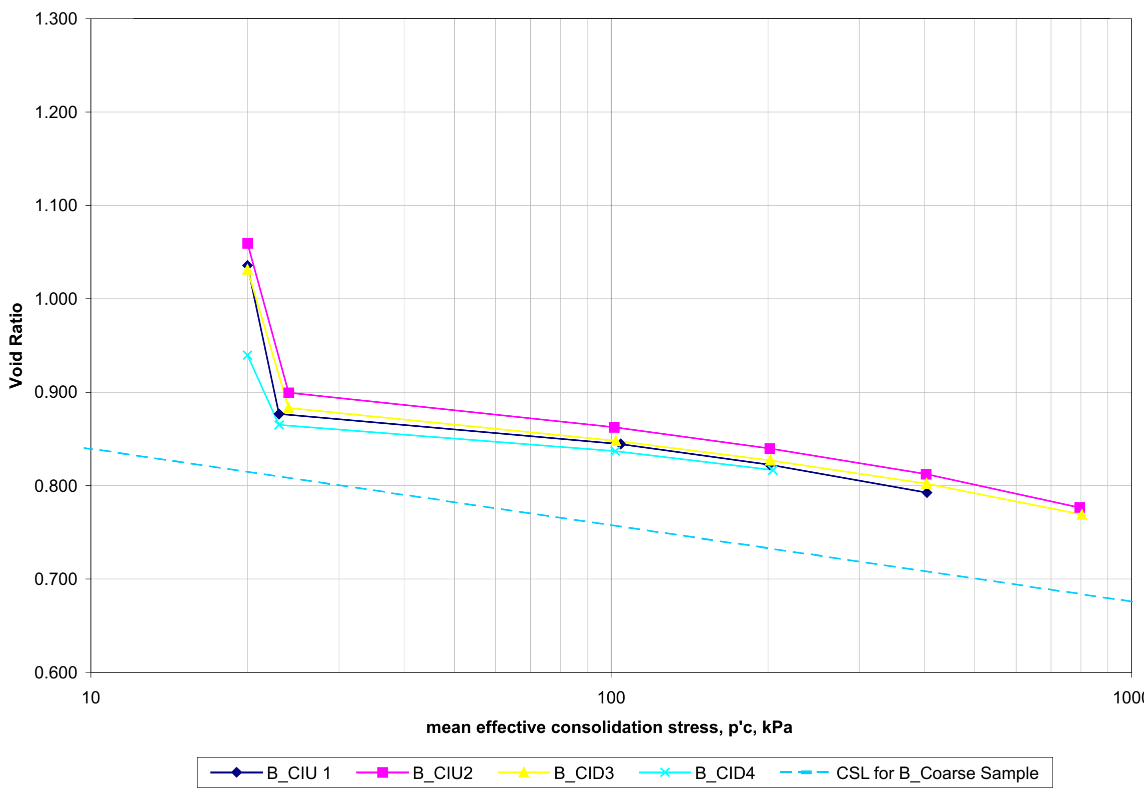
Golder Associates

REVISION DATE: BY: COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 3.cdr



Project No. 03-1413-080 **Drawn** LL **Reviewed** JCC **Date** 05/04/04	Golder Associates	**GRAIN SIZE DISTRIBUTION OF TAILINGS SAMPLES**	**Figure 3.1**





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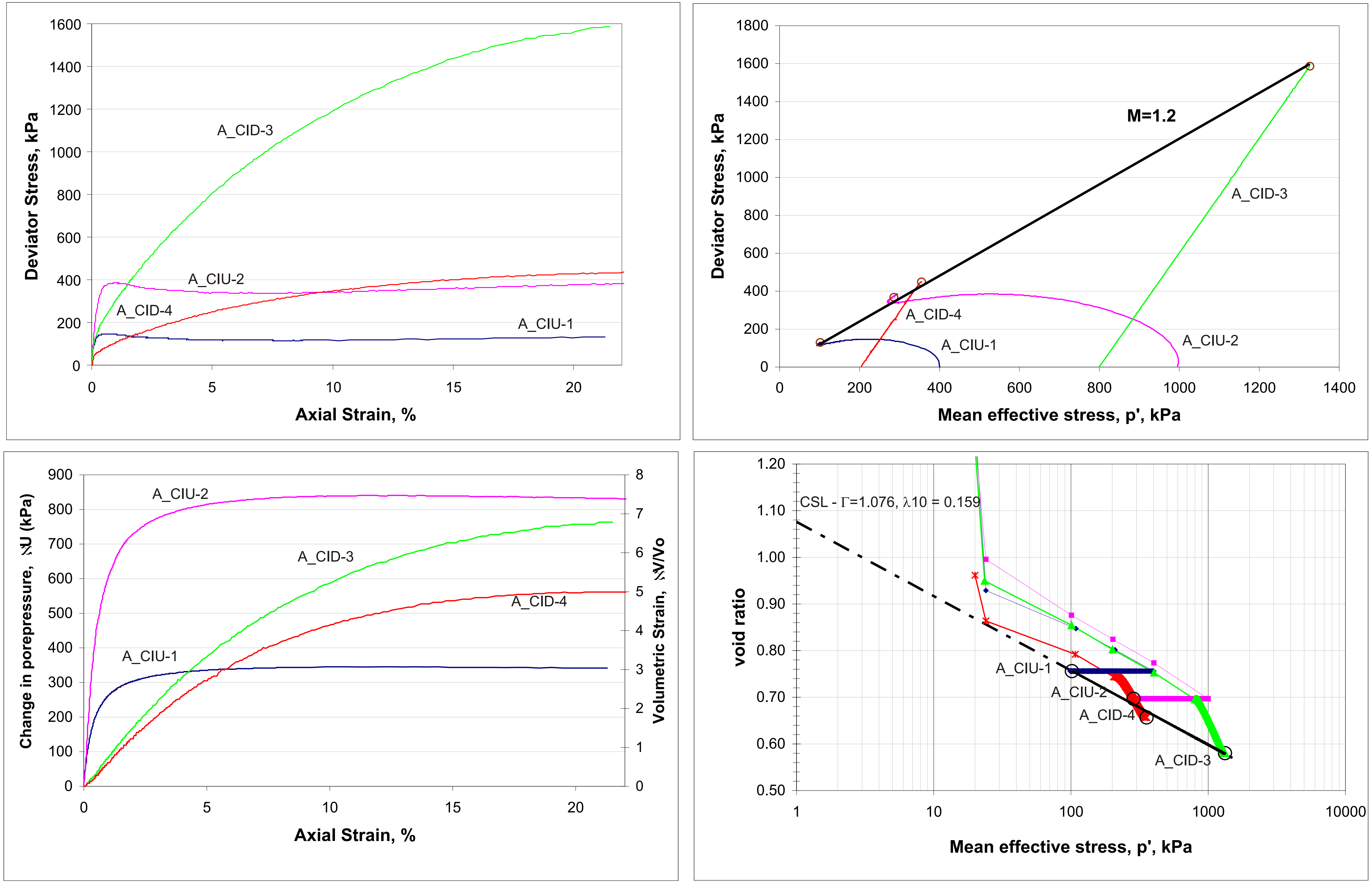
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
NORMAL CONSOLIDATION LINES FROM TRIAXIAL SAMPLES

Golder Associates

PROJECT No.		03-1413-080	FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	**FIGURE 3.2**	
CHECK	JC	06FEB03		
REVIEW				



PROJECT

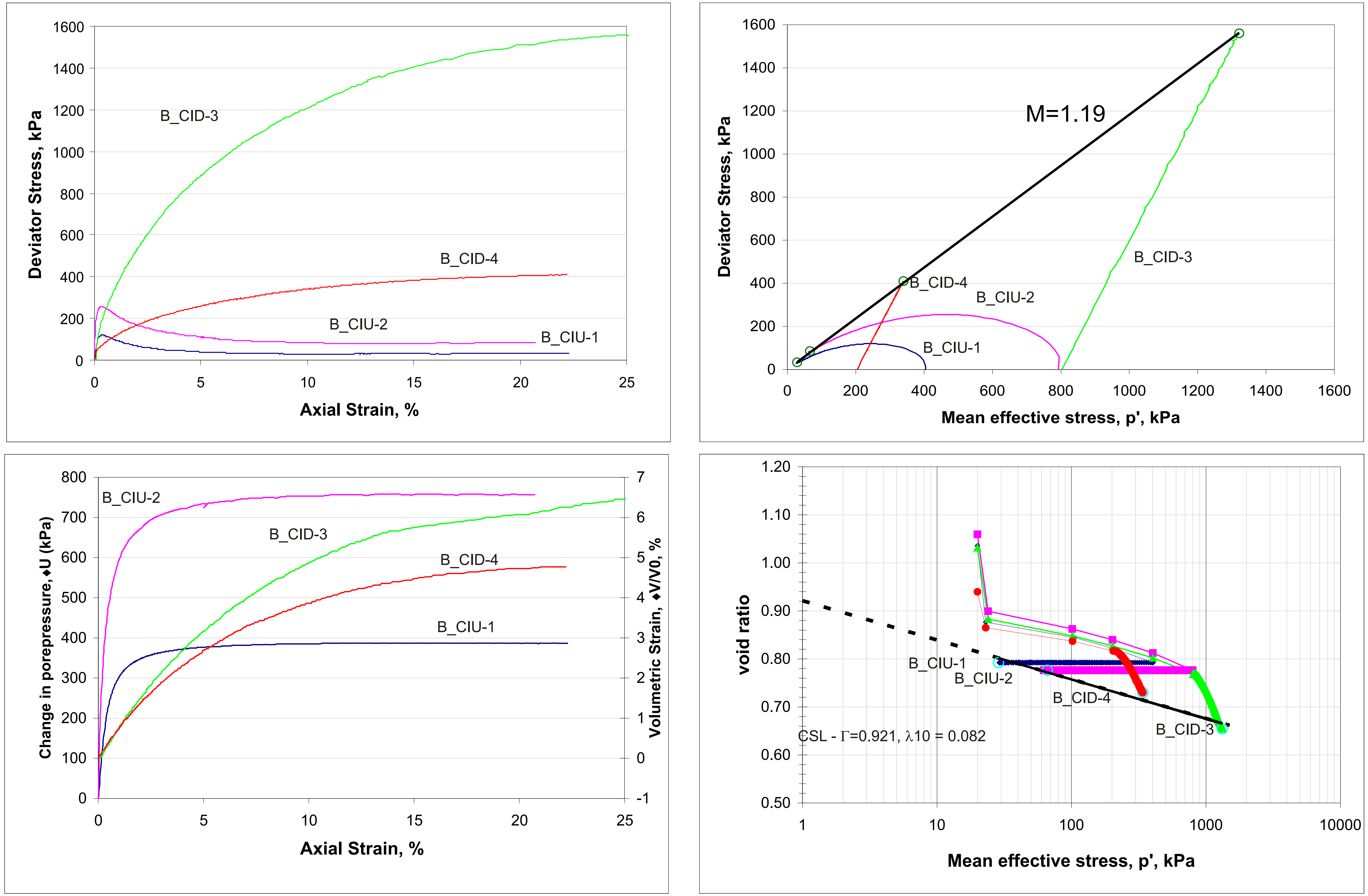
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

SUMMARY OF CRITICAL STATE TESTING
FINE SAMPLE

PROJECT No.	03-1413-080		FILE No. 03-1413-080	
DESIGN	JC	28APR04	SCALE	REV.
CADD	SS	28APR04	**FIGURE 3-3**	
CHECK	JC	28APR04		
REVIEW				

REVISION DATE:28-APR-04 BY: SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 4.cdr



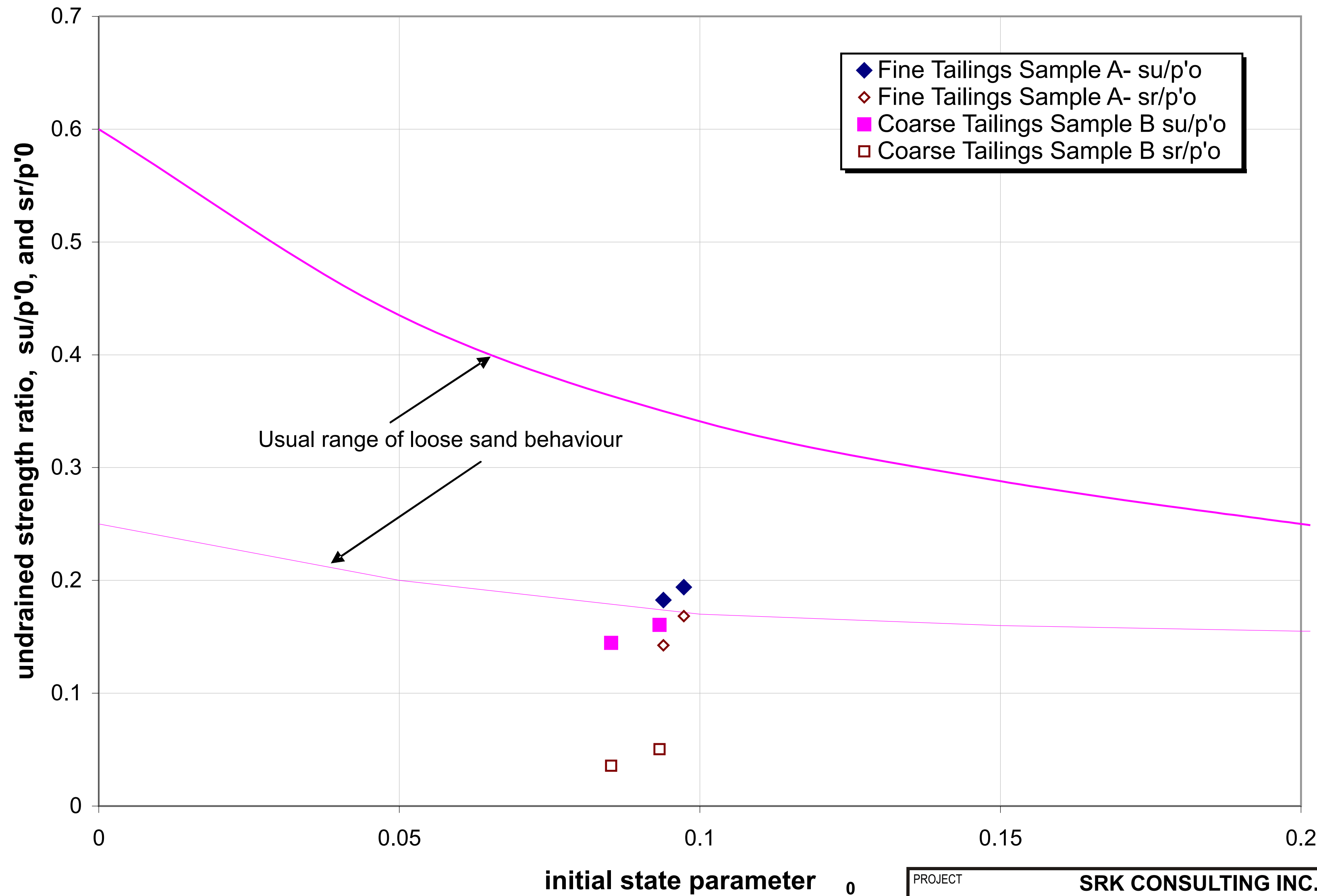
REVISION DATE:28-APR-04 BY: SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 4.cdr

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	SUMMARY OF CRITICAL STATE TESTING COARSE SAMPLE

Golder Associates

PROJECT No.	03-1413-080		FILE No. 03-1413-080
DESIGN	JC	28APR04	SCALE REV.
CADD	SS	28APR04	FIGURE 3-4
CHECK	JC	28APR04	
REVIEW			

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr



PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

UNDRAINED STRENGTH RATIOS

Golder Associates

PROJECT No.	03-1413-078		FILE No. FIGURES 2	
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04	**FIGURE 3.5**	
CHECK	CJ	06FEB04		
REVIEW				

CADD FILE: N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\cad\P1413080-01.dwg

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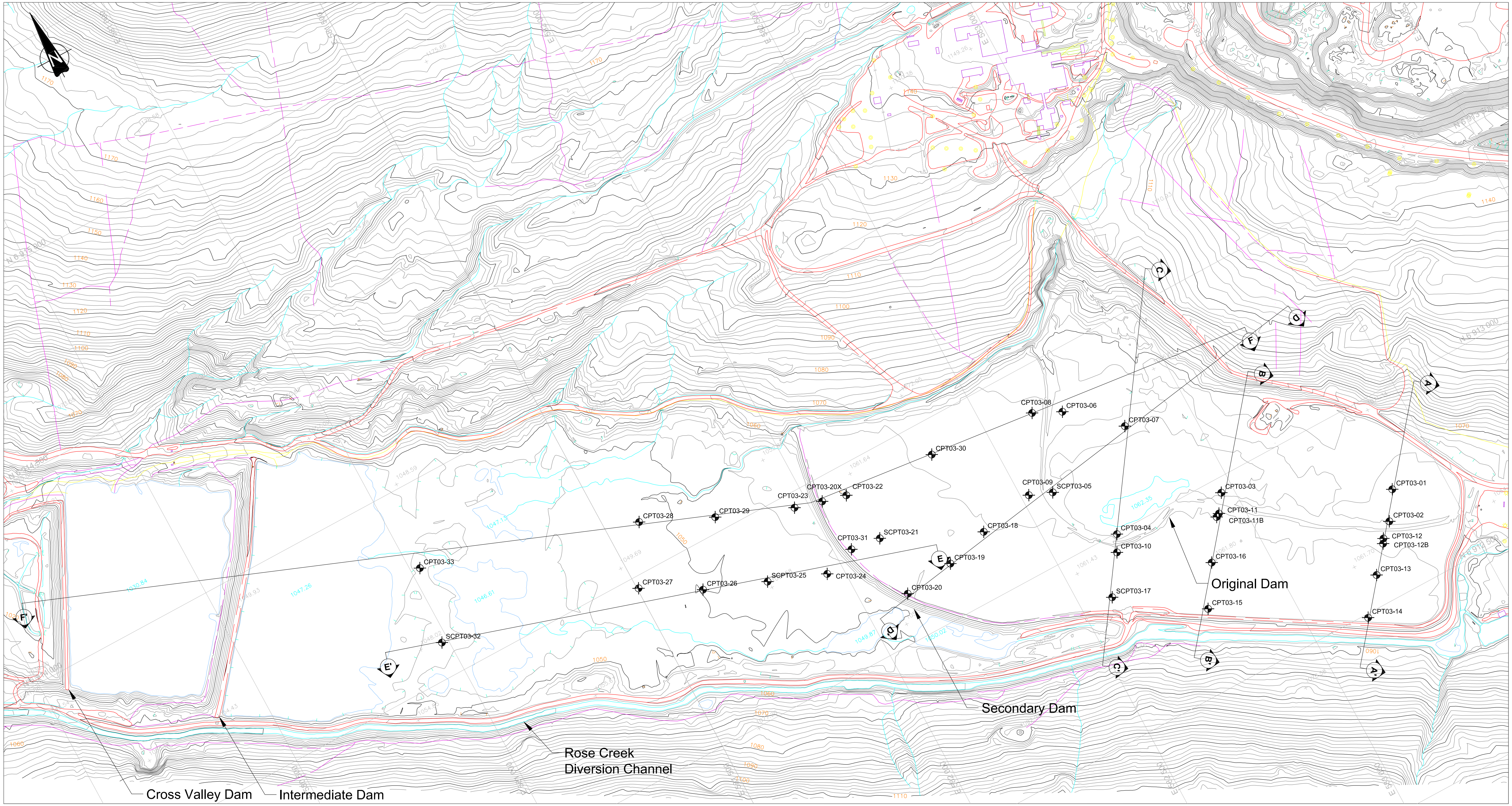


SITE PLAN

0 100 200 300 400 500

Scale in Meters

LEGEND

CPT03-01 Cone Penetration Test

SCPT03-32 Seismic Cone Penetration Test

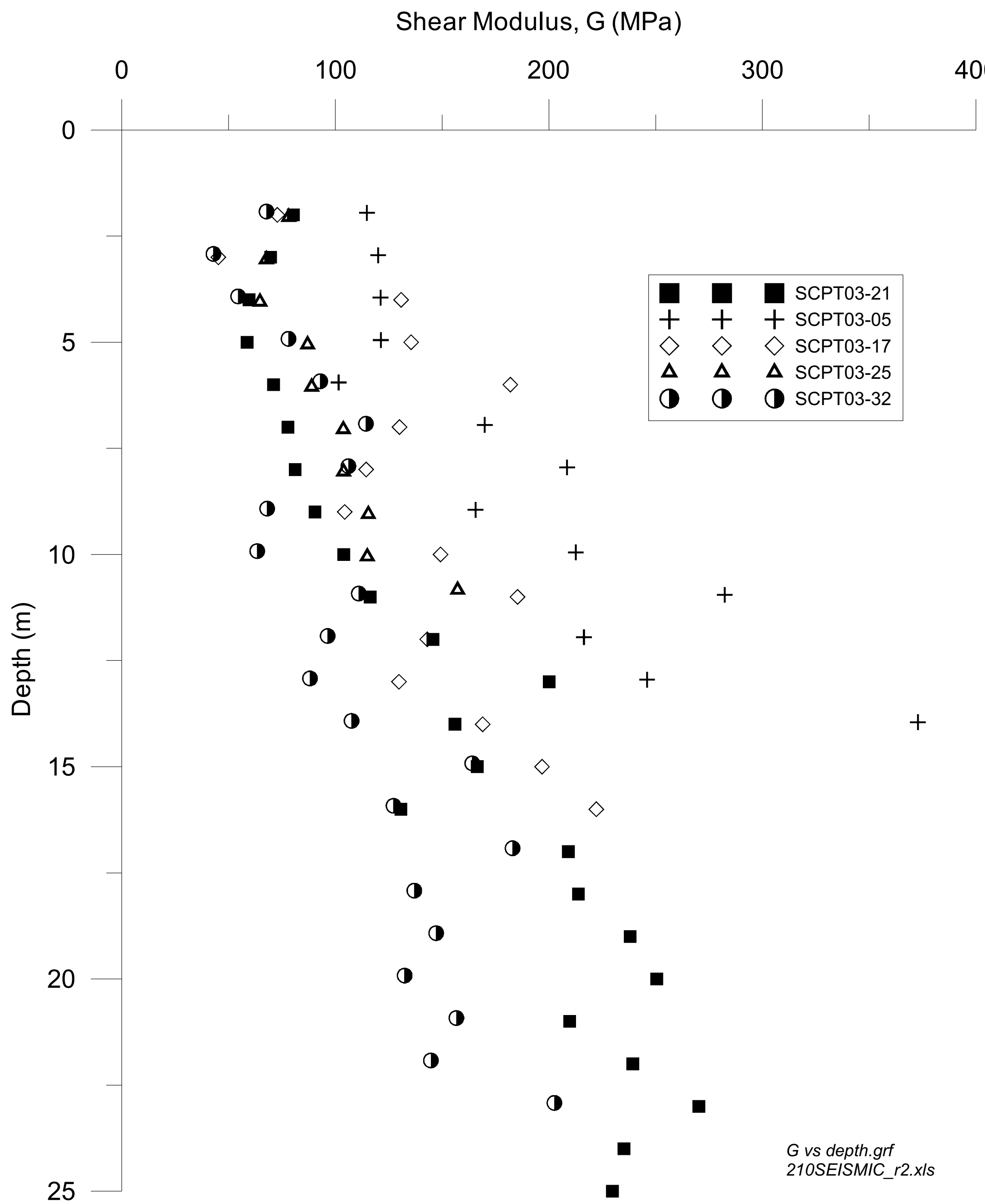
REFERENCES

Base topography from drawing 8856_north.dwg.

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	**SITE PLAN AND CPT LOCATIONS**

Golder Associates

PROJECT No.	03-1413-080		FILE No.	P1413080-01
DESIGN	JCC	29 OCT 03	SCALE AS SHOWN	REV. 0
CADD	SRR	29 OCT 03		
CHECK	JCC	29 OCT 03	**FIGURE 4.1**	
REVIEW				



PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

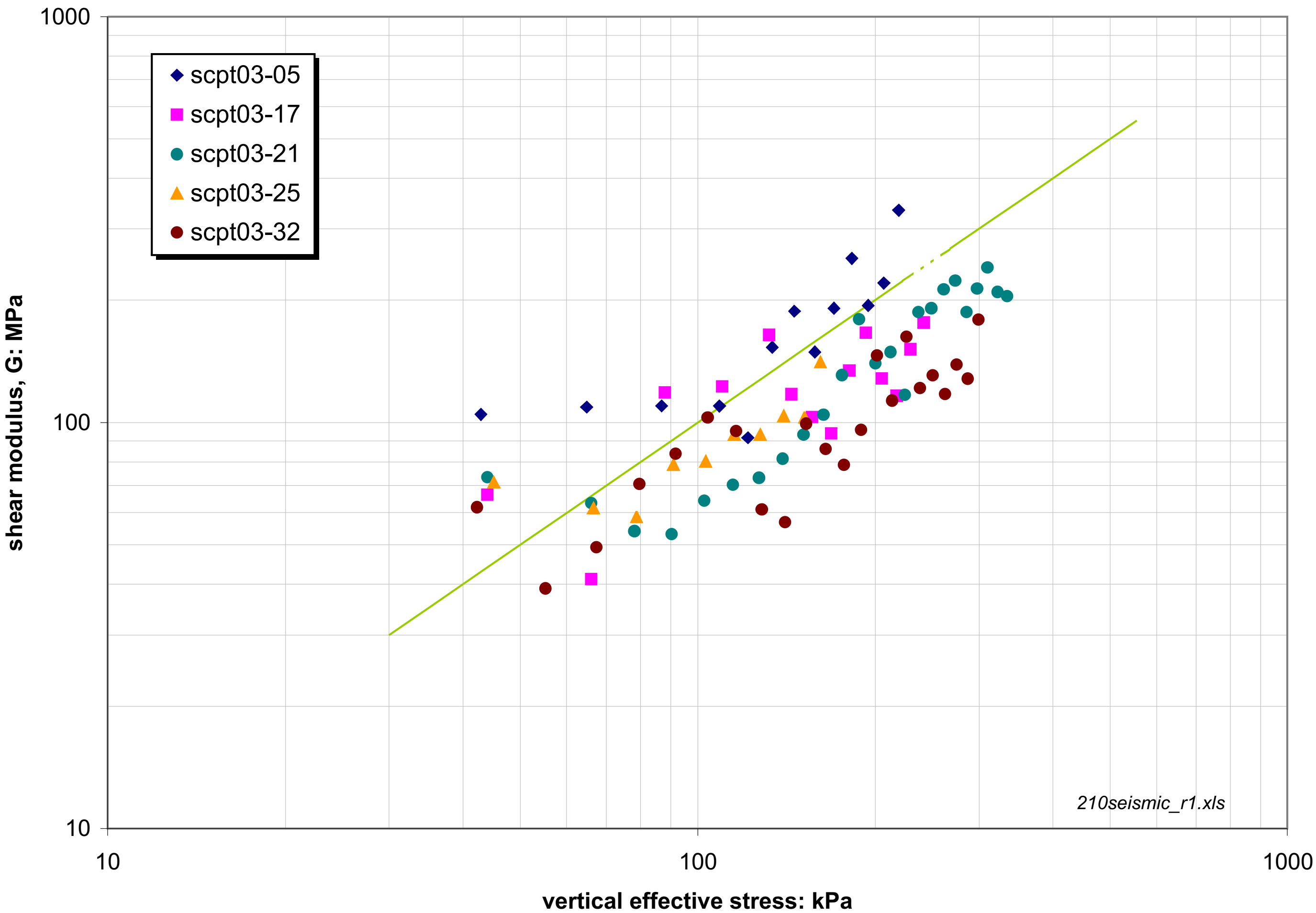
TITLE

SHEAR MODULUS FROM
INSITU GEOPHYSICAL MEASUREMENTS
VERSUS DEPTH

Golder Associates

PROJECT No.	03-1413-080		FILE No.	FIGURES
DESIGN	JC	06FEB03	SCALE NTS	REV.
CADD	SS	06FEB03		
CHECK	JC	06FEB03	**FIGURE 4.2**	
REVIEW				

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



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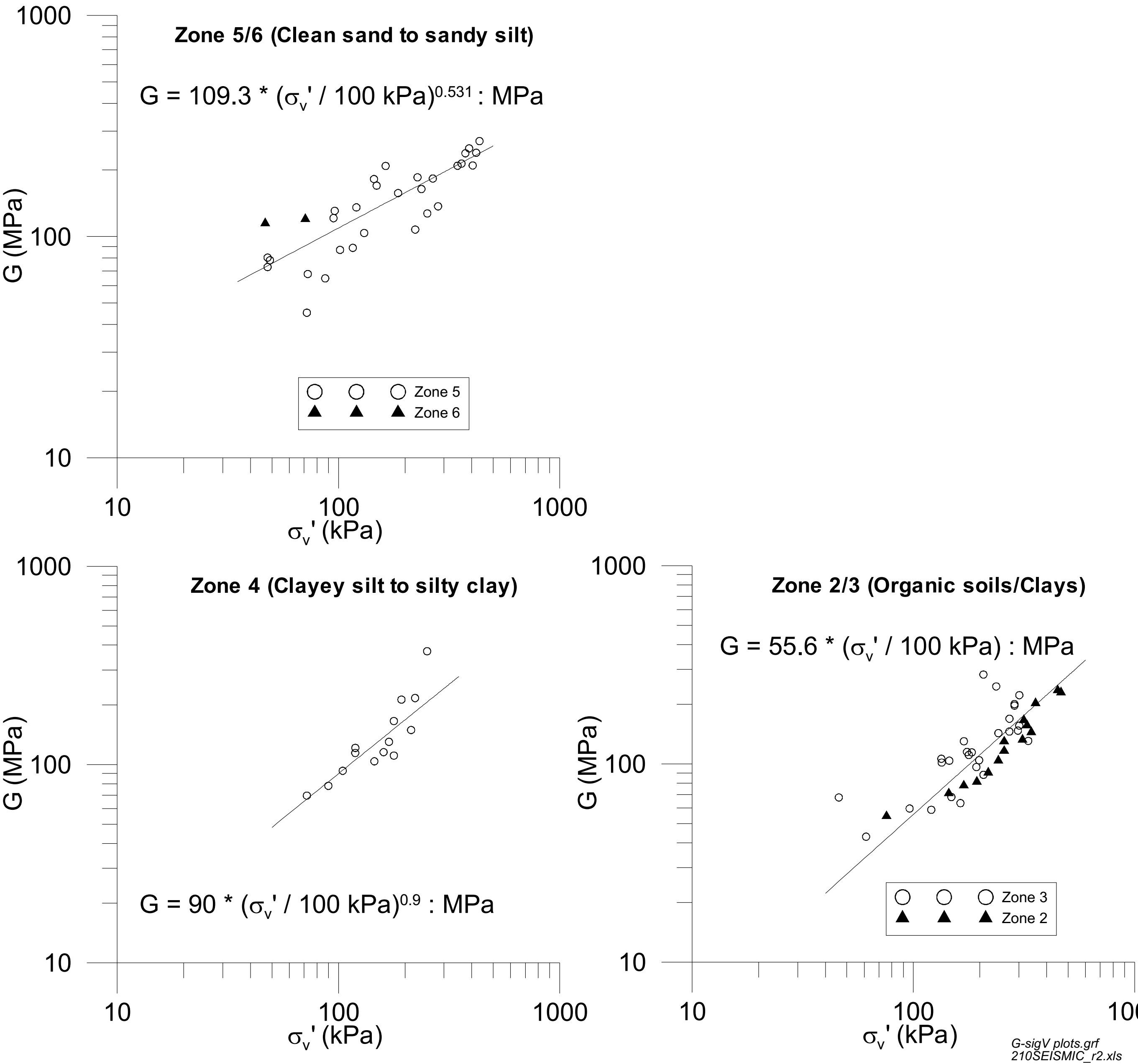
PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	SHEAR MODULUS FROM INSITU GEOPHYSICAL MEASUREMENTS VERSUS VERTICAL EFFECTIVE STRESS			
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES
	DESIGN	JC	06FEB03	SCALE / REV.
	CADD	SS	06FEB03	FIGURE 4.3
	CHECK	JC	06FEB03	
	REVIEW			

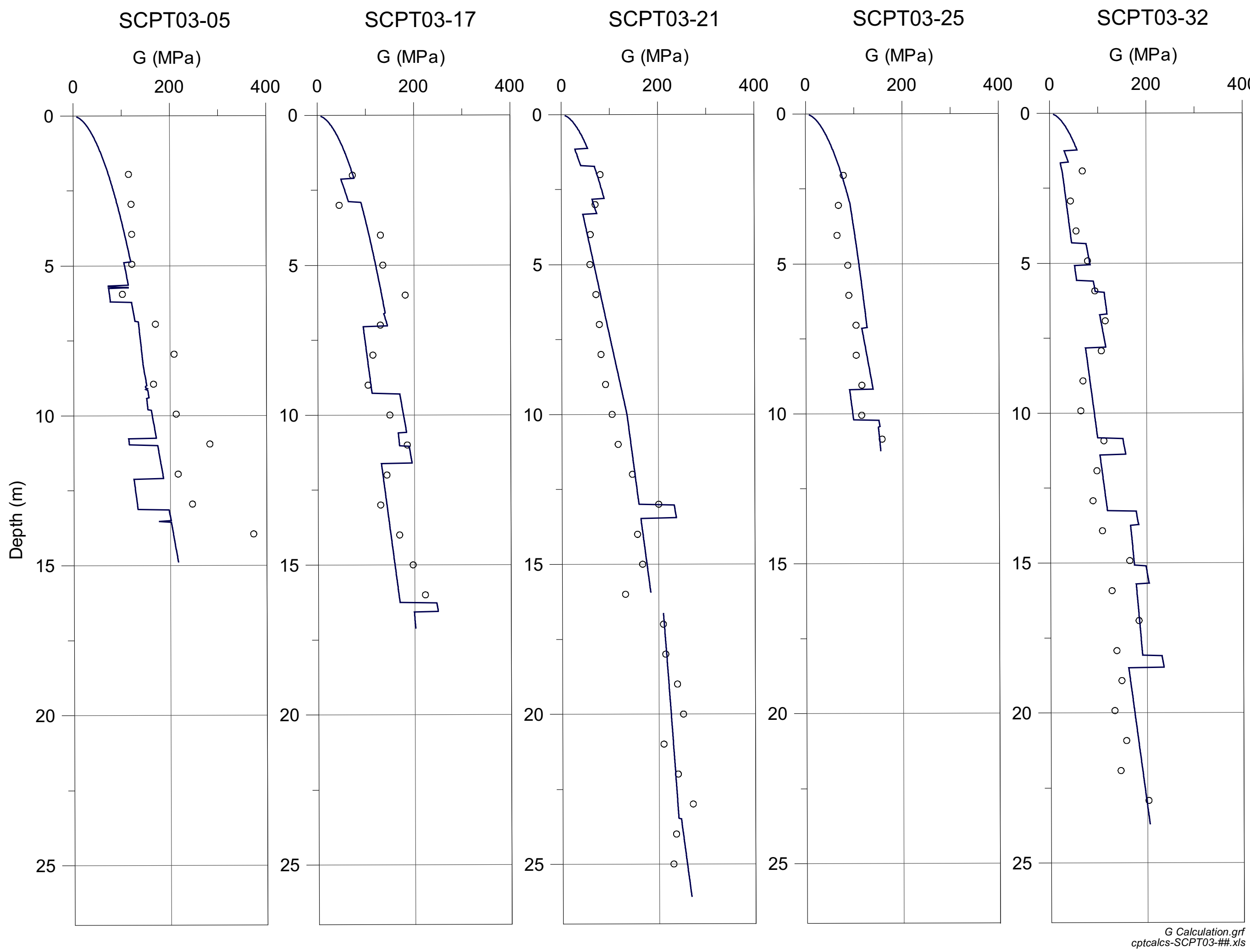


(After Been and Jefferies, 1992)

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	SHEAR MODULUS DATA PLOTTED ACCORDING TO SOIL TYPE CLASSIFICATION			
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES
	DESIGN	JC	06FEB03	SCALE / REV.
	CADD	SS	06FEB03	FIGURE 4.4
	CHECK	JC	06FEB03	
	REVIEW			

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr



PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

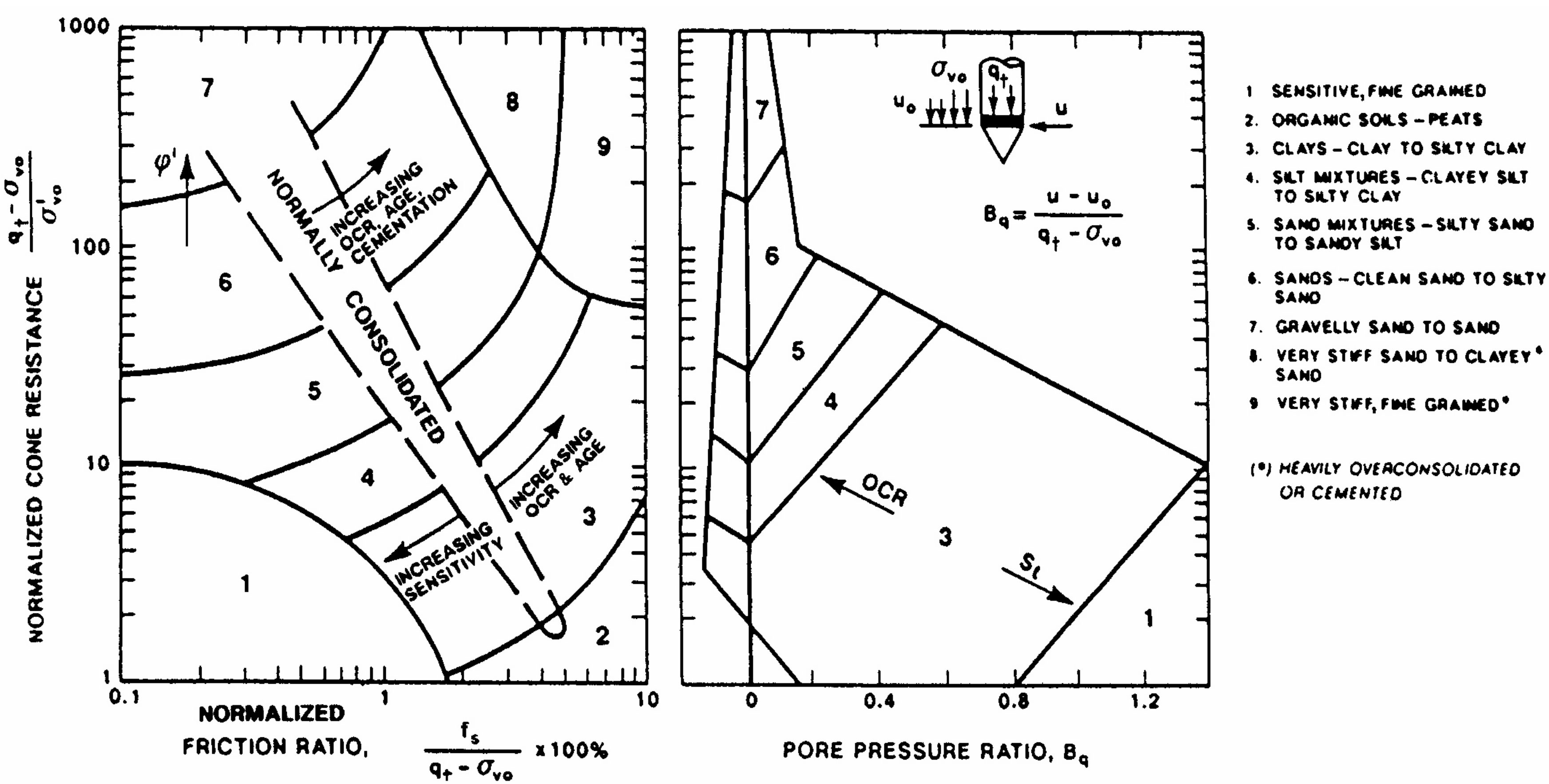
COMPARISON OF CALCULATED AND MEASURED SHEAR MODULUS VALUES

Golder Associates

PROJECT No.	03-1413-078		FILE No.	FIGURES 2
DESIGN	CJ	06FEB04	SCALE NTS	REV.
CADD	SS	06FEB04		
CHECK	CJ	06FEB04		
REVIEW				

FIGURE 4.5

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr

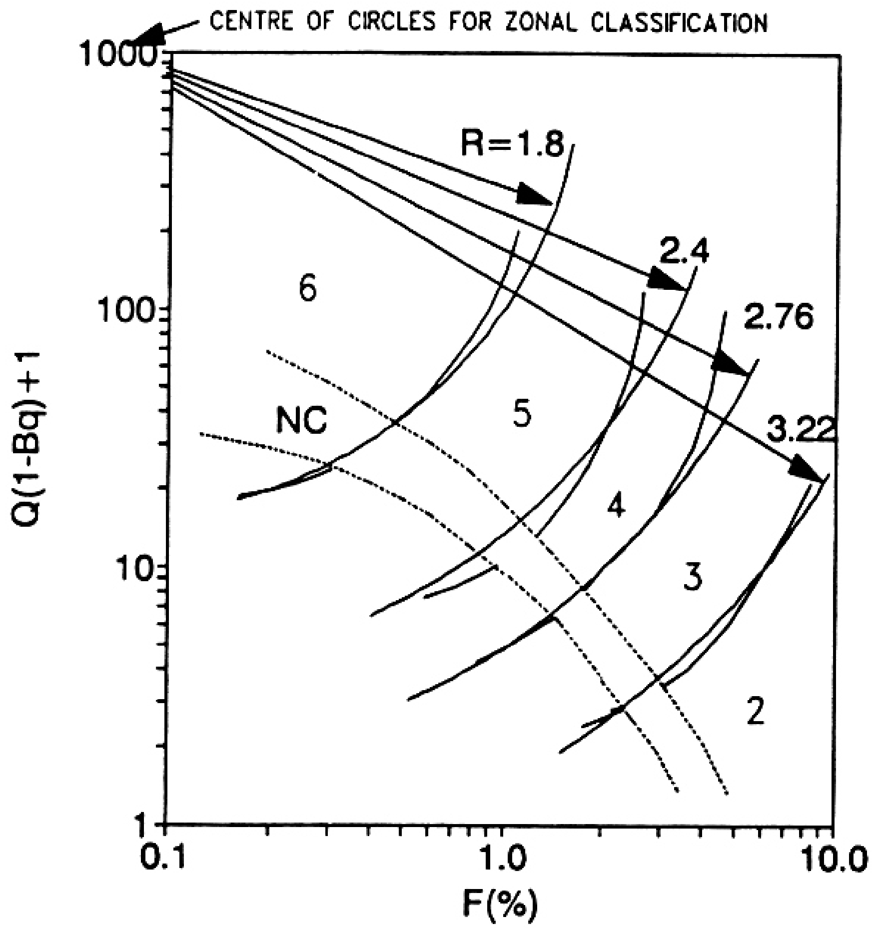


(Robertson & Campanella, 19862)

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	ESTIMATING SOIL TYPE FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-078		FILE No. FIGURES 2
DESIGN	CJ	06FEB04	SCALE / REV.
CADD	SS	06FEB04	FIGURE 5.1
CHECK	CJ	06FEB04	
REVIEW			



(After Been and Jefferies, 1992)

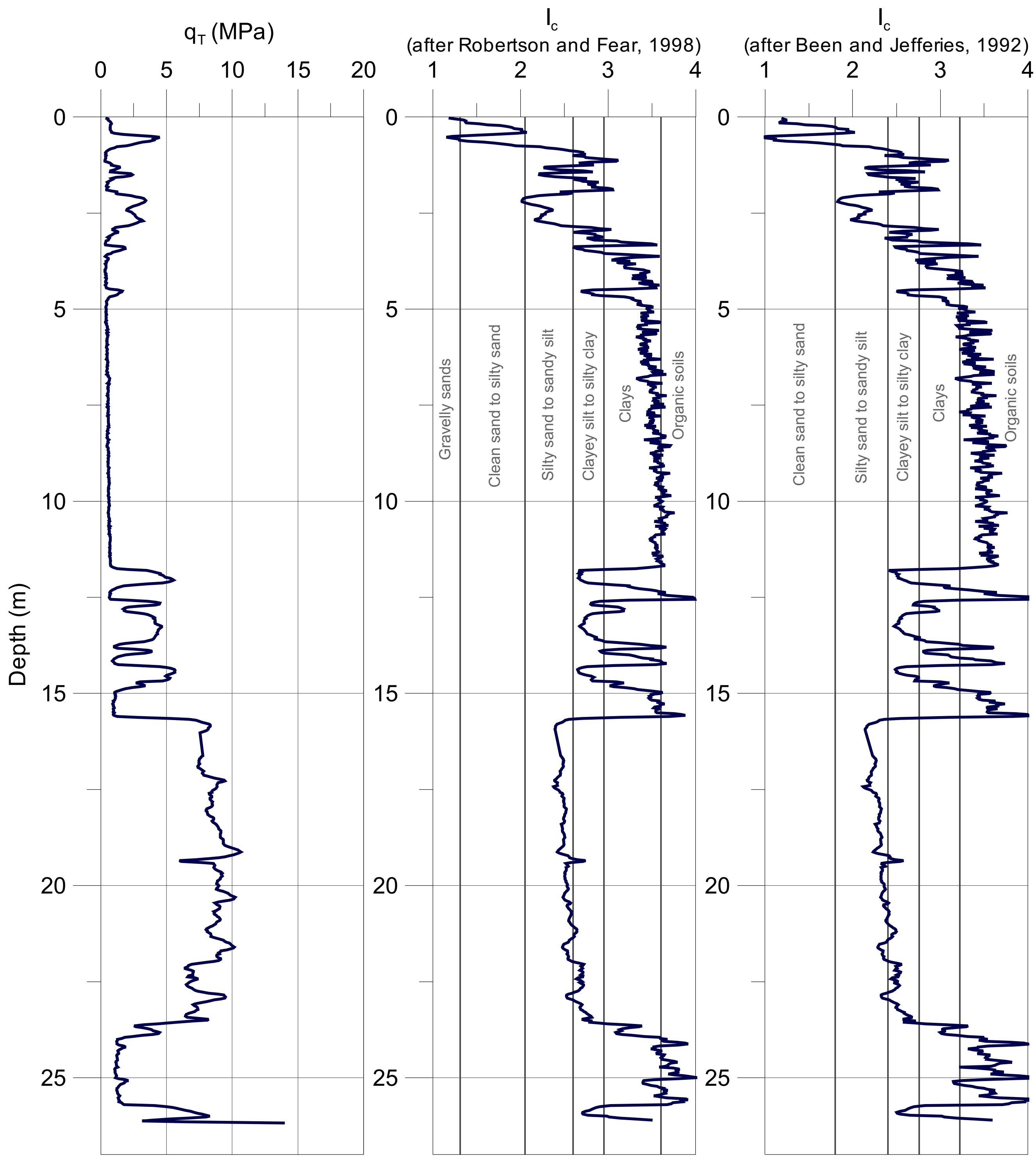
REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
ESTIMATING SOIL TYPE FROM CPT DATA AND Ic VALUES

Golder Associates

PROJECT No.		03-1413-080	FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	**FIGURE 5.2**	
CHECK	JC	06FEB03		
REVIEW				

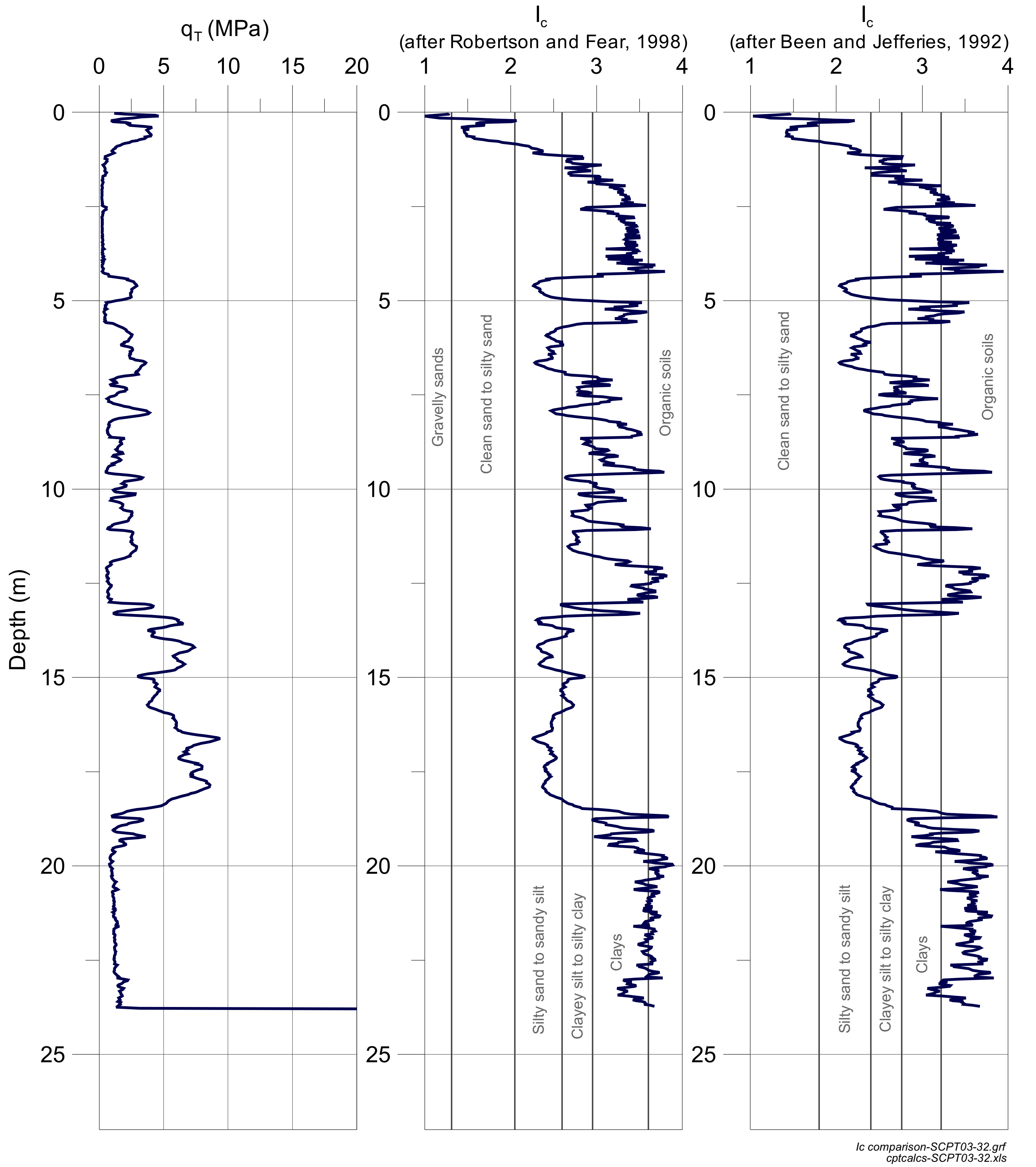


Ic comparison-SCPT03-21.grf
cptcalcs-SCPT03-21.xls

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	COMPARISON OF I_C VALUES SCPT03-21

Golder Associates	PROJECT No.		03-1413-080	FILE No. FIGURES
	DESIGN	JC	06FEB03	SCALE / REV.
	CADD	SS	06FEB03	FIGURE 5.3
	CHECK	JC	06FEB03	
	REVIEW			

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



Ic comparison-SCPT03-32.grf
cptcalcs-SCPT03-32.xls

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

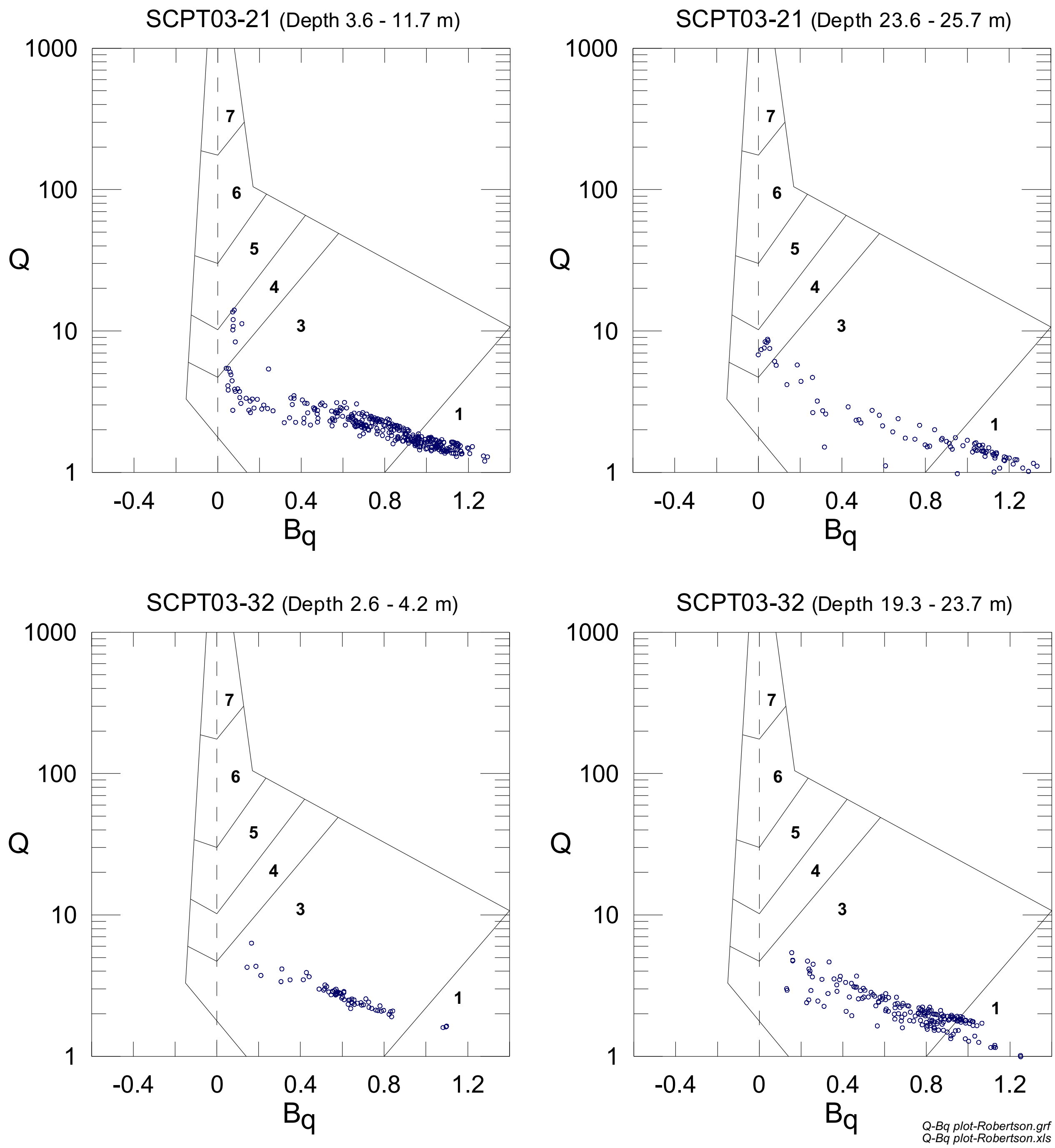
TITLE

COMPARISON OF I_C VALUES
SCPT03-32

Golder Associates

PROJECT No.		03-1413-080	FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	**FIGURE 5.4**	
CHECK	JC	06FEB03		
REVIEW				

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

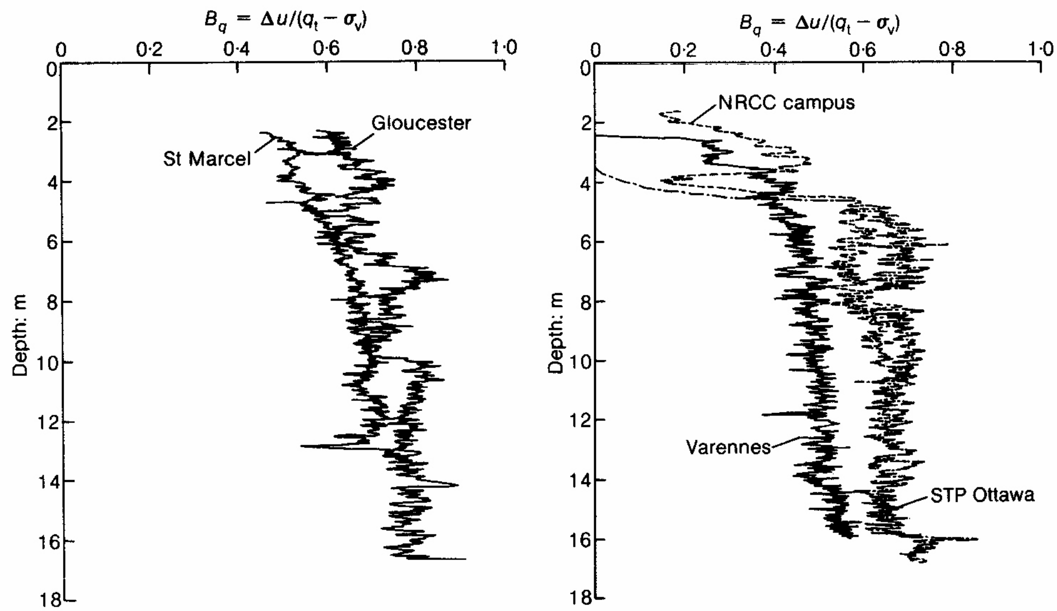


Q-Bq plot-Robertson.grf
Q-Bq plot-Robertson.xls

PROJECT	**SRK CONSULTING INC.** **ROSE CREEK TAILINGS FACILITY** **ANVIL RANGE MINING COMPLEX, YUKON**
TITLE	**CLASSIFICATION OF SOFTER SOILS** **FROM SCPT03-21 AND SCPT03-32** **USING Q-Bq PLOT (ROBERTSON, 1990) ZONES** **AS IN Figure 5.1**

Golder Associates					
PROJECT No.	03-1413-080		FILE No. FIGURES		
DESIGN	JC	06FEB03	SCALE	REV.	
CADD	SS	06FEB03	**FIGURE 5.5**		
CHECK	JC	06FEB03			
REVIEW					

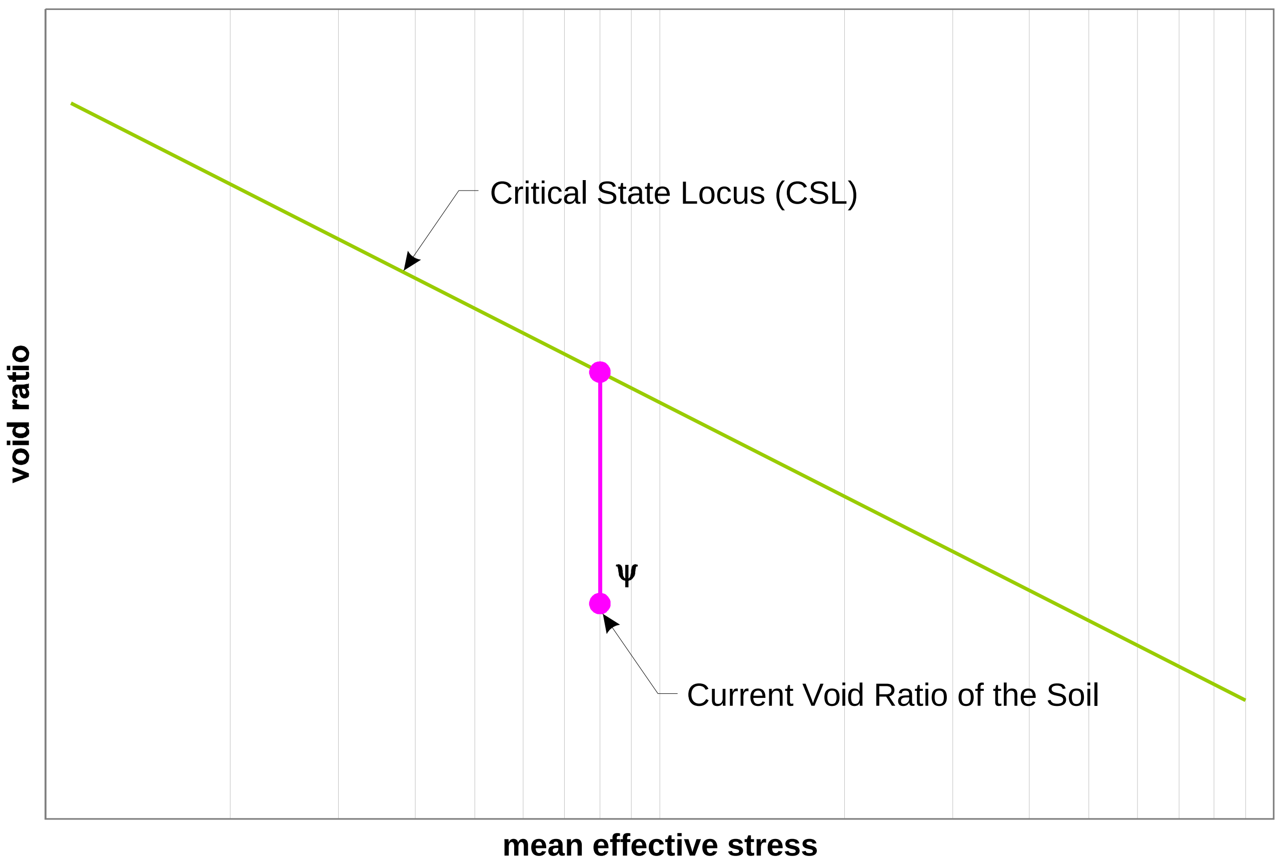
REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



(Konrad and Law, 1987)

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	**PORE PRESSURE PARAMETER B_q vs DEPTH FOR VARIOUS SENSITIVE CLAYS**

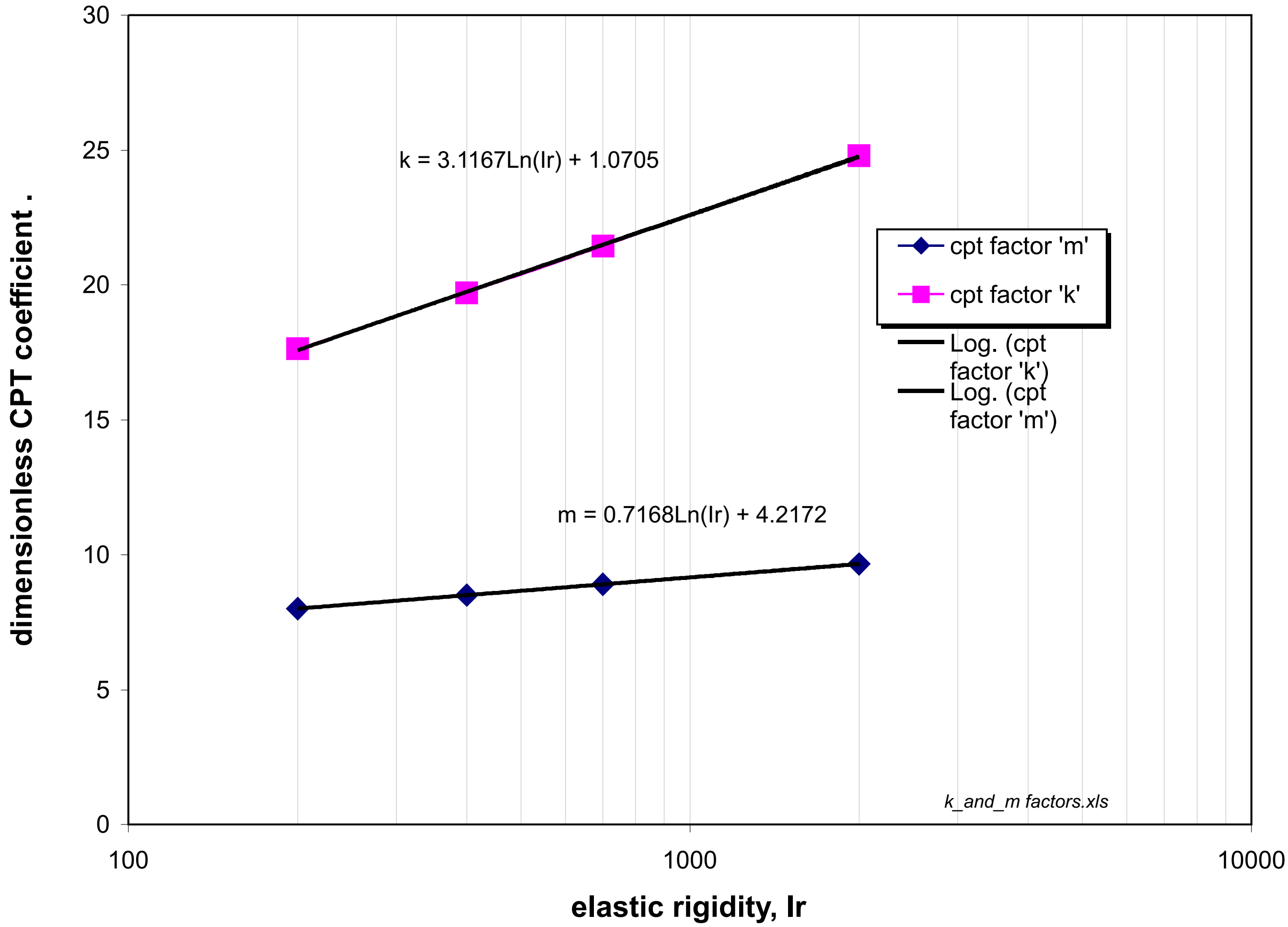
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES	
	DESIGN	JC	06FEB03	SCALE	REV.
	CADD	SS	06FEB03	**FIGURE 5.6**	
	CHECK	JC	06FEB03		
	REVIEW				



PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON		
TITLE	DEFINITION OF THE STATE PARAMETER		
PROJECT No.	03-1413-080	FILE No.	FIGURES
DESIGN	JC	06FEB03	SCALE / REV.
CADD	SS	06FEB03	FIGURE 5.7
CHECK	JC	06FEB03	
REVIEW			

Golder Associates

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

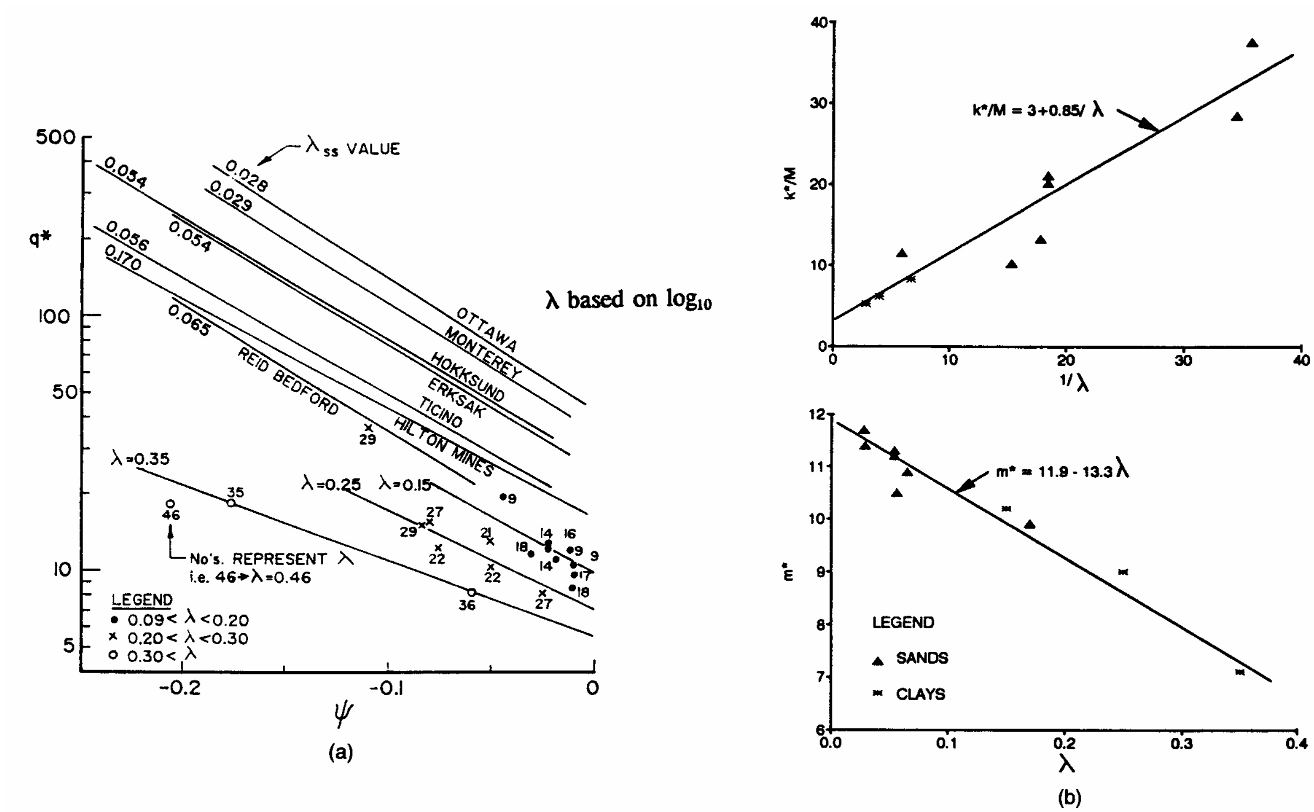


PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
ESTIMATED CPT FACTORS *k, m*
FOR ROSE CREEK TAILINGS

Golder Associates

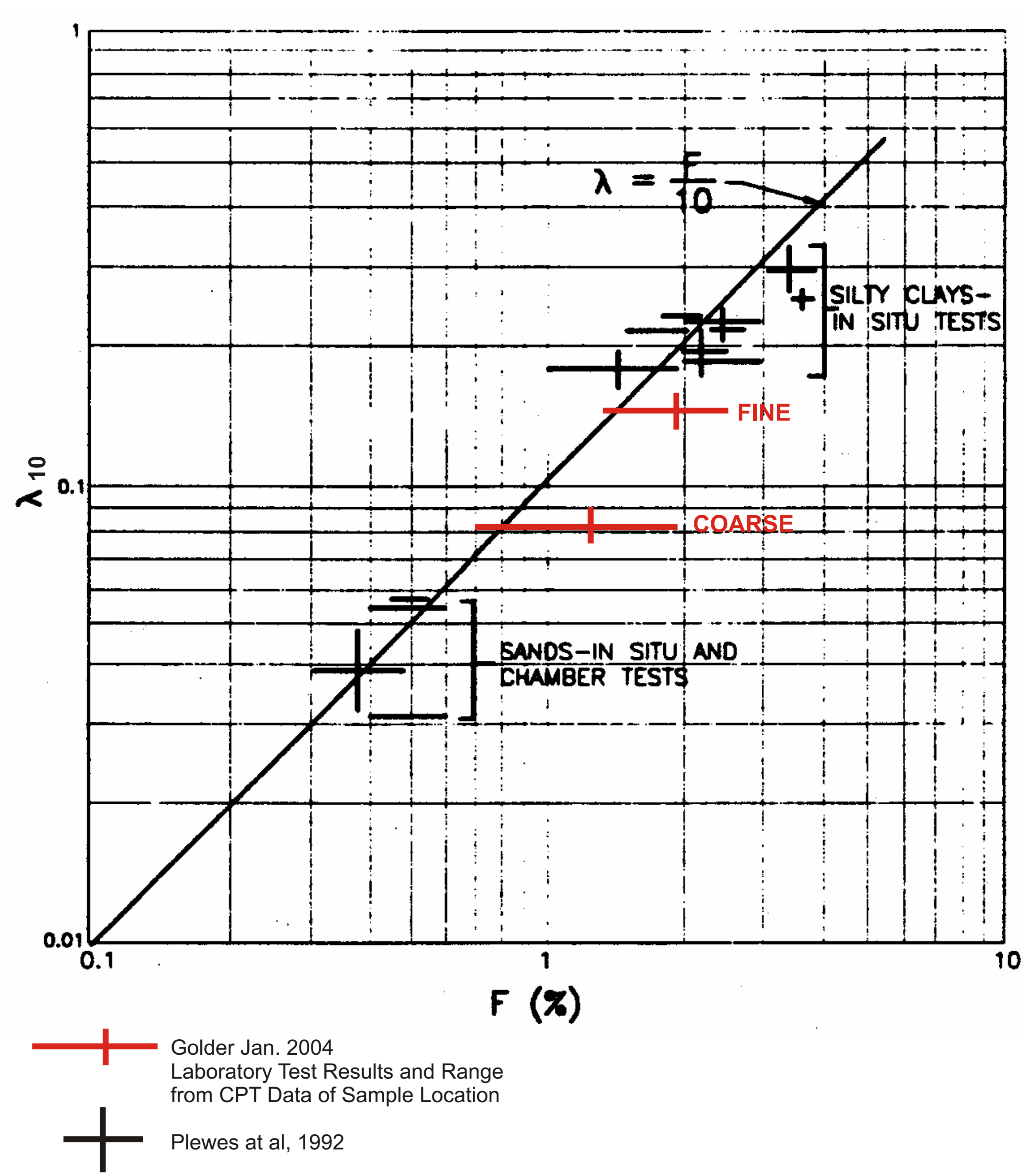
PROJECT No.		03-1413-080	FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	**FIGURE 5.8**	
CHECK	JC	06FEB03		
REVIEW				



PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	TRENDS IN CPT RESISTANCE WITH SOIL TYPE AS REPRESENTED WITH k,m			
PROJECT No.	03-1413-080		FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	FIGURE 5.9	
CHECK	JC	06FEB03		
REVIEW				

Golder Associates

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

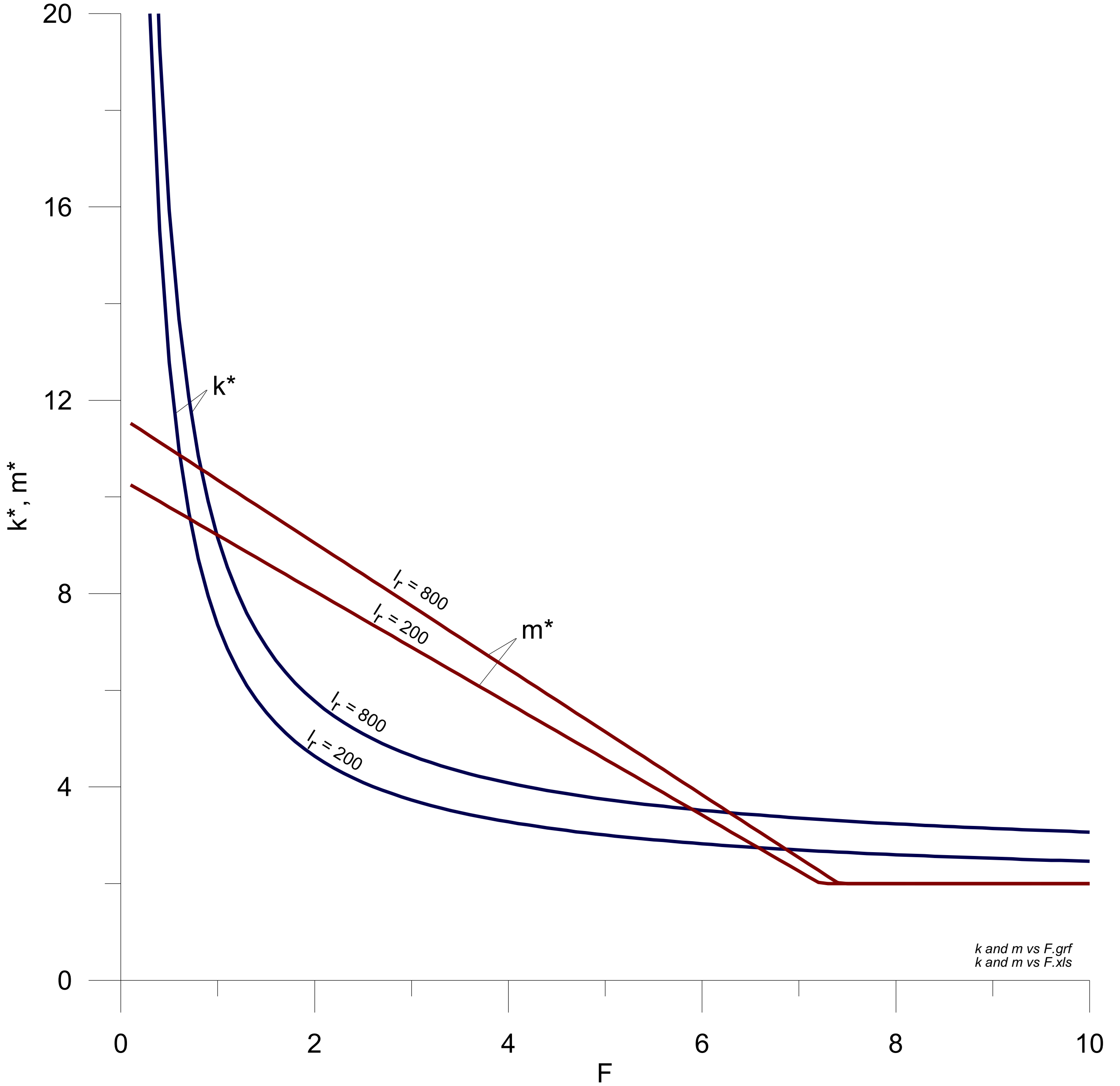


PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	APPROXIMATE RELATIONSHIP BETWEEN λ AND F SUGGESTED BY PLEWES et al (1992)

PROJECT No.	03-1413-080		FILE No. FIGURES
DESIGN	JC	06FEB03	SCALE / REV.
CADD	SS	06FEB03	FIGURE 5.10
CHECK	JC	06FEB03	
REVIEW			

Golder Associates

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



PROJECT

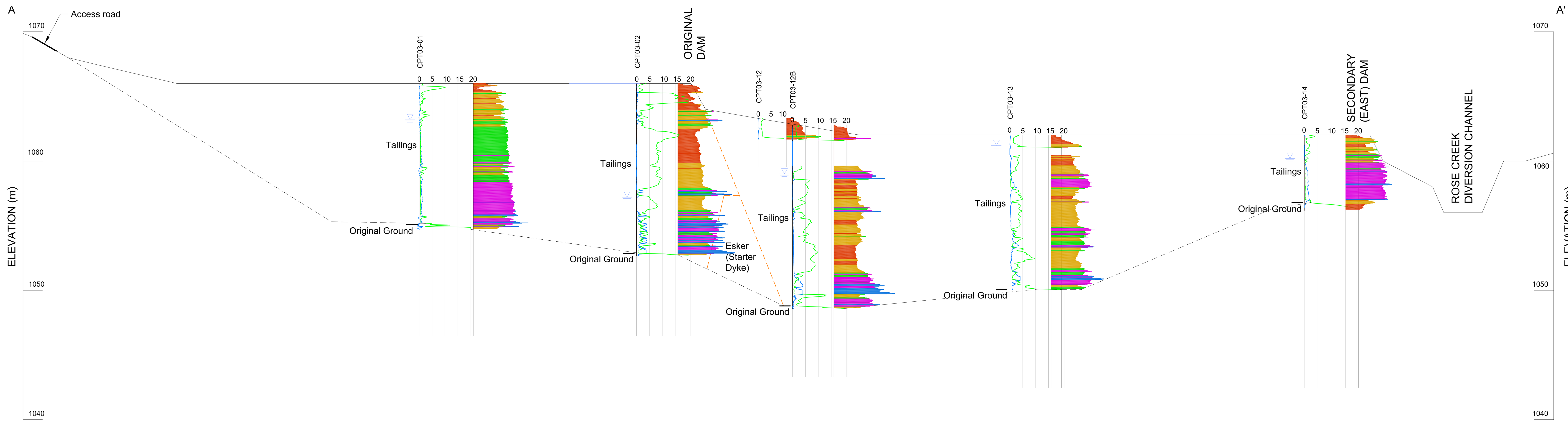
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

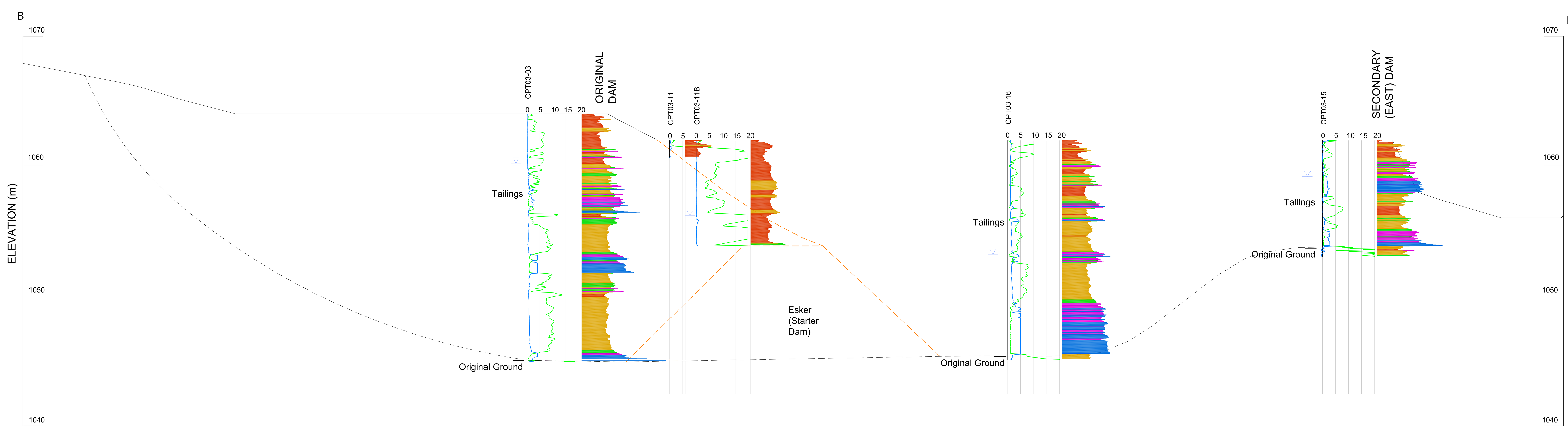
VARIATION OF *k'* AND *m'* WITH *F* AND I_r

PROJECT No.	03-1413-080		FILE No. FIGURES
DESIGN	JC	06FEB03	SCALE / REV.
CADD	SS	06FEB03	**FIGURE 5.11**
CHECK	JC	06FEB03	
REVIEW			

Golder Associates



SECTION A-A'



SECTION B-B'

LEGEND

Ic (after Been and Jefferies 1992)

- Clean Sand to Silty Sand
- Silty Sand to Sandy Silt
- Clayey Silt to Silty Clay
- Clays
- Organic Soils

- qt (MPA) from CPT
- u (m) from CPT
- Approximate Dam Fill
- Inferred Contact Tailings/Original Ground
- Estimated water table elevation in tailings from CPT data.

NOTES

1) Refer to Figure 4.1 for section locations.

2) CPT soundings carried out between October 15 to October 18 2003.

Horizontal Scale in Meters: 0 10 20 30 40 50

Vertical Scale in Meters: 0 2 4 6 8 10

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

ROSE CREEK TAILINGS IMPOUNDMENT CPT SECTIONS A , B

Golder Associates

PROJECT No. 03-1413-080			FILE No. P1413080-03.dwg	
DESIGN	JCC	11 DEC 03	SCALE AS SHOWN	REV. 0
CADD	RCR	11 DEC 03	**FIGURE 5.12**	
CHECK	JCC	11 DEC 03		
REVIEW				

SECTION C-C'

LEGEND

Ic (after Been and Jefferies 1992)

- Clean Sand to Silty Sand
- Silty Sand to Sandy Silt
- Clayey Silt to Silty Clay
- Clays
- Organic Soils

- qt (MPA) from CPT
- u (m) from CPT
- Approximate Dam Fill
- Inferred Contact Tailings/Original Ground
- Estimated water table elevation in tailings from CPT data.

NOTES

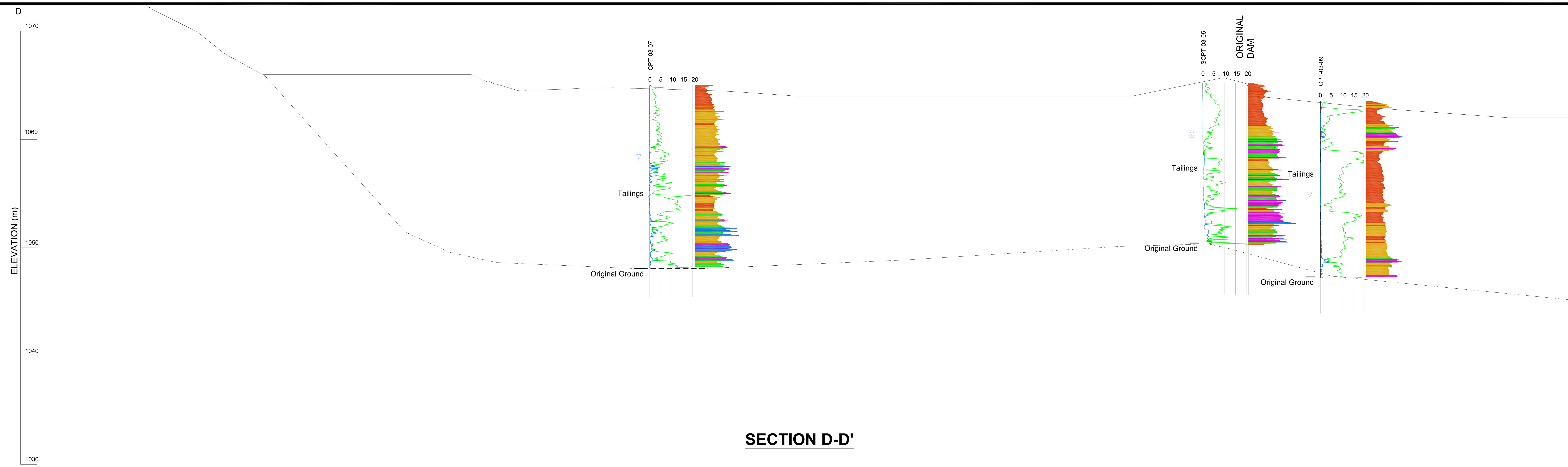
1) Refer to Figure 4.1 for section locations.

2) CPT soundings carried out between October 15 to October 18 2003.

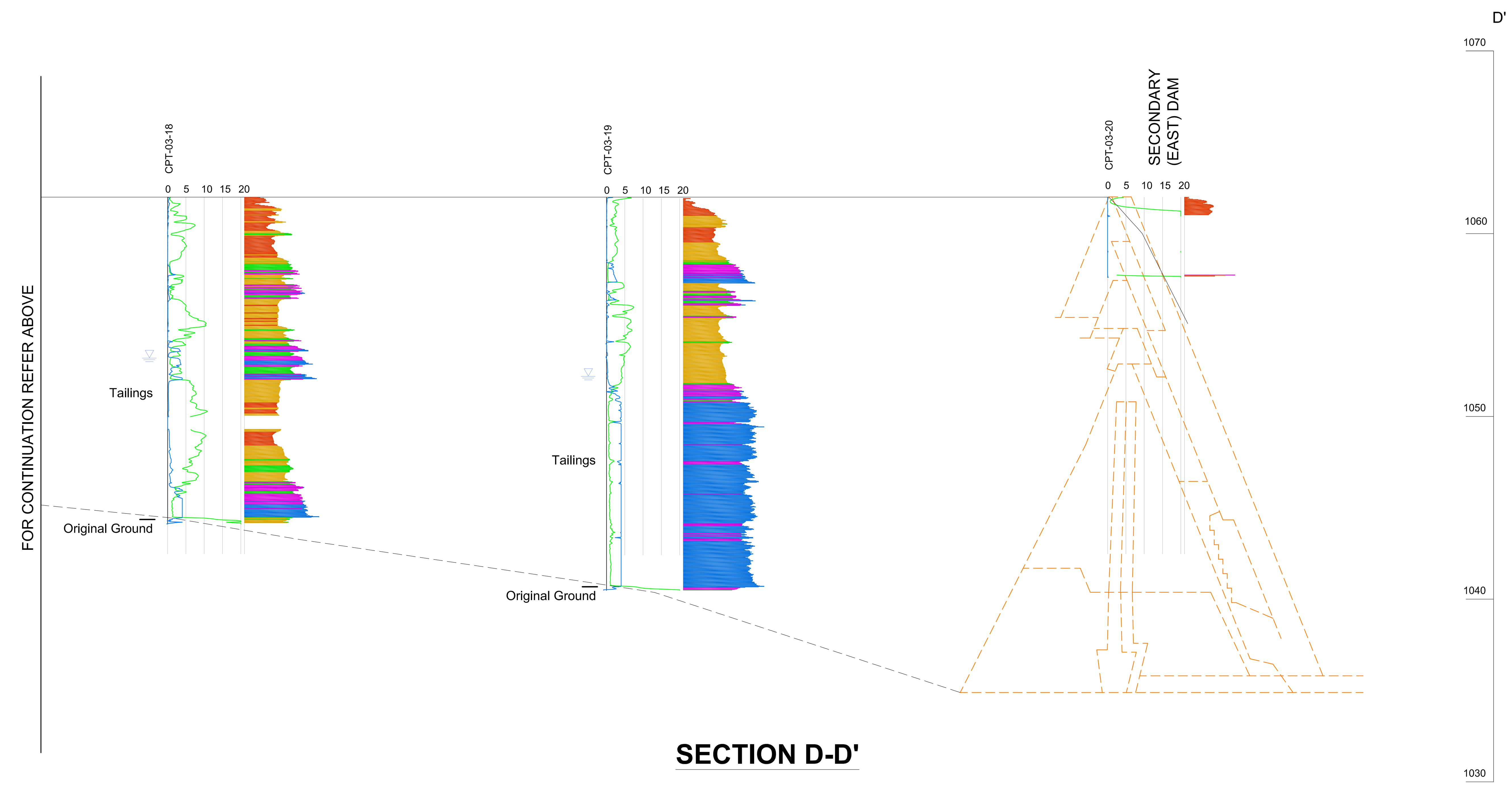
PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	ROSE CREEK TAILINGS IMPOUNDMENT CPT SECTION C

Golder Associates

PROJECT No.	03-1413-080		FILE No.	P1413080-03.dwg
DESIGN	JCC	11 DEC 03	SCALE AS SHOWN	REV. 0
CADD	RCR	11 DEC 03		
CHECK	JCC	11 DEC 03	FIGURE 5.13	
REVIEW				



SECTION D-D'



SECTION D-D'

LEGEND

Ic (after Been and Jefferies 1992)

- Clean Sand to Silty Sand
- Silty Sand to Sandy Silt
- Clayey Silt to Silty Clay
- Clays
- Organic Soils

- qt (MPA) from CPT
- u (m) from CPT
- Approximate Dam Fill
- Inferred Contact Tailings/Original Ground
- Estimated water table elevation in tailings from CPT data.

NOTES

1) Refer to Figure 4.1 for section locations.

2) CPT soundings carried out between October 15 to October 18 2003.

Horizontal Scale in Meters: 0 10 20 30 40 50

Vertical Scale in Meters: 0 2 4 6 8 10

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
ROSE CREEK TAILINGS IMPOUNDMENT
CPT SECTION D

Golder Associates

PROJECT No.	03-1413-080		FILE No.	P1413080-03.dwg
DESIGN	JCC	11 DEC 03	SCALE AS SHOWN	REV. 0
CADD	RCR	11 DEC 03		
CHECK	JCC	11 DEC 03	**FIGURE 5.14**	
REVIEW				

SECTION E-E'

SECTION E-E'

LEGEND

Ic (after Been and Jefferies 1992)

- Clean Sand to Silty Sand
- Silty Sand to Sandy Silt
- Clayey Silt to Silty Clay
- Clays
- Organic Soils

- qt (MPA) from CPT
- u (m) from CPT
- Approximate Dam Fill
- Inferred Contact Tailings/Original Ground
- Estimated water table elevation in tailings from CPT data.

NOTES

1) Refer to Figure 4.1 for section locations.

2) CPT soundings carried out between October 15 to October 18 2003.

0 10 20 30 40 50
Horizontal Scale in Meters

0 2 4 6 8 10
Vertical Scale in Meters

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
ROSE CREEK TAILINGS IMPOUNDMENT
CPT SECTION E

Golder Associates

PROJECT No. 03-1413-080			FILE No. P1413080-03.dwg
DESIGN	JCC	11 DEC 03	SCALE AS SHOWN REV. 0
CADD	RCR	11 DEC 03	
CHECK	JCC	11 DEC 03	**FIGURE 5.15**
REVIEW			

SECTION F-F'

SECTION F-F'

LEGEND

Ic (after Been and Jefferies 1992)

- Clean Sand to Silty Sand
- Silty Sand to Sandy Silt
- Clayey Silt to Silty Clay
- Clays
- Organic Soils

- qt (MPA) from CPT
- u (m) from CPT
- Approximate Dam Fill
- Inferred Contact Tailings/Original Ground
- Estimated water table elevation in tailings from CPT data.

NOTES

1) Refer to Figure 4.1 for section locations.

2) CPT soundings carried out between October 15 to October 18 2003.

0 10 20 30 40 50
Horizontal Scale in Meters

0 2 4 6 8 10
Vertical Scale in Meters

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

**ROSE CREEK TAILINGS IMPOUNDMENT
CPT SECTION F**

Golder Associates

PROJECT No.	03-1413-080		FILE No.	P1413080-03.dwg
DESIGN	JCC	11 DEC 03	SCALE AS SHOWN	REV. 0
CADD	RCR	11 DEC 03		
CHECK	JCC	11 DEC 03		
REVIEW				

FIGURE 5.16

SECTION F-F' (cont.)

LEGEND

Ic (after Been and Jefferies 1992)

- Clean Sand to Silty Sand
- Silty Sand to Sandy Silt
- Clayey Silt to Silty Clay
- Clays
- Organic Soils

- qt (MPA) from CPT
- u (m) from CPT
- Approximate Dam Fill
- Inferred Contact Tailings/Original Ground
- Estimated water table elevation in tailings from CPT data.

NOTES

1) Refer to Figure 4.1 for section locations.

2) CPT soundings carried out between October 15 to October 18 2003.

0 10 20 30 40 50
Horizontal Scale in Meters

0 2 4 6 8 10
Vertical Scale in Meters

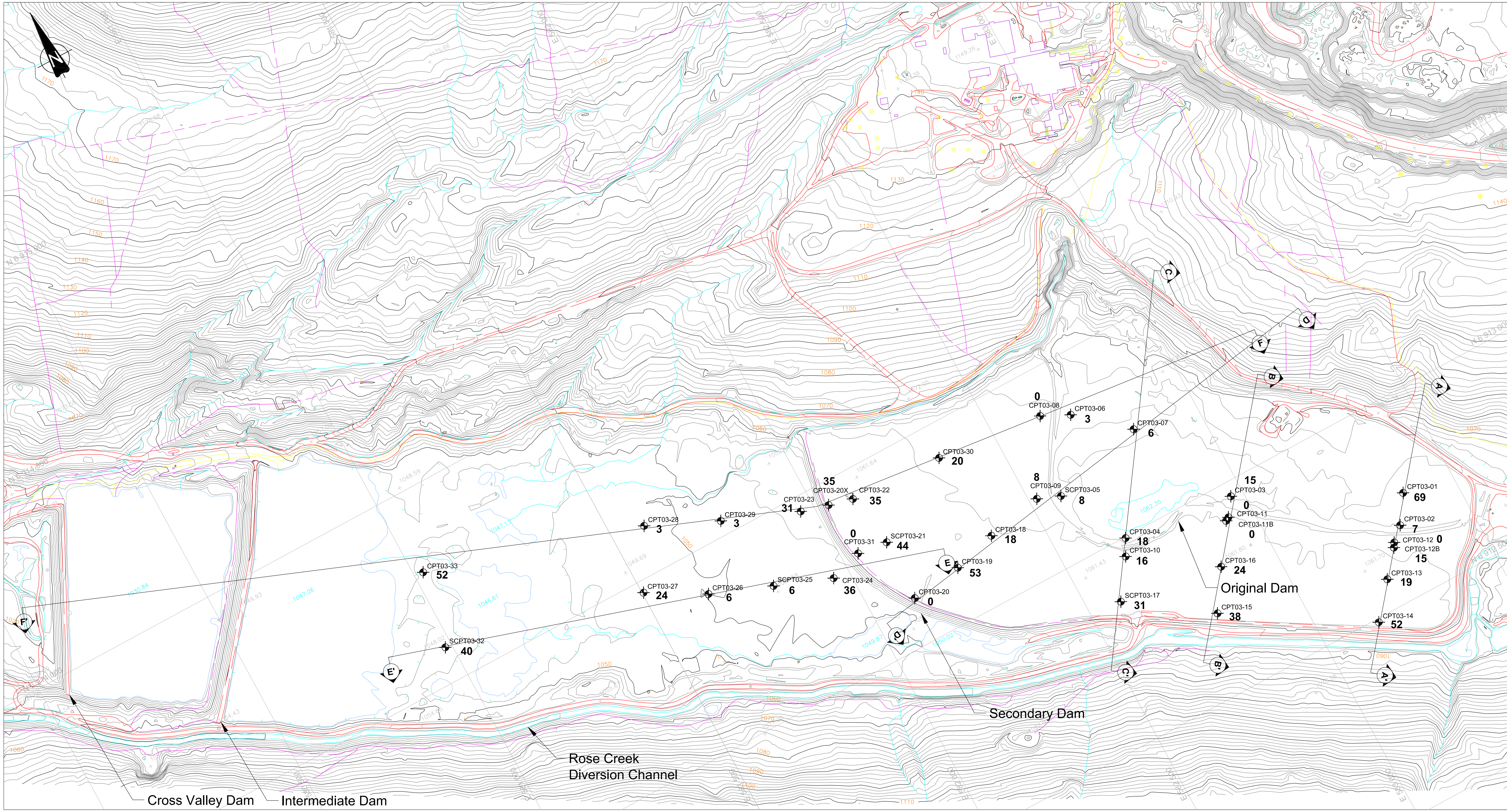
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
ROSE CREEK TAILINGS IMPOUNDMENT
CPT SECTION F (CONT.)

Golder Associates

PROJECT No.	03-1413-080		FILE No.	P1413080-03.dwg
DESIGN	JCC	11 DEC 03	SCALE AS SHOWN	REV. 0
CADD	RCR	11 DEC 03		
CHECK	JCC	11 DEC 03		
REVIEW				

FIGURE 5.17



SITE PLAN

0 100 200 300 400 500

Scale in Meters

LEGEND

CPT03-01	Cone Penetration Test
SCPT03-32	Seismic Cone Penetration Test
40	%ψ>0 (contractive) in CPT Sounding

REFERENCES

Base topography from drawing 8856_north.dwg.

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

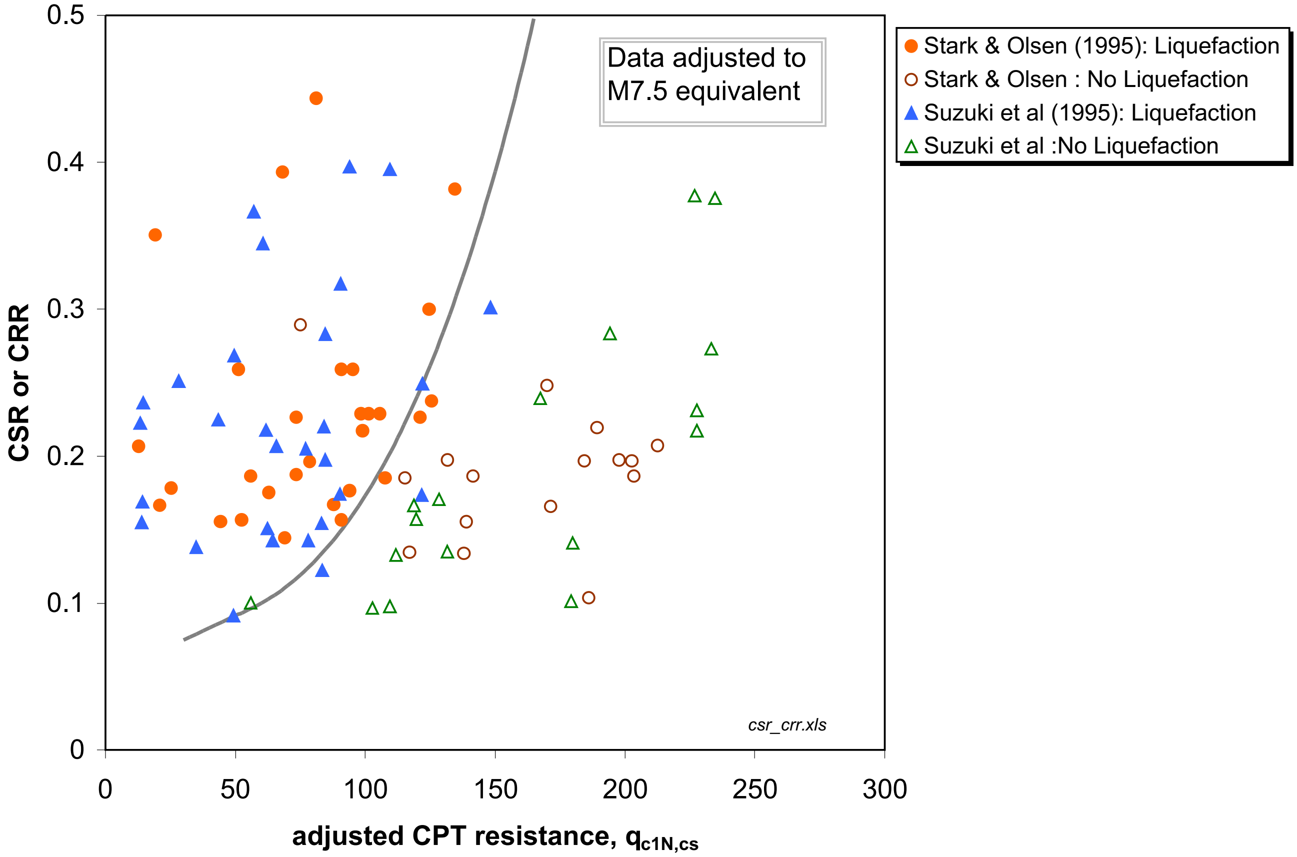
% CONTRACTIVE ZONE AT EACH CPT LOCATION

Golder Associates

PROJECT No.	03-1413-080		FILE No.	P1413080-01
DESIGN	JCC	29 OCT 03	SCALE AS SHOWN	REV. 0
CADD	SRR	29 OCT 03		
CHECK	JCC	29 OCT 03	FIGURE 5.18	
REVIEW				

CADD FILE: N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\cad\P1413080-04.dwg

REVISION DATE: 04/06/10 11:07am By: rroque



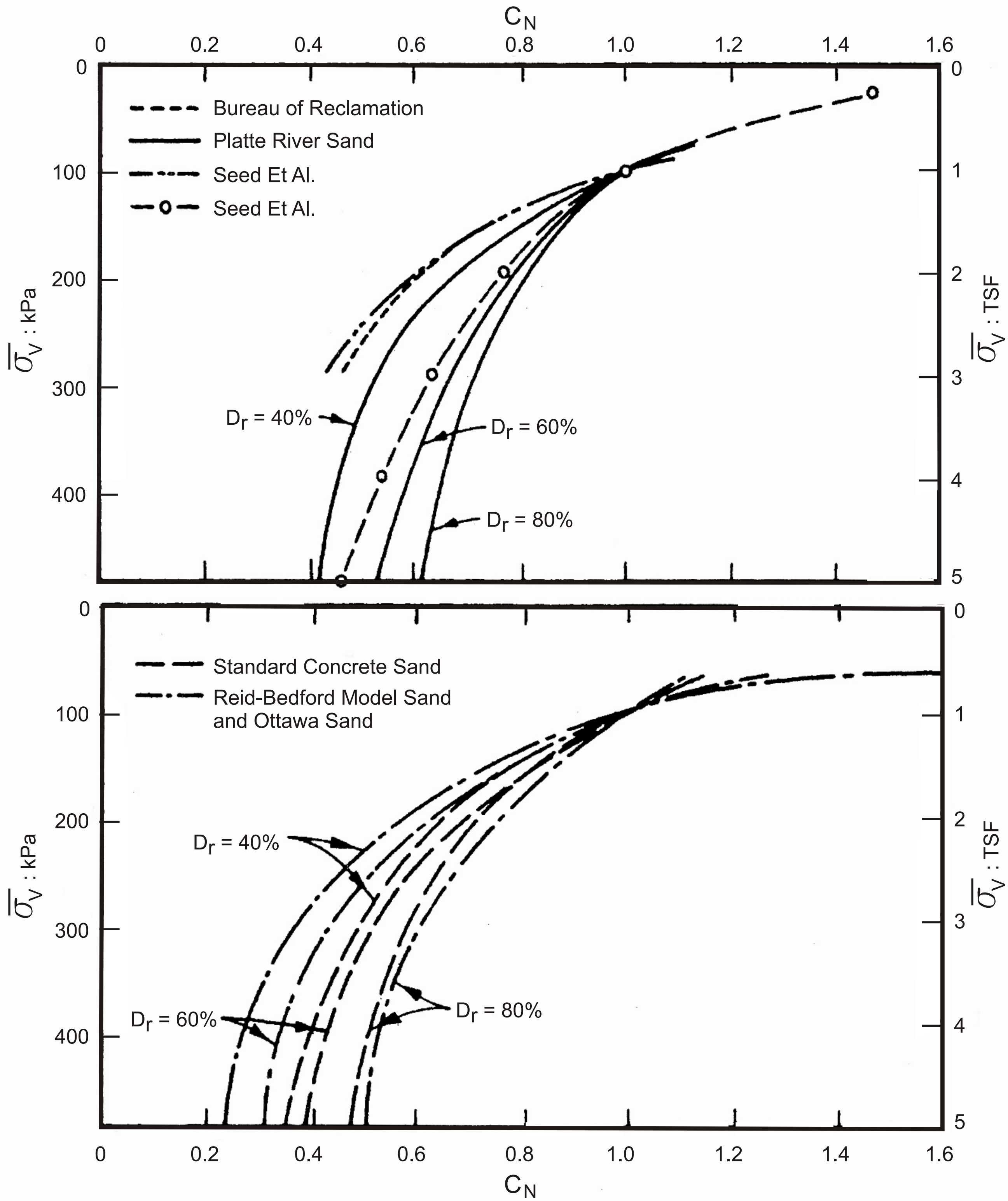
After Robertson & Wride (1998)

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	CYCLIC STRESS RATIO (CSR) IN LIQUEFACTION CASE HISTORIES AND PROPOSED CYCLIC STRENGTH RATION (CRR)

Golder Associates

PROJECT No.	03-1413-080		FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	**FIGURE 6.1**	
CHECK	JC	06FEB03		
REVIEW				

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

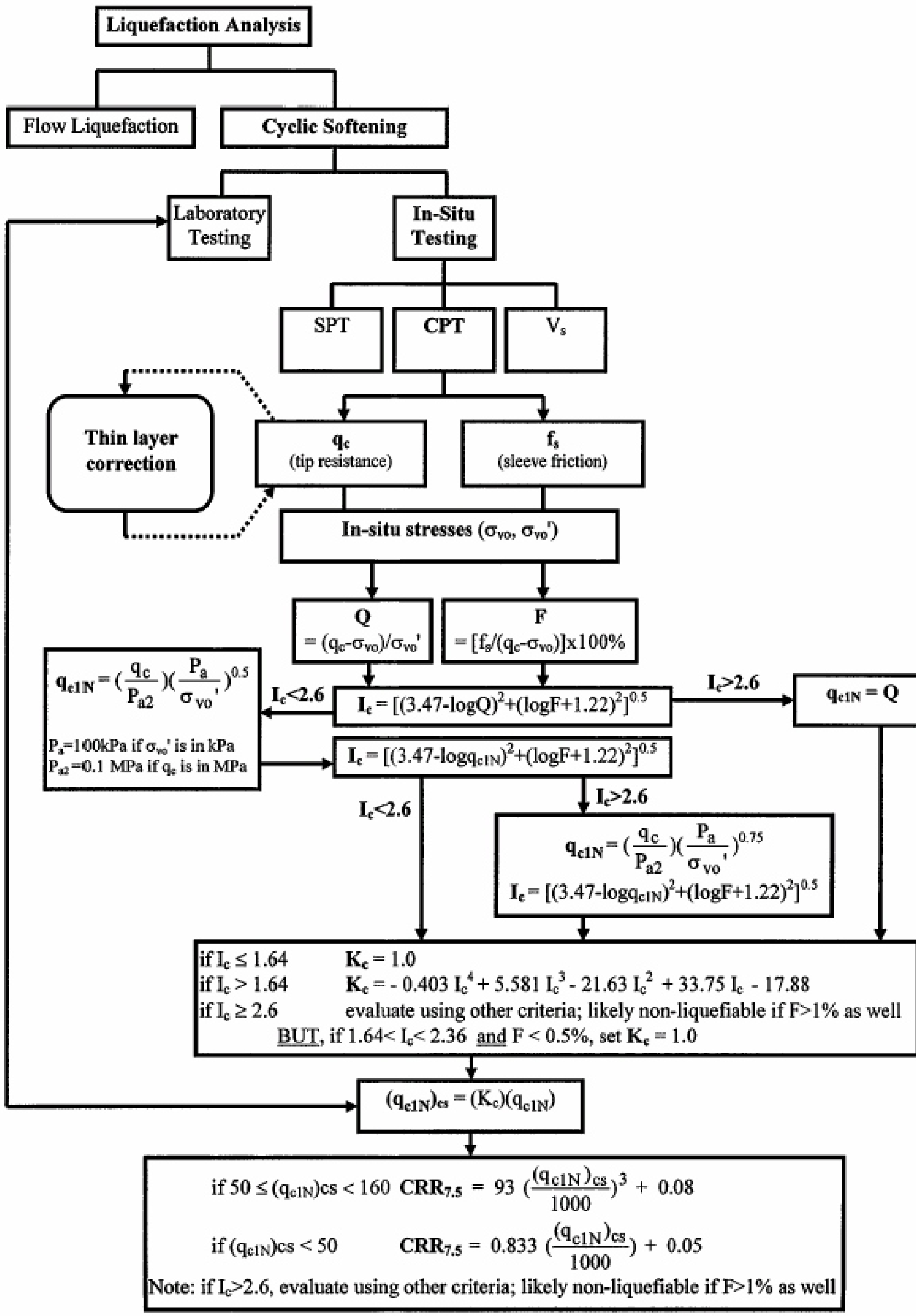


PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	C_N FROM MARCUSON AND BIEGANOVSKY (1977)

Golder Associates

PROJECT No.		03-1413-080	FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	FIGURE 6.2	
CHECK	JC	06FEB03		
REVIEW				

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



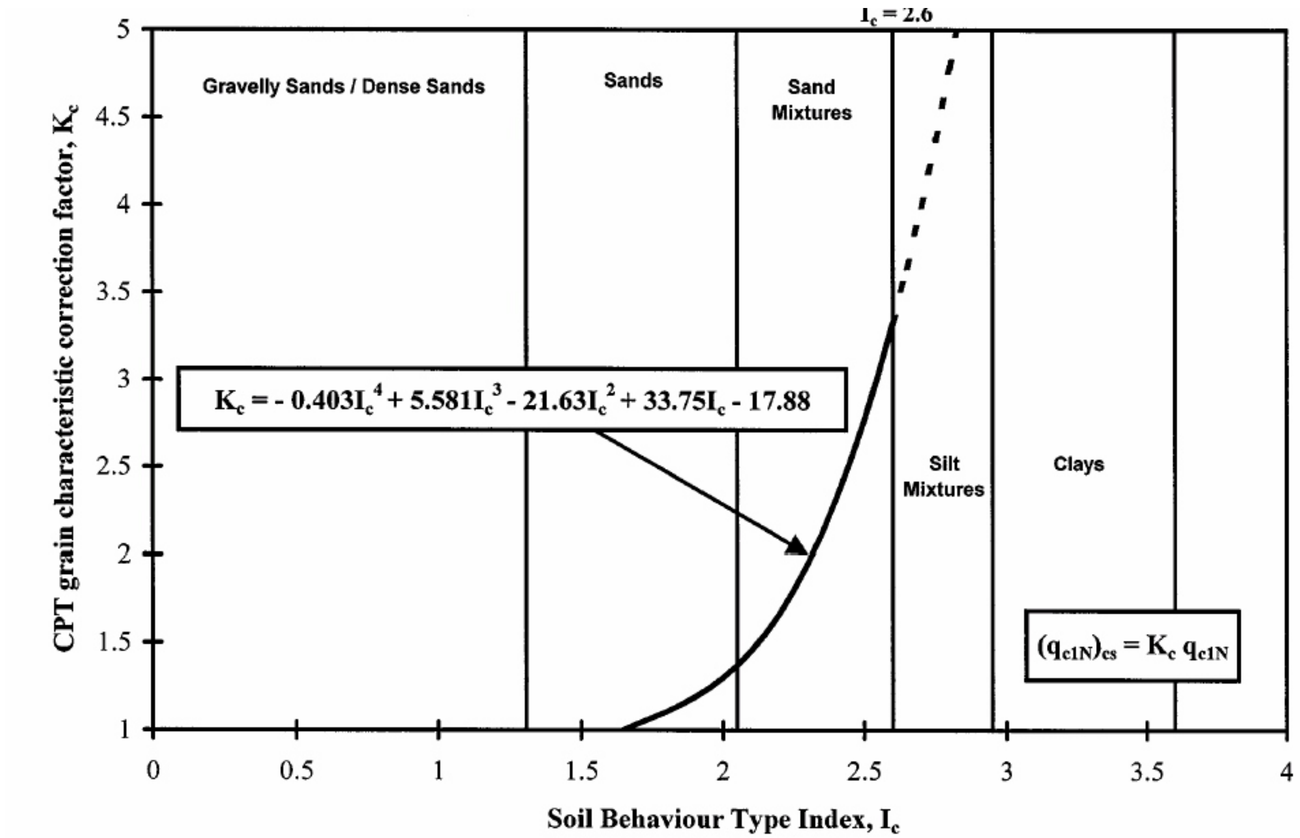
After Robertson & Wride (1998)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
CYCLIC RESISTANCE RATIO CALCULATION

Golder Associates

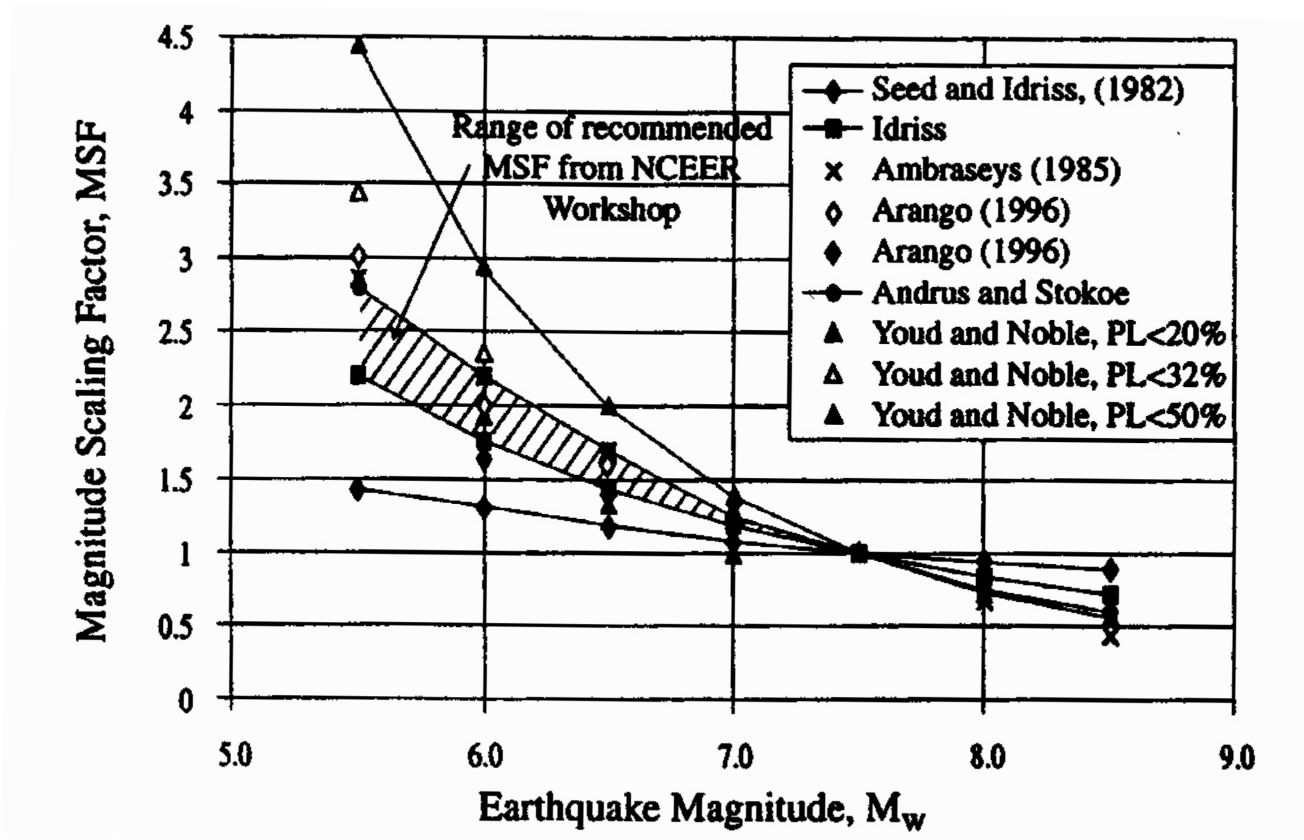
PROJECT No.		03-1413-080	FILE No. FIGURES	
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	FIGURE 6.3	
CHECK	JC	06FEB03		
REVIEW				



After Robertson & Wride (1998)

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON				
TITLE	"CLEAN SAND EQUIVALENT" CORRECTION FACTOR Kc				
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES	
	DESIGN	JC	06FEB03	SCALE	REV.
	CADD	SS	06FEB03	FIGURE 6.4	
	CHECK	JC	06FEB03		
	REVIEW				

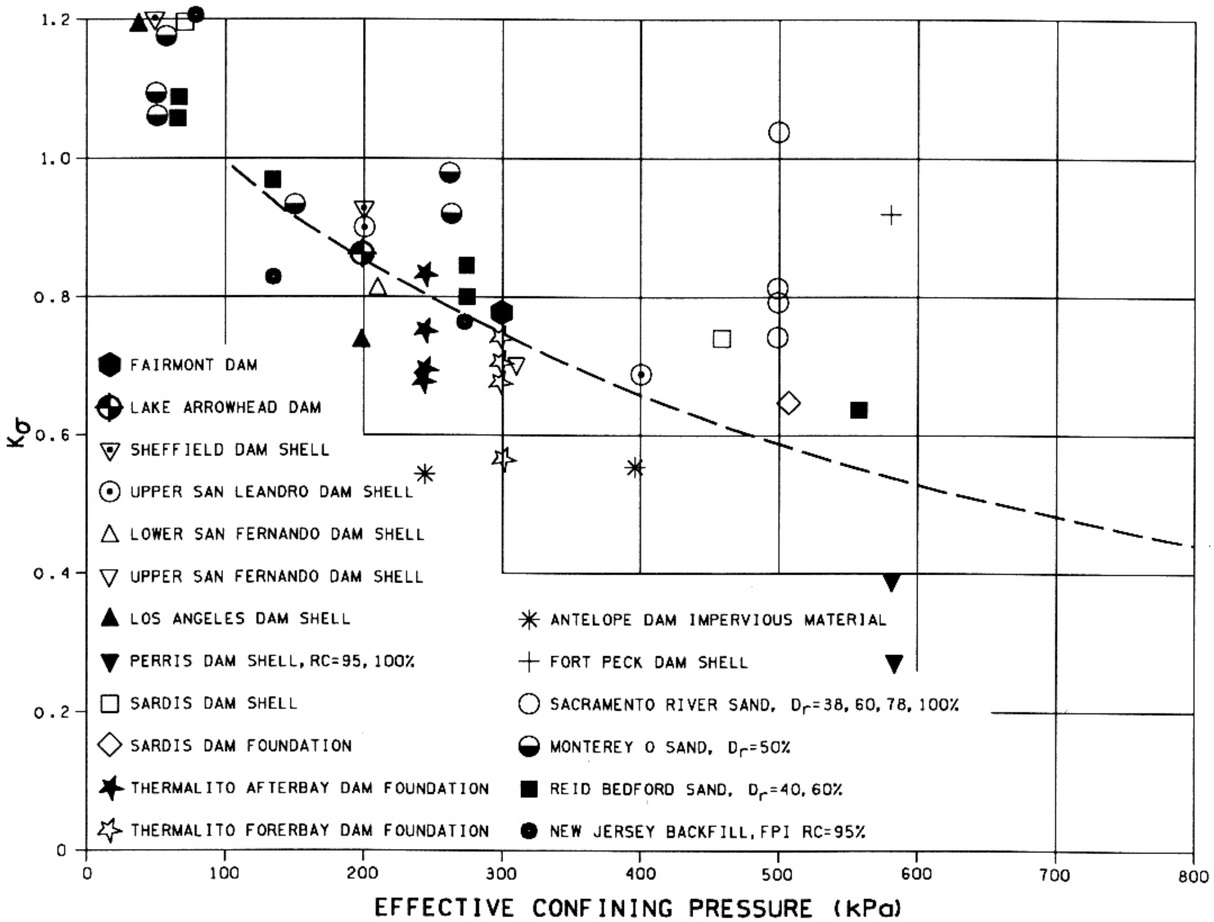
REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



After Youd and Noble (1997)

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	EARTHQUAKE MAGNITUDE SCALING FACTOR K_M			
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES
	DESIGN	JC	06FEB03	SCALE / REV.
	CADD	SS	06FEB03	FIGURE 6.5
	CHECK	JC	06FEB03	
	REVIEW			

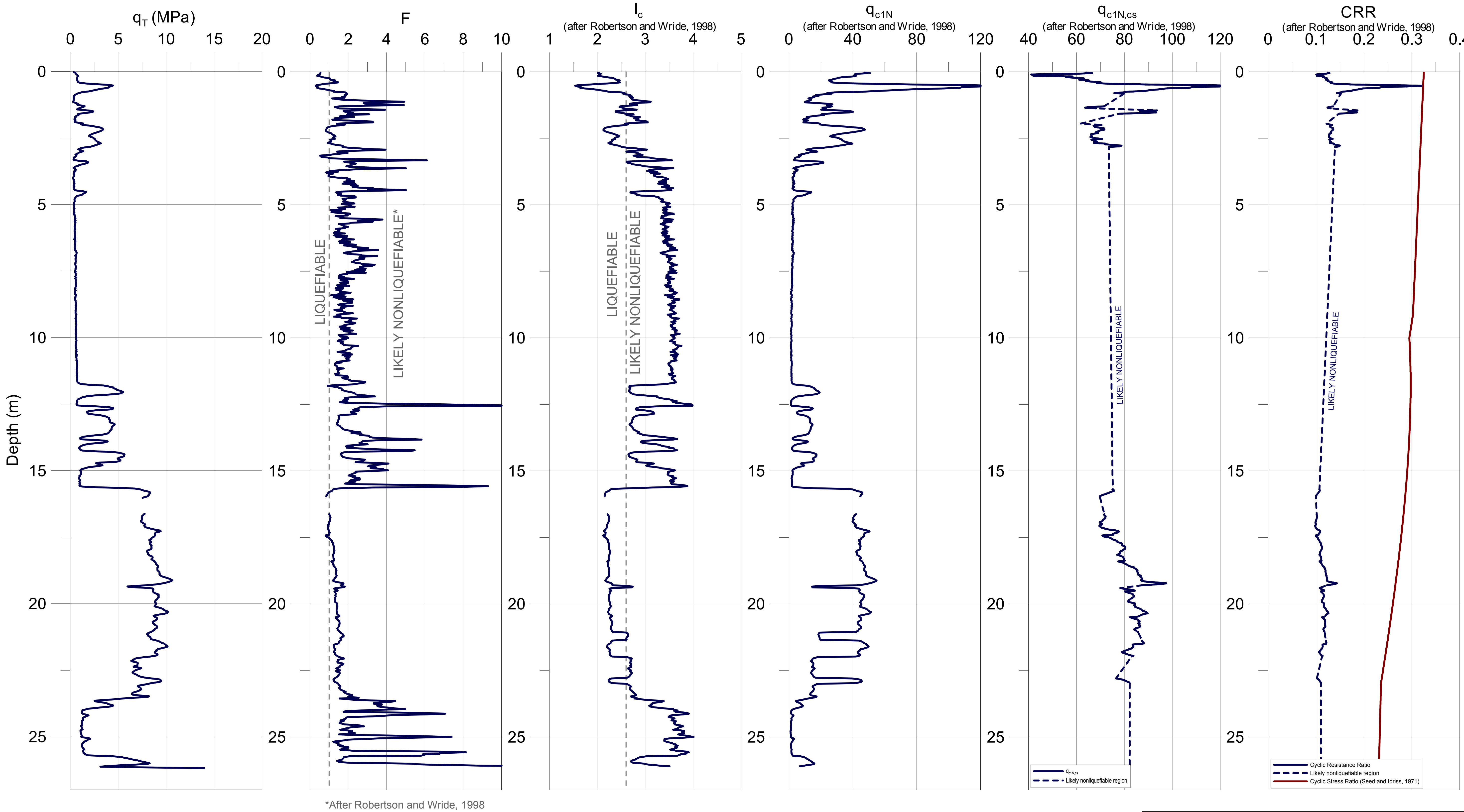
REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



After Harder & Seed 1990,
as updated by Byrne et al 1994

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON				
TITLE	SUMMARY OF RESULTS FOR K_σ				
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES	
	DESIGN	JC	06FEB03	SCALE	REV.
	CADD	SS	06FEB03	FIGURE 6.6	
	CHECK	JC	06FEB03		
	REVIEW				

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr



*After Robertson and Wride, 1998

Notes

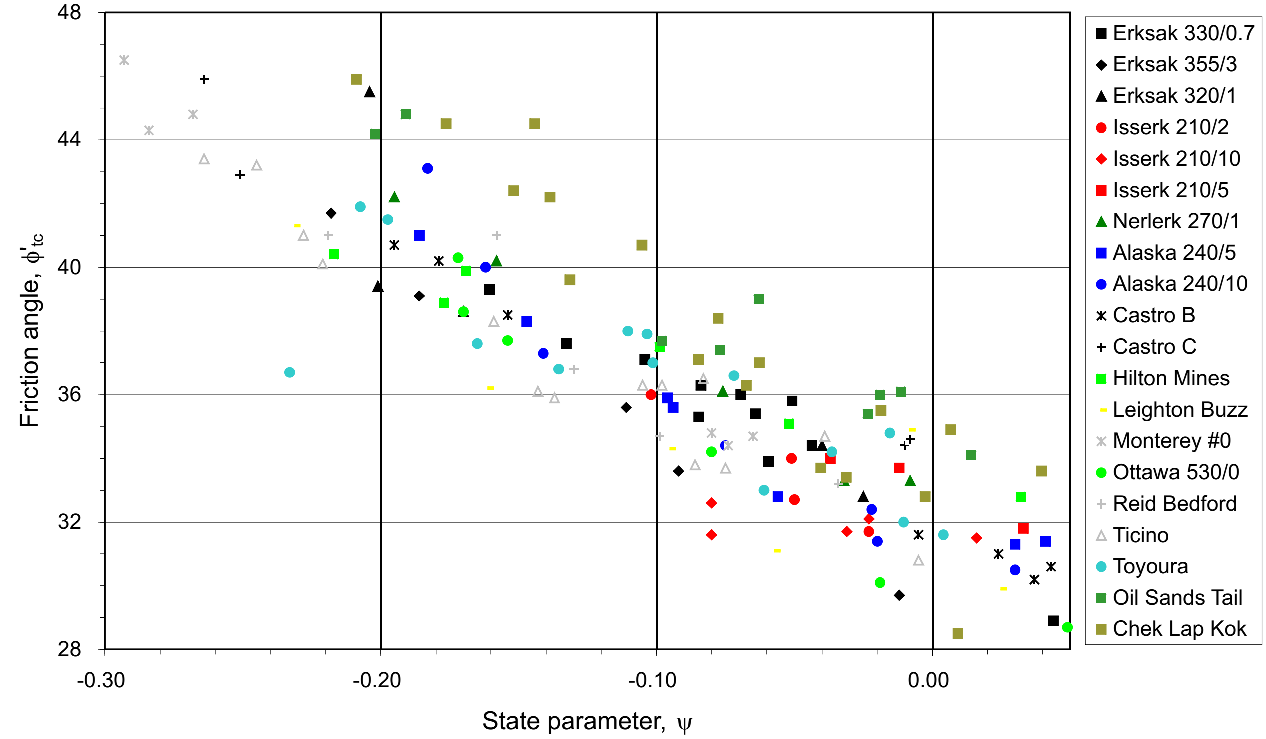
1. I_c, q_{c1n}, $q_{c1n,cs}$, and *CRR* values according to Robertson, P.K., and Wride, C.E. (Fear). 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, **35**: 442-459
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6

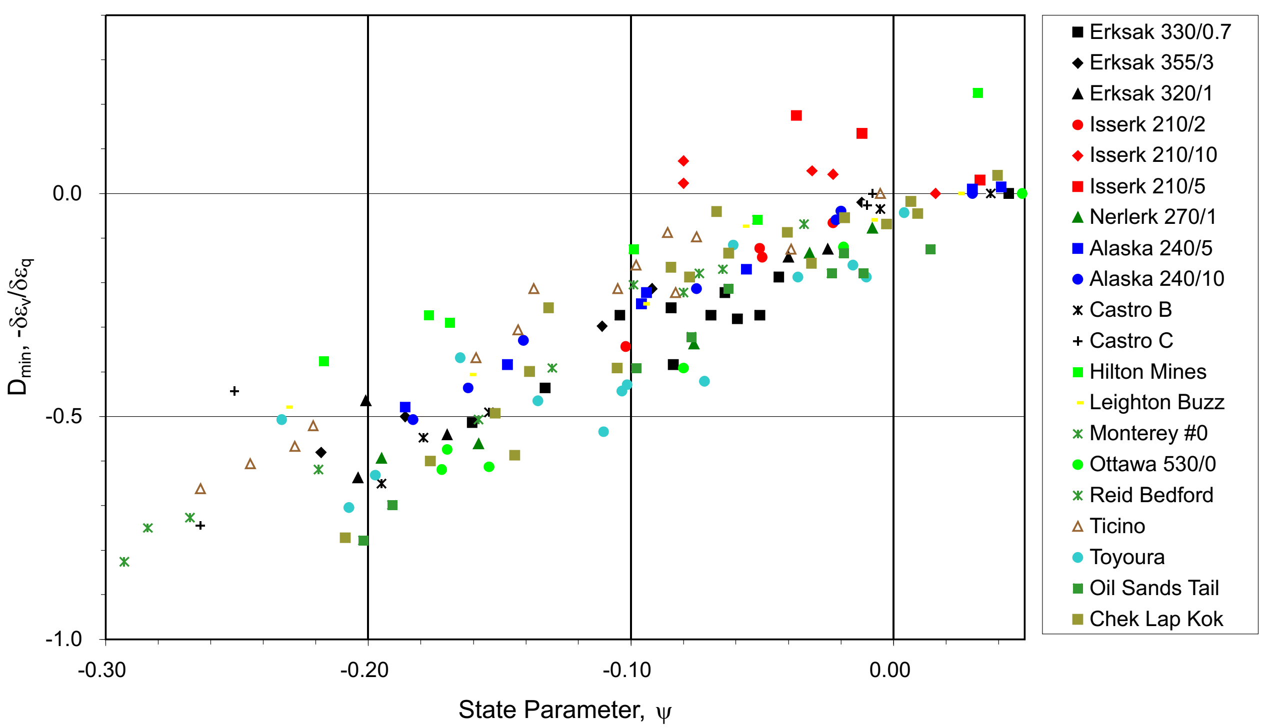
REVISION DATE: 4 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\reporting\figures\plots-SCPT03-21-CRR.grf

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	SCPT03-21: SEISMIC CALCULATIONS

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-SCPT03-21-CRR
DESIGN	DRK	19 JAN 04	SCALE NTS REV.
CADD	DRK	19 JAN 04	FIGURE 6.7
CHECK	--	--	
REVIEW	--	--	





REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

EXAMPLES OF SOIL BEHAVIOUR
(PEAK STRENGTH AND MAXIMUM DILATANCY)
AS FUNCTION OF STATE PARAMETER

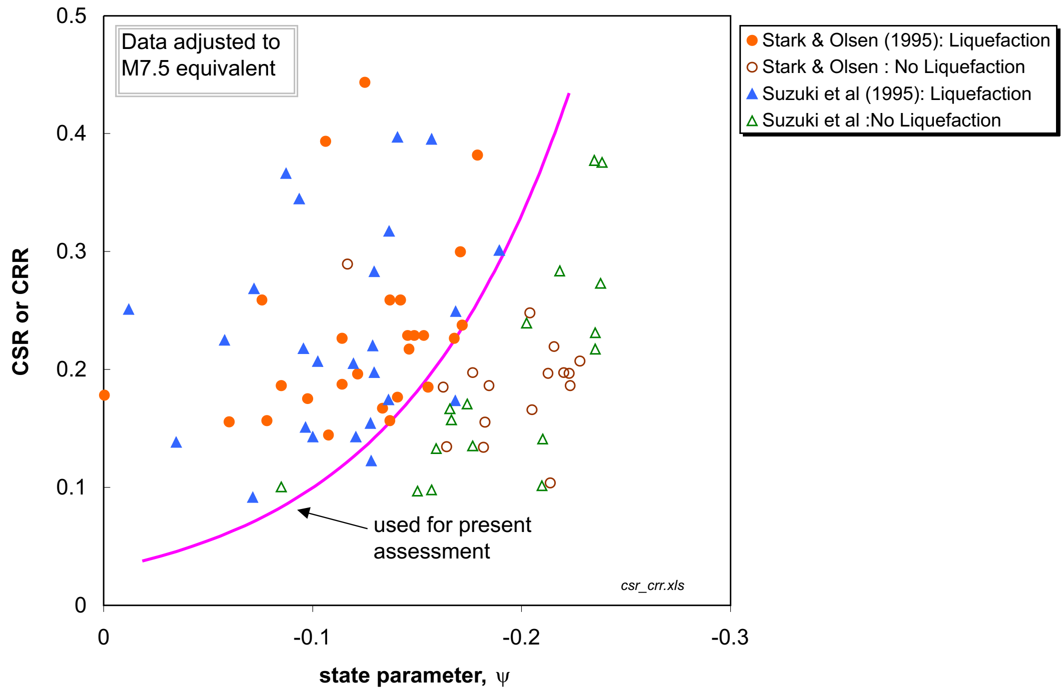
Golder Associates

PROJECT No.		03-1413-080
DESIGN	JC	06FEB03
CADD	SS	06FEB03
CHECK	JC	06FEB03
REVIEW		

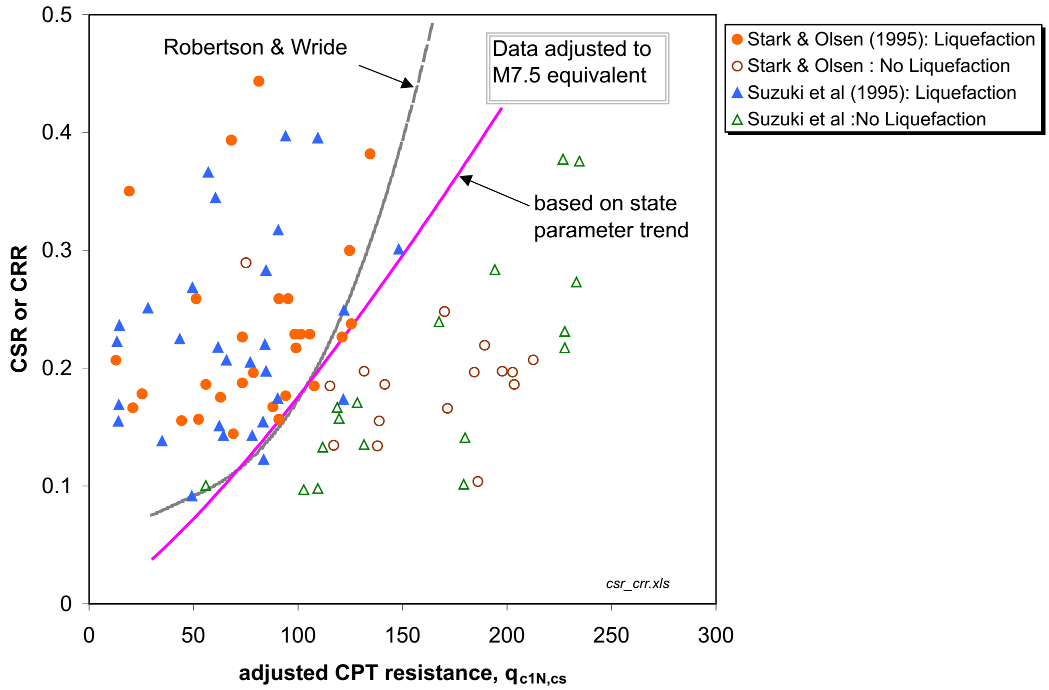
FILE No. FIGURES
SCALE REV.

FIGURE 6.8

(a)



(b)



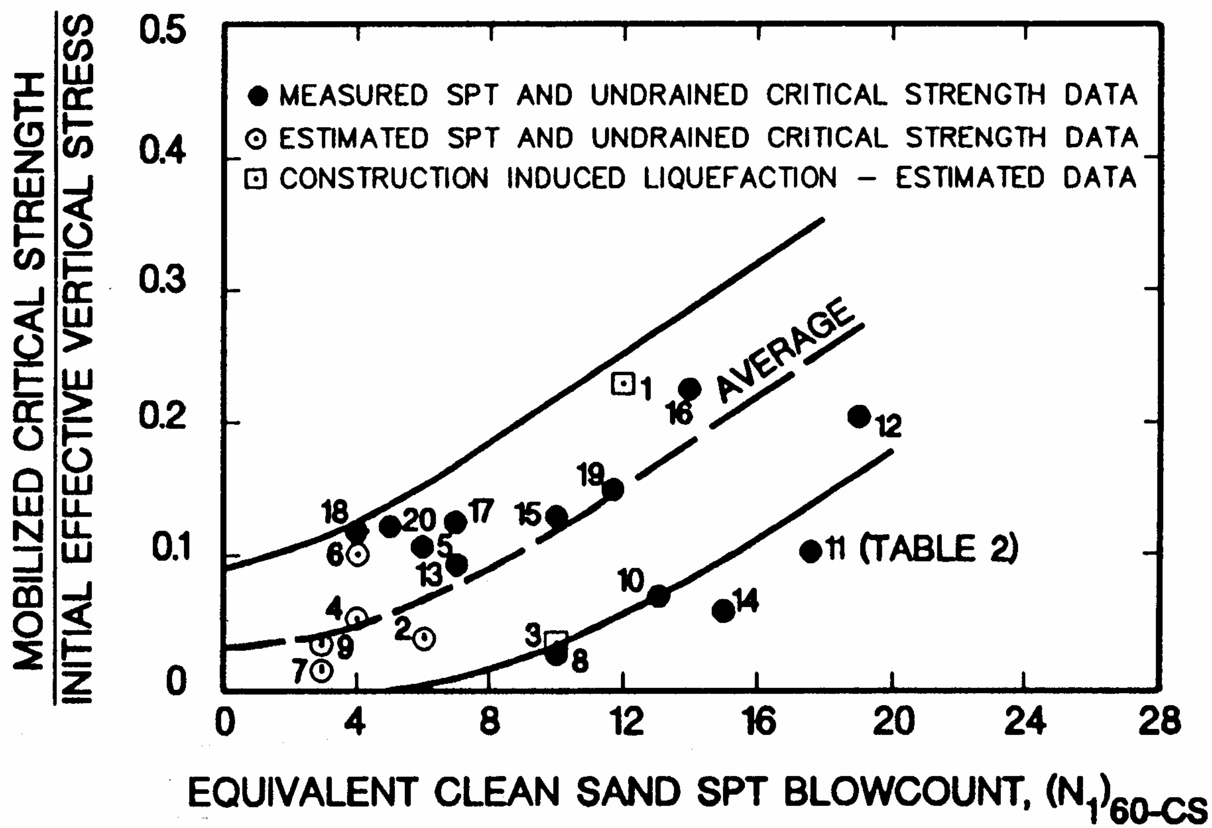
PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	CRR FROM STATE PARAMETER TREND

PROJECT No.	03-1413-080		FILE No.	FIGURES
DESIGN	JC	06FEB03	SCALE	REV.
CADD	SS	06FEB03	FIGURE 6.9	
CHECK	JC	06FEB03		
REVIEW				

Golder Associates

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr



PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

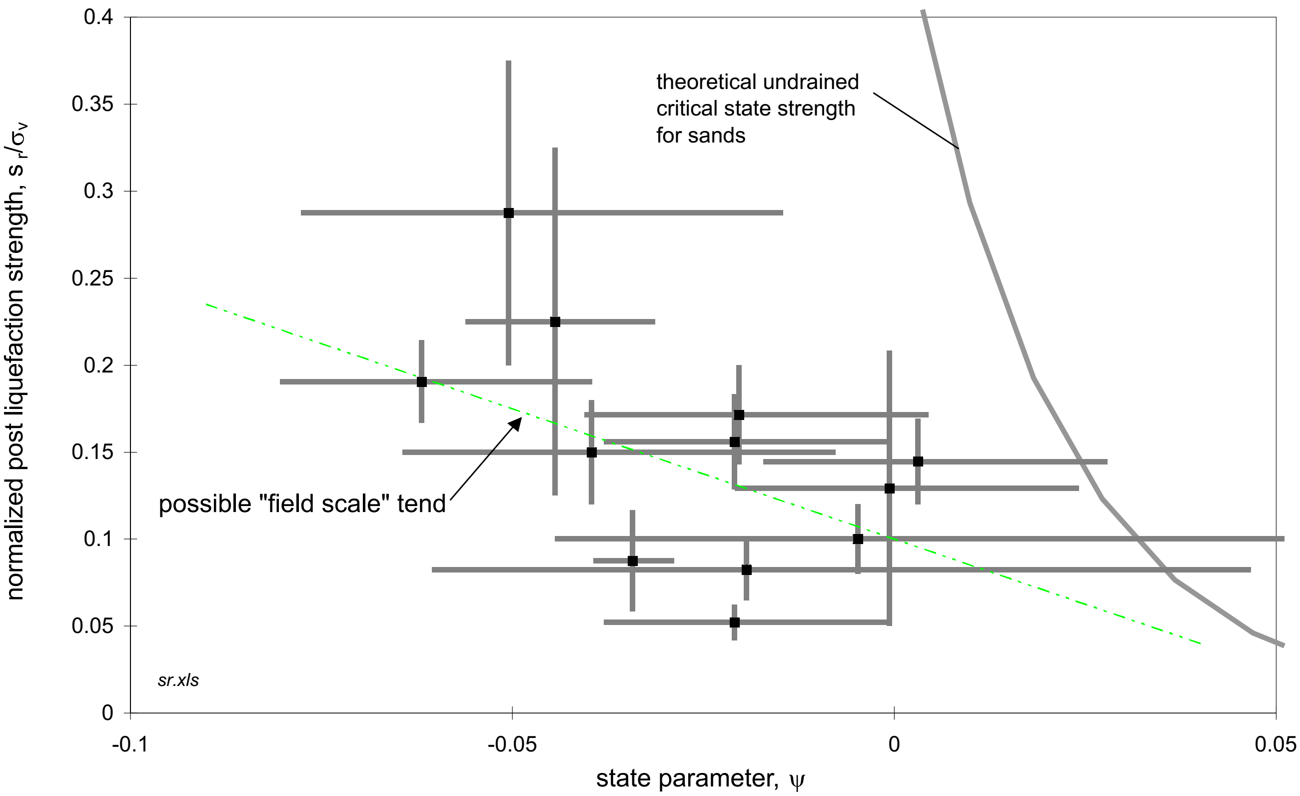
TITLE
MOBILIZED CRITICAL STRENGTH FROM SPT

Golder Associates

PROJECT No.	03-1413-078	
DESIGN	CJ	06FEB04
CADD	SS	06FEB04
CHECK	CJ	06FEB04
REVIEW		

FILE No. FIGURES 2
SCALE
REV.

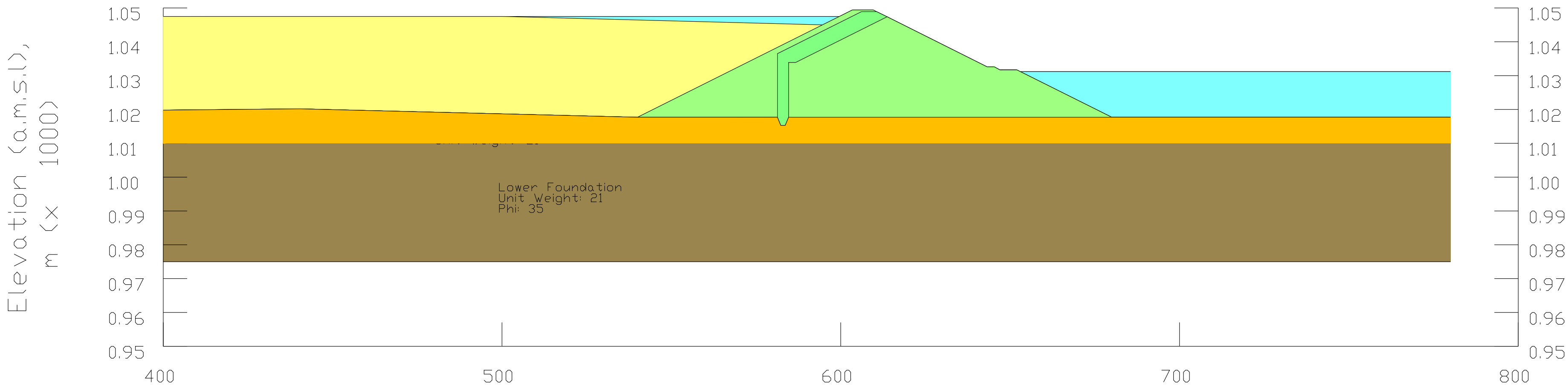
FIGURE 6.10



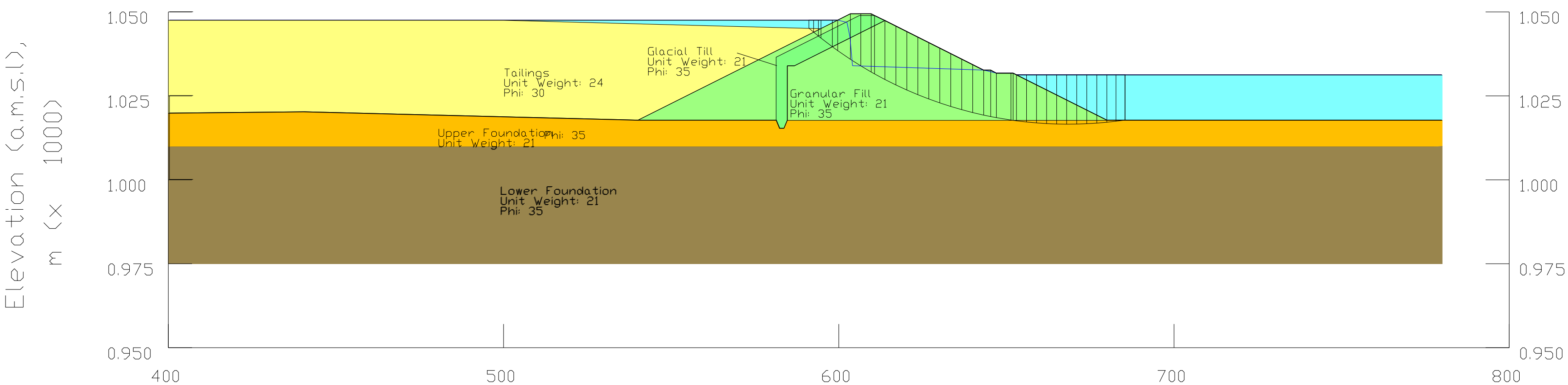
REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures.cdr

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	**RESIDUAL (POST LIQUEFACTION) STRENGTH FROM FIELD CASE HISTORIES (BARS SHOW UNCERTAINTY IN CHARACTERISTIC STATE AND MOBILIZED STRENGTH RATIO)**			
Golder Associates	PROJECT No.	03-1413-080		FILE No. FIGURES
	DESIGN	JC	06FEB03	SCALE / REV.
	CADD	SS	06FEB03	**FIGURE 6.11**
	CHECK	JC	06FEB03	
	REVIEW			

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr



a) Dam Cross Section



b) Typical Slip Surface Considered
FOS = 1 ah = 0.18g

PROJECT

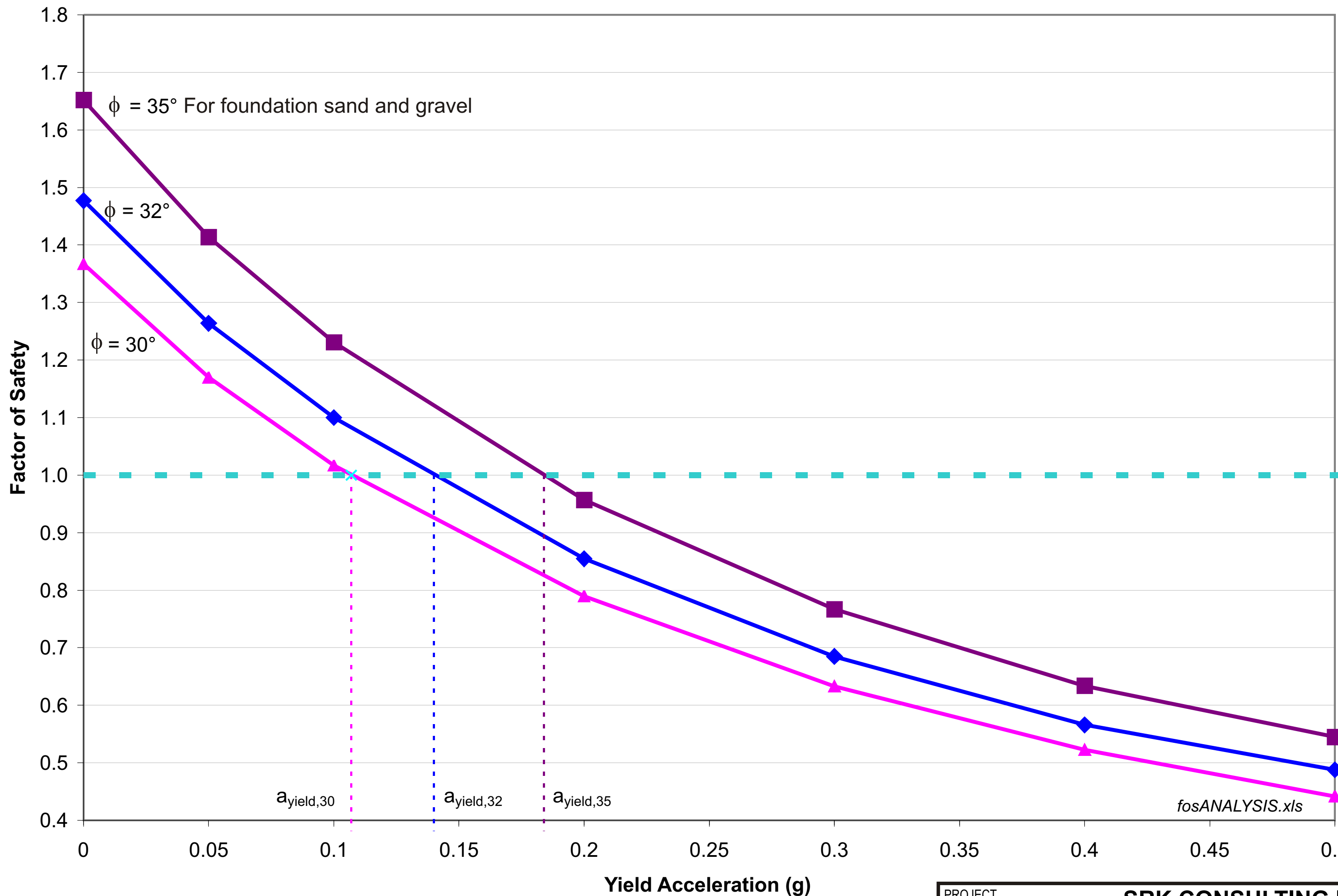
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

IDEALIZED DAM CROSS SECTION
CONSIDERED IN ANALYSES

Golder Associates

PROJECT No.	03-1413-080		FILE No. FIGURES 2	
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04	**FIGURE 8.1**	
CHECK	CJ	06FEB04		
REVIEW				



PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

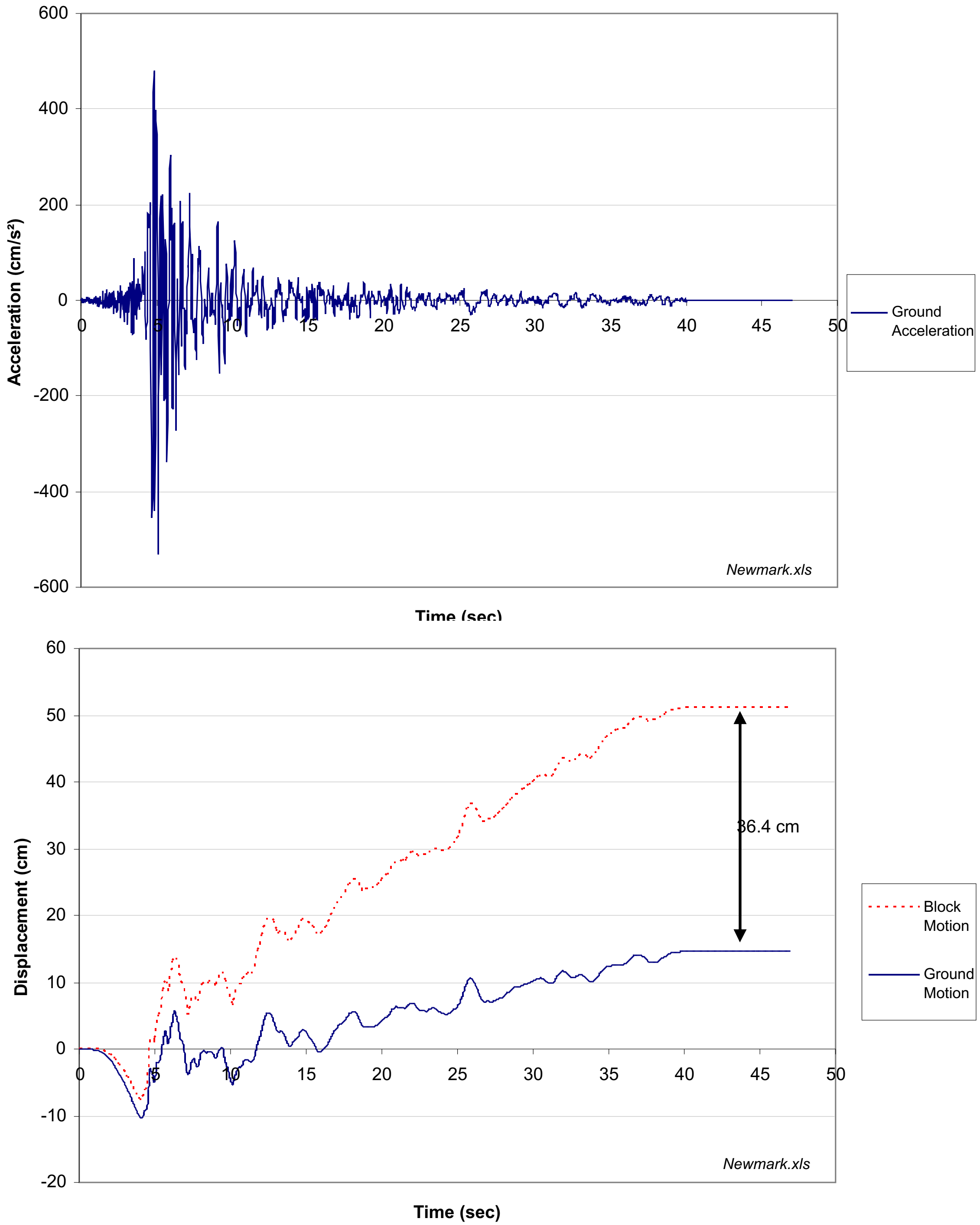
TITLE

DETERMINATION OF YIELD ACCELERATION FROM SLOPE STABILITY ANALYSIS

Golder Associates

PROJECT No.	03-1413-078		FILE No. FIGURES 2	
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04	**FIGURE 8.2**	
CHECK	CJ	06FEB04		
REVIEW				

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr

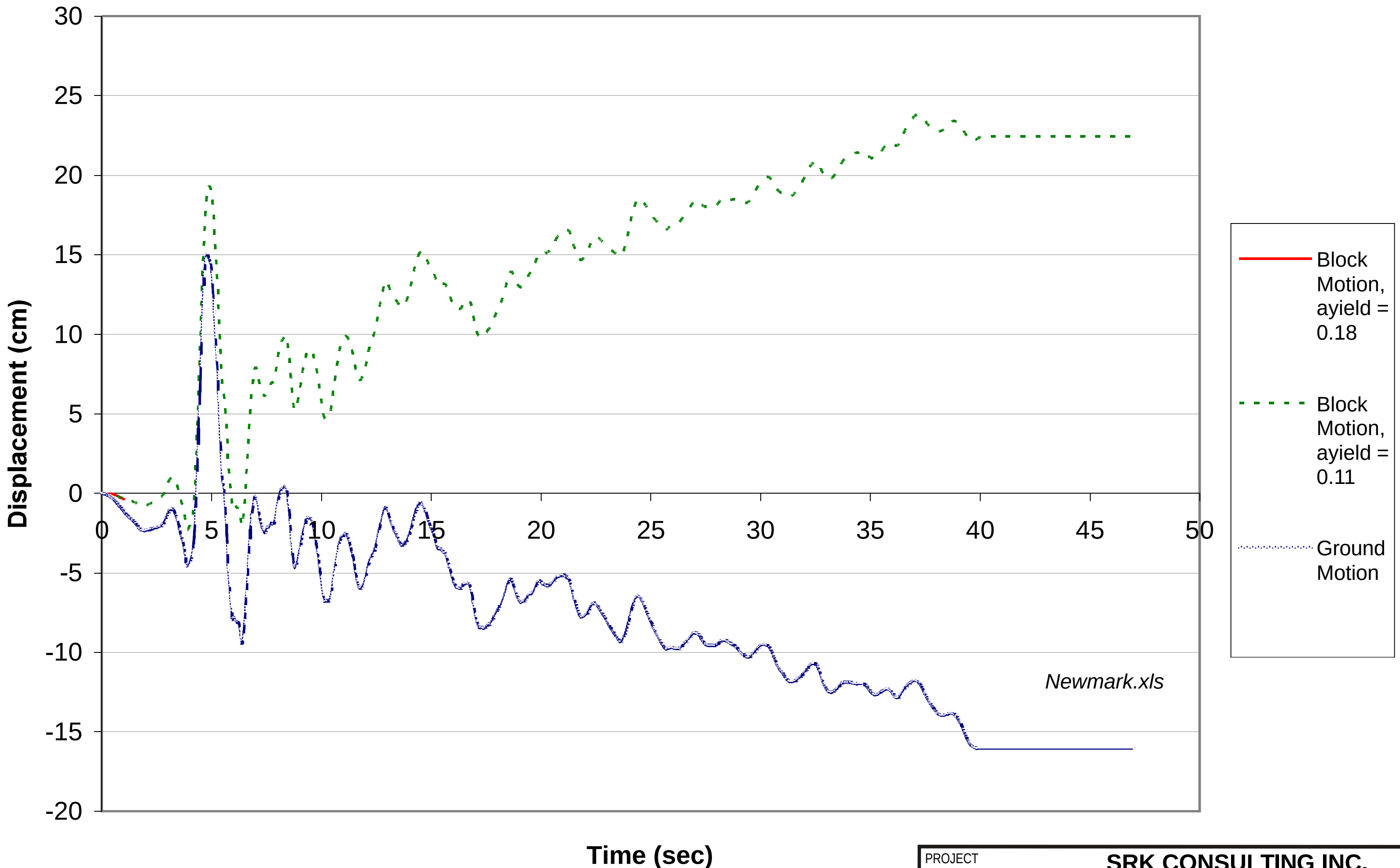


PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
EXAMPLE OF COMPUTED DAM DISPLACEMENT RESPONSE IN LOMA PRIETA EARTHQUAKE (GILROY SEWAGE PLANT, 0°) FOR ayield 8= 0.18g

Golder Associates

PROJECT No.	03-1413-078		FILE No. FIGURES 2	
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04	**FIGURE 8.3**	
CHECK	CJ	06FEB04		
REVIEW				

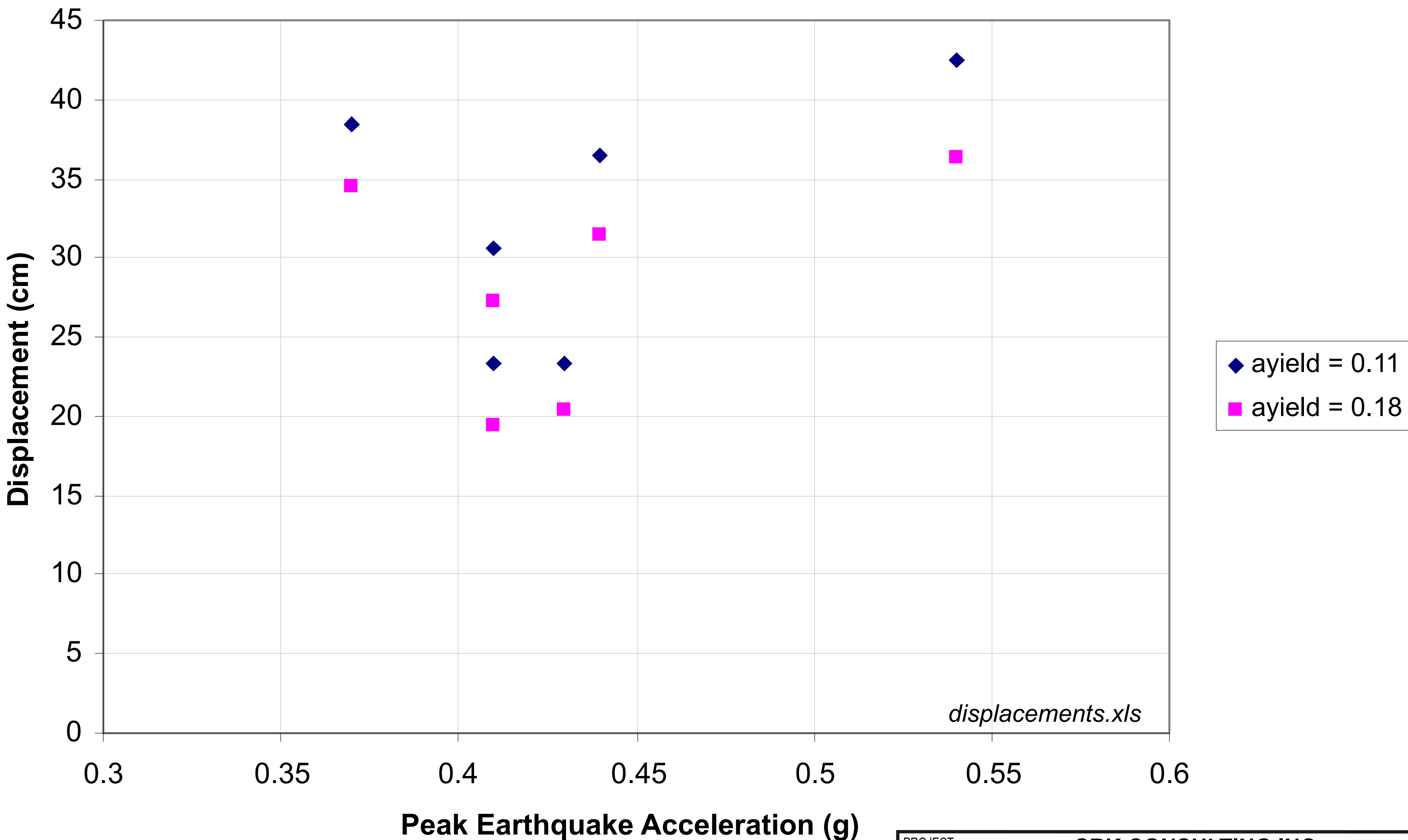


PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	EFFECT OF VARYING YIELD ACCELERATION ON DAM DISPLACEMENT

PROJECT No.	03-1413-078		FILE No. FIGURES 2
DESIGN	CJ	06FEB04	SCALE / REV.
CADD	SS	06FEB04	FIGURE 8.4
CHECK	CJ	06FEB04	
REVIEW			

Golder Associates

REVISION DATE:06-FEB-04 BY:SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 2.cdr



PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

COMPUTED MAXIMUM DISPLACEMENT OF BLOCK FOR VARIOUS EARTHQUAKES

Golder Associates

PROJECT No.	03-1413-078		FILE No. FIGURES 2	
DESIGN	CJ	06FEB04	SCALE	REV.
CADD	SS	06FEB04	**FIGURE 8.5**	
CHECK	CJ	06FEB04		
REVIEW				

APPENDIX I

DATA REVIEW DOCUMENTS SUMMARY

Appendix I - Table 1 - Summary of Golder Reports Related to the Rose Creek Tailings Impoundment

Report Title	Project #	Date	Summary
Tailings Containment Facility Anvil Mine	V 72337	October, 1973	Geotechnical Investigation for the stability assessment of the Original tailings dyke. Static Stability acceptable - comment on seismic stability- identified potential tailings liquefaction and displacement, with recommendation to construct engineered tailings embankment downstream as part of mine expansion plans. Presents concept to develop additional tailings disposal area downstream of the present facility (to be the secondary dam and impoundment). Presents 2 options for rose creek diversion (channel and culvert), 10 borehole logs at original dyke, 2 for secondary dyke. Good summary of material laboratory testing (tailings and fills). Comments on under seepage in this unlined facility over permeable granular. One recommendation is to place slimes over native ground ahead of tailings deposition to blind off foundation. Includes 1972 air photo of facility.
Preliminary Report on Proposed New Tailing Facilities Anvil Mine	V 74011	June, 1974	Recommendation concerning creation of new tailings storage area downstream of Original area (to be the secondary dam and impoundment). Diversion Rose Creek in a channel is the preferred options. Selected scheme involves construction of both upstream and downstream dams of zoned fills.
An Internal Factual Report of Soils Conditions Relating to Tailings Containment Expansion Cyprus Anvil Faro, Yukon	V74011 / C75704	May, 1975	Expansion plan was undertaken to increase mine production, and Golder was retained to provide detailed investigation involving boreholes and test pits. Includes figure 3 large sheet with site photo (could be 1972 photo, not ref). Shows secondary dam boreholes.
Summary of 1975 Construction Tailing Dam Structure	V 74011	June, 1976	Brief account of earthwork construction during summer of 1975 including quantities of material excavated and of fill material to be placed for the West and East Secondary Dams (starter dam to 3547 feet elev. crest) E size sheet drawing shows as constructed section
A Preliminary Statement of Engineering Considerations For the Proposed Cyprus Anvil Tailings Storage Expansion Project Faro, Yukon	792-2025	October, 1979	First report Describing the nature, scope, and geotechnical settings and preliminary design and identify earthwork quantities. For a proposed cross valley dam and intermediate dam scheme to store tailings and water downstream of the secondary dam . Includes Cyprus Anvil's submission to Government concerning approval for construction. Includes borehole data, geophysics results, Hydrocon report on hydrology and hydraulics. Air photo from 1979
Preliminary Materials Data Package for Tenderer's Information Cyprus Anvil 1980 Embankment Dam Raising, Yukon	792-2041	April, 1980	Tenderer's Info for the Stage III raise of the Secondary dam. Provides summary of the two part of the Stage II raise of the dam (first 15 feet then 5 feet raises). Shows air photo, year unknown.
Geotechnical Considerations Tailings Containment Construction 1980 Embankment Dam Raising	792-2041	May, 1980	Design of Secondary dam raising and geotechnical considerations. (stage III raise) . 4 boreholes, Recent inspection indicated acceptable performance of dam, and new design has evolved based on recommendations provided in new guidelines. 4 boreholes , 1975 air photo, typical dam sections, Grainsizes for soil classes,
An update study concerning Design and construction of tailings retaining structures at Cyprus Anvil Mine	C78702		Included as Appendix to the 792-2041 May 1980 - presents 1978 boreholes, photo from 1975, piezo sections.
Final Design Recommendations for the Down Valley Tailings Disposal Project Faro, Yukon (Vol 1)	792-2025	June, 1980	Golder was retained by Cyprus Anvil to develop an appropriate design for the cross valley dam and the intermediate dam and the extension of rose creek diversion channel. The report discusses various stages and procedures of the investigation and its findings and recommended design for construction Air photo from 1979, long sections . Cross valley to 1065 m, Intermediate dam to 1068 m crests.
Final Design Recommendations for the Down Valley Tailings Disposal Project Faro, Yukon (Vol 2)	792-2025	June, 1980	Appendix I: Rose Creek Flow Data / II: Supplementary Climatic Data for Faro and Anvil / III:Test Pit and Borehole Information / Visited Photographs / V: Test Excavation
Final Design Recommendations for the Down Valley Tailings Disposal Project Faro, Yukon (Vol 3)	792-2025	June, 1980	Appendix VI: Cross Valley Dam Design / VII: Borrow Pit Investigations / VII: Diversions Canal Geothermal, Seepage and Stability Considerations, IX: Hydrologic and Hydraulic Design Report / Appendix 1: Rose Creek - Flow Data / Figures / X: Earthquake Data
Construction Report Cyprus Anvil 1980 Raising of the Tailings Dam Faro, Yukon (Vol 1)	802-2024	July, 1981	Summary and description of the work and on-site engineering services provided for the Stage III raise of the Secondary Dam (to crest 3585 feet or so) , as-built cross sections of dam during and after construction, and discussion concerning installation of piezometer and foundation monitoring
Construction Report Cyprus Anvil 1980 Raising of the Tailings Dam Faro, Yukon (Vol 2)	802-2024	July, 1981	Appendix IV: West Dam (Station 0 to Station 27+28W)
Construction Report Cyprus Anvil 1980 Raising of the Tailings Dam Faro, Yukon (Vol 3)	802-2024	July, 1981	Appendix IV Continued
Down Valley Tailings Containment Project 1980-81 Construction Volume I		February, 1982	Summary of construction specs, drawings and as builts

Down Valley Tailings Containment Project 1980-81 Construction Volume II		February, 1982	photos
Down Valley Tailings Containment Project 1980-81 Construction Volume III		February, 1982	
1982 Performance Monitoring of the Down Valley Tailings Project (Vol 1)	822-2021	March, 1983	First year report on Geotechnical and thermal performance of the Down Valley Tailings Project. Construction of tailings storage and creek diversion systems are performing well.
1982 Performance Monitoring of the Down Valley Tailings Project (Vol 2)	822-2021	March, 1983	Intermediate Dam, Cross Valley Dam, Canal Thermistors, Canal Piezometers, etc.
1988 Performance Monitoring and Additional Work on the Down Valley Tailings Project Far, Yukon (Vol 1)	882-2412 (872-2408/882-2410/882-2409)	February, 1989	Annual Geotechnical inspection of the Fresh Water Supply Dam and stability evaluation is presented. Fresh Water Supply Dam was added to the terms of reference, and concerns regarding cracking was addressed.
Intermediate Dam 1989 Raising and Fresh Water Supply Dam Toe Berm (Tender, Appendix)	822-2413	May, 1989	1989 Intermediate Dam Raising (by 5 m) and Fresh Water Supply Dam Toe Berm Construction
Intermediate Dam 1989 Raising and Fresh Water Supply Dam Toe Berm (Appendix A)	822-2413	May, 1989	Data for Tenderers, including test pit logs (borrow areas) fill material grain size curves.
1989 Performance Monitoring of the Down Valley Tailings Project Faro Mine (Vol 1)	892-2406 (882-2410/882-209)	March, 1990	Findings of annual inspection and data review including Fresh Water Supply Reservoir dam. It is operating safely, but continued observation is necessary.
1989 Performance Monitoring of the Down Valley Tailings Project Faro Mine (Vol 2)	892-2406 (882-2410/882-209)	March, 1990	Appendix III: Instrumentation Observation Data (Cross Valley Dam, Intermediate Dam, Diversion Canal, Fresh Water Dam, Misc. Piezometer)
Summarizing the Down Valley Tailings 1988 and 1989 Construction Projects (Vol 1)	822-2143C (822-2406C)	April, 1990	Construction comprised raising 5m of intermediate dam in 1988 and further raising in 1989. Construction of toe berm at Fresh Water Supply Dam and toe berm drainage system at toe of Diversion Canal Dyke.
Summarizing the Down Valley Tailings 1988 and 1989 Construction Projects (Vol 2)	822-2143C (822-2406C)	April, 1990	Appendix I: 1988 Construction / II: 1989 Construction / III: Instrumentation - Specs, drawings, photos, BH logs
Faro Mine Down Valley Tailings Project Cross Valley Dam Toe Berm Design Report	892-2410	June, 1990	Design recommendations concerning berming of downstream toe of Cross Valley Dam, to provide confinement to area of toe seepage without frustrating free drainage to exit channels
Construction Specifications Cross Valley Dam Toe Drain Curragh Resources Inc. Faro, Yukon	892-2410A	June, 1990	Construction Specs: works involve clearing surface material, repair of haul roads, widening of ditch, etc.
Faro Mine Annual Inspection 1994	942-2430	September, 1994	Pictures and illustrations obtained from Geotechnical inspections
1997 Annual Geotechnical Performance Evaluation and Instrumentation Data Report Faro (Vol 1)	972-2446-5300	June, 1998	Proposed scope of work includes field geotechnical inspection and preparation of maintenance recommendations and synthesis and interpretation of field instrumentation data.

APPENDIX I - TABLE 2

**LISTING OF FOUNDATION BOREHOLES
ROSE CREEK TAILINGS FACILITY
FARO MINE**

Structure		Borehole	Boring Date	Report
Original Impoundment Dam		BH A-1	July, 1973	V72337 (October, 1973)
		BH A-2	July, 1973	
		BH A-3	July, 1973	
		BH A-4	July, 1973	
		BH B-1	July, 1973	
		BH B-2	July, 1973	
		BH B-3	July, 1973	
		BH B-4	July, 1973	
		BH C-1	July, 1973	
		BH C-2	August, 1973	
Second Impoundment Dam	West Dam	BH D-1	August, 1973	V72337 (October, 1973)
		BH D-2	August, 1973	
		BH 3	June/July, 1974	V74011/C75704 (May, 1975)
		BH 4	June/July, 1974	
		BH 5	June/July, 1974	
		BH 11	June/July, 1974	
		BH 12	June/July, 1974	
		BH 13	June/July, 1974	
		BH 78-1	March, 1978	C78702 (April, 1978)
		BH 78-2	March, 1978	
		BH 78-3	March, 1978	
		BH 78-4	March, 1978	
		BH 78-5	March, 1978	
		BH 78-6	March, 1978	
		BH P81-01	April, 1981	802-2024 (July, 1981)
		BH P81-02	April, 1981	
		BH P81-03	April, 1981	
		BH P81-04	April, 1981	
		BH P81-05	April, 1981	
	East Dam	BH P81-06	April, 1981	
		BH P81-07	April, 1981	
		BH P81-08	April, 1981	
Intermediate Dam		BH 79-27	November, 1979	792-2025 (June, 1980)
		BH 79-28	December, 1979	
		BH 79-29	December, 1979	
		BH 79-30	December, 1979	
		BH 79-31	December, 1979	
		BH 79-33	December, 1979	
		BH 80-35	February, 1980	
		BH 80-36	February, 1980	
		BH 80-37	February, 1980	
		BH 80-46	February, 1980	
Cross Valley Dam		BH 79-1	July, 1979	792-2025 (June, 1980)
		BH 79-5	July, 1979	
		BH 79-6	July, 1979	
		BH 79-7	July, 1979	
		BH 79-8	August, 1979	
		BH 79-15	August, 1979	
		BH 79-16	August, 1979	
		BH 79-17	August, 1979	
		BH 79-18	August, 1979	
		BH 79-19	August, 1979	
		BH 79-20	August, 1979	
		BH 79-21	August, 1979	
		BH 79-22	August, 1979	
		BH 79-22A	August, 1979	
		BH 79-26	August, 1979	
		BH 79-34	December, 1979	
		BH 80-38	February, 1980	
		BH 80-38A	February, 1980	
		BH 80-39	February, 1980	
		BH 80-40	February, 1980	
		BH 80-41	February, 1980	
		BH 81-98	August, 1981	802-2038 (February, 1982)
		BH 81-99	August, 1981	
		BH 81-100	August, 1981	
		BH 81-101	August, 1981	
		BH 81-102	August, 1981	
		BH 81-103	August, 1981	
		BH 81-104	August, 1981	
		BH 81-105	August, 1981	
		BH 81-106	August, 1981	
		BH 81-107	August, 1981	
		BH 81-108	August, 1981	
		BH 81-109	August, 1981	
		BH 81-110	August, 1981	
		BH 81-111	August, 1981	
		BH 81-112	August, 1981	
		BH 81-113	August, 1981	
		BH 81-114	August, 1981	
		BH 81-115	August, 1981	
		BH 81-116	August, 1981	
		BH 81-117	August, 1981	
		BH 81-118	August, 1981	
		BH 81-119	August, 1981	
		BH 81-120	August, 1981	
		BH 81-121	August, 1981	
		BH 81-122	August, 1981	
		BH 81-123	August, 1981	
		BH 81-124	August, 1981	

APPENDIX II

LABORATORY TEST DATA

1.0 LABORATORY MEASUREMENT OF CRITICAL STATE PROPERTIES

The following describes the geotechnical laboratory testing program carried out on the Rose Creek tailings samples for the measurement of the critical state properties.

Two bulk samples of the rose creek tailings with different gradations were prepared in the laboratory. Index tests were carried out to determine the grain size distribution, specific gravity of solids (ASTM standard D854), and maximum and minimum densities for each sample (ASTM standards D4253 and D4254, respectively).

Reconstituted samples prepared with moist tamping method were made for triaxial testing. Following is a description of the triaxial testing program.

1.1 Triaxial Test Equipment

The triaxial testing was carried out using a Wykeham Farrance loading frame, a Karol-Warner manufactured triaxial cell and a Brainard Kilman pressure regulator and volume change panel at the Golder Burnaby laboratory.

Two Tyco AB-200 pore pressure transducers with a range of up to 1380 kPa were used, one for cell pressure and one for pore pressure measurements. Accuracy of the pore pressure transducers were 1% of the best-fit linear line. Displacement during shear loading was measured with a M+M 50 mm linear displacement sensor with an accuracy of 0.03%. Axial load was measured at the top of the loading frame with either a Tyco JP-2000 or a Sensotec 2000 lb load cell, each with a range of up to 8.9 kN and an accuracy of +/-0.15% FSO.

All electronic measurement devices were connected to the Electro-Numerics Micro-p Signal Conditioner and then into a DataTaker DT-100 Data Logger running on the laboratory PC to monitor and record displacement, pore pressure, cell pressure and load during the shearing of samples at 15 bit resolution. The sampling frequency during shear loading was initially set to record all data once per 10 seconds until about 4% strain, then reduced to once per 60 seconds to the end of the test.

Prior to the testing program, all equipment was calibrated including the load cells, pore pressure transducers, triaxial system volume change, and displacement transducers.

The triaxial cell was equipped with frictionless end platens. This consisted of an end platen oversized compared to the sample. Each oversized end was fitted with two-ring membranes that were separated with a smear of grease. The platens were of a diameter of 74 mm and the samples prepared with a split mold with diameter of 71.5 mm. The split mold height was 142.8 mm for a sample height to diameter ratio of 2:1. Latex rubber membranes with a 0.3 mm thickness were used.

All tests were carried out displacement controlled. The drained and undrained tests were all carried out at a displacement rate of 0.254 mm/min.

1.2 Procedures

Reconstituted samples were prepared using the moist tamping method. Prior to starting a test, a target consolidation pressure and void ratio were assigned. Using estimates of the saturation and consolidation volume changes, the required preparation void ratio was calculated, which determined the required preparation dry soil mass.

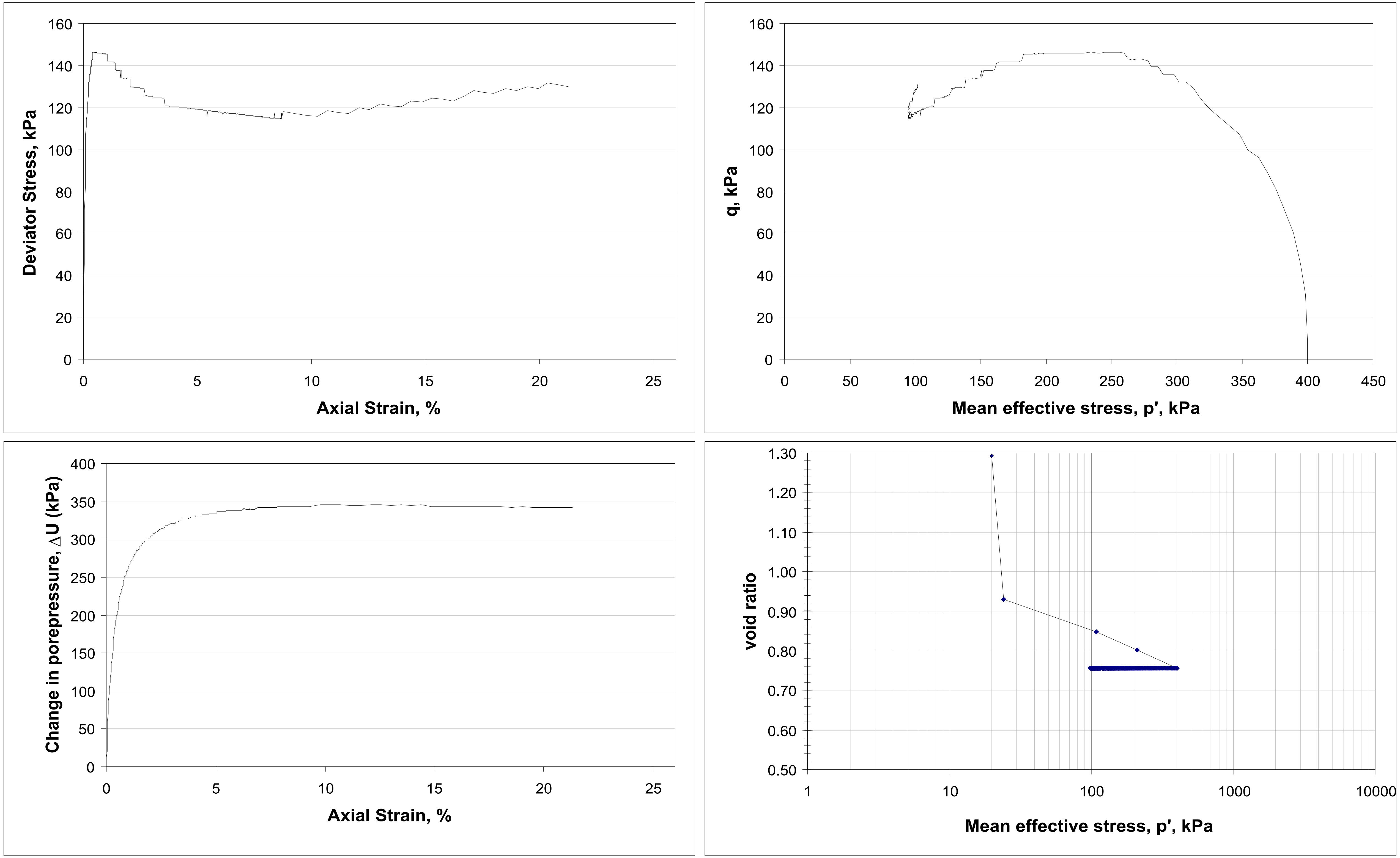
A total dry sample weight of between 800 g to 1100 g was mixed with water to achieve a moisture content of around 10% for the coarse A sample and around 12.5% for the fine B sample. The previously calculated dry soil mass was corrected to a wet soil mass based on the actual moisture content of this prepared sample and then divided into six equal weight portions required for the preparation void ratio.

The vacuum split mold was attached to the triaxial cell base with a membrane connected to the bottom platen. The top of the membrane was folded over the split mold and the membrane was held open with a vacuum pressure. The six equal mass portions of the moist soil were then compacted with the tamping rod in six equal thickness lifts (each 1/6 the total sample height in the mold). This helps ensure that each lift and thus the entire sample is prepared at the same density and void ratio. Prior to the next lift being placed, the surface of the previous lift was scarified to a depth of 2 mm to 4 mm. After tamping of the 6^{th} lift, the top cap platen was placed on sample, the membrane turned up and the o-rings applied to seal the membrane to the top cap. Then the vacuum was switched from the split mold to the sample through the drainage port. At this point a vacuum pressure of between 15 kPa to 20 kPa was applied.

While under the vacuum pressure, accurate sample height and diameter were measured. Following this the triaxial cell was assembled, filled with water and a 20 kPa confining pressure applied to the cell. Then, $C0_2$ gas was slowly percolated through the sample for one to two hours. Following $C0_2$, de-aired water was allowed to flow through the sample under less than 1 m driving head. During this time the changes in the cell volume were recorded for an estimate of the sample saturation volume changes.

The de-aired water was percolated through the sample for some four to eight hours. Following this initial saturation, the sample pore pressure lines were connected to the volume change device and back pressure applied. Then back pressure saturation was carried out by increasing the cell pressure and measuring the response in pore pressure. Increments of cell pressure were made until a $\bar{B} > 0.97$ was achieved, where $\bar{B} = \Delta u / \Delta \sigma 3$.

After achieving sample saturation consolidation to the required pressure was carried out in increments. Data from the consolidation tests was used to define the normal compression line (NCL) behaviour. After consolidation, the sample was sheared. At the end of all tests, drainage to the cell was locked (even for drained tests) and the cell drained. After disassembly of the test frame, the sample on the base pedestal with top cap and intact membrane (and thus with the sample still at its end of test water content because of the locked drainage lines) was removed to a freezer. The moisture content (w) of the frozen sample was then measured, after removing the frozen soil from the pedestal, by oven drying for 24 hours and calculating void ratio as $e = w^*G_s$ (recall the sample was fully saturated by the test procedures so S =1). This frozen sample technique provides by far the most accurate void ratio measurement (Sladen & Hanford, 1987), so minimizing uncertainty in CSL void ratio.



PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

TRIAXIAL TEST RESULT
A_CIU-1

Golder Associates

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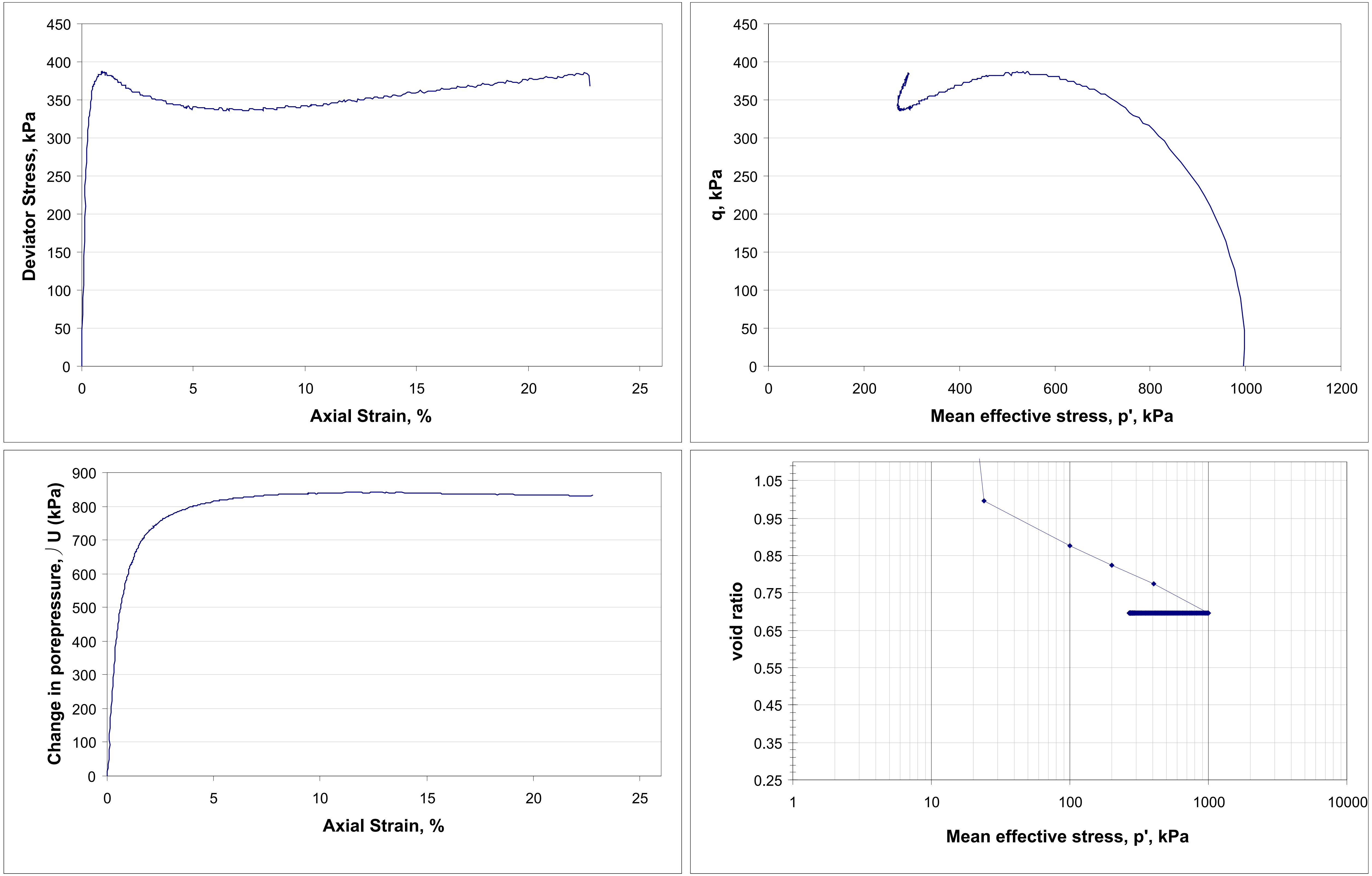
REVIEW				



PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
TRIAXIAL TEST RESULT
A_CIU-2

Golder Associates

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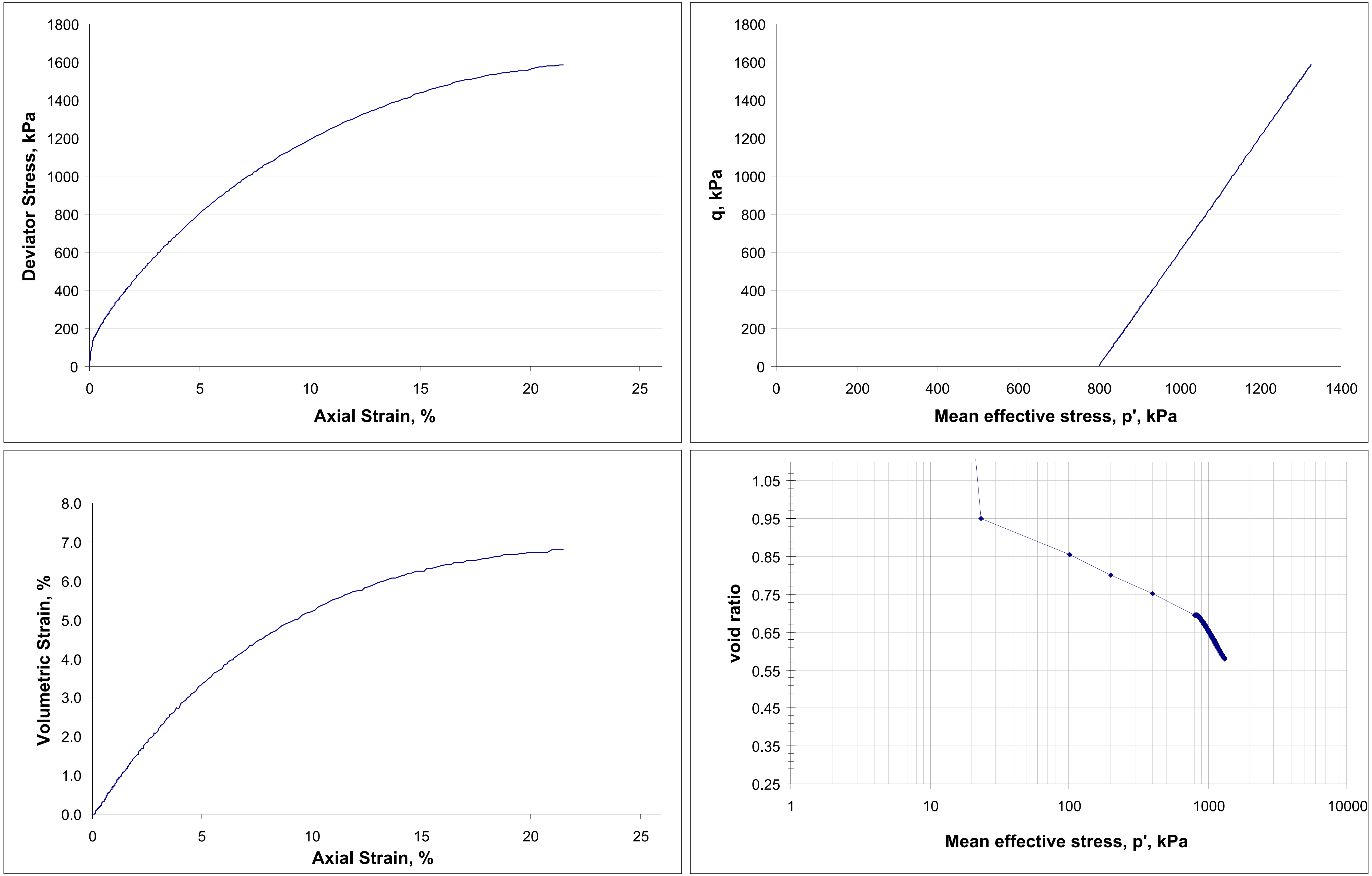

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

TRIAXIAL TEST RESULT
A_CID-3

Golder Associates

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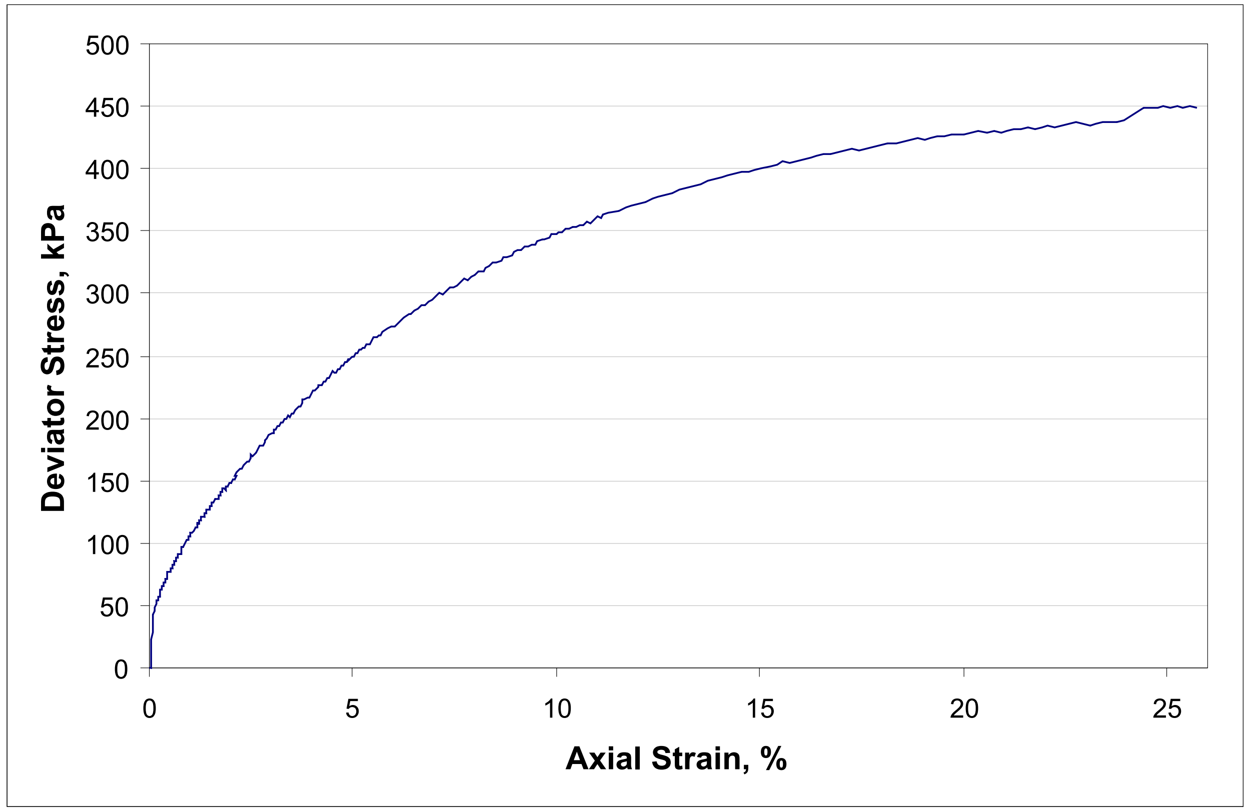

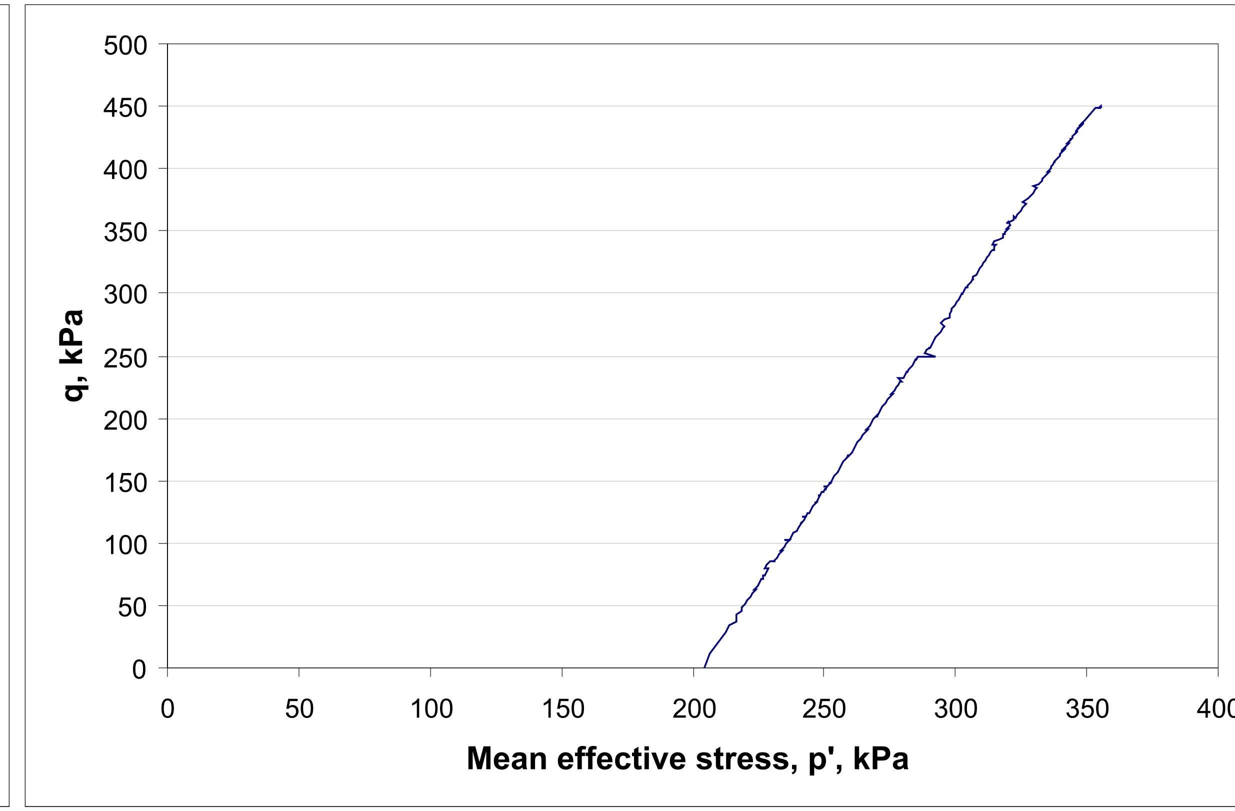

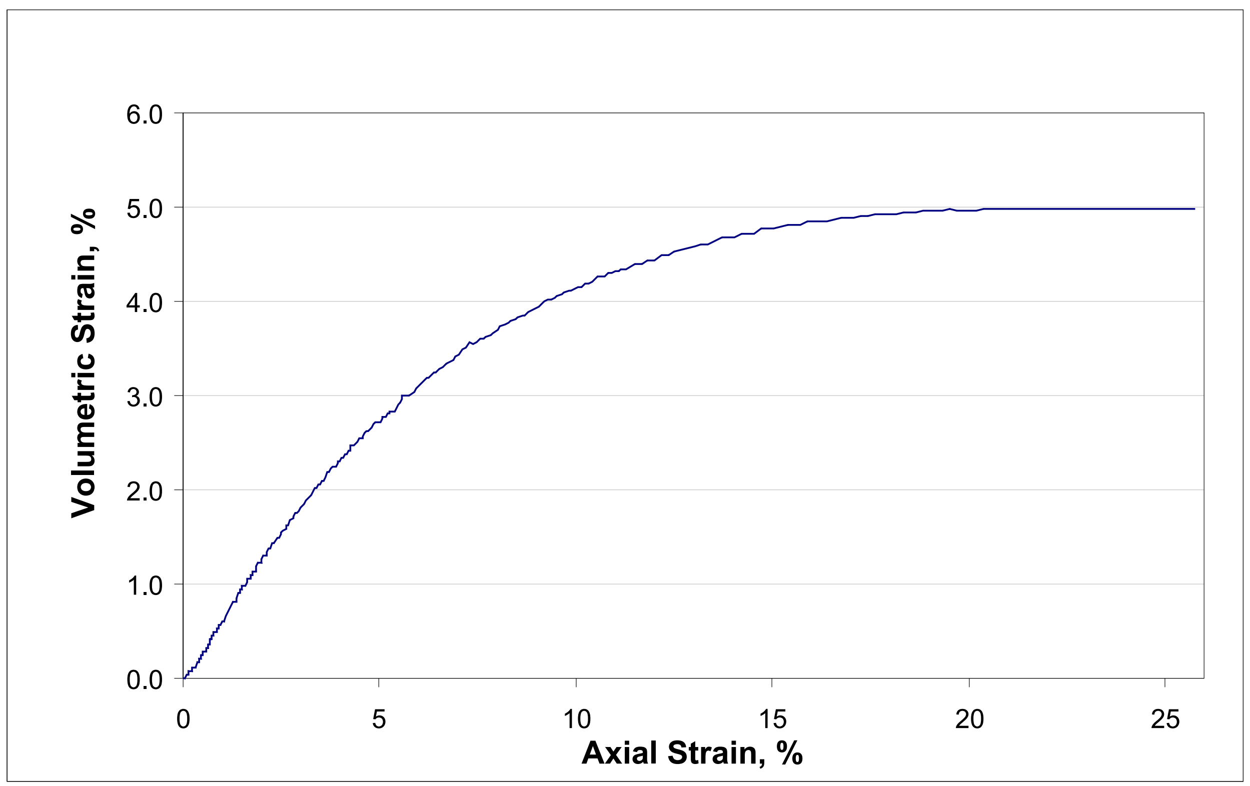

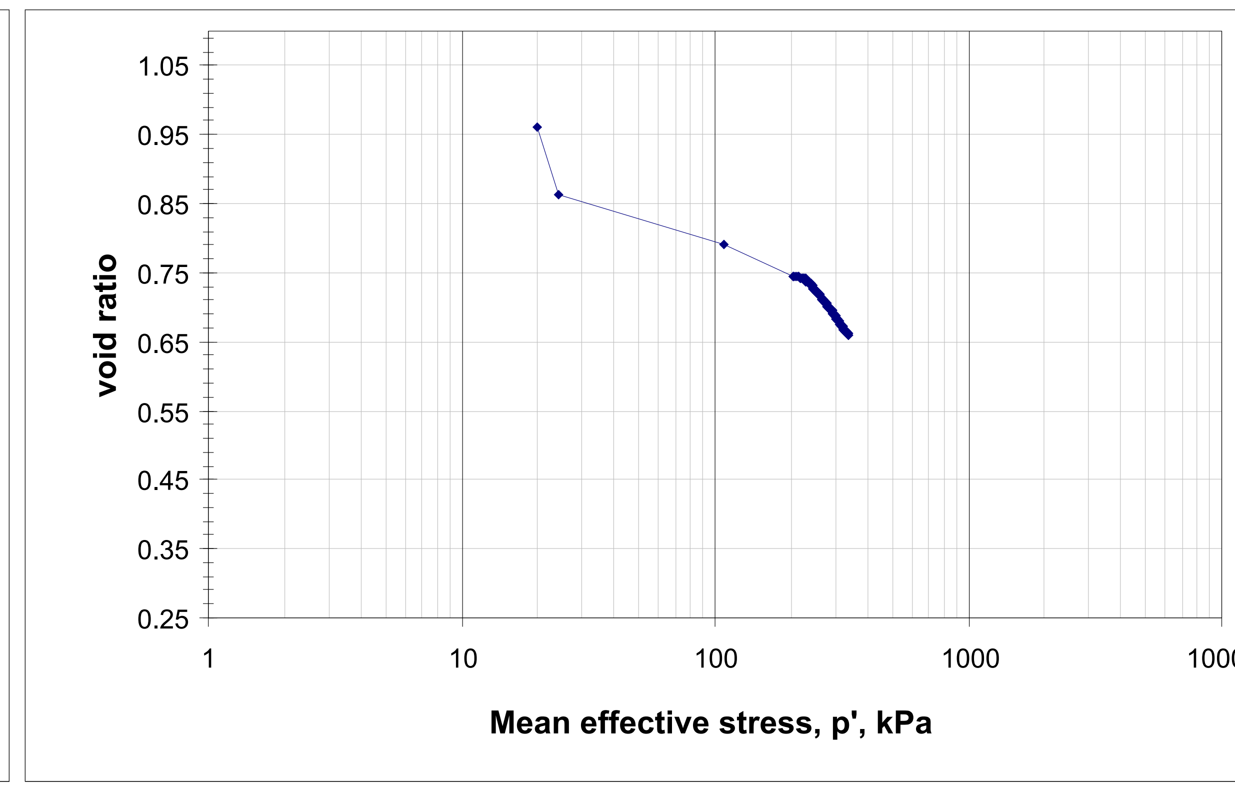

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

TRIAXIAL TEST RESULT
A_CID-4

Golder Associates

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CHECK	JC	28APR04	
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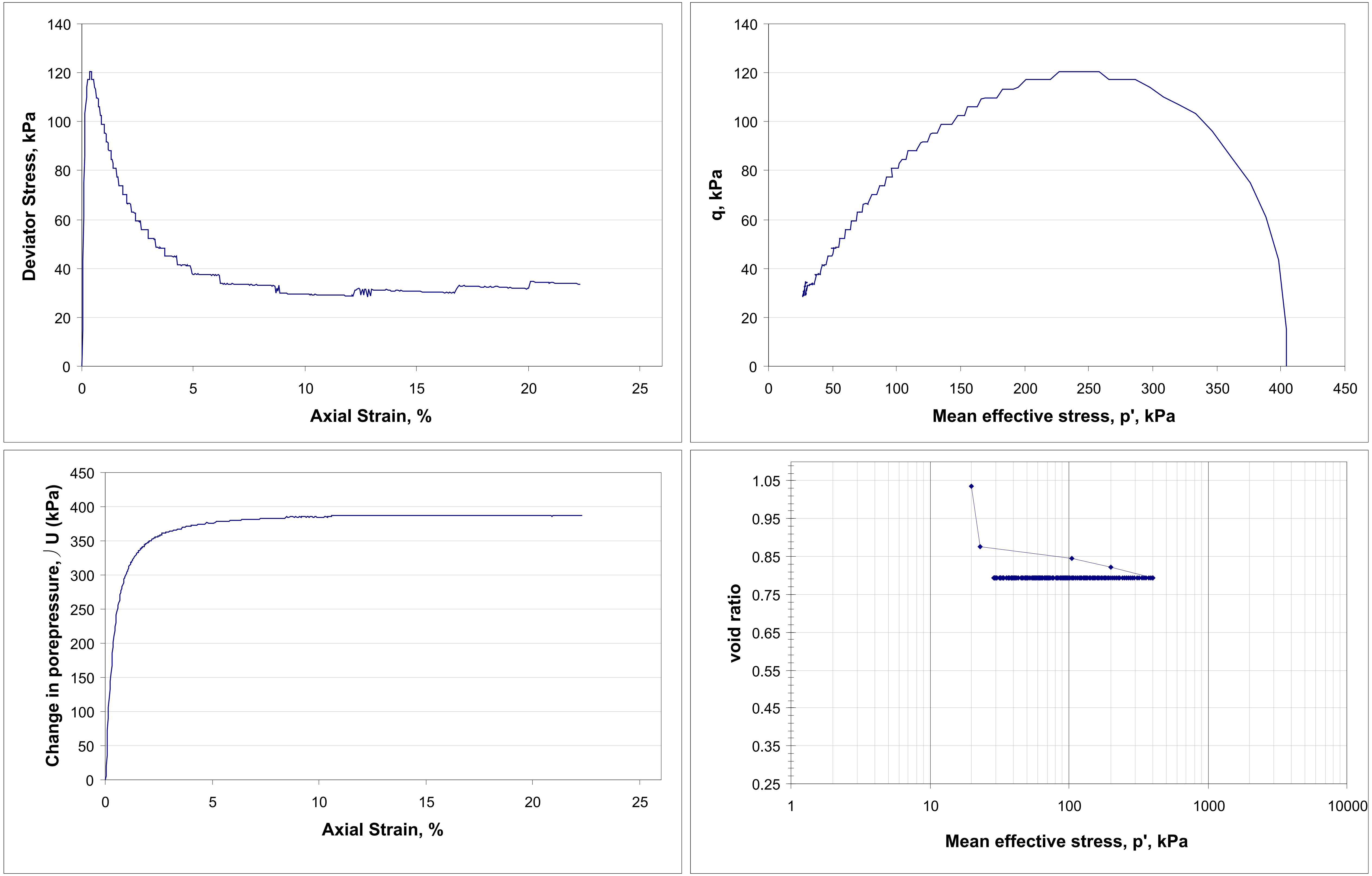

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	**TRIAXIAL TEST RESULT B_CIU-1**

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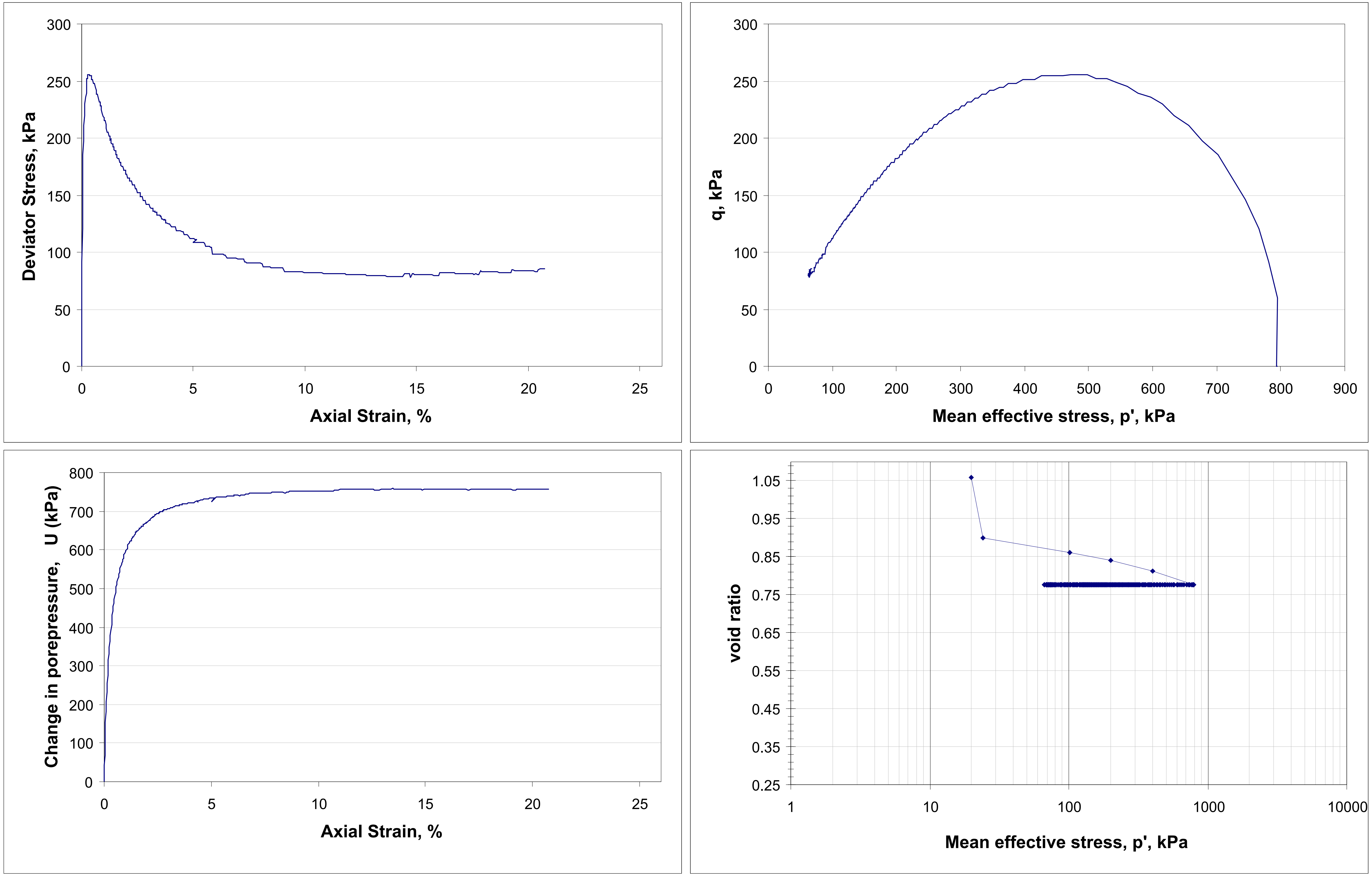
CHECK	JC	28APR04	
REVIEW			

Golder Associates

REVISION DATE:28-APR-04 BY: SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 4.cdr



PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
TRIAXIAL TEST RESULT
B_CIU-2

Golder Associates

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REVIEW			

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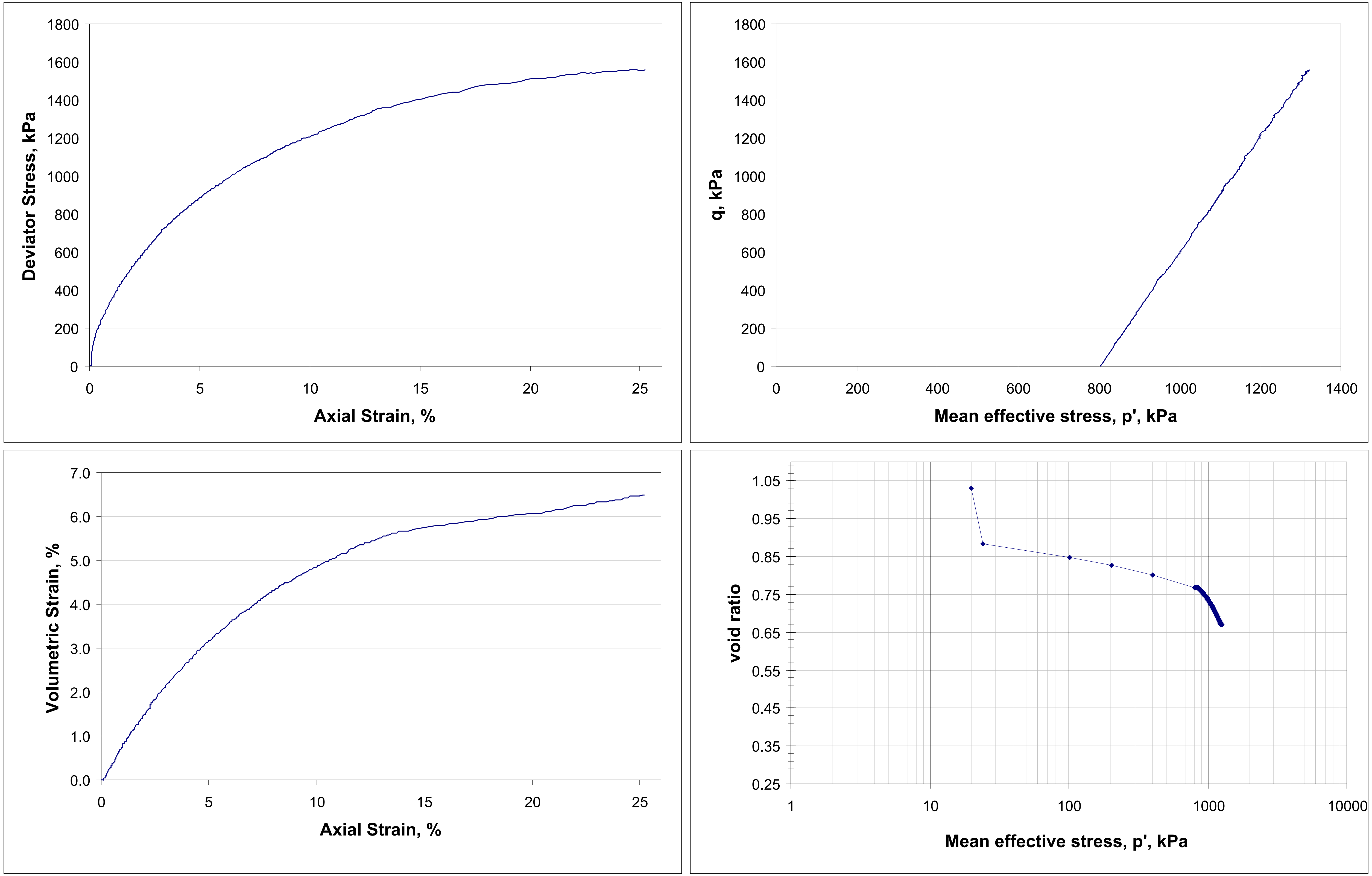

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
TRIAXIAL TEST RESULT
B_CID-3

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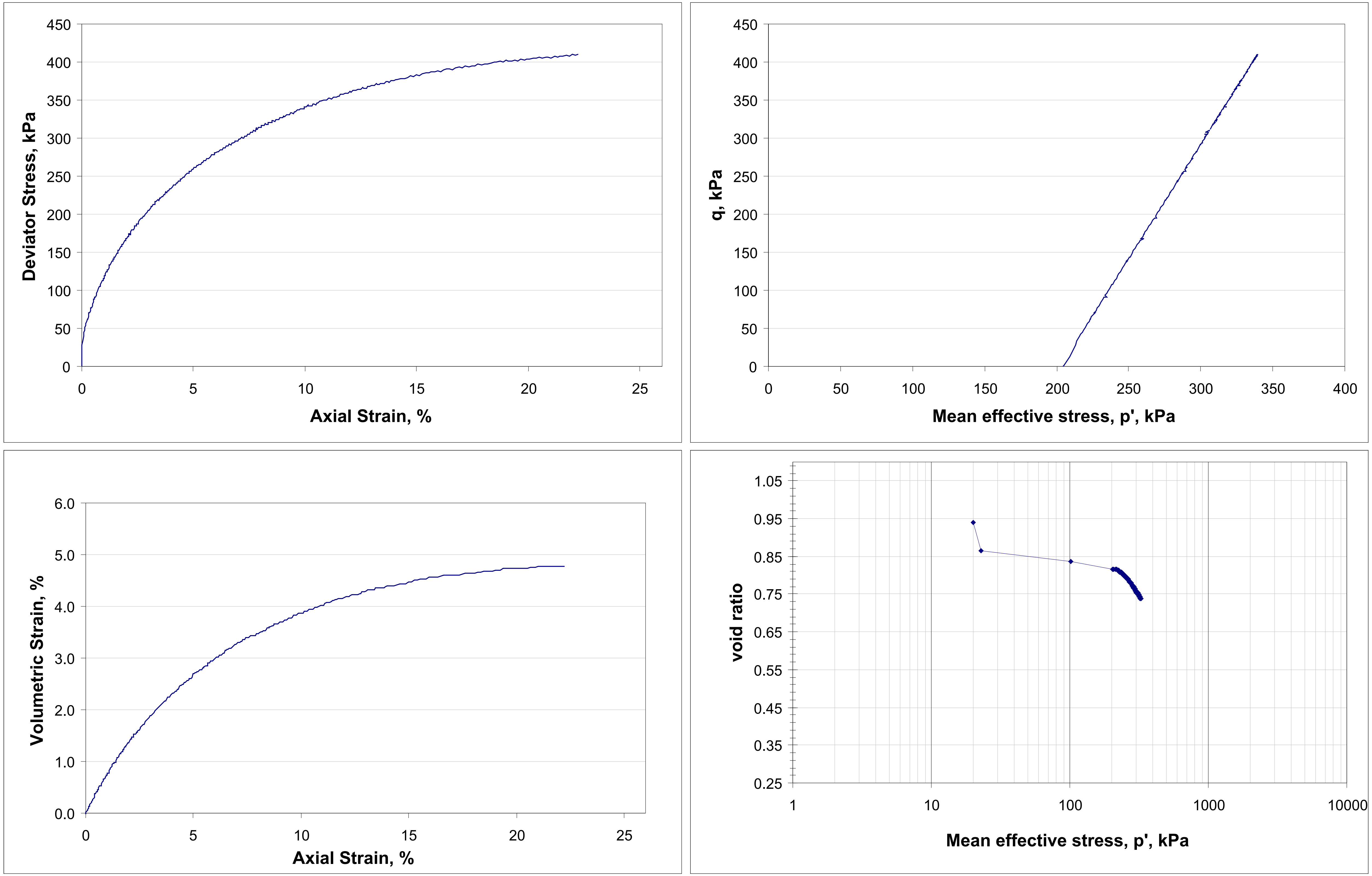
REVIEW			

Golder Associates

REVISION DATE:28-APR-04 BY: SS COREL FILE:N:\Bur-Graphics\Projects\2003\1413\03-1413-080\Drafting\corel\Figures 4.cdr



PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	TRIAXIAL TEST RESULT B_CID-4

Golder Associates

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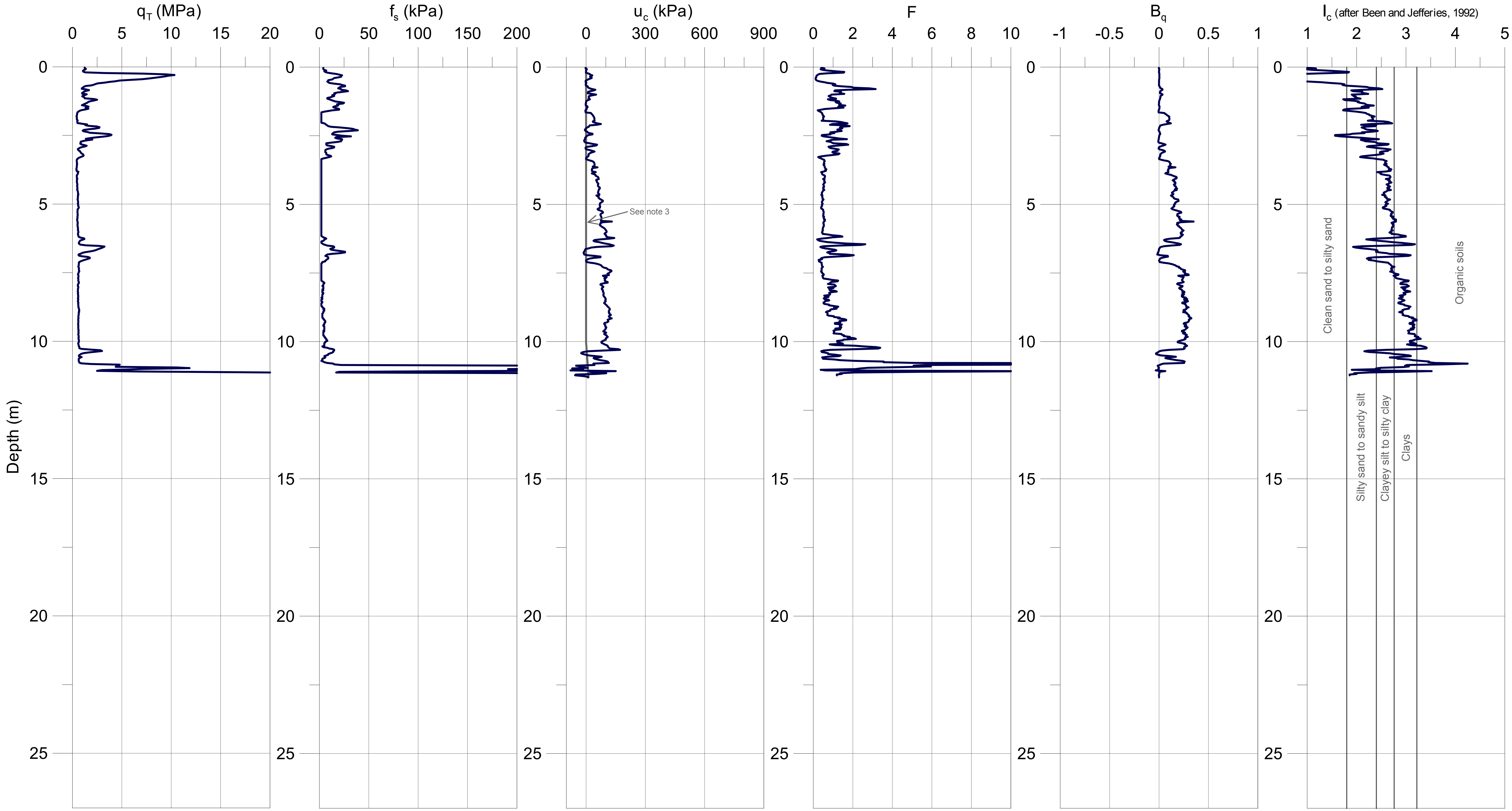
REVIEW			

APPENDIX III

CPT LOGS



Notes

1. I_C values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

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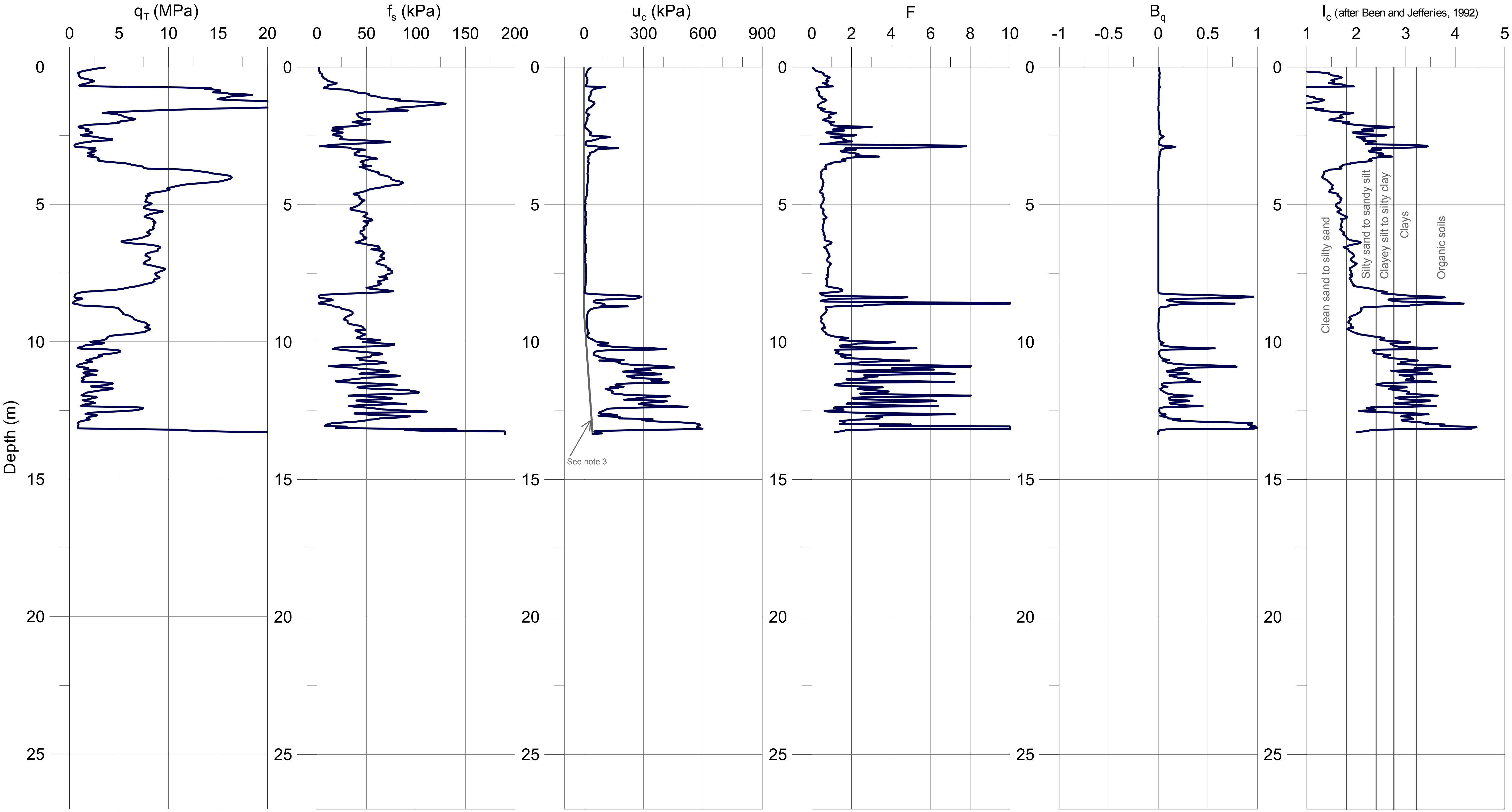

Notes

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2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

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PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

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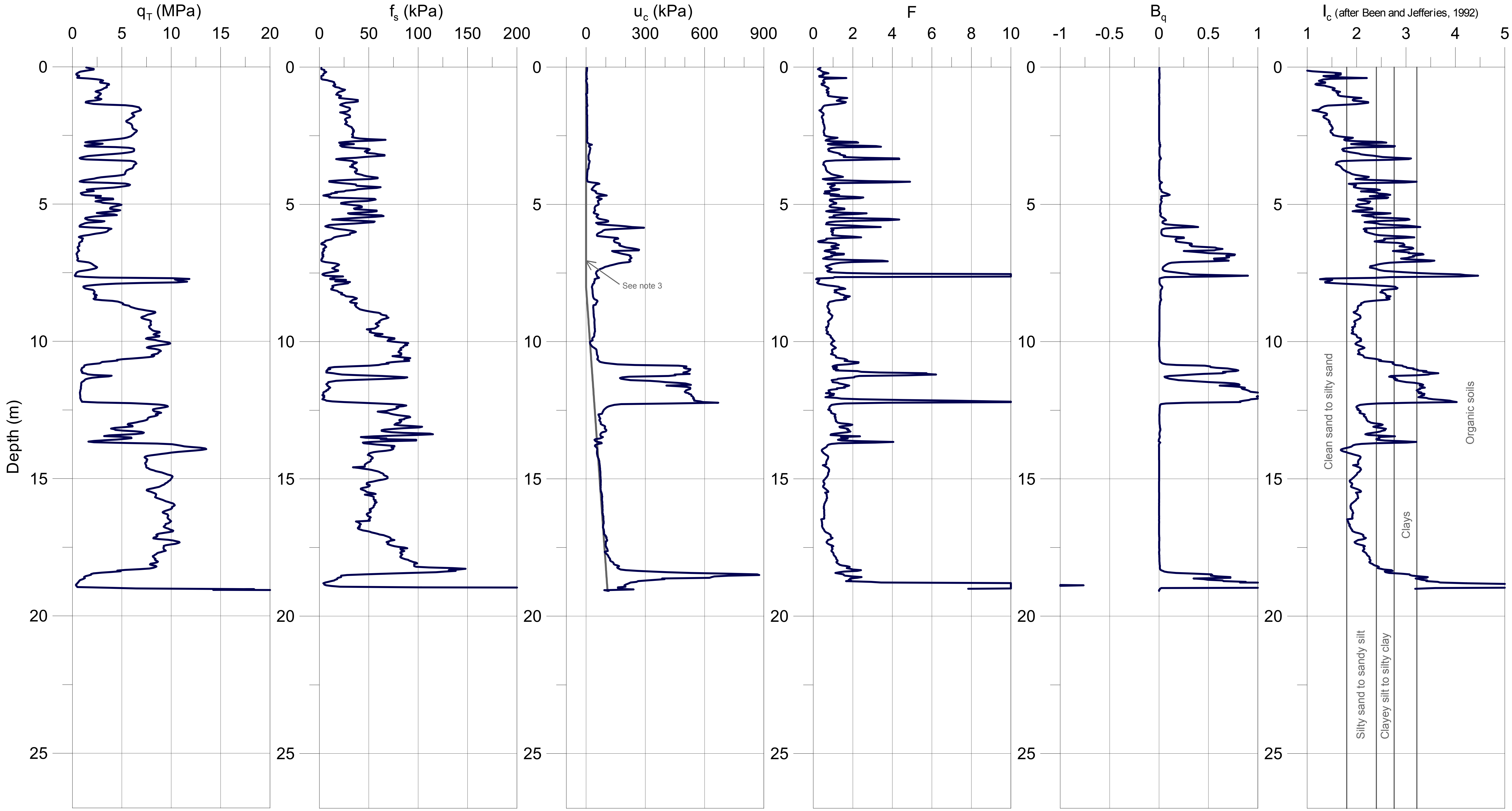

Notes

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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

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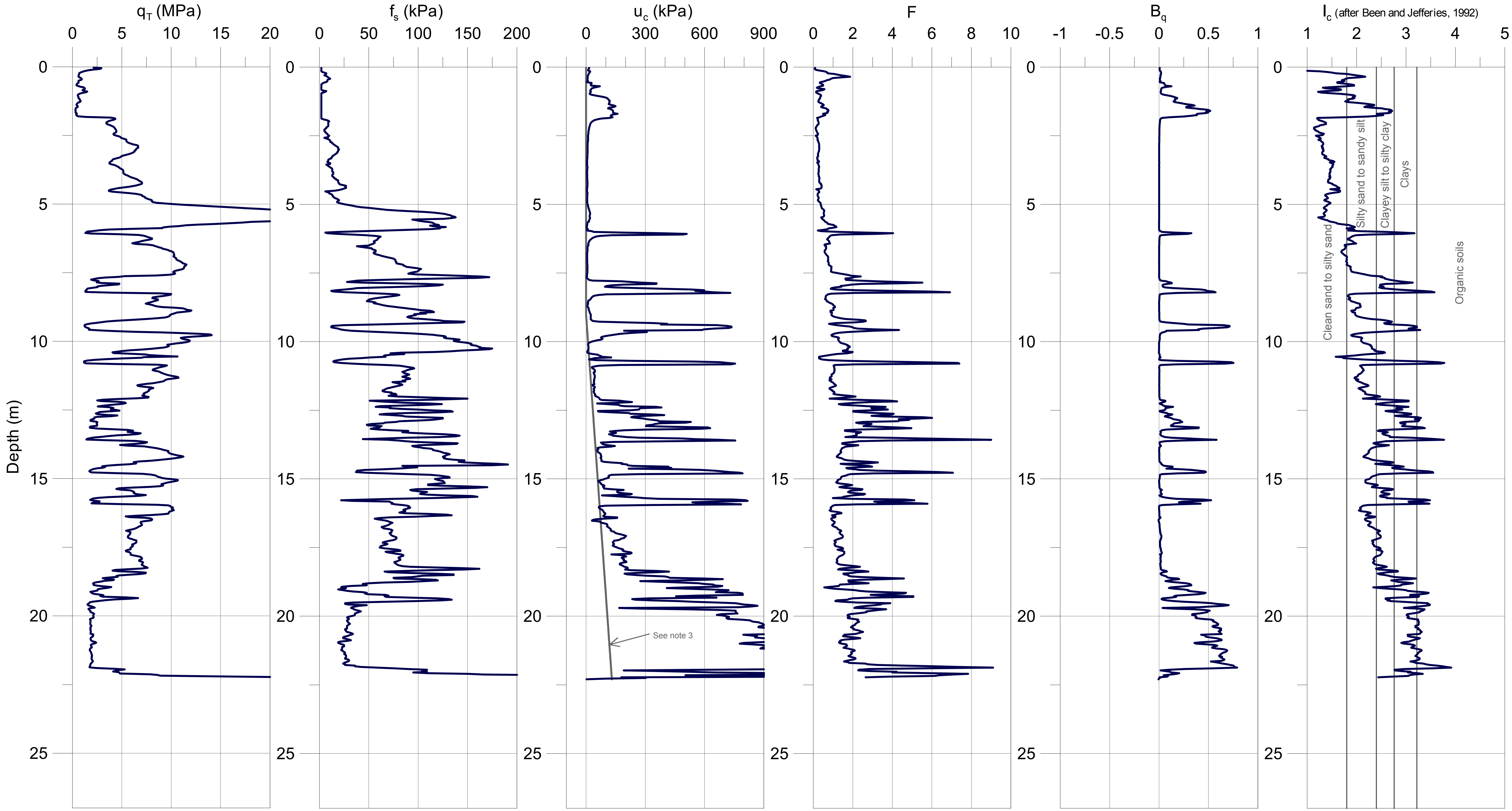

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1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

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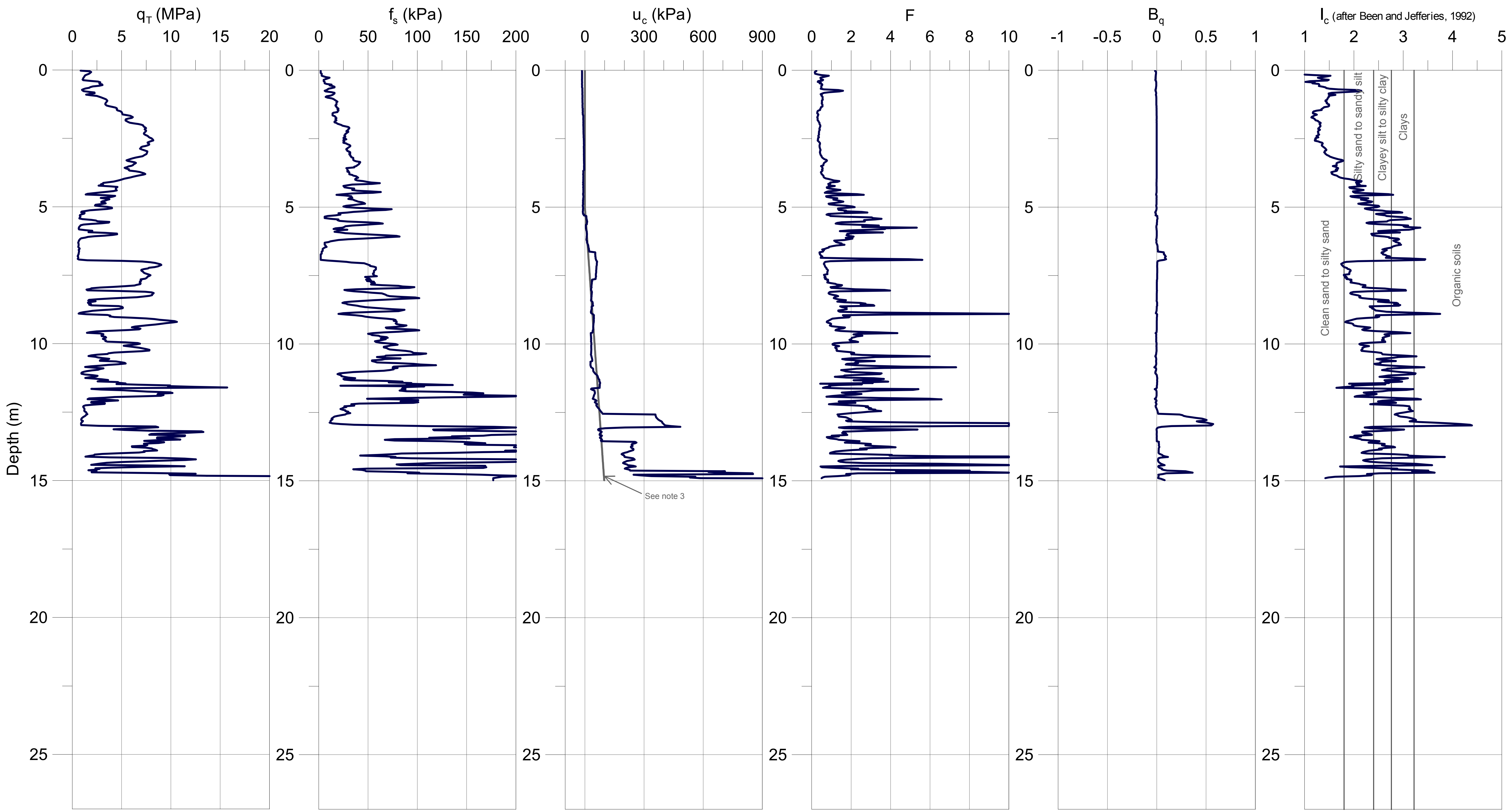

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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

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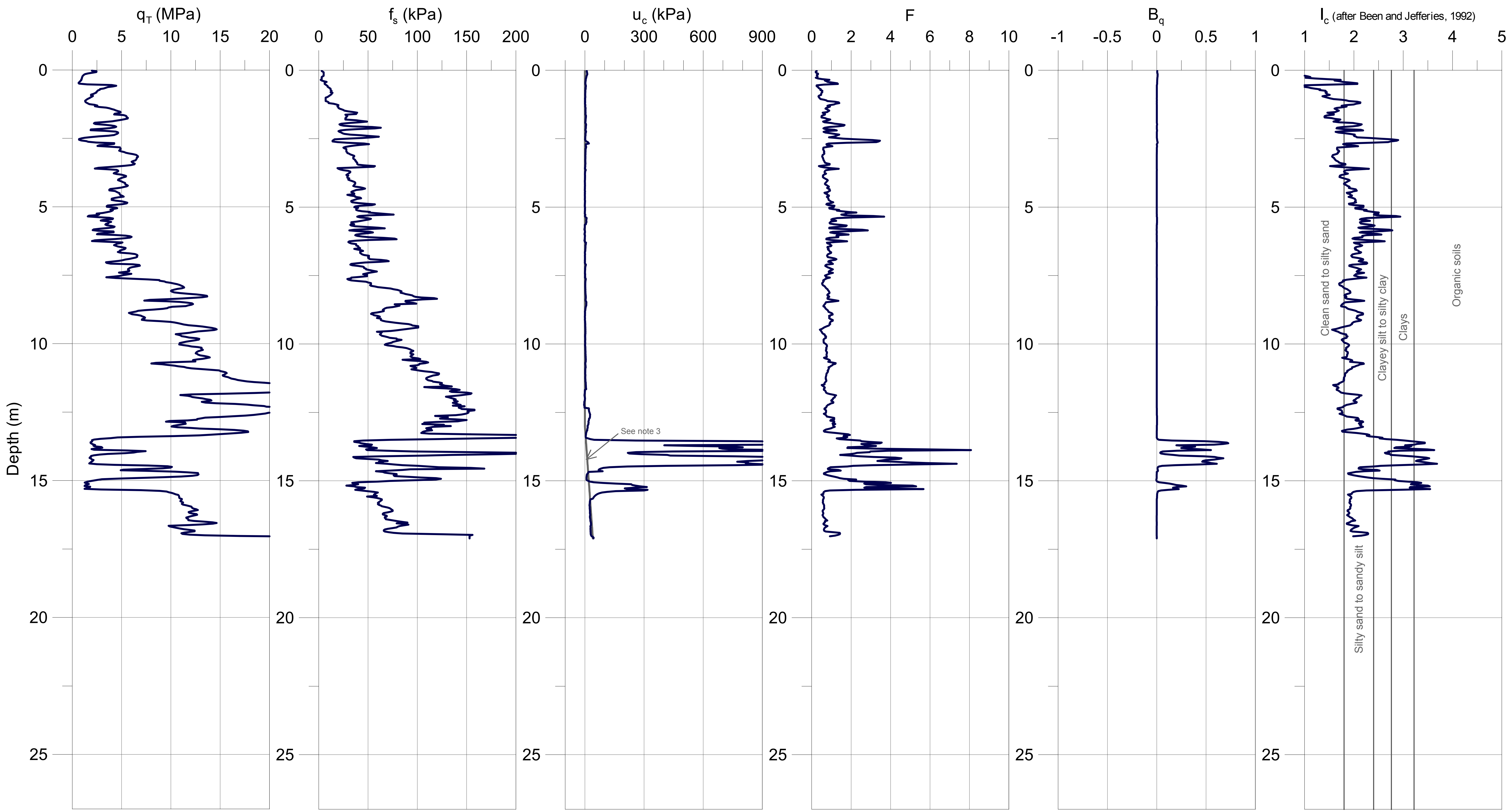

Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

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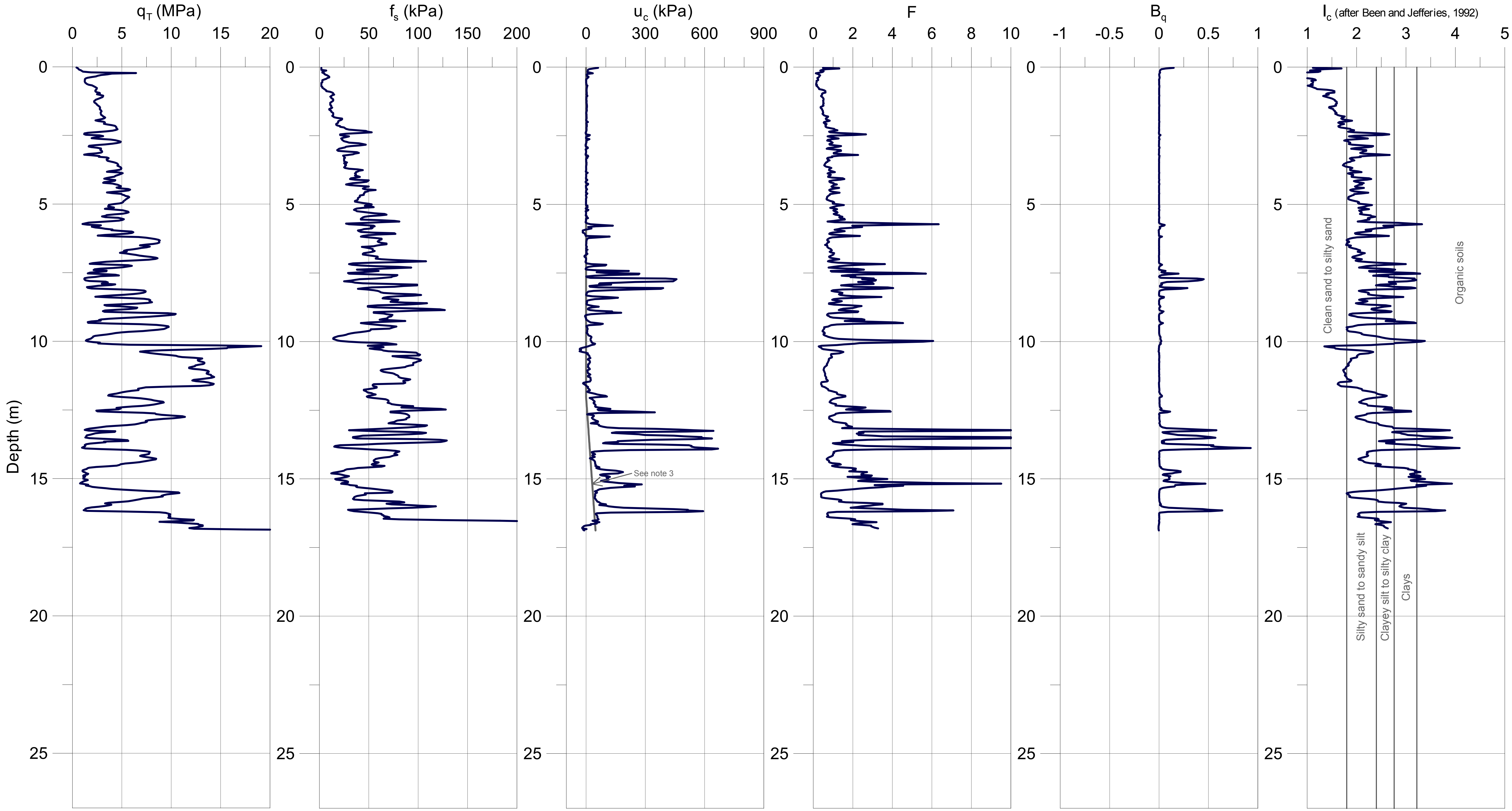

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2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

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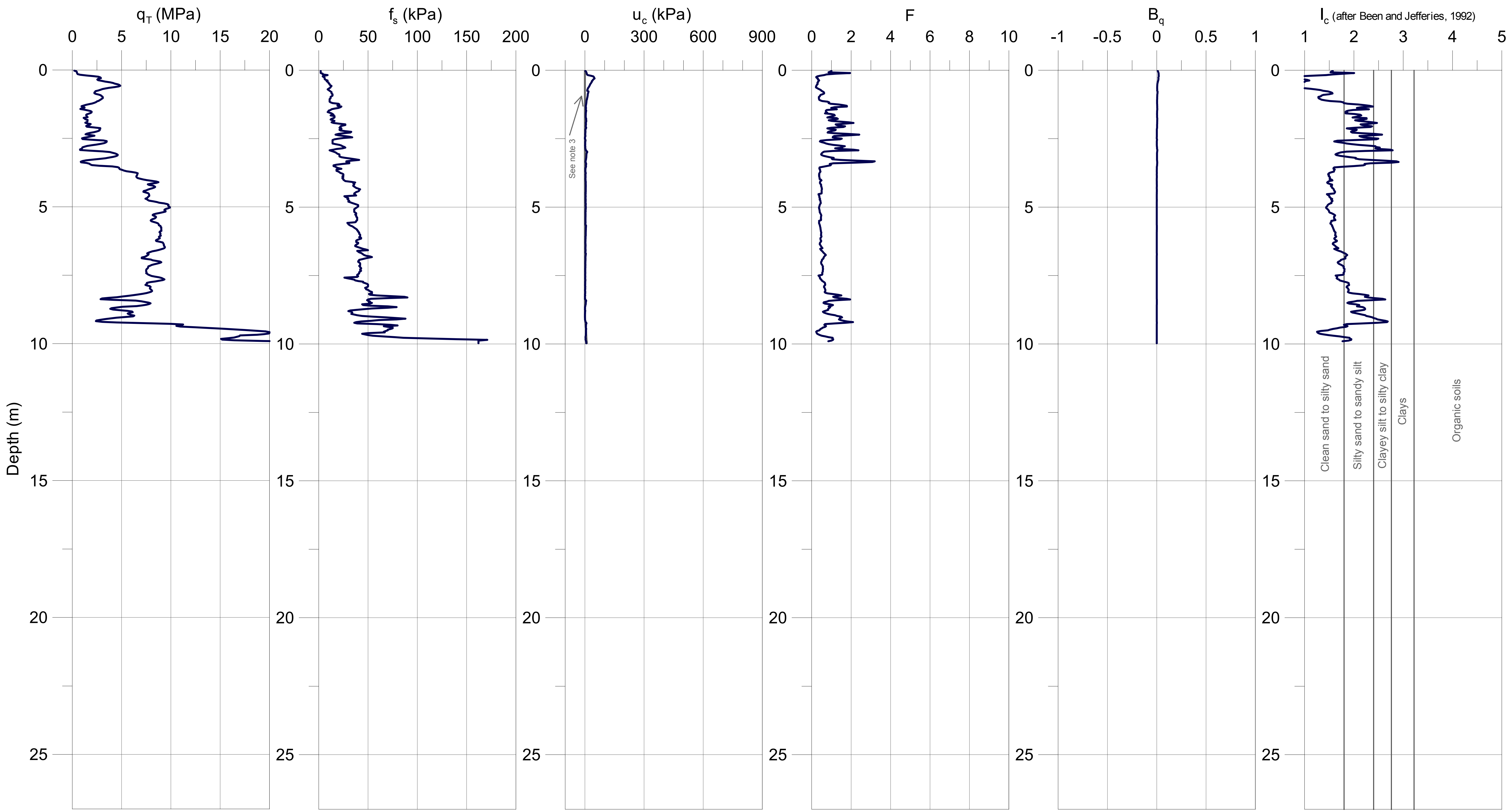

Notes

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2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES			
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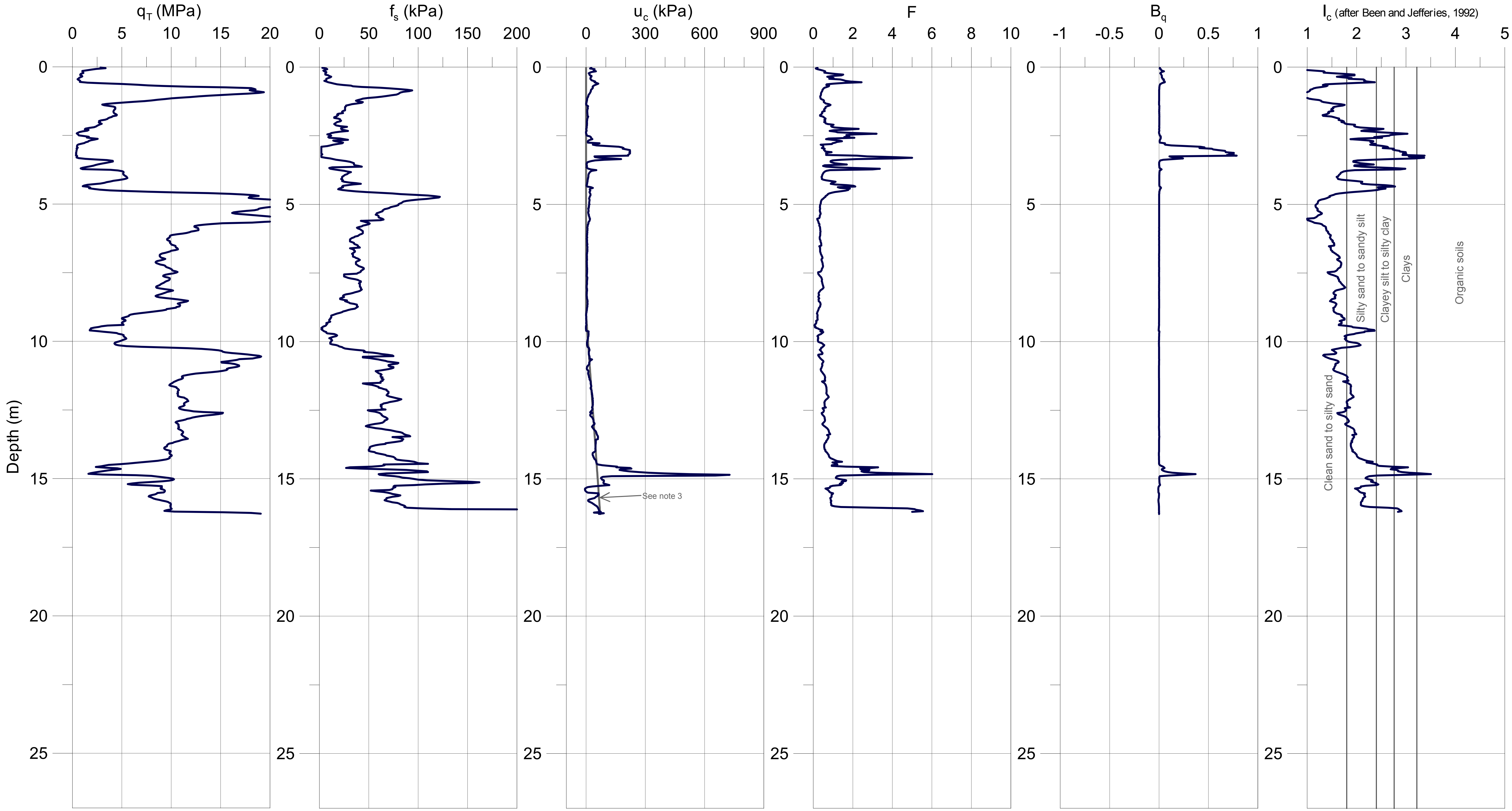

Notes

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2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

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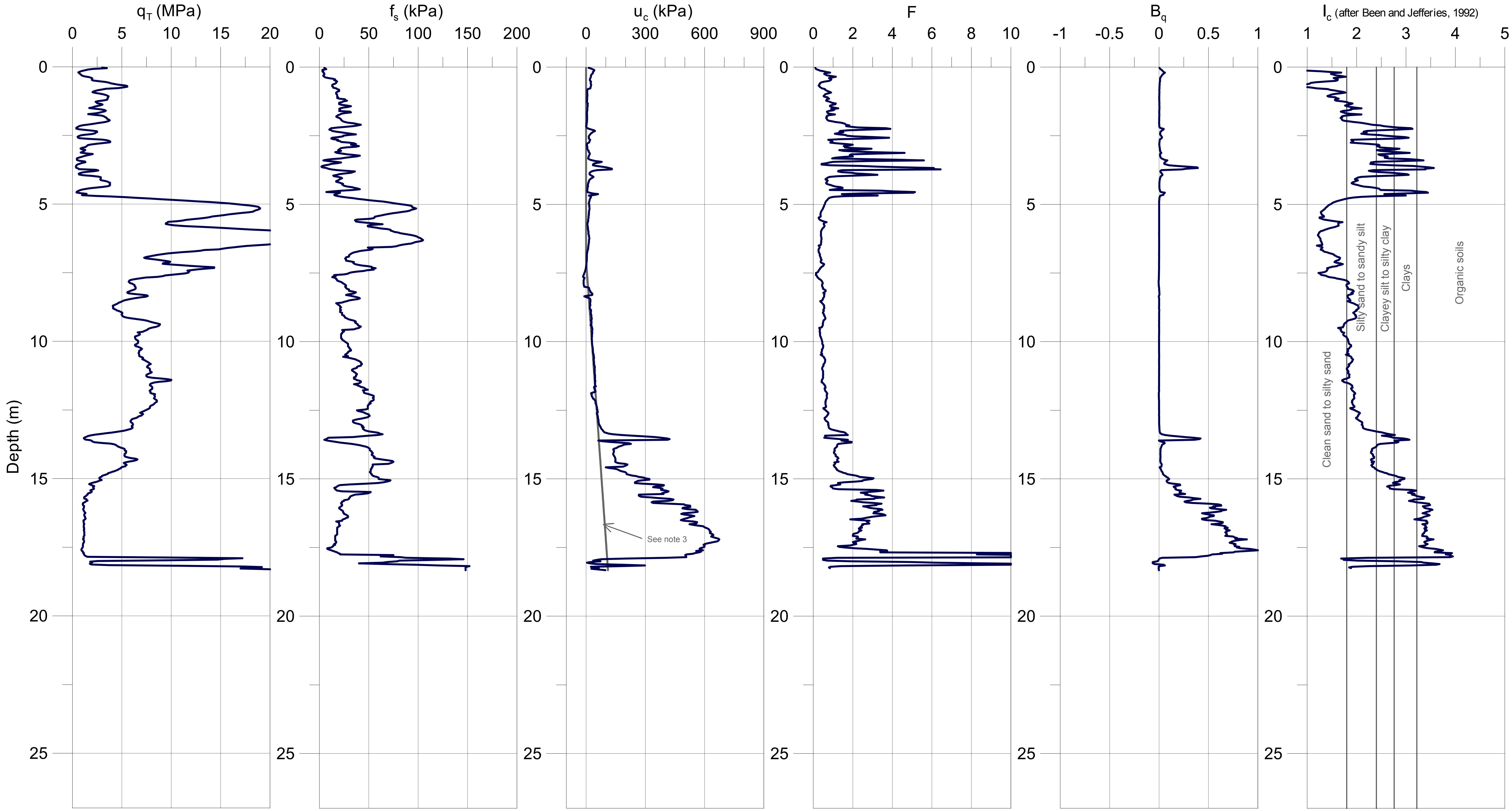

Notes

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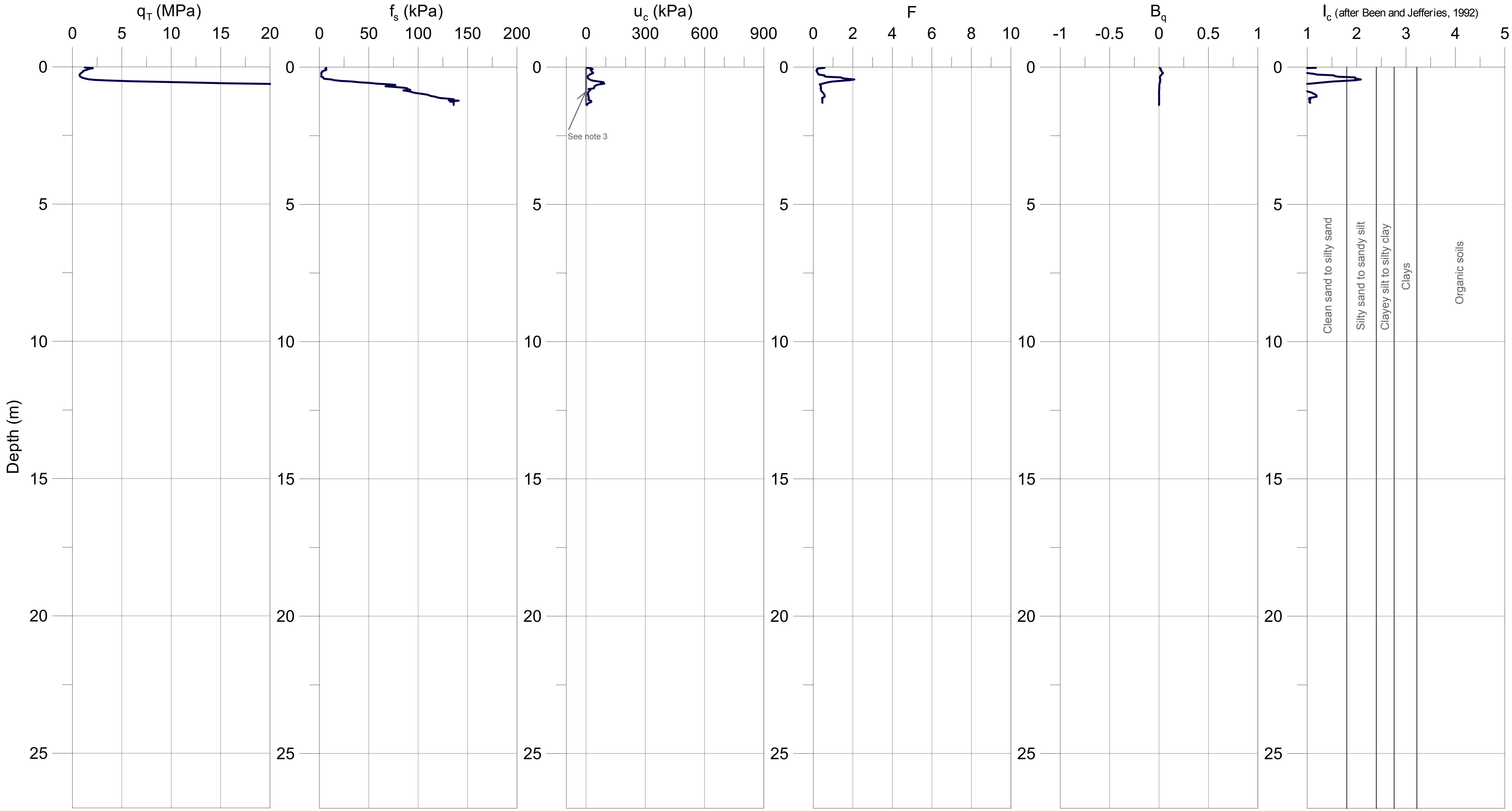
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON	
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES	

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-10
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	CPT03-10
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-10.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

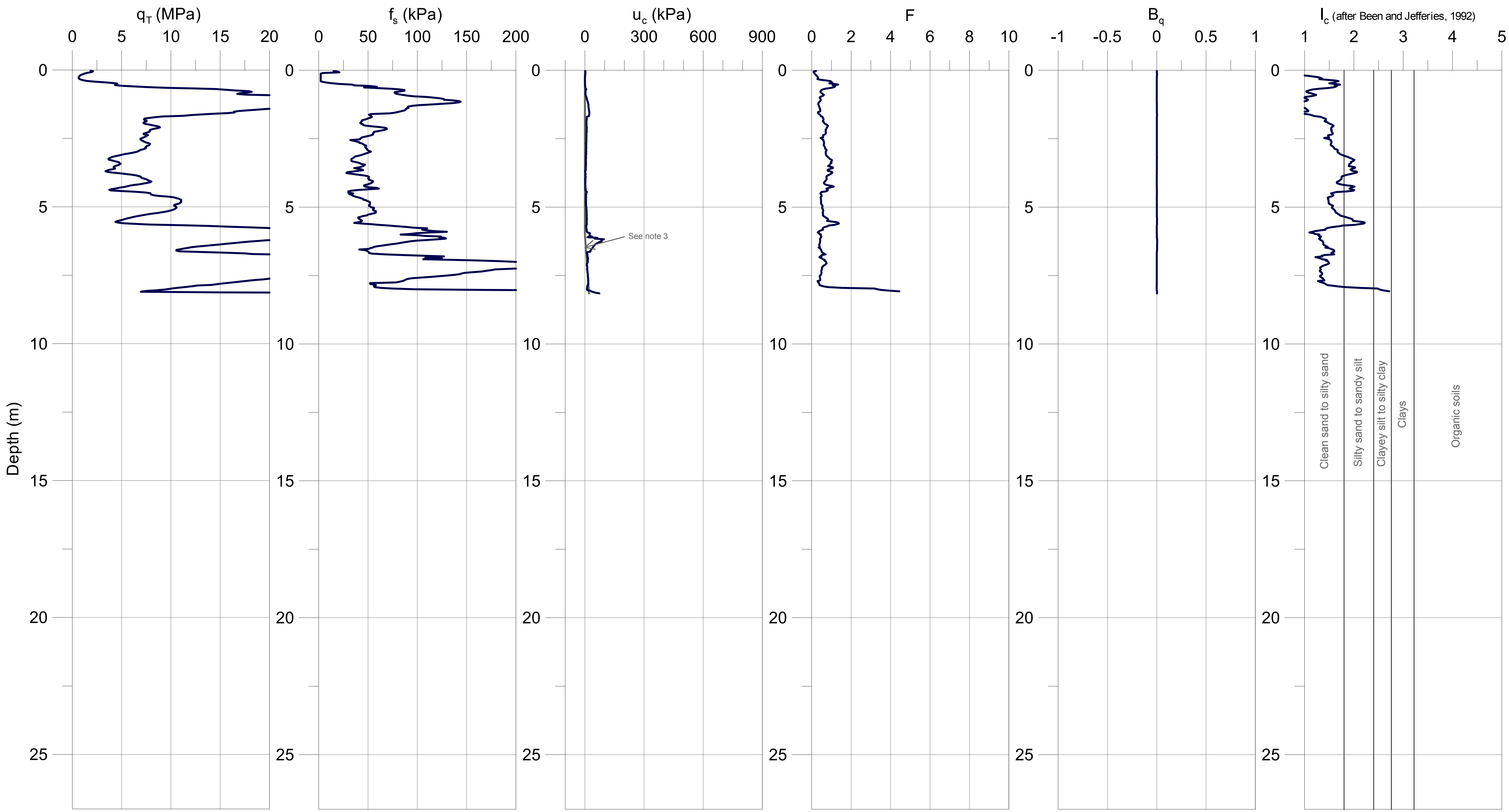
Golder Associates

PROJECT No.	03-1413-080	
DESIGN	DRK	21 JAN 04
CADD	DRK	21 JAN 04
CHECK	--	--
REVIEW	--	--

FILE No. plots-CPT03-11
SCALE NTS REV.

CPT03-11

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-11.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-11B.grf

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No.	plots-CPT03-11B
DESIGN	DRK	21 JAN 04	SCALE NTS	REV.
CADD	DRK	21 JAN 04		
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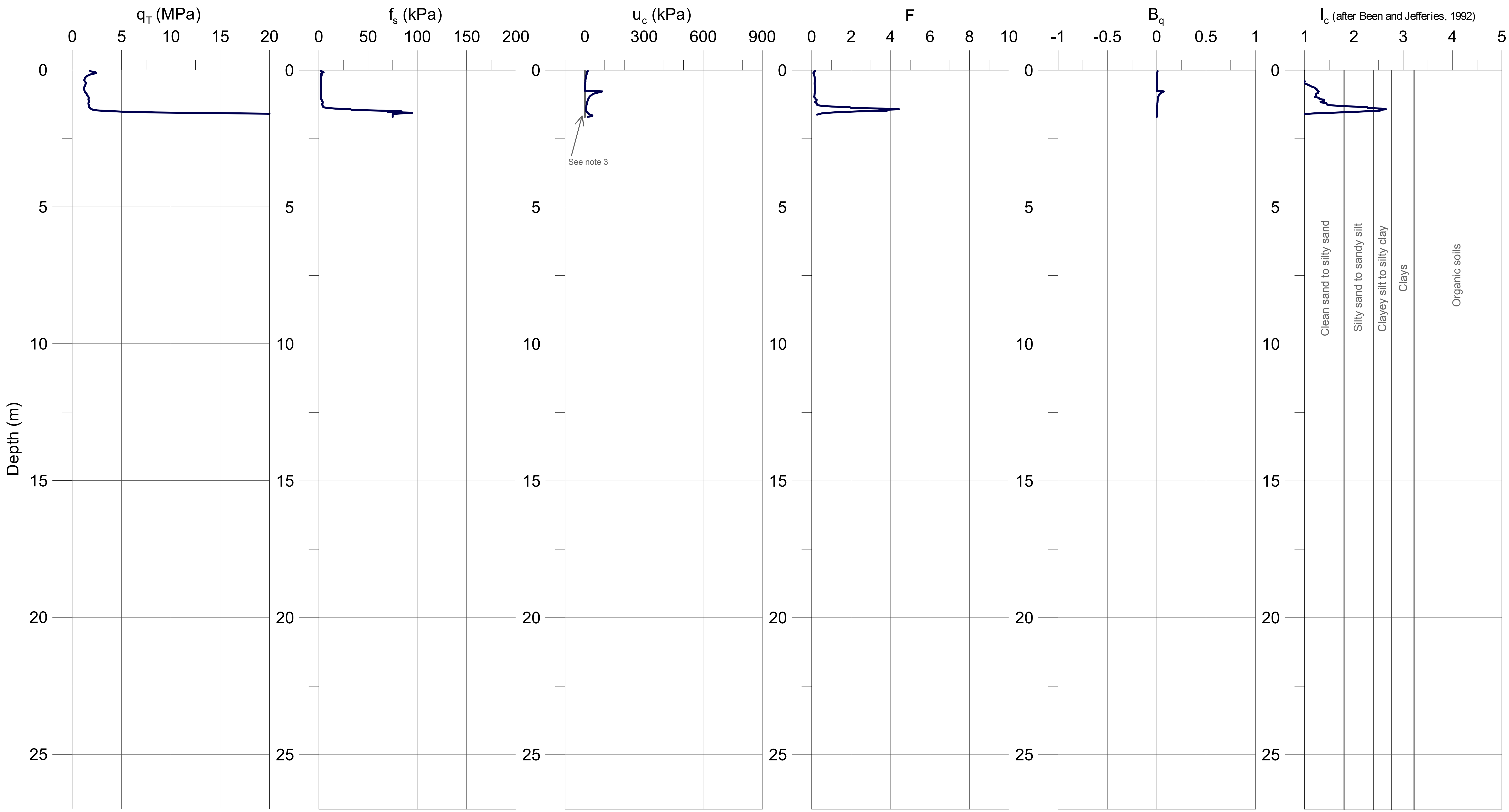
REVIEW	--	--		



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
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PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

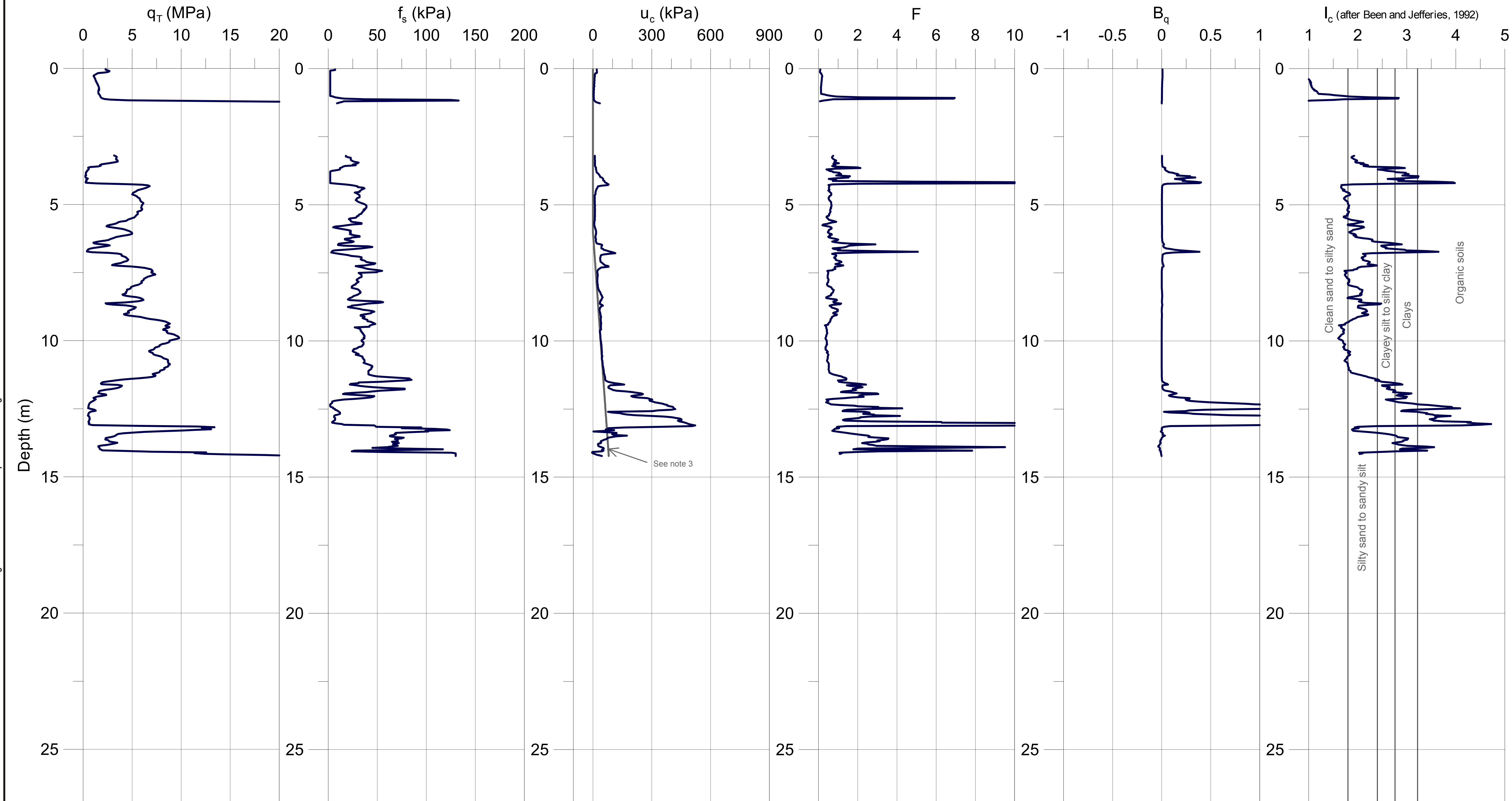
TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-12
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-12**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-12.grf



REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-12B.grf

Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON			
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES			
Golder Associates	PROJECT No.	03-1413-080	FILE No. plots-CPT03-12B	
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	CADD	DRK 21 JAN 04	CPT03-12B	
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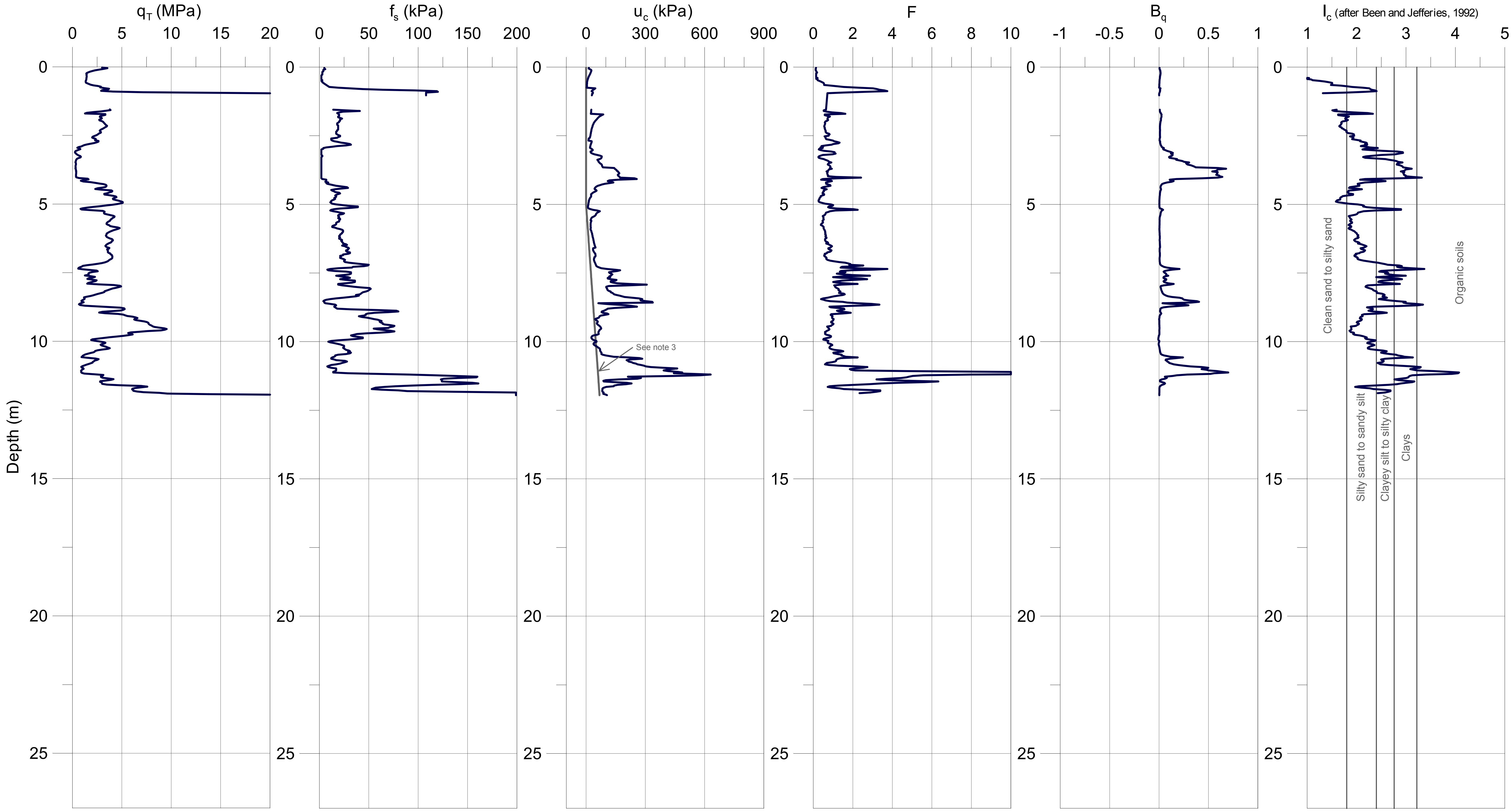
	REVIEW	-- --		



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

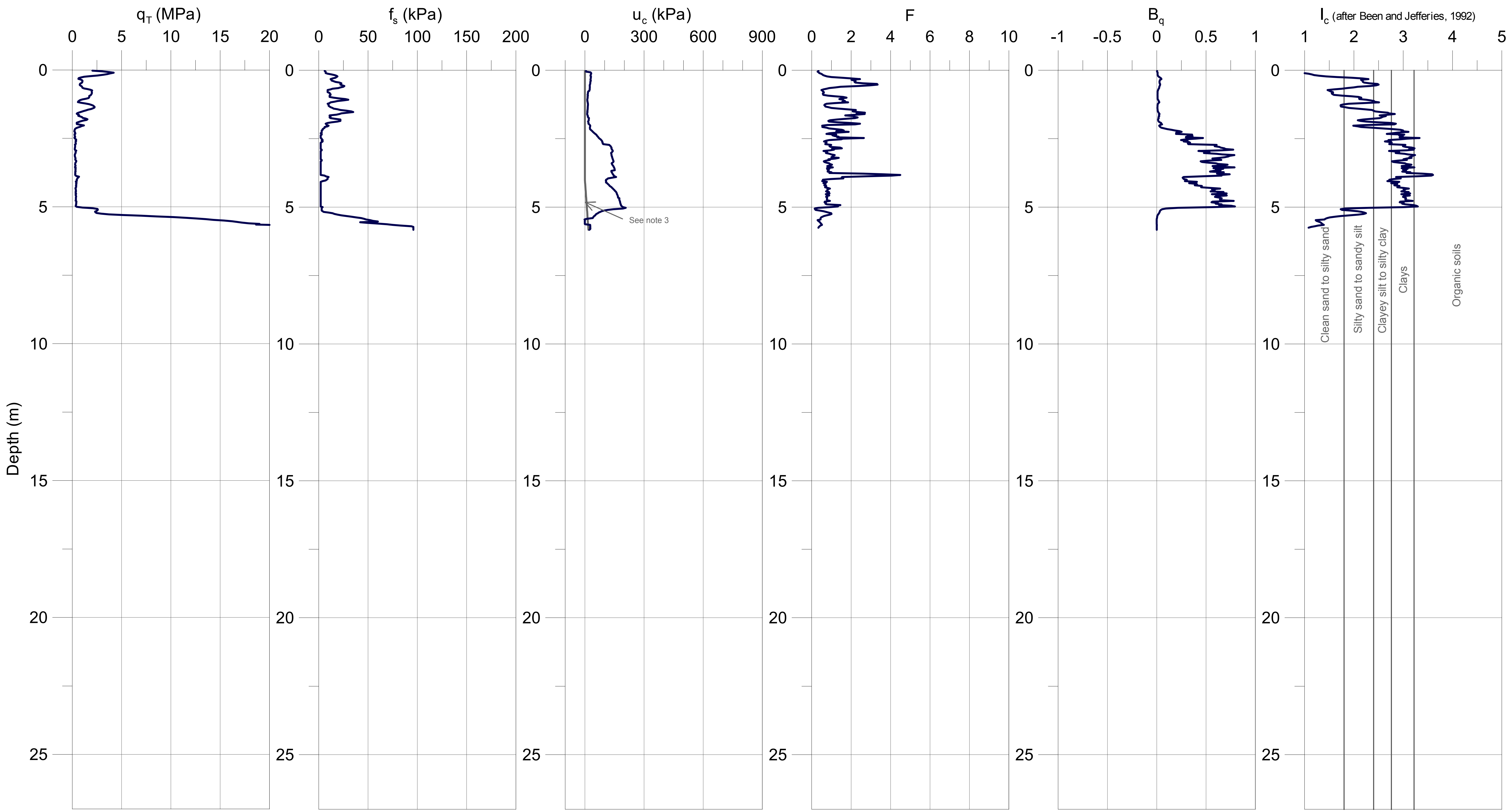
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-13
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-13**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-13.grf



REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-14.grf

Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

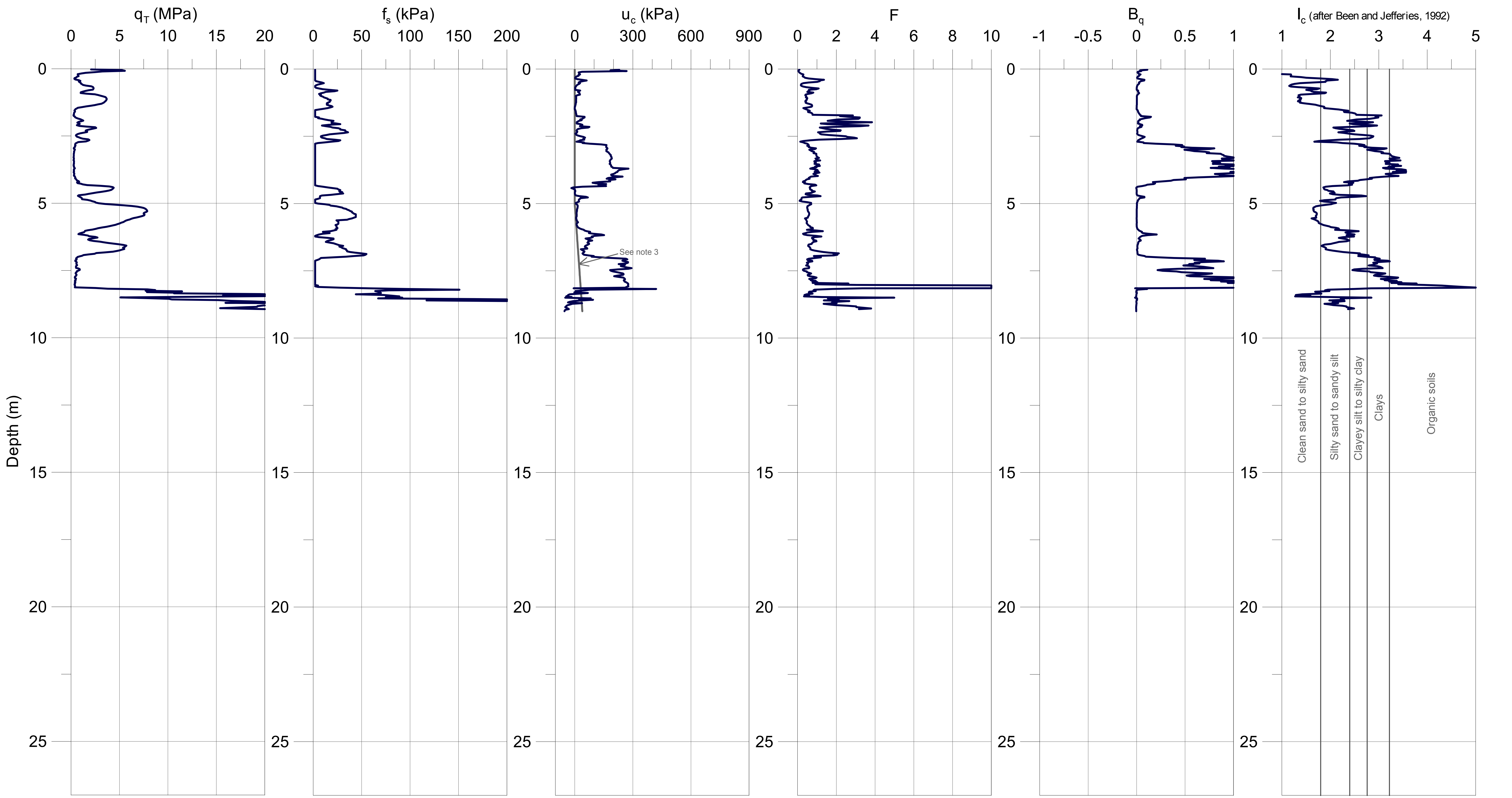
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-14
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-14**
CHECK	--	--	
REVIEW	--	--	



Notes

1. I_C values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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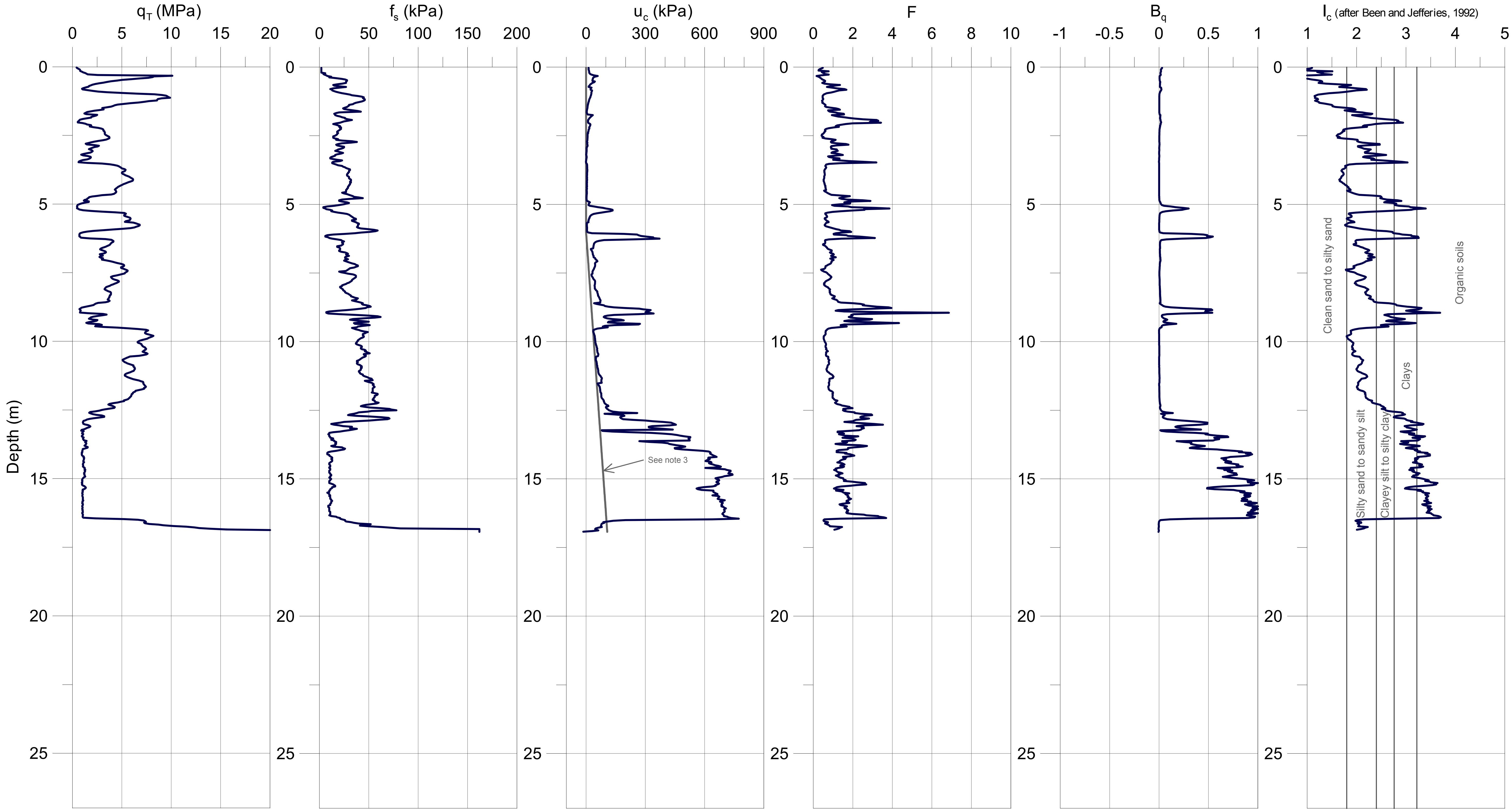
PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates				
PROJECT No.	03-1413-080		FILE No.	plots-CPT03-15
DESIGN	DRK	21 JAN 04	SCALE NTS	REV.
CADD	DRK	21 JAN 04	CPT03-15	
CHECK	--	--		
REVIEW	--	--		

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-15.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

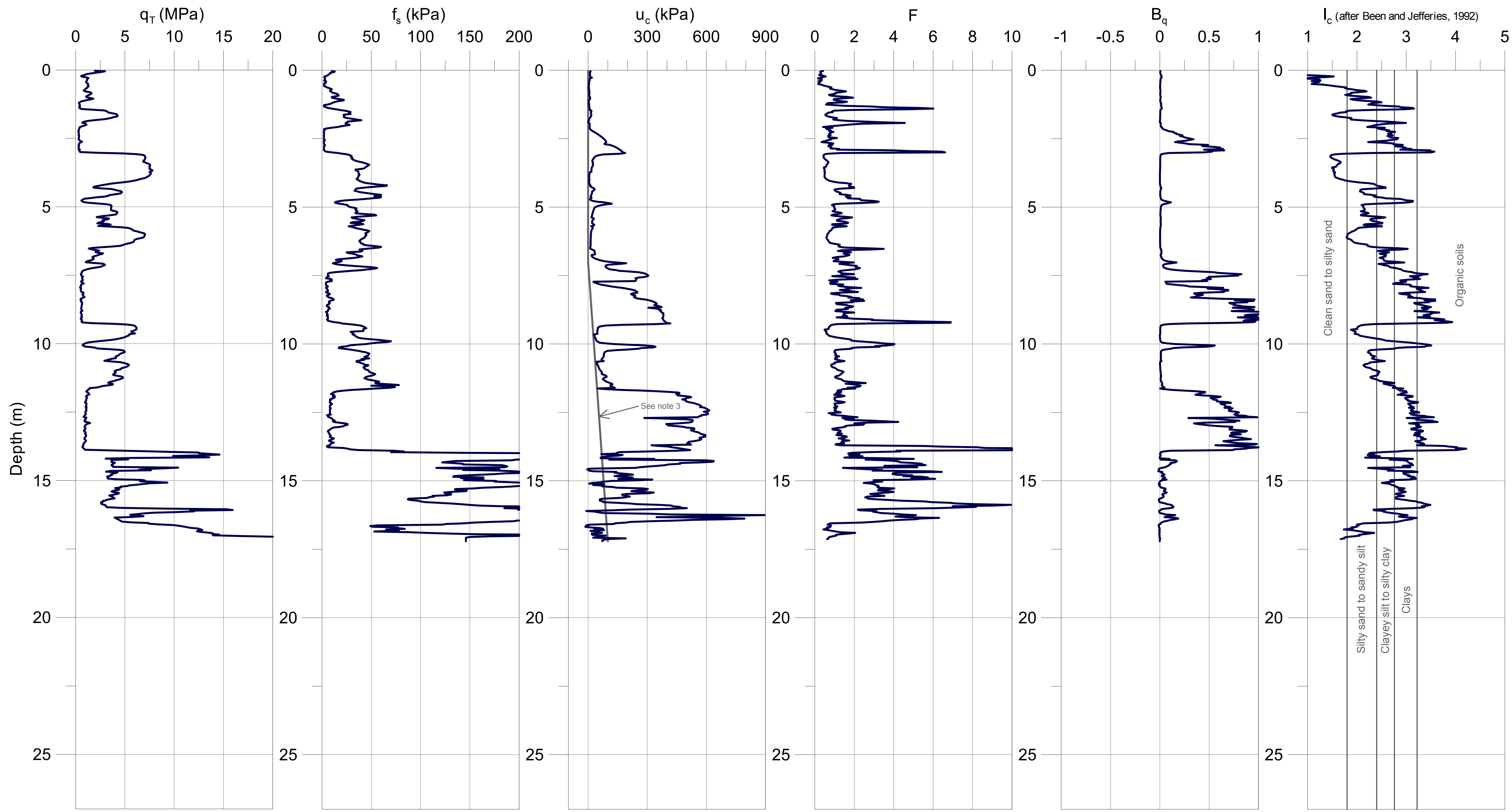
TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-16
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	
CHECK	--	--	**CPT03-16**
REVIEW	--	--	

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-16.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
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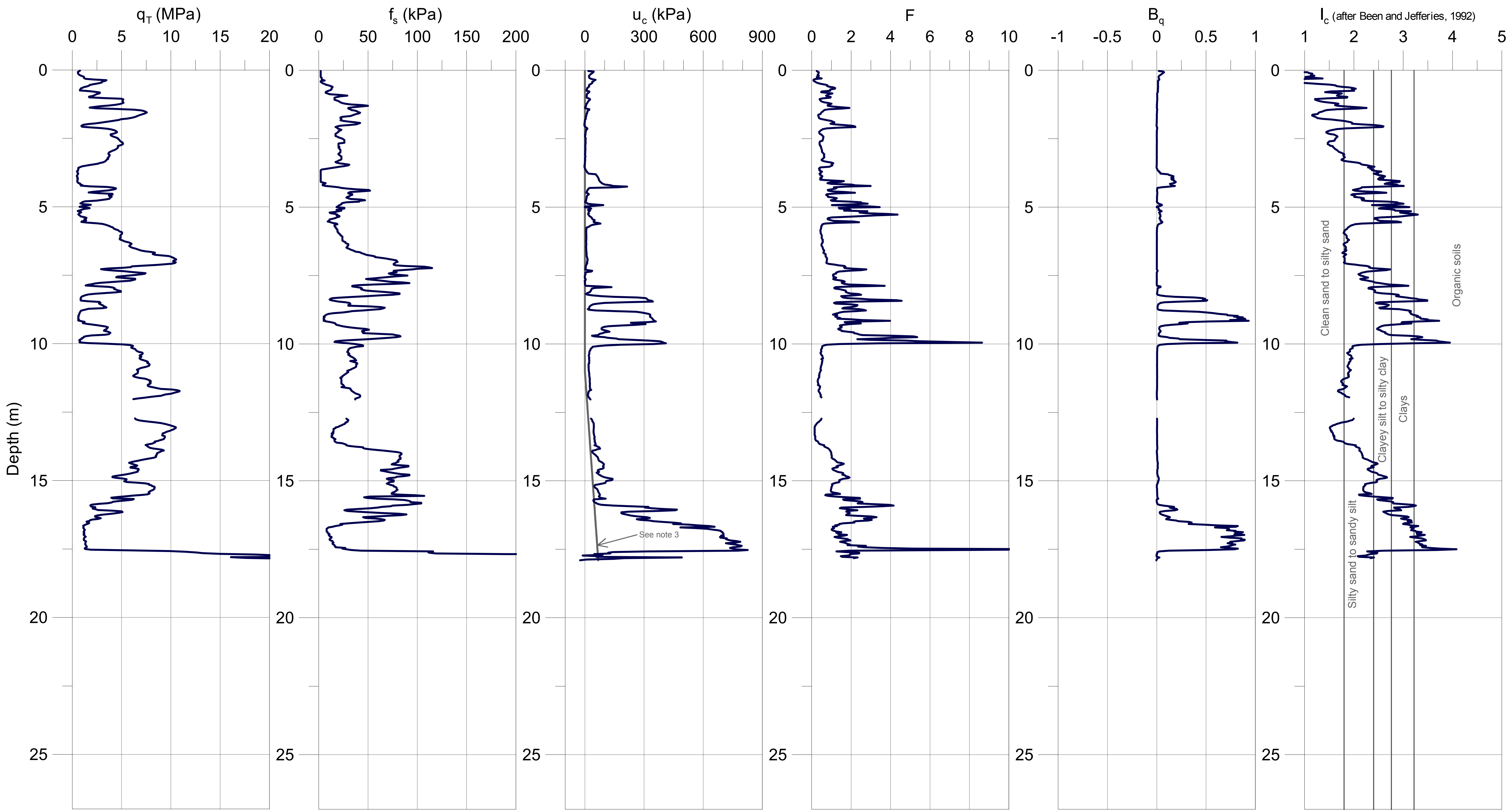
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots-SCPT03-17
DESIGN DRK	21 JAN 04	SCALE NTS REV.
CADD DRK	21 JAN 04	
CHECK --	--	**SCPT03-17**
REVIEW --	--	

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-SCPT03-17.grf



Notes

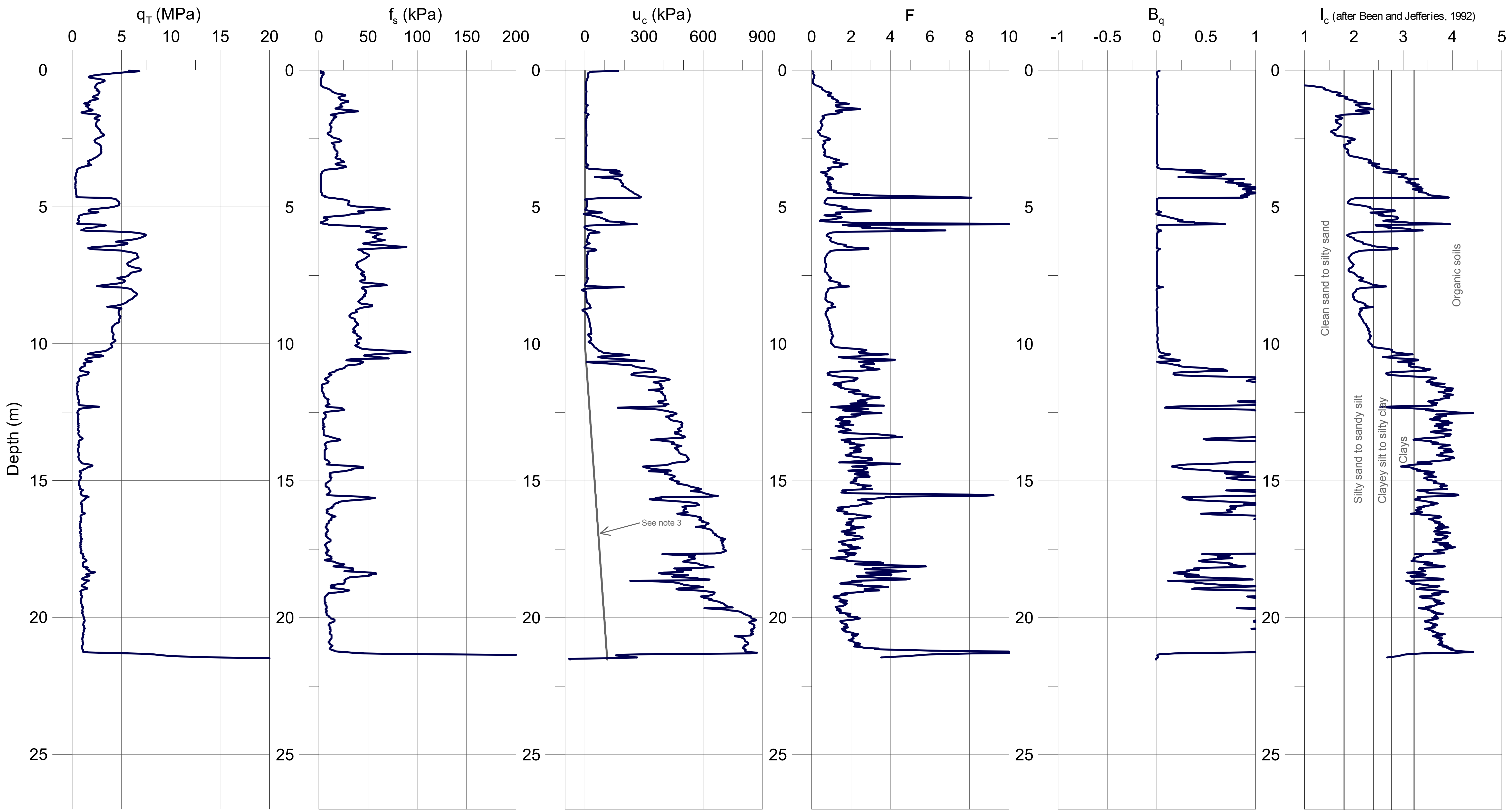
1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	**RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES**

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-18	
DESIGN	DRK	21 JAN 04	SCALE NTS	REV.
CADD	DRK	21 JAN 04	**CPT03-18**	
CHECK	--	--		
REVIEW	--	--		

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-18.grf



REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-19.grf

Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-19	
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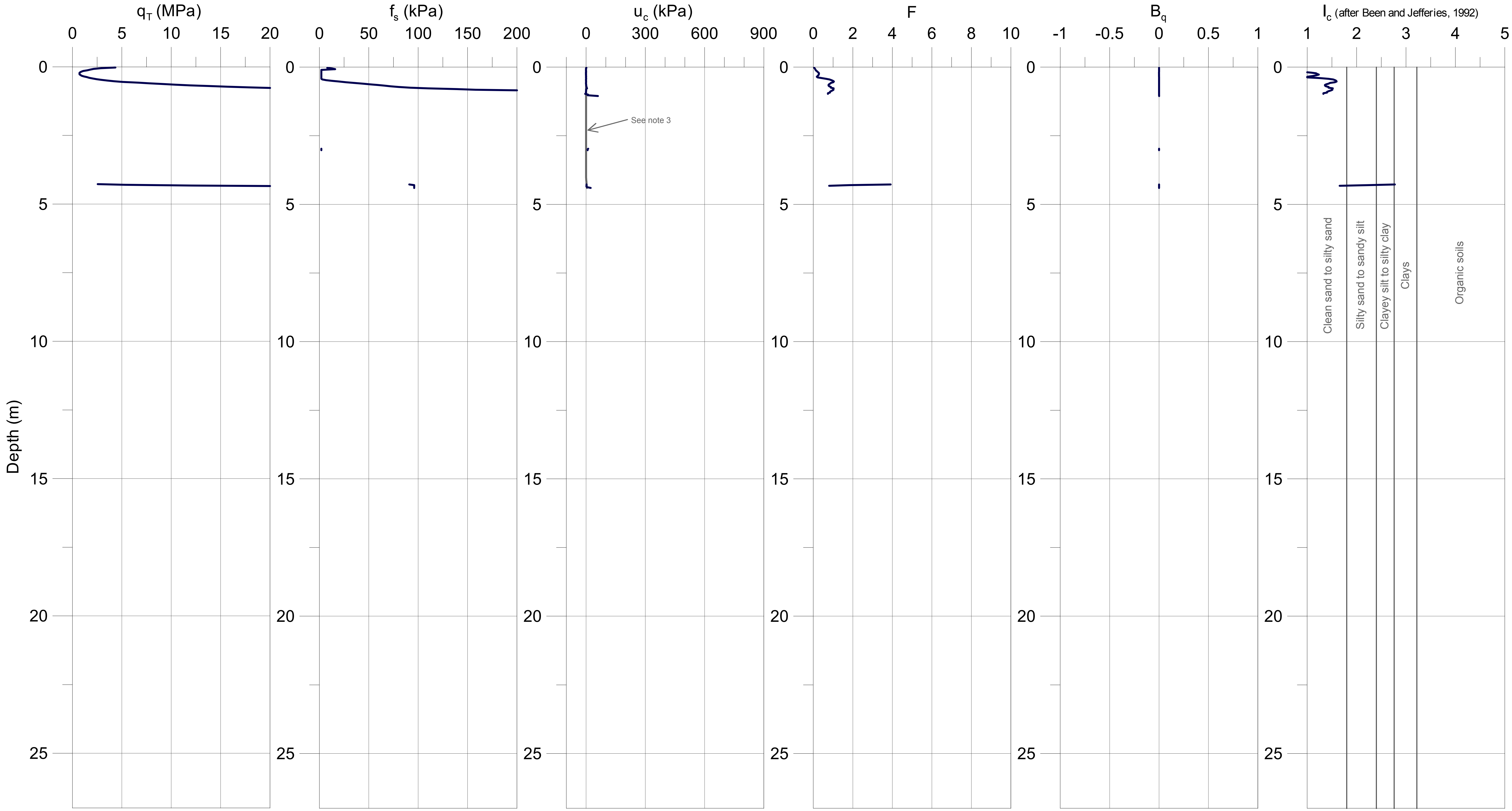
CADD	DRK	21 JAN 04	**CPT03-19**	
CHECK	--	--		
REVIEW	--	--		



REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-20.grf

Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

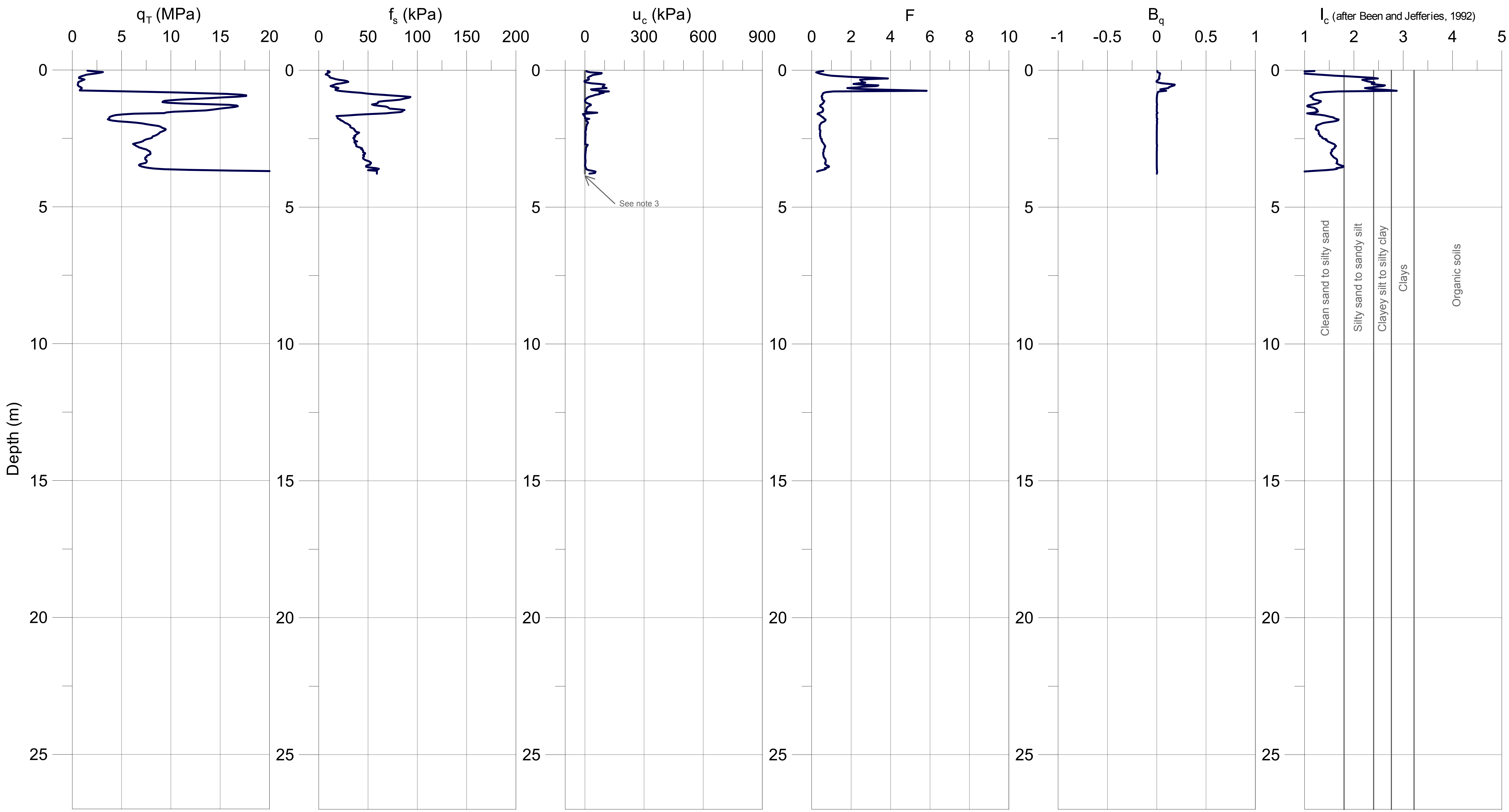
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-20
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	
CHECK	--	--	**CPT03-20**
REVIEW	--	--	

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-20X.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

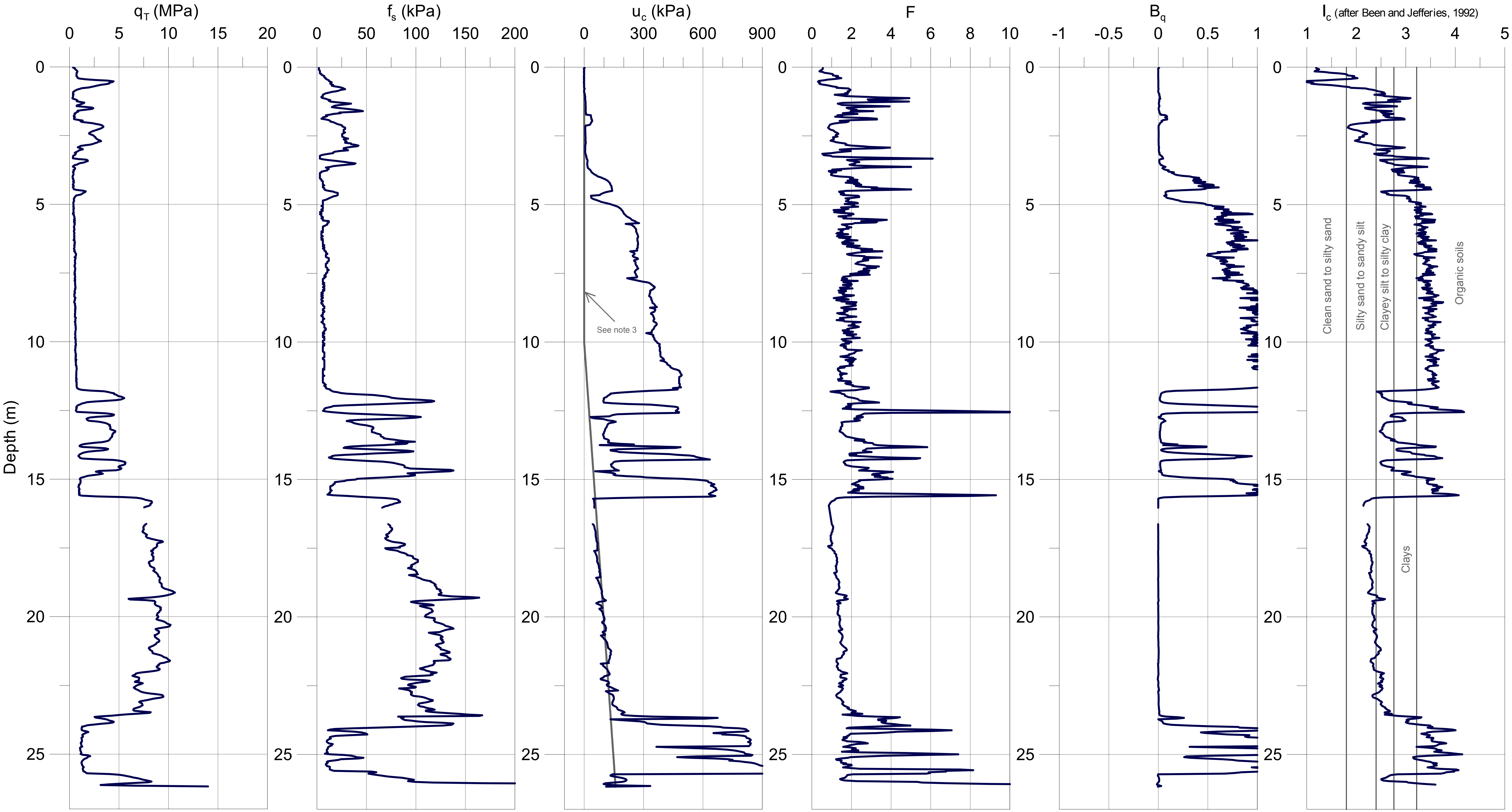
TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080	
DESIGN	DRK	21 JAN 04
CADD	DRK	21 JAN 04
CHECK	--	--
REVIEW	--	--

FILE No. plots-CPT03-20X
SCALE NTS REV.

CPT03-20X



REVISION DATE: 20 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-SCPT03-21.grf

Notes

1. I_C values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-SCPT03-21	
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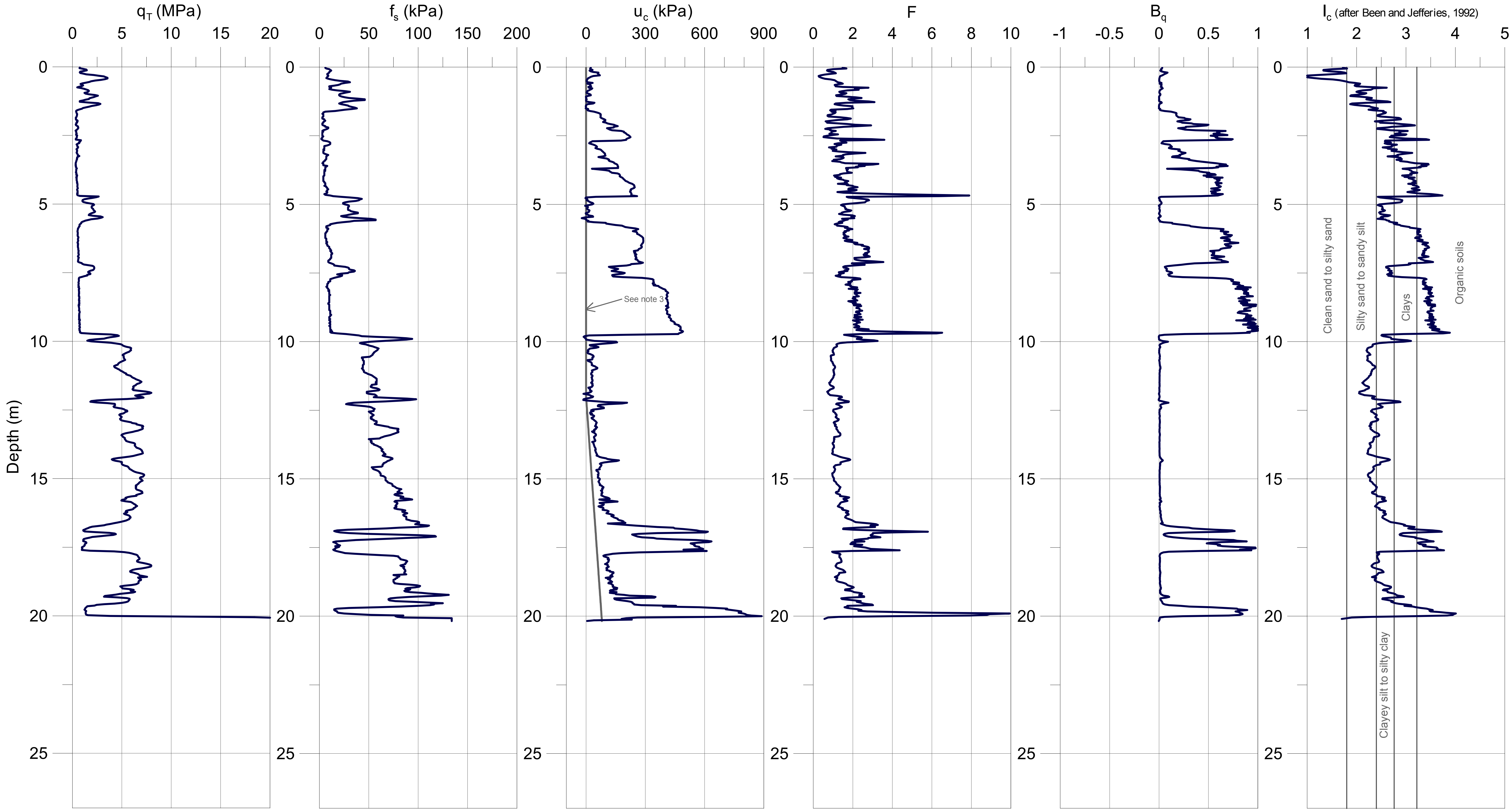
CADD	DRK	14 JAN 04	**SCPT03-21**	
CHECK	--	--		
REVIEW	--	--		



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

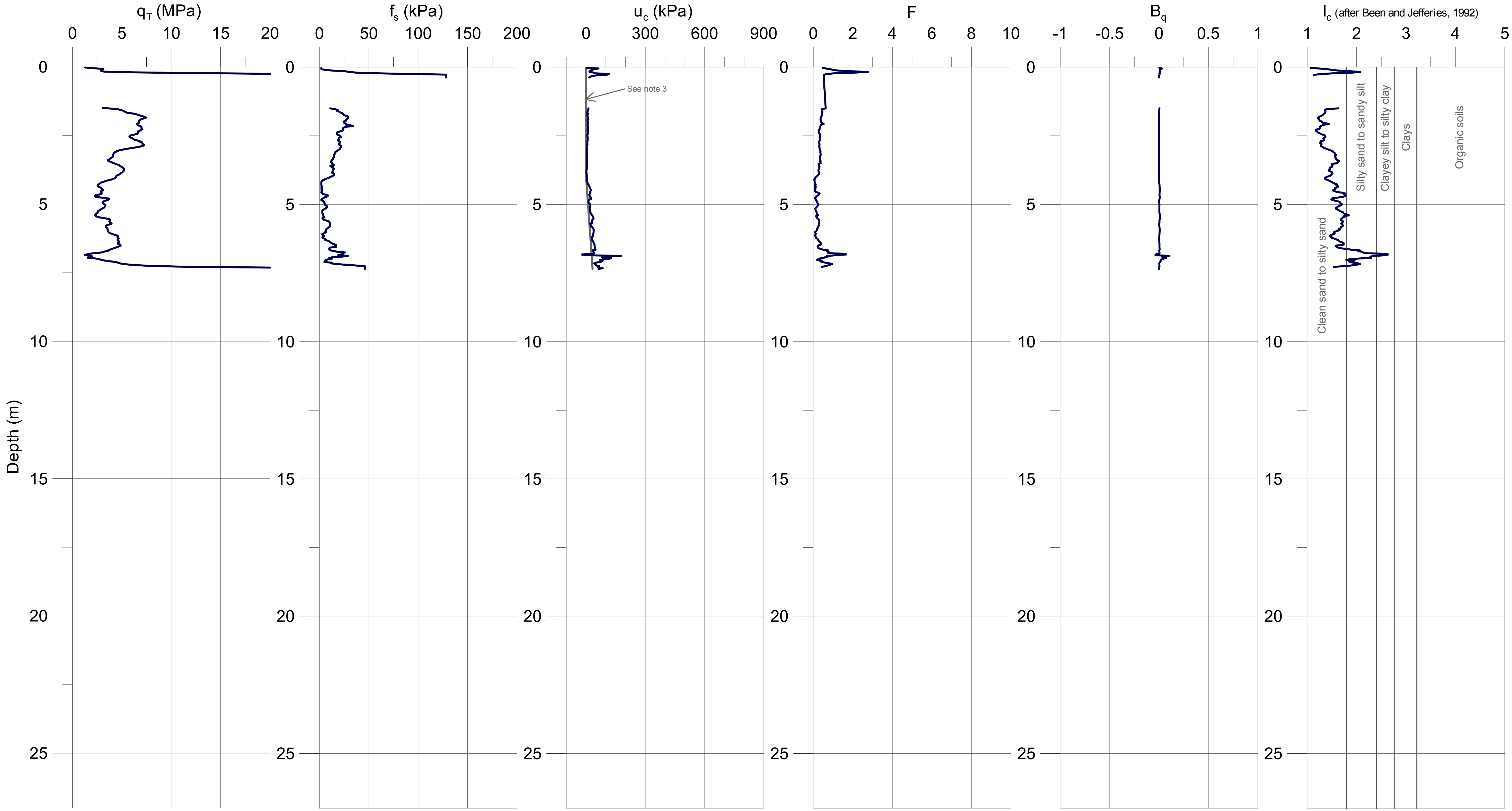
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-22
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-22**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-22.grf



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Notes

1. I_C values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

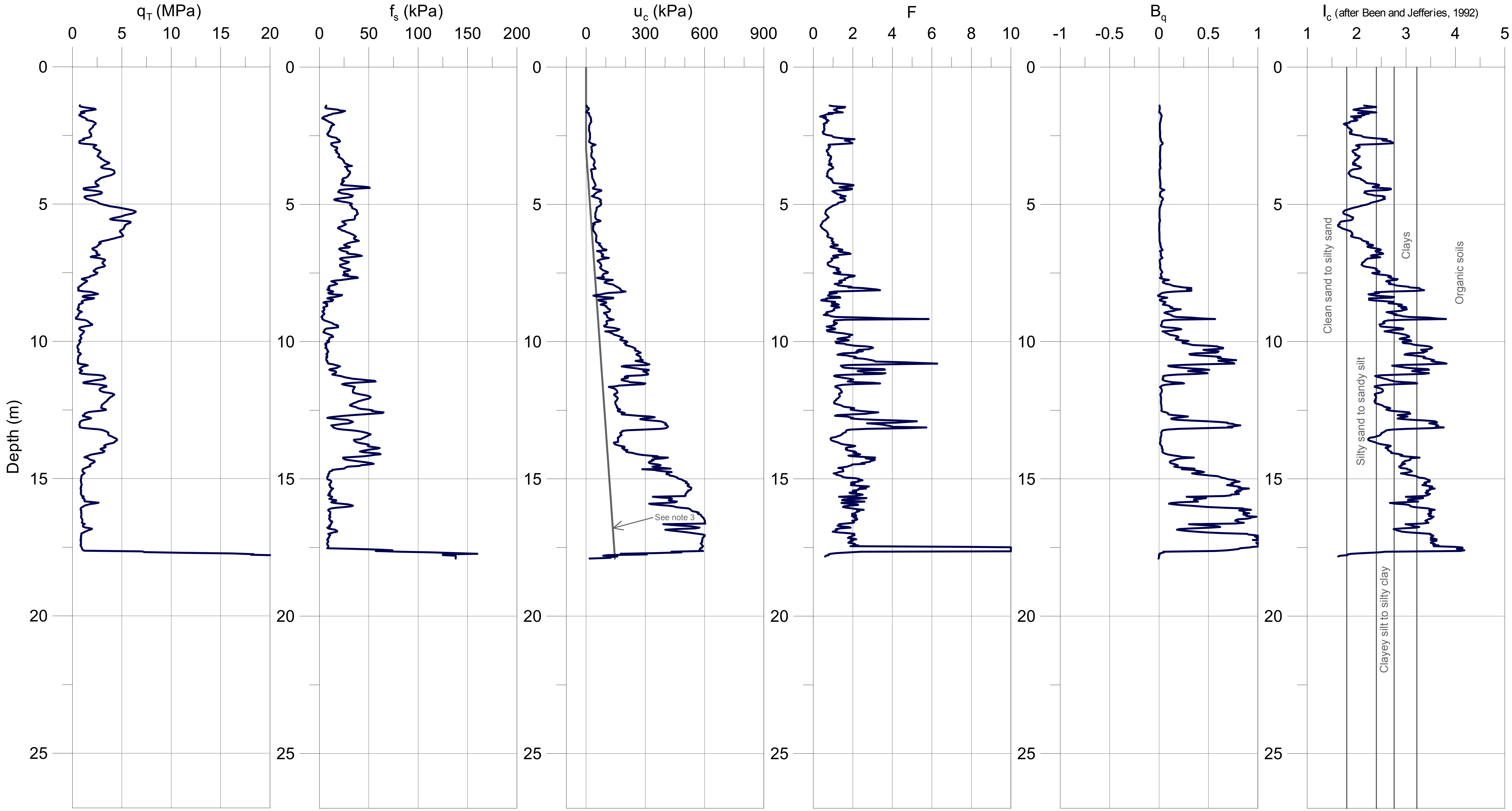
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-23
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	
CHECK	--	--	**CPT03-23**
REVIEW	--	--	



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

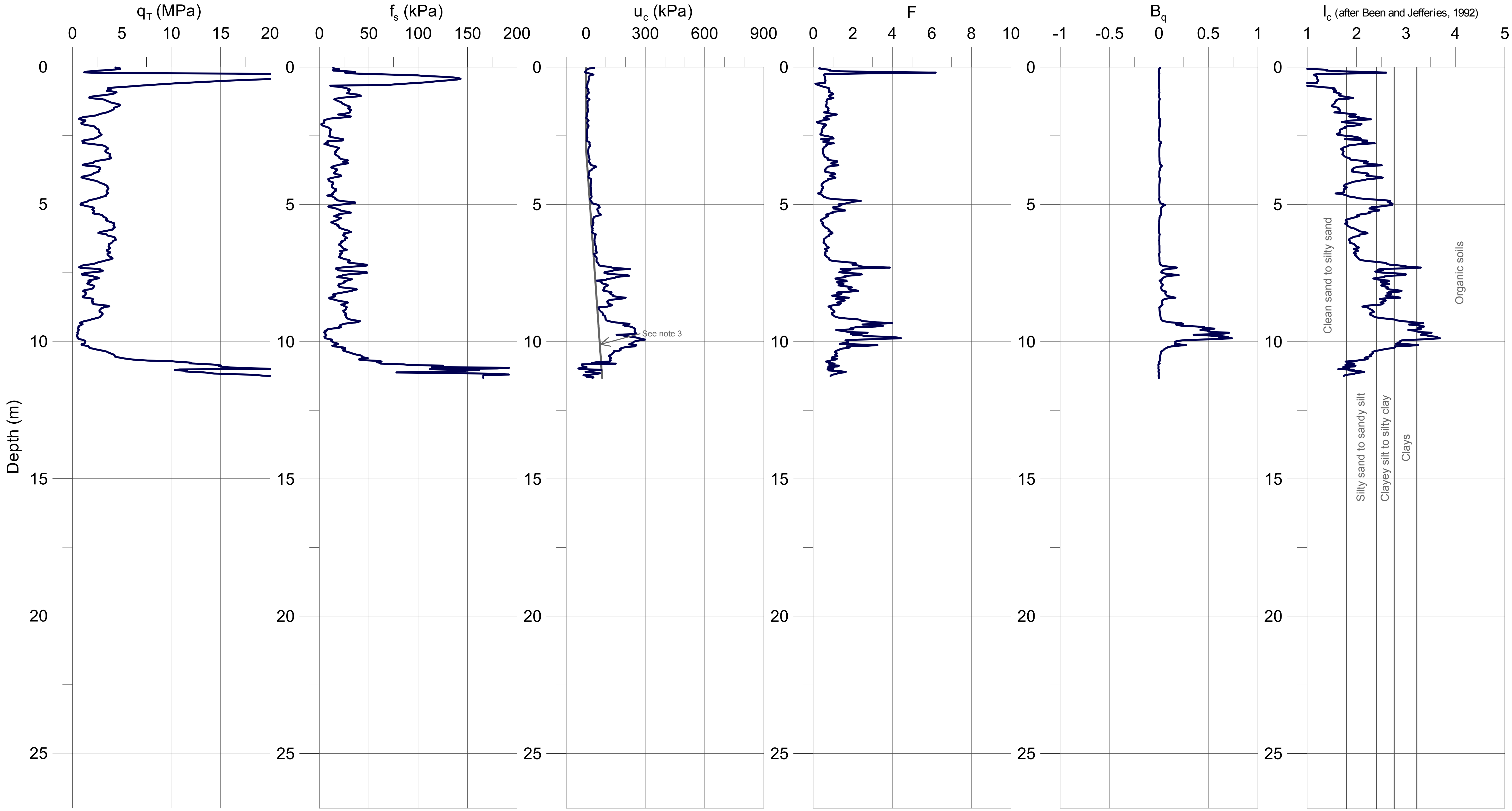
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-24
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-24**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-24.grf



REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-SCPT03-25.grf

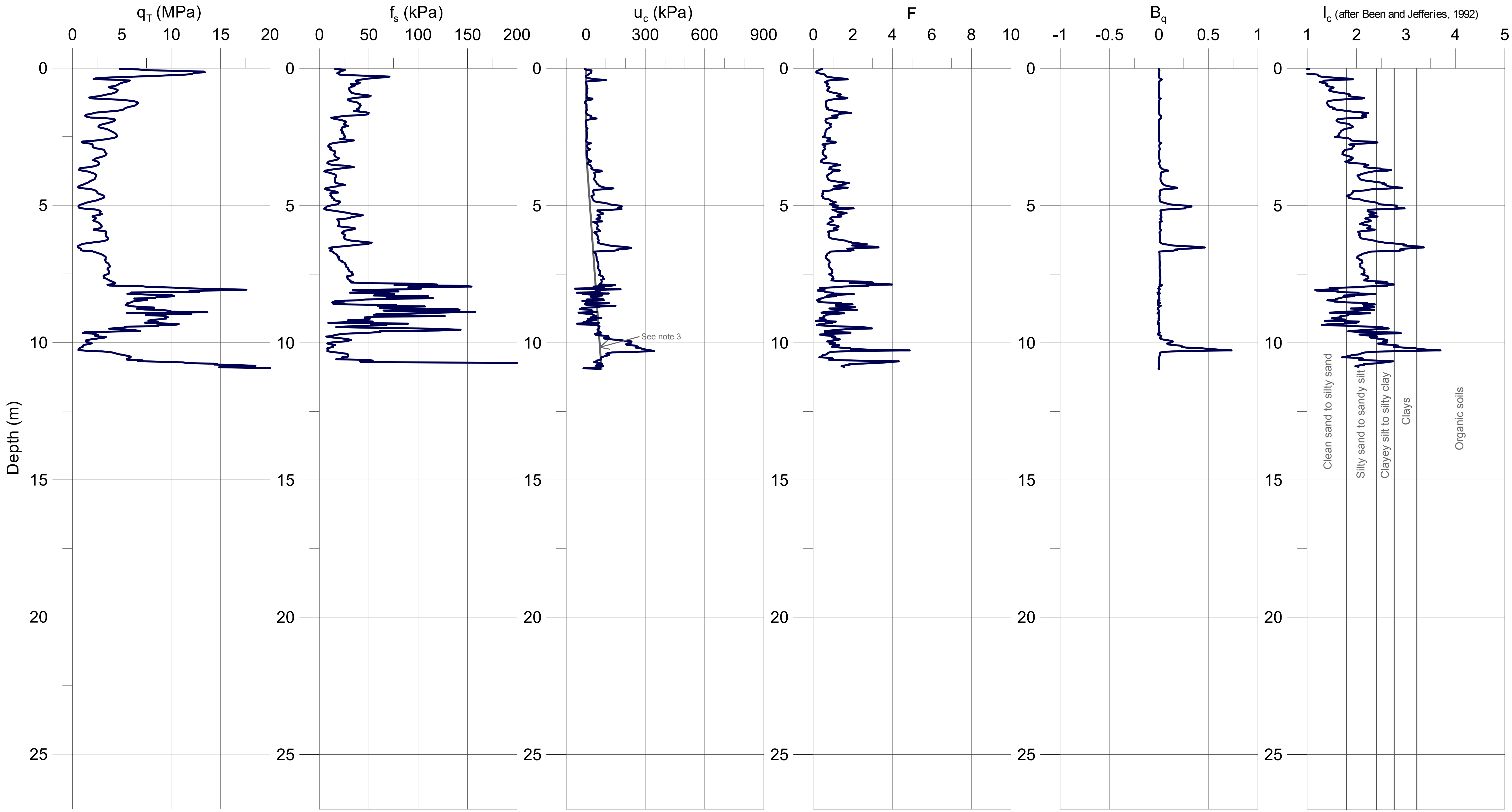
Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-SCPT03-25	
DESIGN	DRK	21 JAN 04	SCALE NTS	REV.
CADD	DRK	21 JAN 04	SCPT03-25	
CHECK	--	--		
REVIEW	--	--		



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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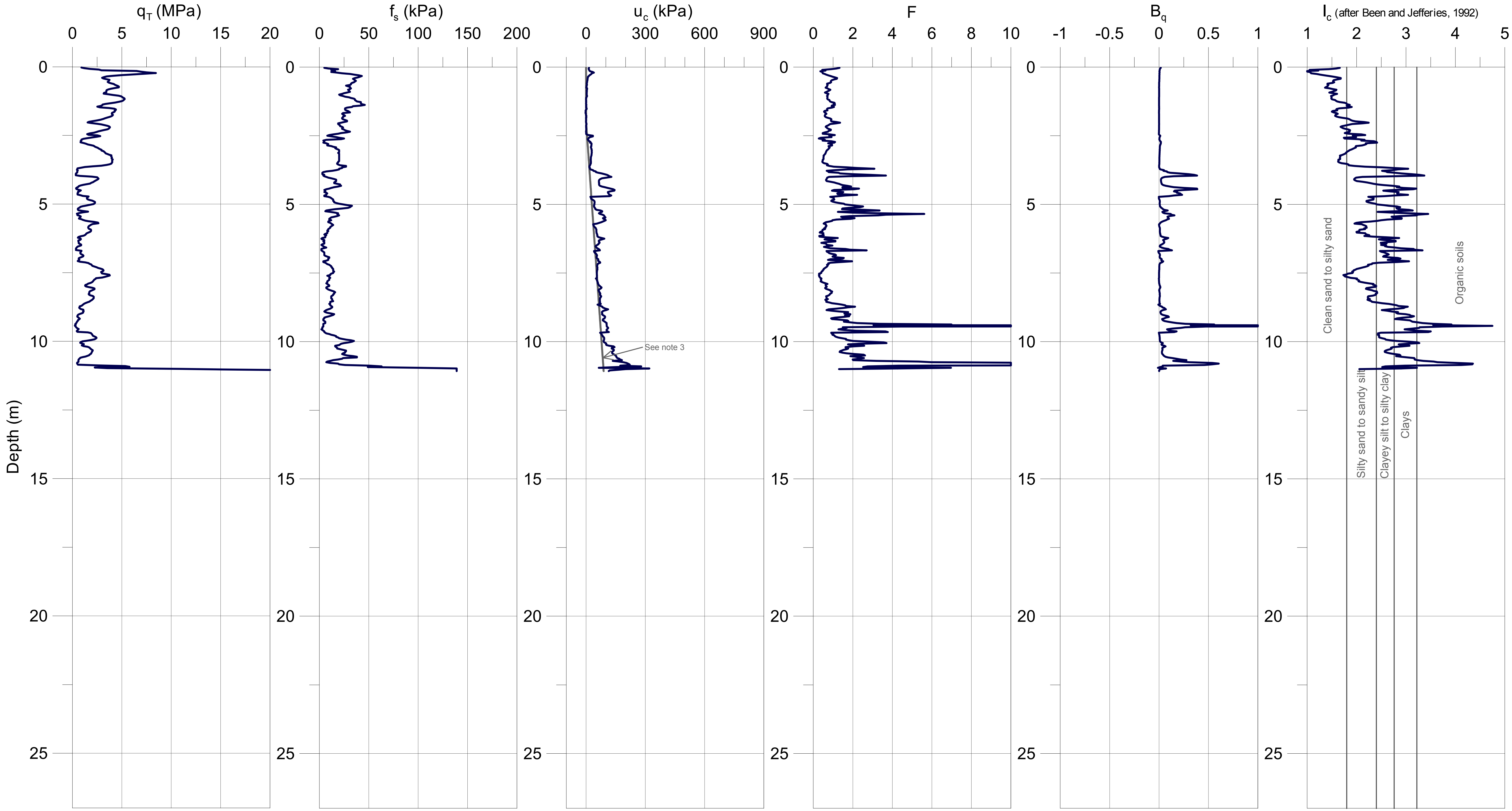
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-26
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	
CHECK	--	--	**CPT03-26**
REVIEW	--	--	

REVISION DATE: 21 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-26.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-27.grf

PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-27
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CADD	DRK	21 JAN 04	CPT03-27
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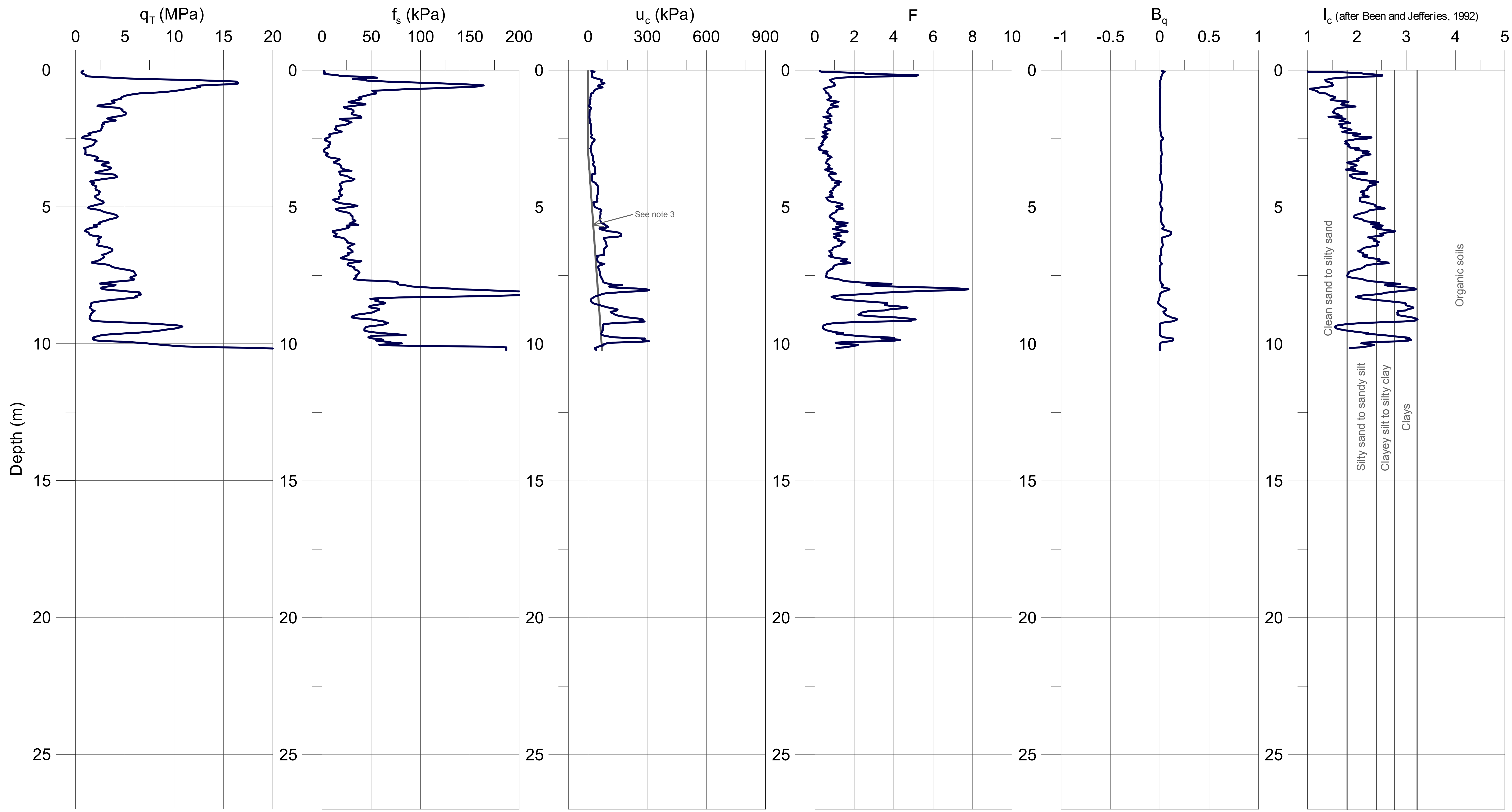
REVIEW	--	--	



REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-28.grf

Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-28
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-28**
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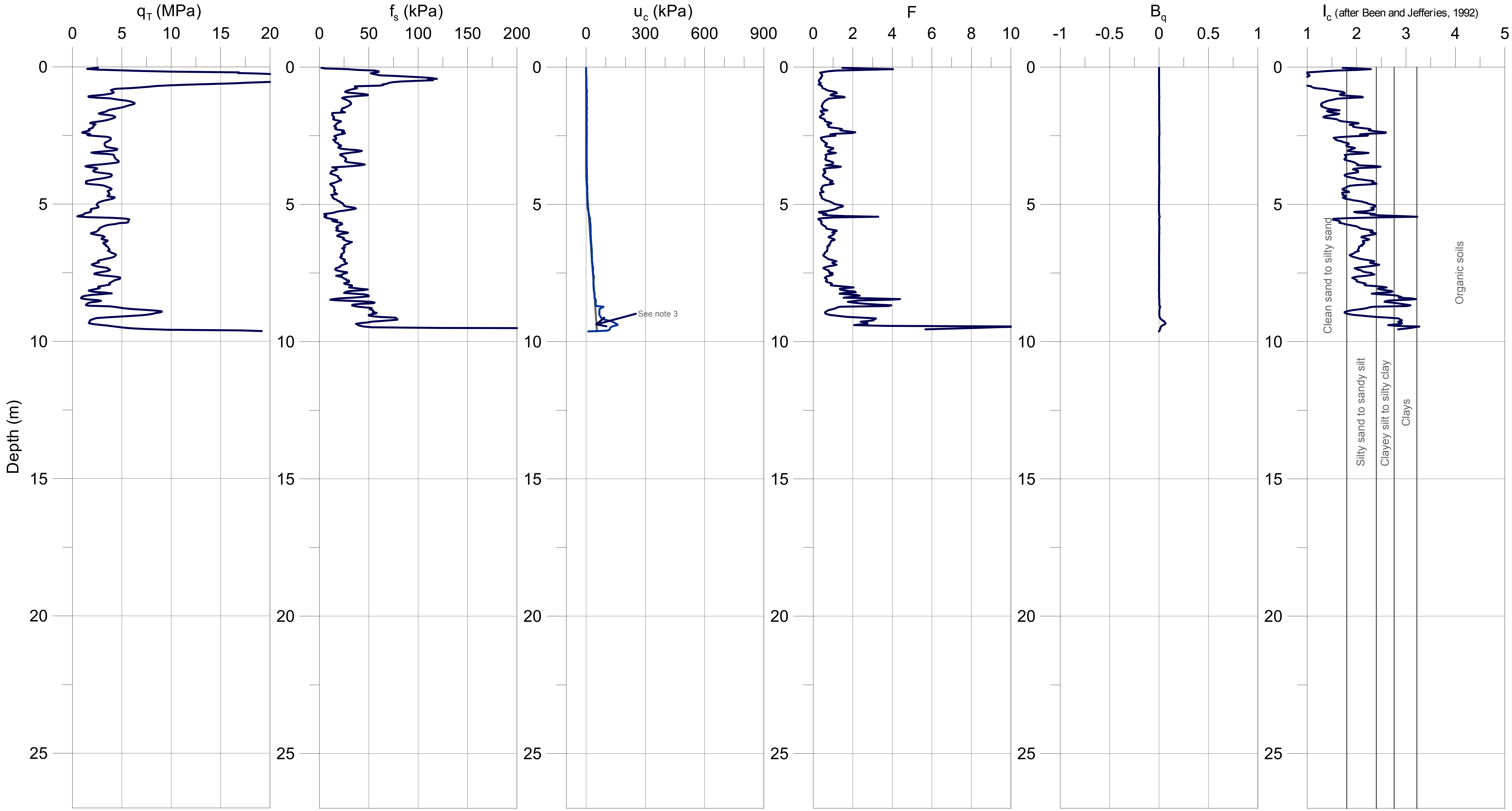
REVIEW	--	--	



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

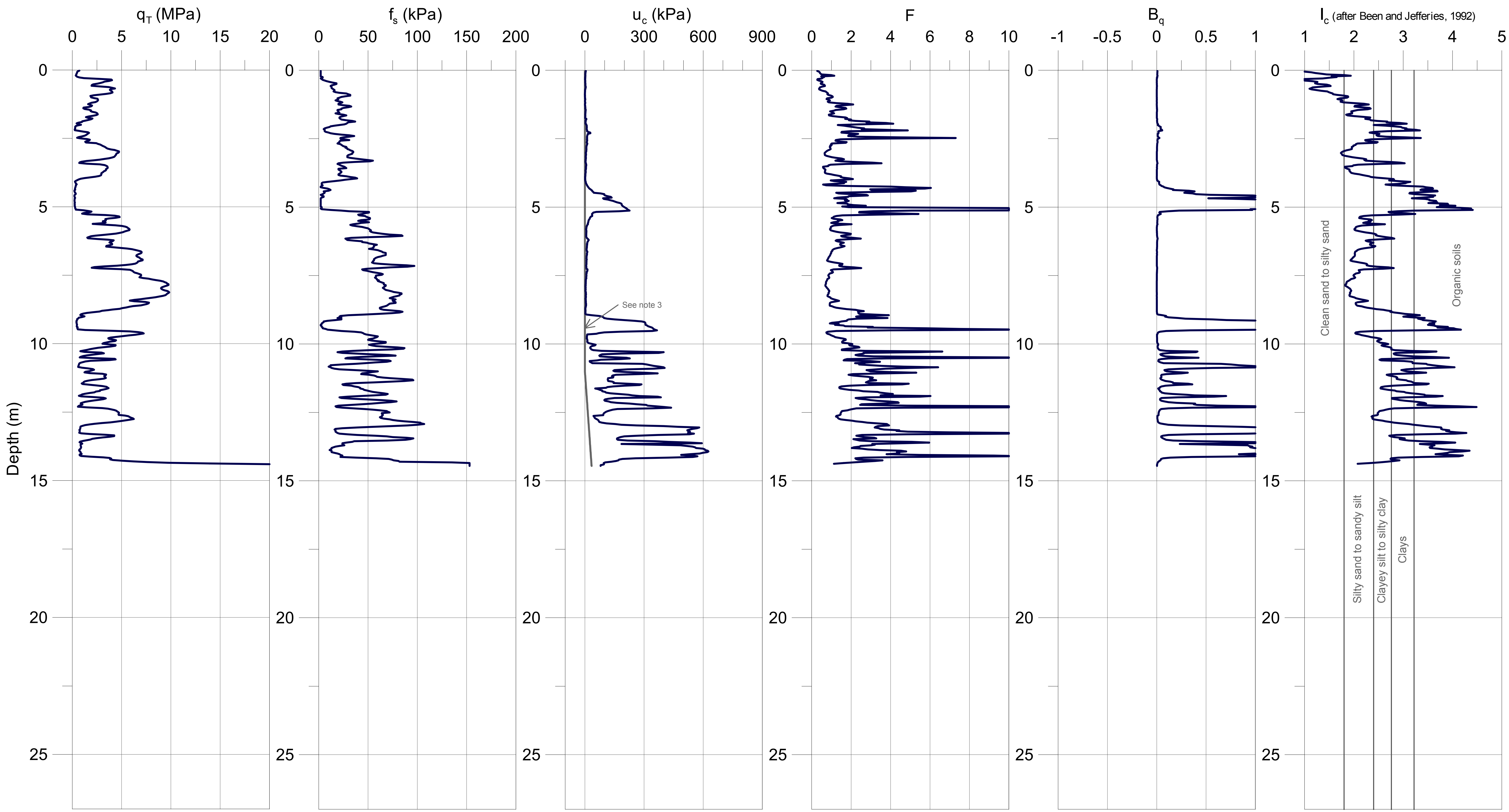
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.		03-1413-080	FILE No. plots-CPT03-29
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-29**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-29.grf



REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-30.grf

Notes

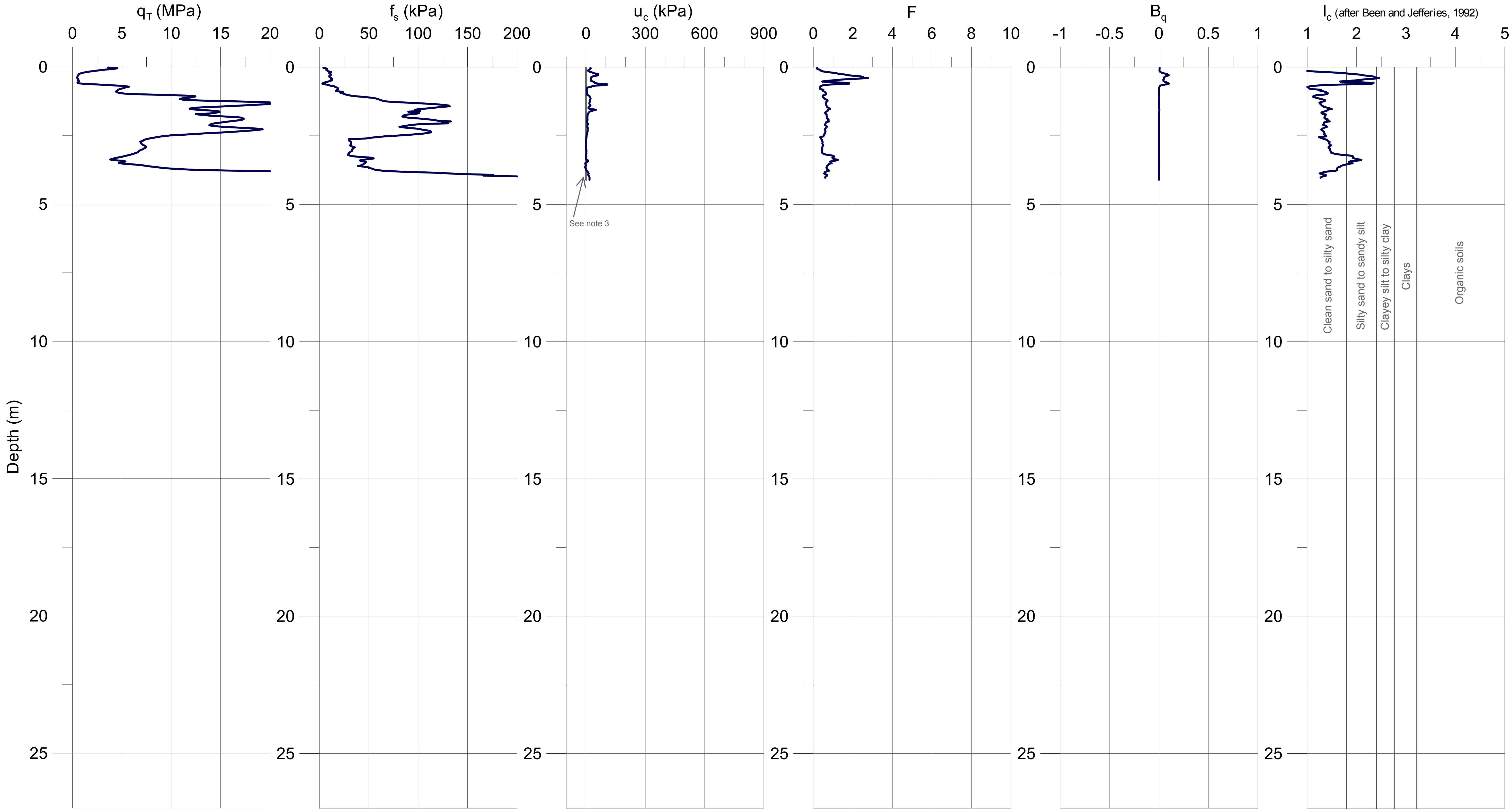
1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-30
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-30**
CHECK	--	--	
REVIEW	--	--	



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

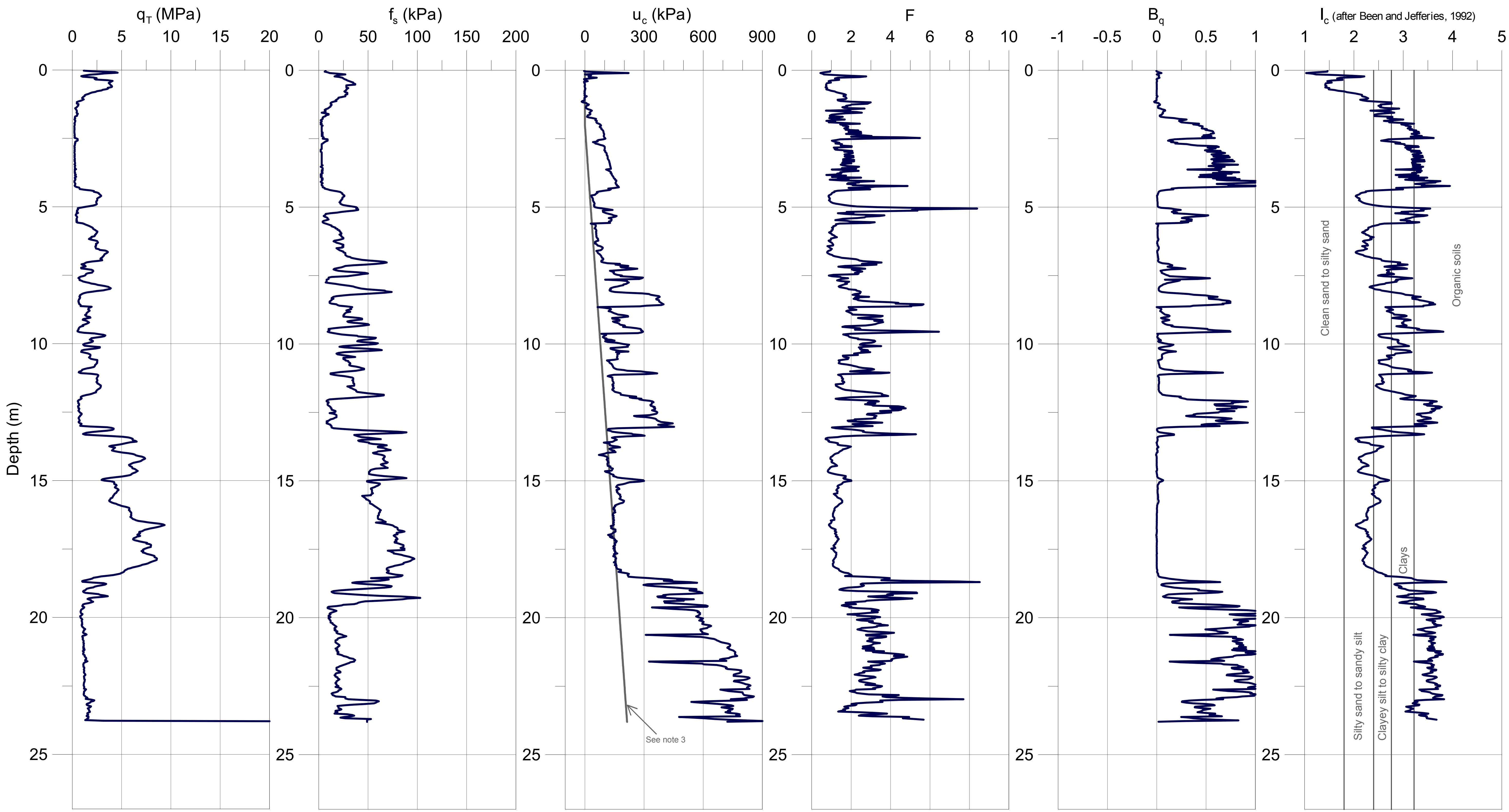
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-CPT03-31
DESIGN	DRK	21 JAN 04	SCALE NTS REV.
CADD	DRK	21 JAN 04	**CPT03-31**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-31.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

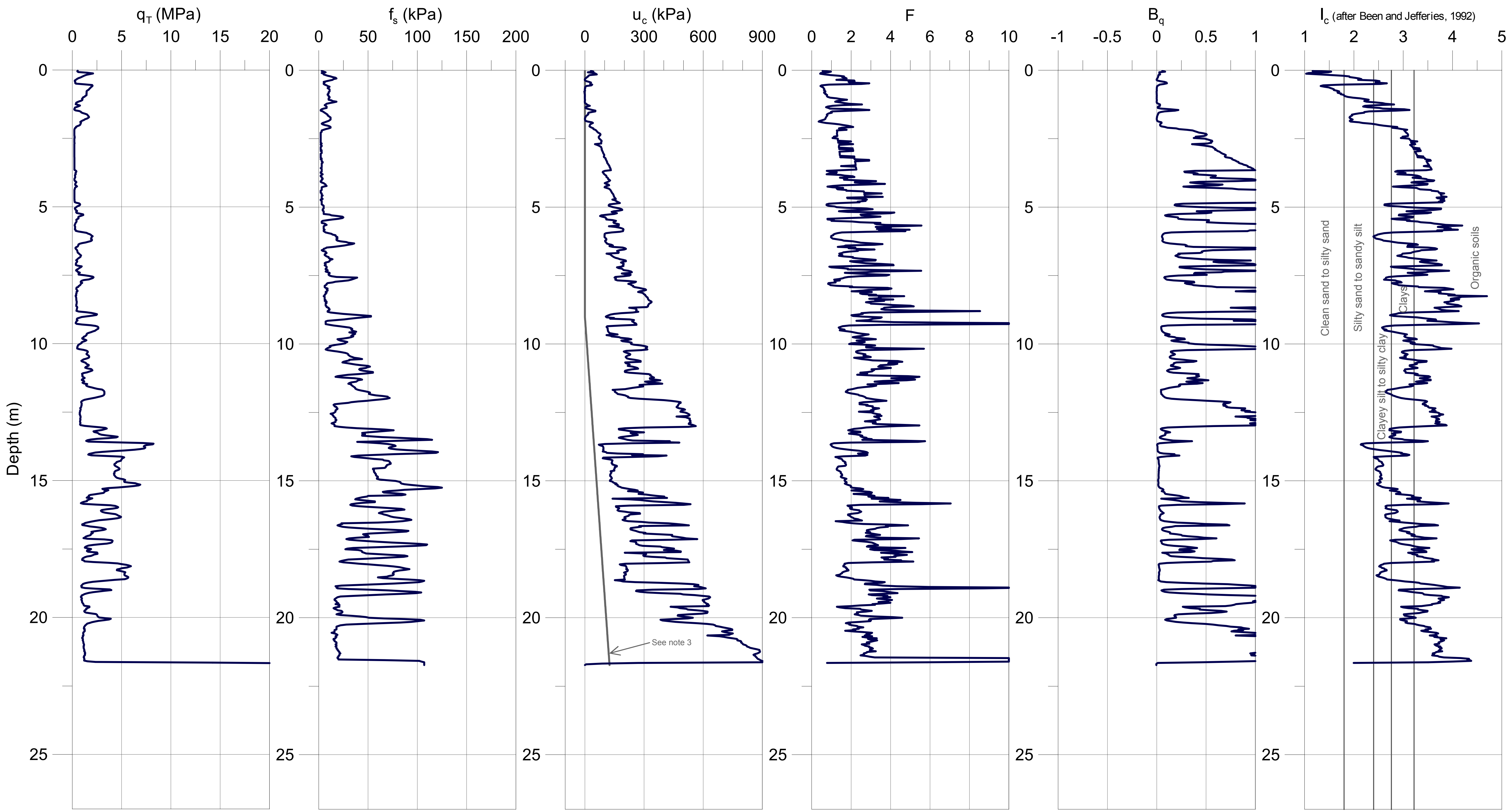
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots-SCPT03-32
DESIGN	DRK	20 JAN 04	SCALE NTS REV.
CADD	DRK	20 JAN 04	**SCPT03-32**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 20 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-SCPT03-32.grf



Notes

1. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
2. Density varies with depth using assumed C_c = 0.1, and G_s = 3.6
3. Hydrostatic ground water profile estimated as shown

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
RECORD OF CPT SOUNDINGS
WITH F, Bq, AND, Ic PROFILES

Golder Associates

PROJECT No.	03-1413-080	
DESIGN	DRK	21 JAN 04
CADD	DRK	21 JAN 04
CHECK	--	--
REVIEW	--	--

FILE No. plots-CPT03-33
SCALE NTS REV.

CPT03-33

REVISION DATE: 22 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots-CPT03-33.grf

APPENDIX IV

SHEAR WAVE VELOCITY LOGS

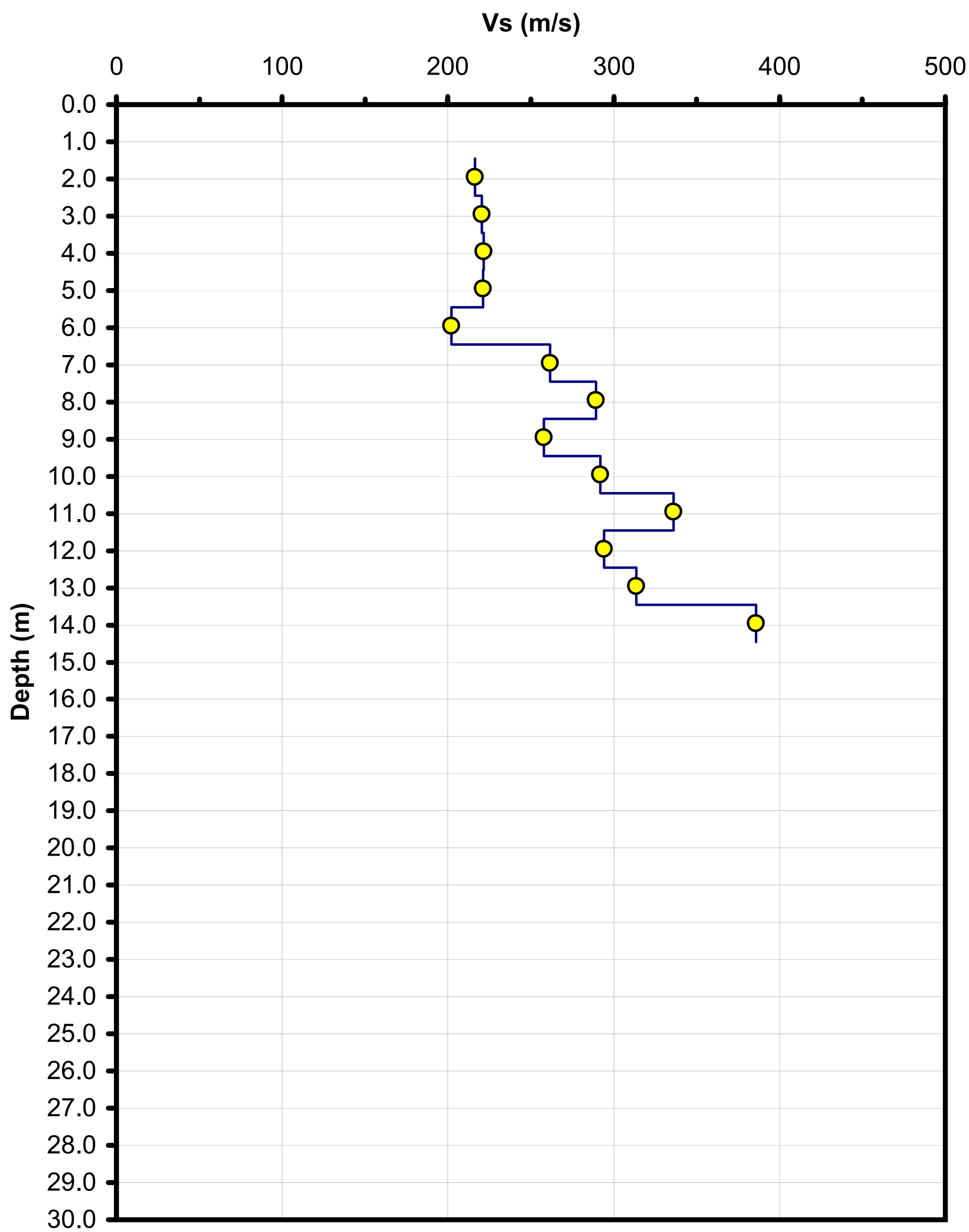
Job No.: 03-210
Client: Golder Associates Ltd.
Project: Faro Tailings Facility, Yukon
Sounding: SCPT-05
Date: October 15th, 2003

Seismic Source: Beam
Source Offset (m): 0.60
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SEISMIC

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
1.65	1.45	1.57				
2.65	2.45	2.52	0.95	4.41	216	1.95
3.65	3.45	3.50	0.98	4.44	221	2.95
4.65	4.45	4.49	0.99	4.47	221	3.95
5.65	5.45	5.48	0.99	4.49	221	4.95
6.65	6.45	6.48	0.99	4.92	202	5.95
7.65	7.45	7.47	1.00	3.81	261	6.95
8.65	8.45	8.47	1.00	3.45	289	7.95
9.65	9.45	9.47	1.00	3.87	258	8.95
10.65	10.45	10.47	1.00	3.42	292	9.95
11.65	11.45	11.47	1.00	2.97	336	10.95
12.65	12.45	12.46	1.00	3.40	294	11.95
13.65	13.45	13.46	1.00	3.19	313	12.95
14.65	14.45	14.46	1.00	2.59	386	13.95

Job No: 03-210
Client: Golder Associates Ltd.
Location: Faro Tailings Facility, Yukon
Sounding: SCPT-05
Sounding Date: October 15th, 2003



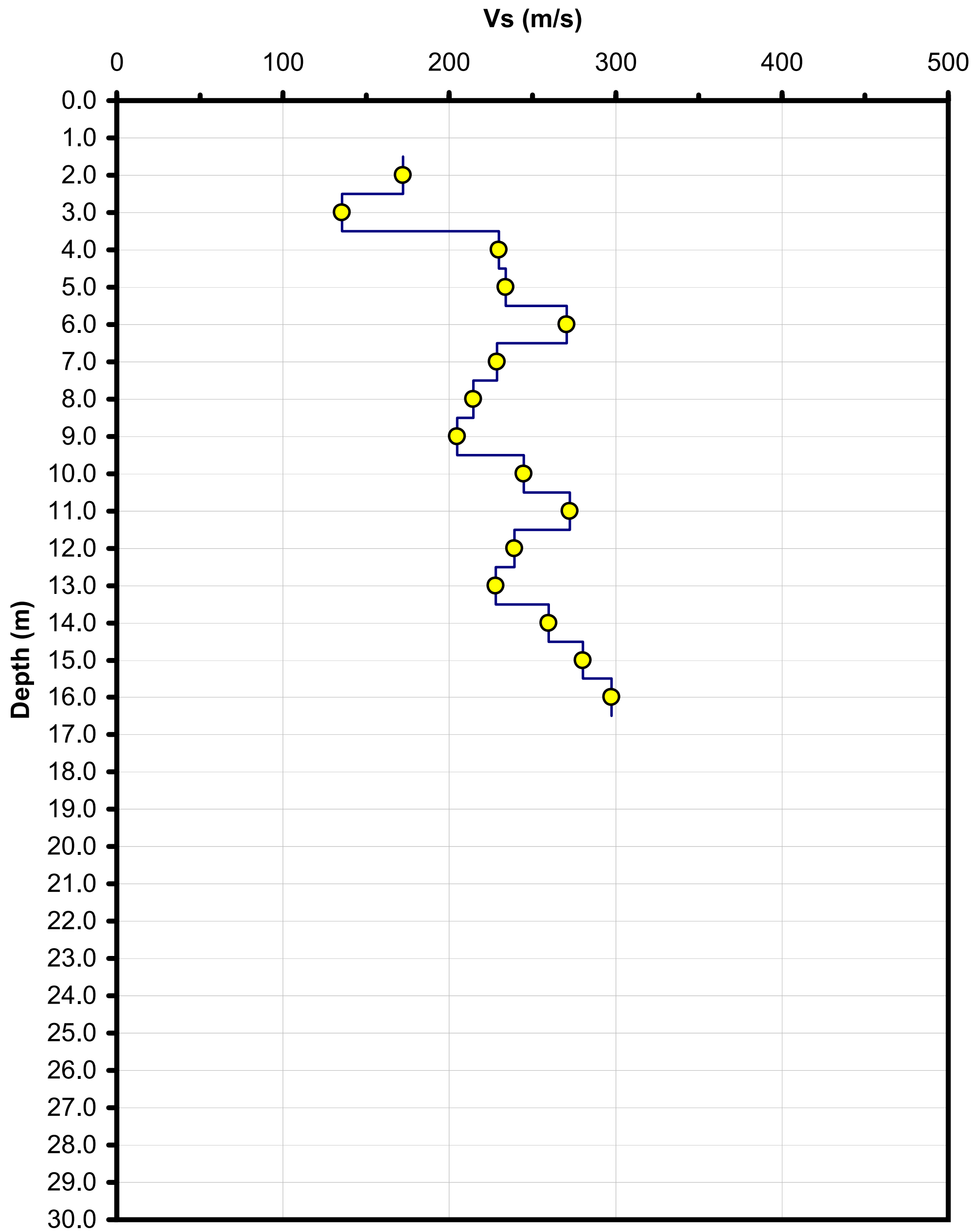
Job No.: 03-210
Client: Golder Associates Ltd.
Project: Faro Tailings Facility, Yukon
Sounding: SCPT-17
Date: October 16th, 2003

Seismic Source: Beam
Source Offset (m): 0.60
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SEISMIC

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
1.70	1.50	1.62				
2.70	2.50	2.57	0.96	5.55	172	2.00
3.70	3.50	3.55	0.98	7.24	135	3.00
4.70	4.50	4.54	0.99	4.30	230	4.00
5.70	5.50	5.53	0.99	4.25	234	5.00
6.70	6.50	6.53	1.00	3.68	271	6.00
7.70	7.50	7.52	1.00	4.36	229	7.00
8.70	8.50	8.52	1.00	4.65	214	8.00
9.70	9.50	9.52	1.00	4.88	205	9.00
10.70	10.50	10.52	1.00	4.08	245	10.00
11.70	11.50	11.52	1.00	3.67	272	11.00
12.70	12.50	12.51	1.00	4.18	239	12.00
13.70	13.50	13.51	1.00	4.39	228	13.00
14.70	14.50	14.51	1.00	3.85	260	14.00
15.70	15.50	15.51	1.00	3.57	280	15.00
16.70	16.50	16.51	1.00	3.36	298	16.00

Job No: 03-210
Client: Golder Associates Ltd.
Location: Faro Tailings Facility, Yukon
Sounding: SCPT-17
Sounding Date: October 16th, 2003



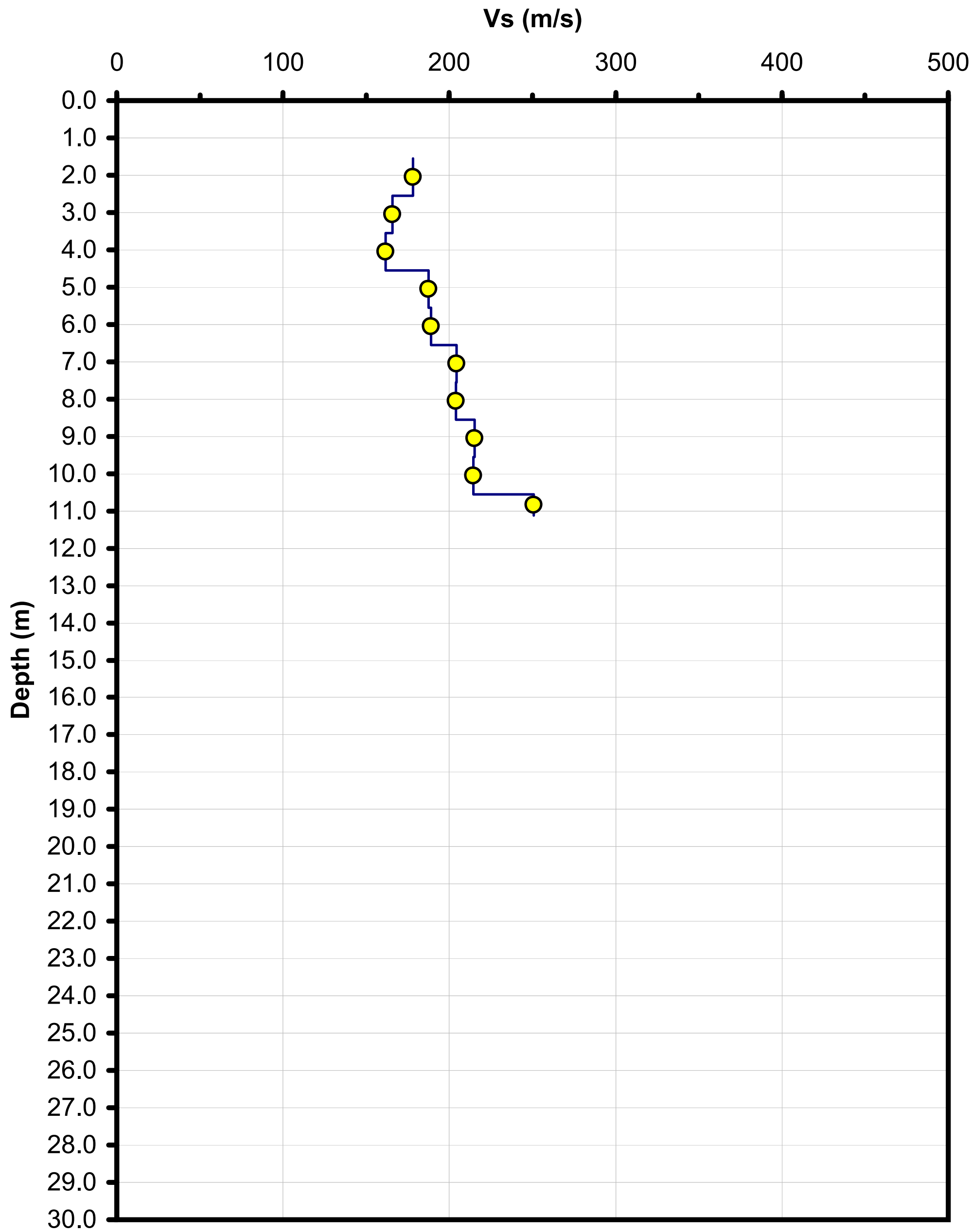
Job No.:	03-210
Client:	Golder Associates Ltd.
Project:	Faro Tailings Facility, Yukon
Sounding:	SCPT-25
Date:	October 17th, 2003

Seismic Source:	Beam
Source Offset (m):	0.60
Source Depth (m):	0.00
Geophone Offset (m):	0.20

SEISMIC

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
1.75	1.55	1.66				
2.75	2.55	2.62	0.96	5.37	178	2.05
3.75	3.55	3.60	0.98	5.92	166	3.05
4.75	4.55	4.59	0.99	6.12	162	4.05
5.75	5.55	5.58	0.99	5.30	187	5.05
6.75	6.55	6.58	1.00	5.26	189	6.05
7.75	7.55	7.57	1.00	4.88	204	7.05
8.75	8.55	8.57	1.00	4.89	204	8.05
9.75	9.55	9.57	1.00	4.64	215	9.05
10.75	10.55	10.57	1.00	4.66	214	10.05
11.32	11.12	11.14	0.57	2.27	251	10.84

Job No: 03-210
Client: Golder Associates Ltd.
Location: Faro Tailings Facility, Yukon
Sounding: SCPT-25
Sounding Date: October 17th, 2003



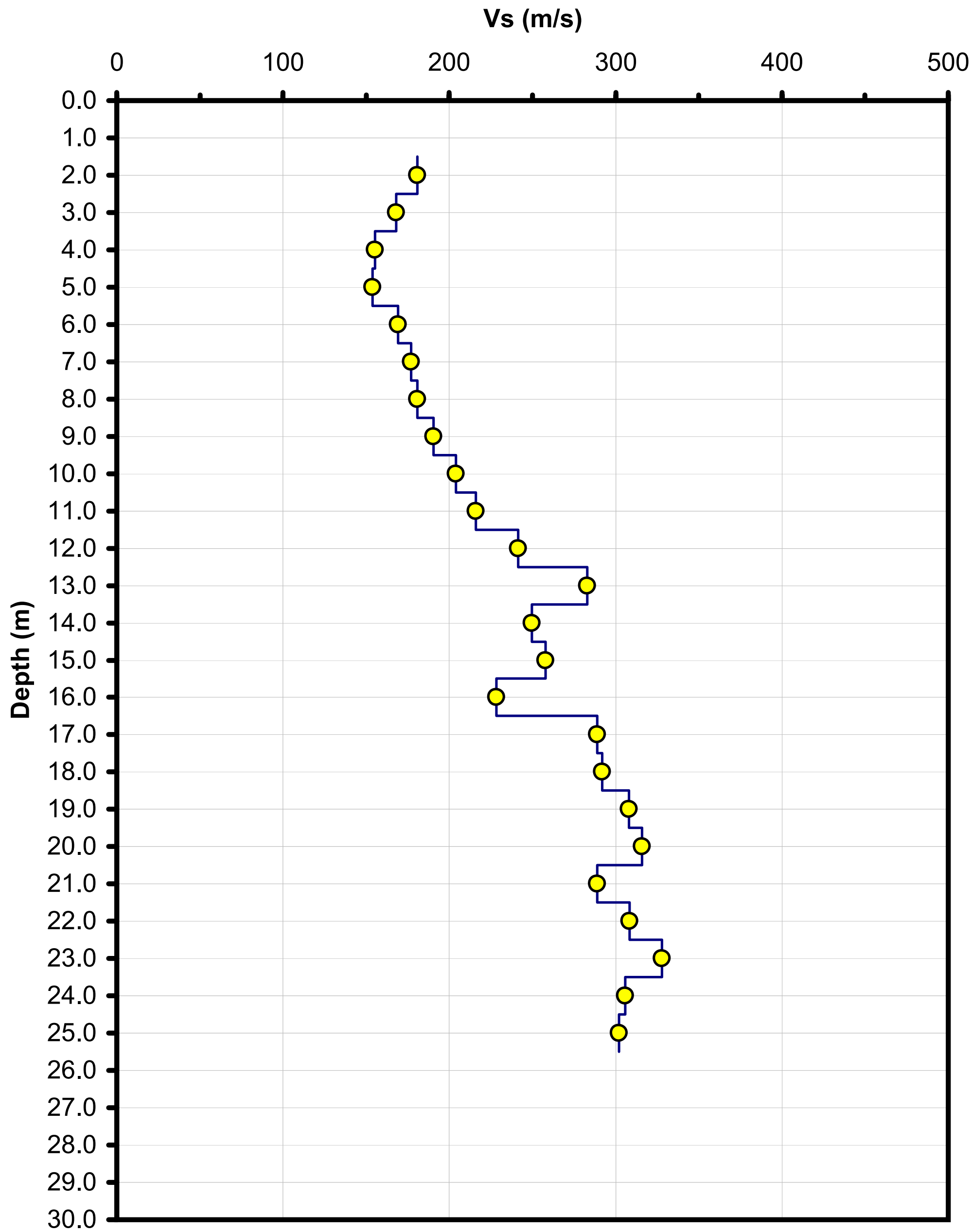
Job No.: 03-210
Client: Golder Associates Ltd.
Project: Faro Tailings Facility, Yukon
Sounding: SCPT-21
Date: October 18th, 2003

Seismic Source: Beam
Source Offset (m): 0.60
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SEISMIC

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
1.70	1.50	1.62				
2.70	2.50	2.57	0.96	5.29	181	2.00
3.70	3.50	3.55	0.98	5.83	168	3.00
4.70	4.50	4.54	0.99	6.37	155	4.00
5.70	5.50	5.53	0.99	6.45	154	5.00
6.70	6.50	6.53	1.00	5.88	169	6.00
7.70	7.50	7.52	1.00	5.63	177	7.00
8.70	8.50	8.52	1.00	5.52	181	8.00
9.70	9.50	9.52	1.00	5.24	191	9.00
10.70	10.50	10.52	1.00	4.89	204	10.00
11.70	11.50	11.52	1.00	4.63	216	11.00
12.70	12.50	12.51	1.00	4.14	242	12.00
13.70	13.50	13.51	1.00	3.53	283	13.00
14.70	14.50	14.51	1.00	4.00	250	14.00
15.70	15.50	15.51	1.00	3.88	258	15.00
16.70	16.50	16.51	1.00	4.38	228	16.00
17.70	17.50	17.51	1.00	3.46	289	17.00
18.70	18.50	18.51	1.00	3.42	292	18.00
19.70	19.50	19.51	1.00	3.25	308	19.00
20.70	20.50	20.51	1.00	3.17	316	20.00
21.70	21.50	21.51	1.00	3.46	289	21.00
22.70	22.50	22.51	1.00	3.24	309	22.00
23.70	23.50	23.51	1.00	3.05	328	23.00
24.70	24.50	24.51	1.00	3.27	306	24.00
25.70	25.50	25.51	1.00	3.31	302	25.00

Job No: 03-210
Client: Golder Associates Ltd.
Location: Faro Tailings Facility, Yukon
Sounding: SCPT-21
Sounding Date: October 18th, 2003



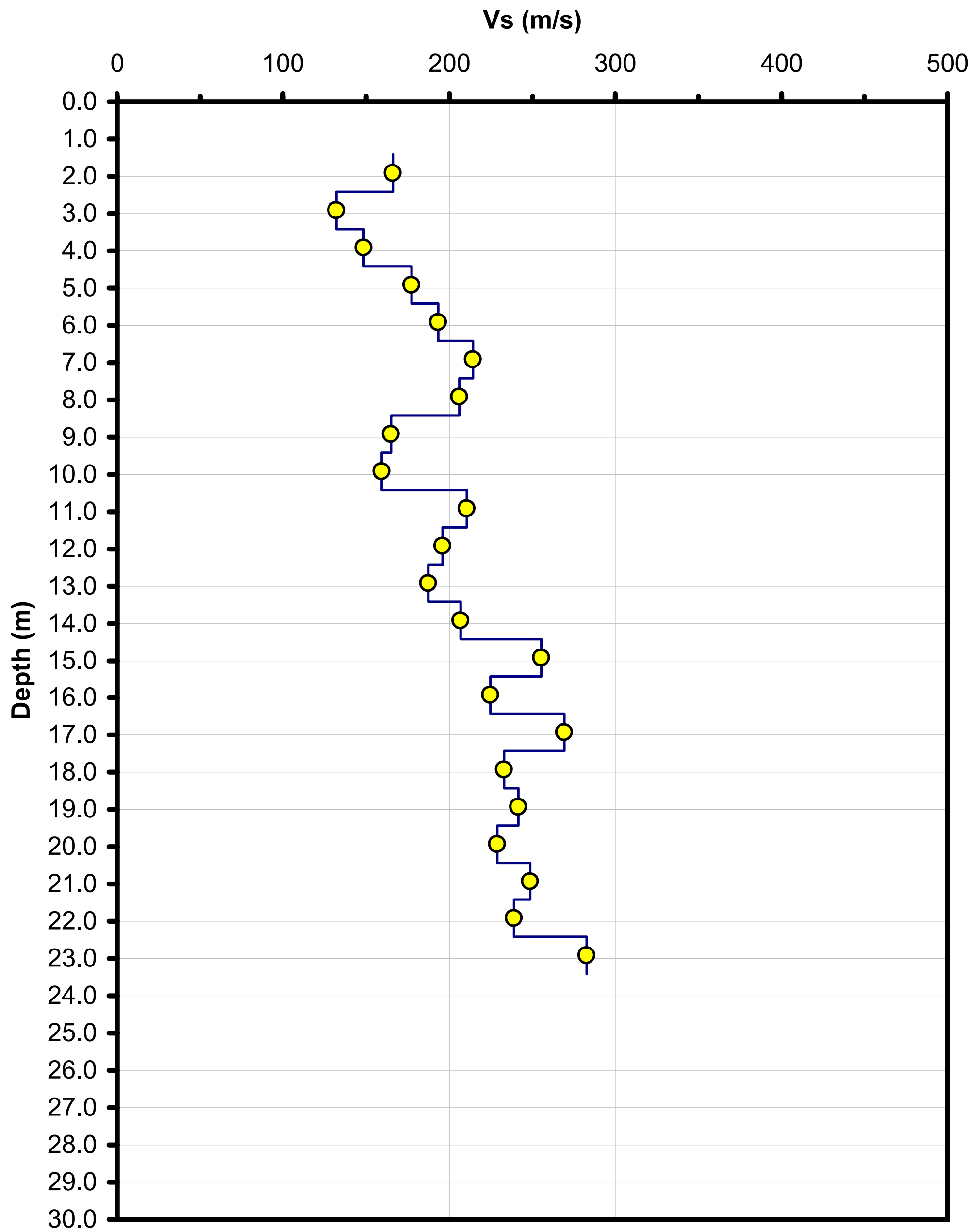
Job No.: 03-210
Client: Golder Associates Ltd.
Project: Faro Tailings Facility, Yukon
Sounding: SCPT-32
Date: October 18th, 2003

Seismic Source: Beam
Source Offset (m): 0.60
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SEISMIC

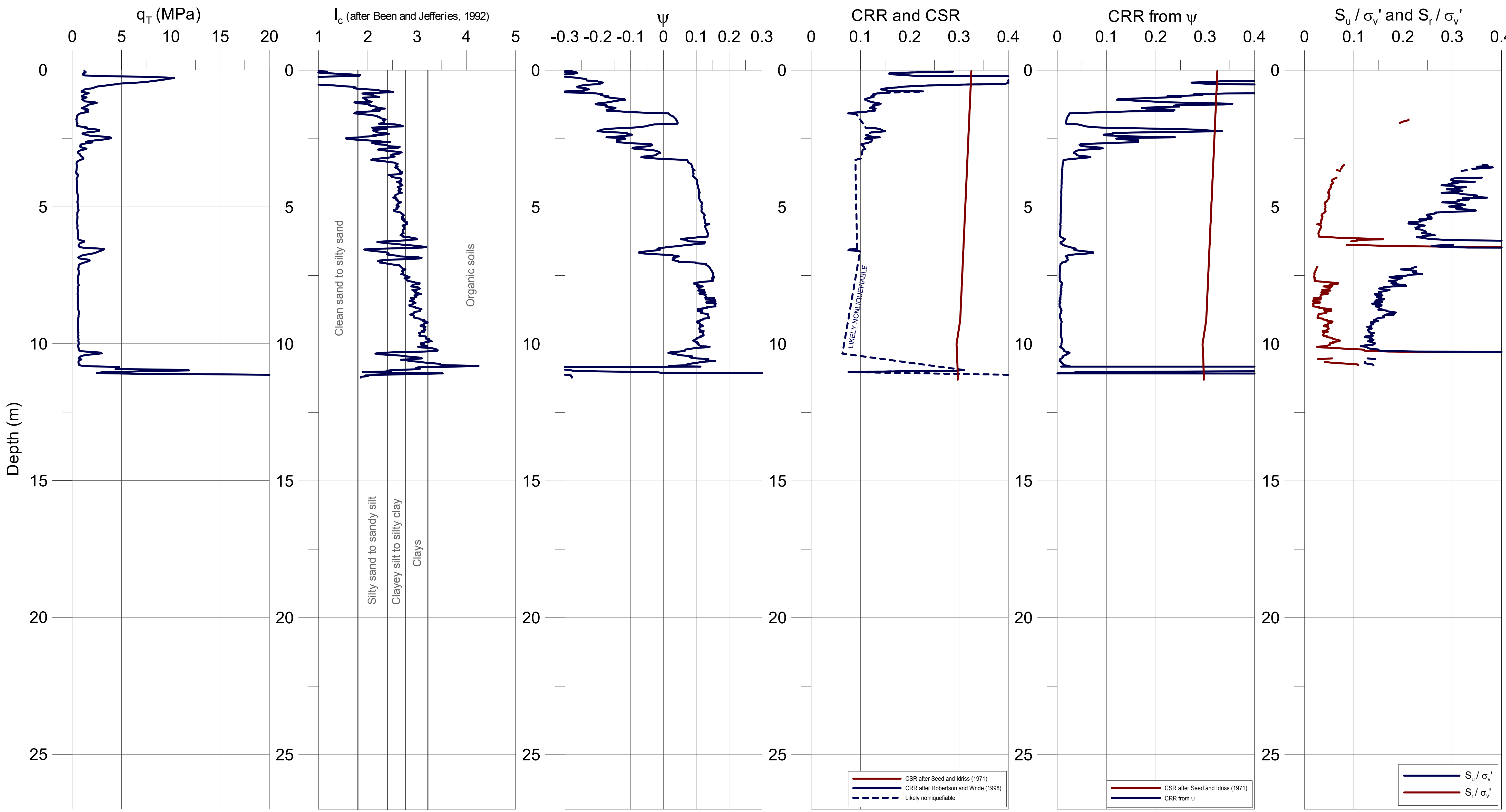
Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Depth Interval (m)	Time Interval (ms)	Vs (m/s)	Mid Layer (m)
1.62	1.42	1.54				
2.62	2.42	2.49	0.95	5.73	166	1.92
3.62	3.42	3.47	0.98	7.42	132	2.92
4.62	4.42	4.46	0.99	6.66	148	3.92
5.62	5.42	5.45	0.99	5.60	177	4.92
6.62	6.42	6.45	0.99	5.15	193	5.92
7.62	7.42	7.44	1.00	4.65	214	6.92
8.62	8.42	8.44	1.00	4.84	206	7.92
9.62	9.42	9.44	1.00	6.05	165	8.92
10.62	10.42	10.44	1.00	6.27	159	9.92
11.62	11.42	11.44	1.00	4.75	210	10.92
12.62	12.42	12.43	1.00	5.10	196	11.92
13.62	13.42	13.43	1.00	5.33	187	12.92
14.62	14.42	14.43	1.00	4.83	207	13.92
15.62	15.42	15.43	1.00	3.91	255	14.92
16.62	16.42	16.43	1.00	4.45	225	15.92
17.62	17.42	17.43	1.00	3.71	269	16.92
18.62	18.42	18.43	1.00	4.29	233	17.92
19.62	19.42	19.43	1.00	4.14	241	18.92
20.62	20.42	20.43	1.00	4.37	229	19.92
21.62	21.42	21.43	1.00	4.02	249	20.92
22.62	22.42	22.43	1.00	4.18	239	21.92
23.62	23.42	23.43	1.00	3.54	283	22.92

Job No: 03-210
Client: Golder Associates Ltd.
Location: Faro Tailings Facility, Yukon
Sounding: SCPT-32
Sounding Date: October 18th, 2003



APPENDIX V

LOGS OF SOIL TYPE AND STATE FROM CPT DATA



REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-01.grf

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
4. Seismic results based on a_{max} / g = 0.5 and magnitude 7.0 (Gail Atkinson, "Draft Seismic Hazard Assessment for Faro, YK", Dec. 23, 2003)
4. CSR after Seed, H.B., and Idriss, I.M. 1971. Simplified procedure for evaluating soil liquefaction potential. Journal of the Soil Mechanics and Foundations Division, ASCE, **107**(SM9): 1249-1274.
5. CRR after Robertson, P.K., and Wride (Fear), C.E. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, **35**: 442–459.
6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

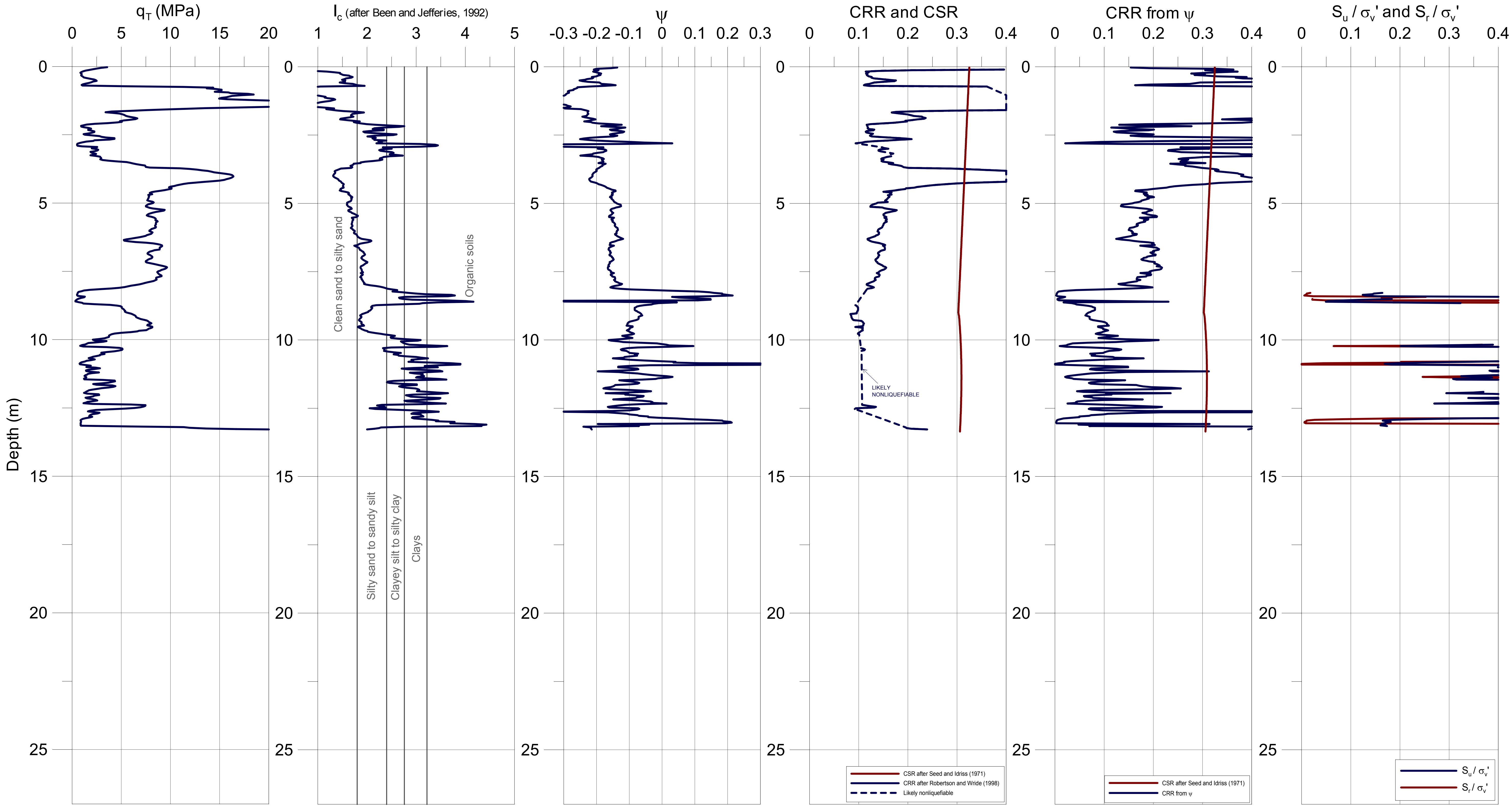
TITLE

LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-01	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04	**CPT03-01**	
CHECK	--	--		
REVIEW	--	--		

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-02.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
4. Seismic results based on a_{max} / g = 0.5 and magnitude 7.0 (Gail Atkinson, "Draft Seismic Hazard Assessment for Faro, YK", Dec. 23, 2003)
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6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-02	
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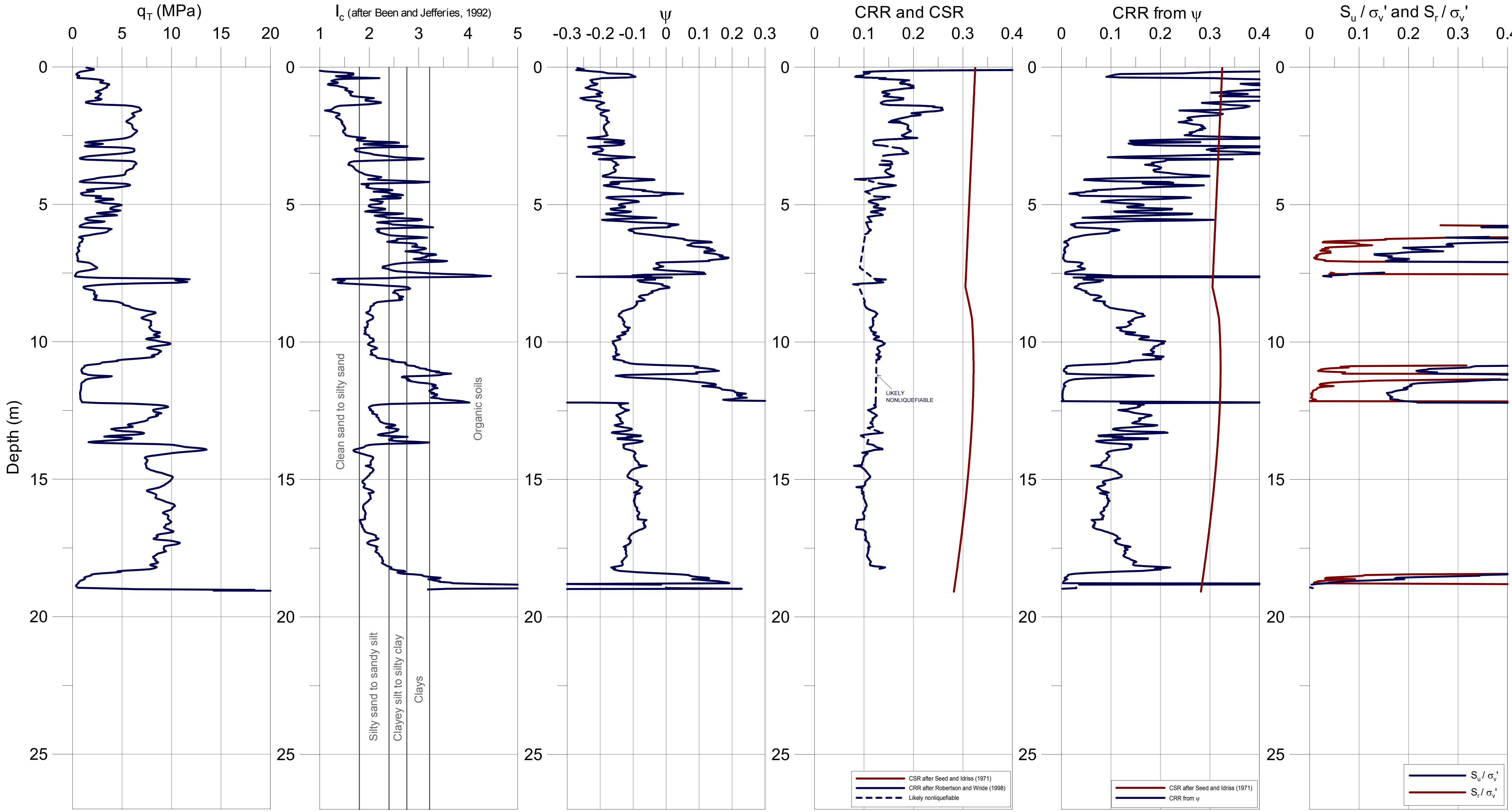
REVIEW	--	--	



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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6. CRR estimated from ψ (see text of report)
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-03
DESIGN DRK	29 JAN 04	SCALE NTS REV.
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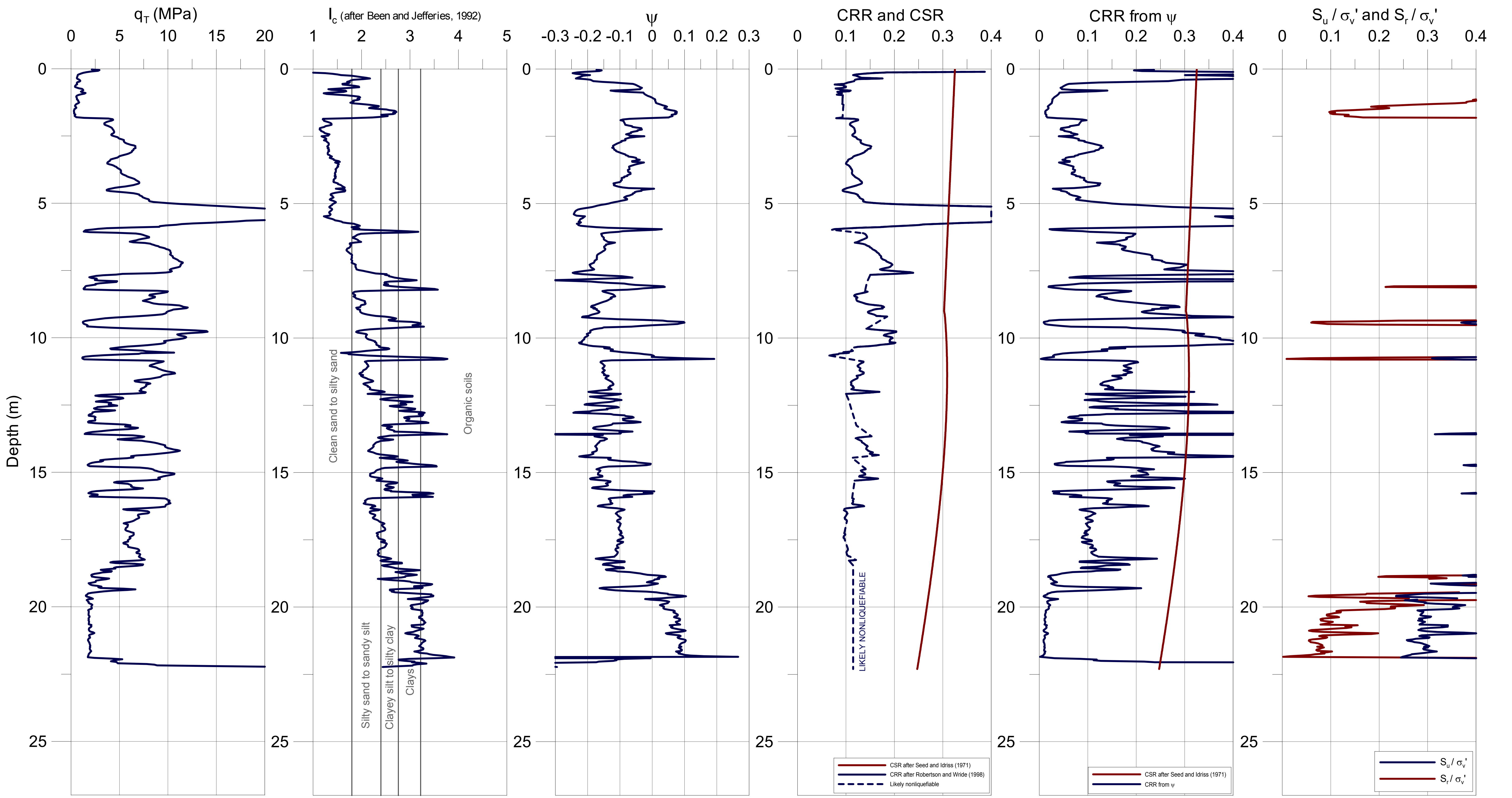
CHECK --	--	**CPT03-03**
REVIEW --	--	

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-03.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-04	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04	**CPT03-04**	
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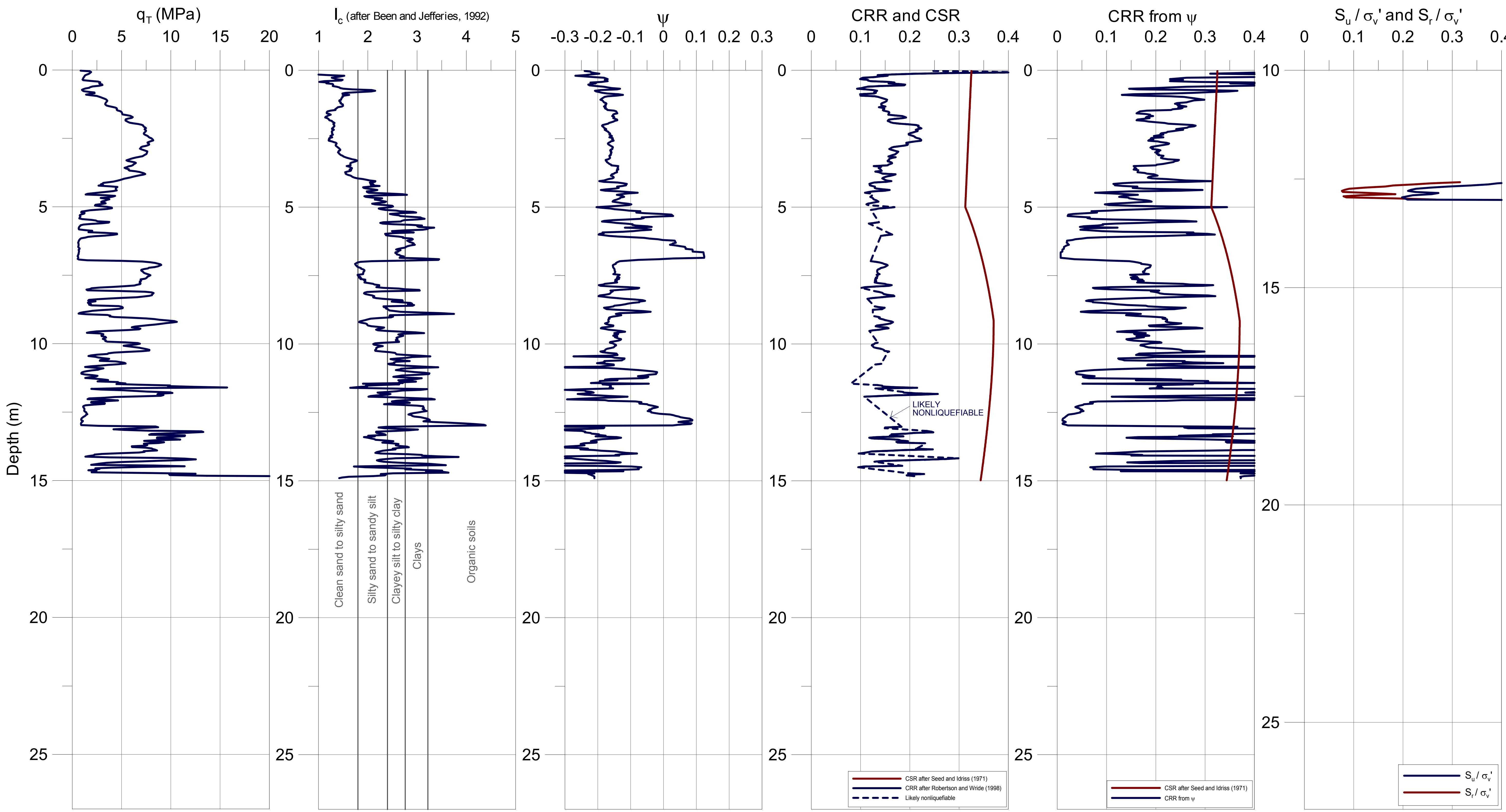
REVIEW	--	--		

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-04.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-SCPT03-05
DESIGN	DRK	29 JAN 04	SCALE NTS REV.
CADD	DRK	29 JAN 04	
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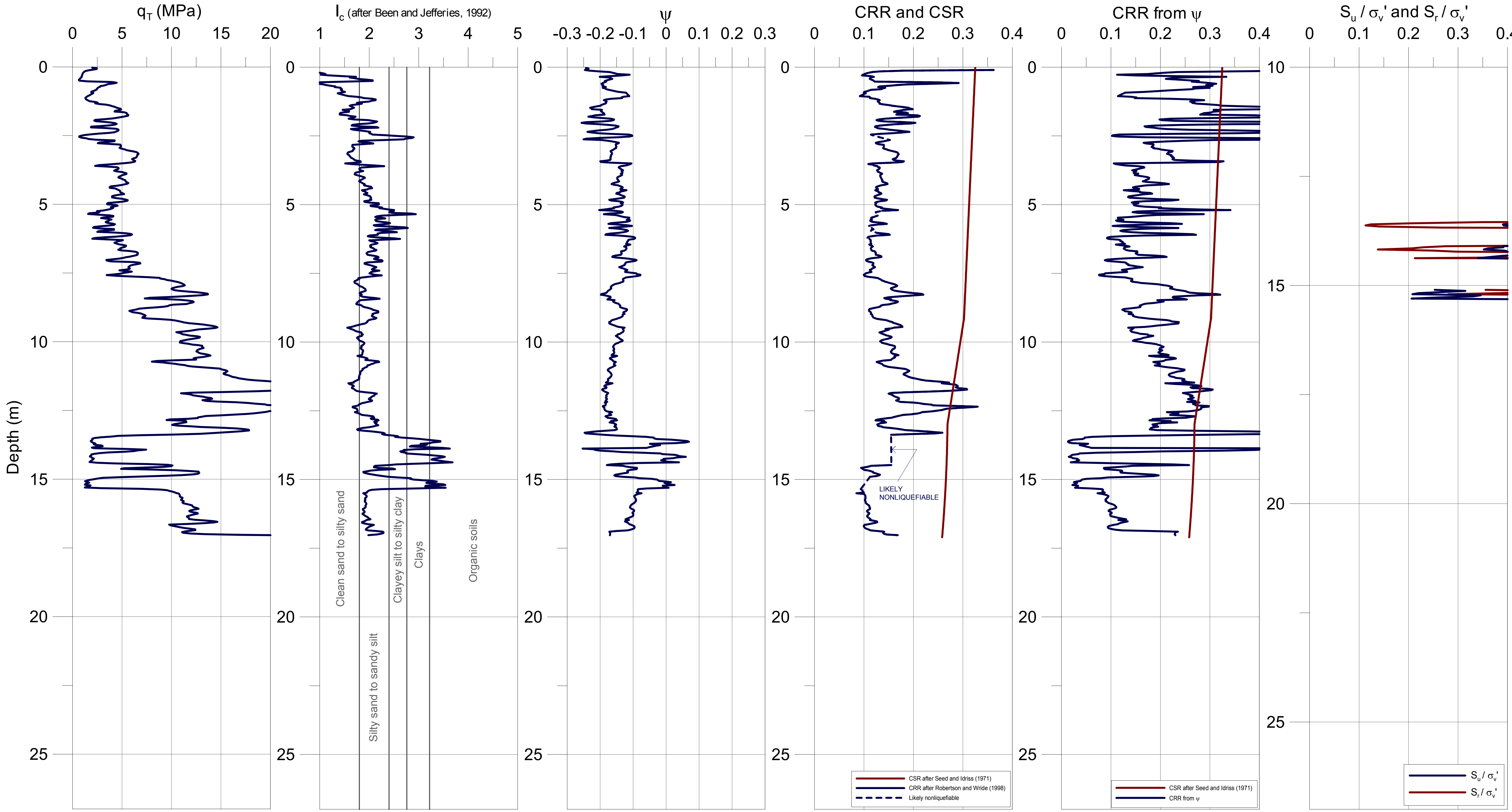
REVIEW	--	--	

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-SCPT03-05.grf



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Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-06	
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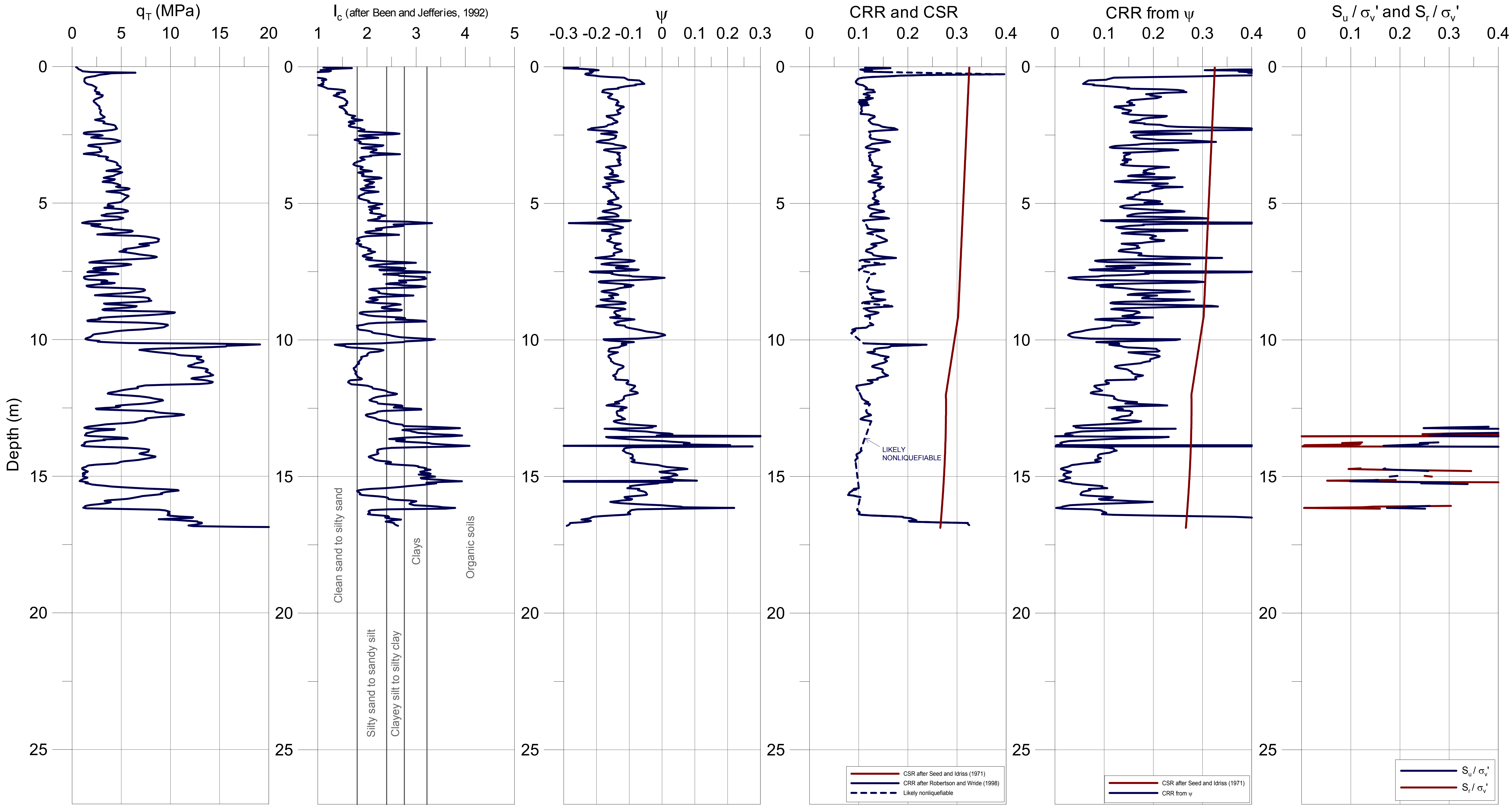
REVIEW	--	--	



Notes

1. For record of CPT see Appendix III
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-07	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
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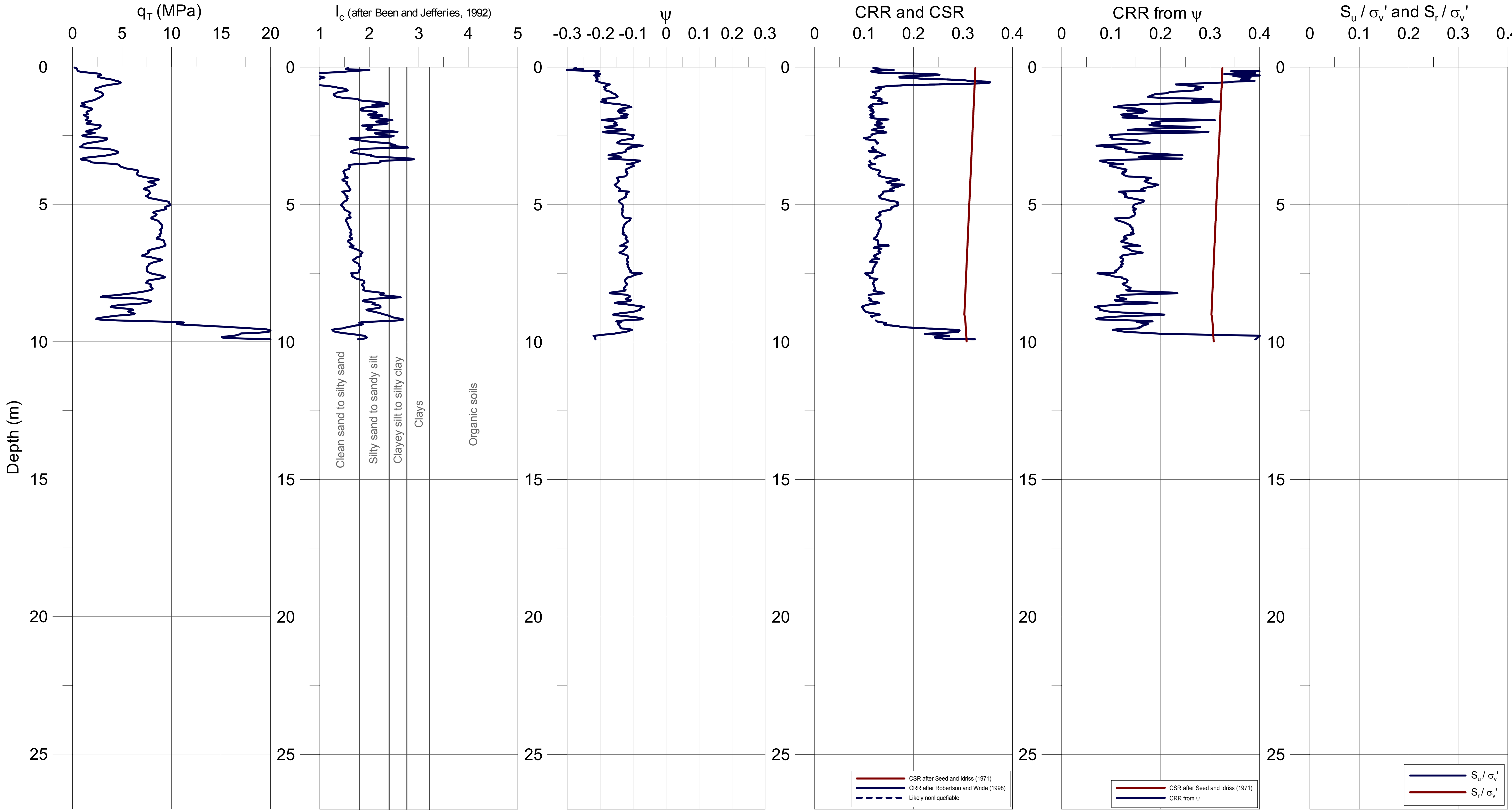
REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-07.grf



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Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-08	
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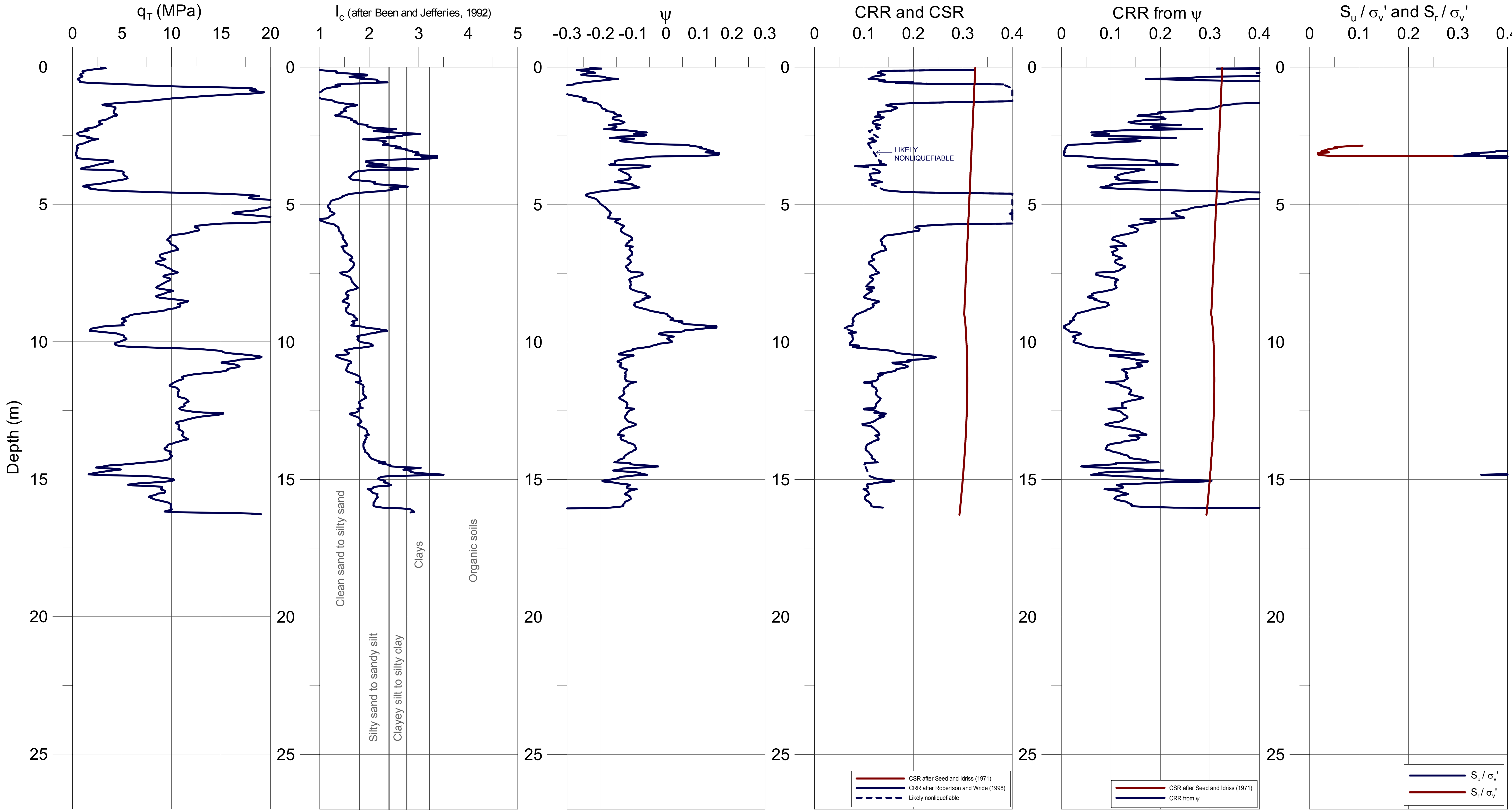

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Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT	**SRK CONSULTING INC.** **ROSE CREEK TAILINGS FACILITY** **ANVIL RANGE MINING COMPLEX, YUKON**
TITLE	**LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA**

Golder Associates			
PROJECT No.	03-1413-080	FILE No. plots2-CPT03-09	
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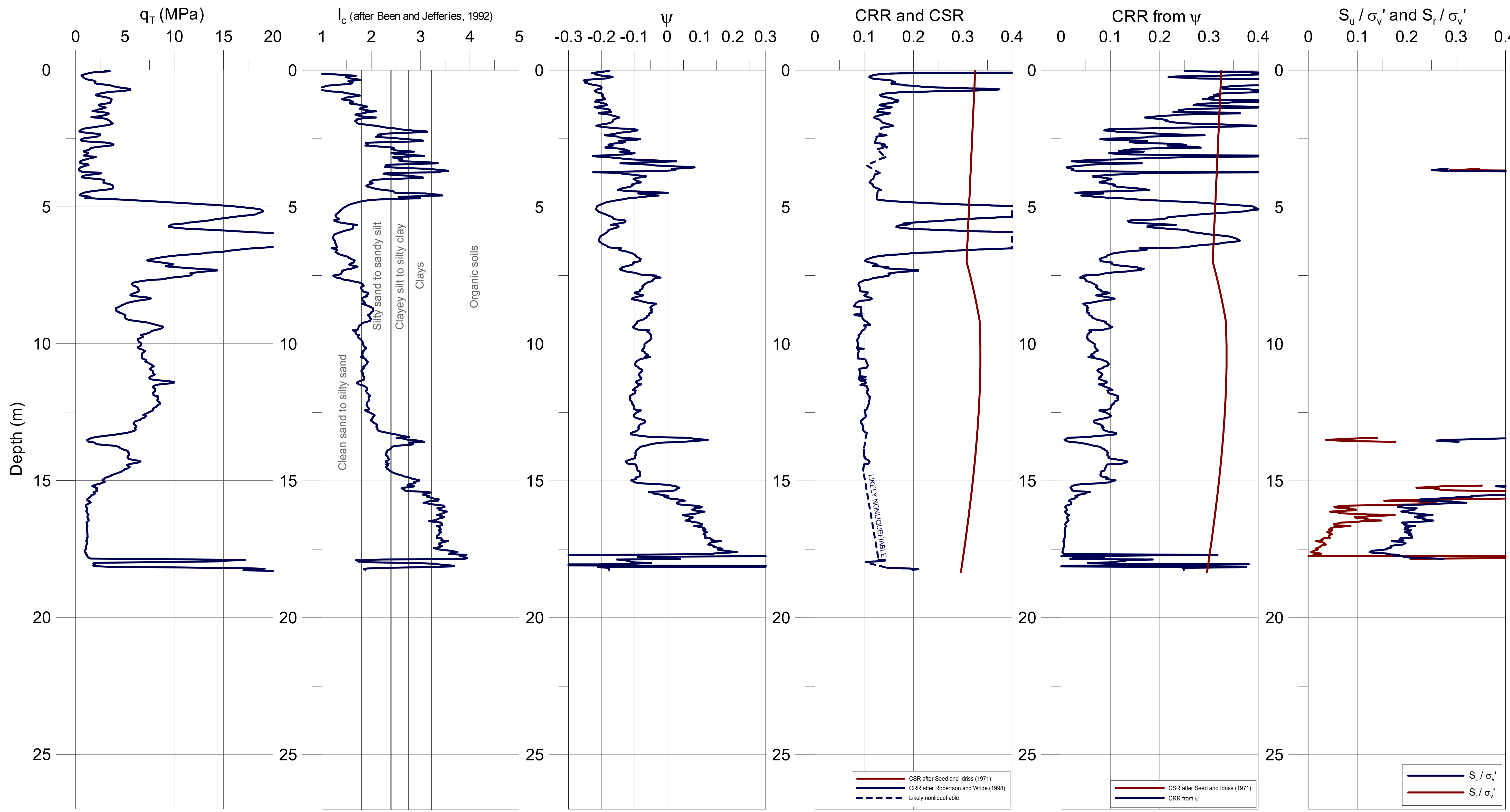

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Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-10
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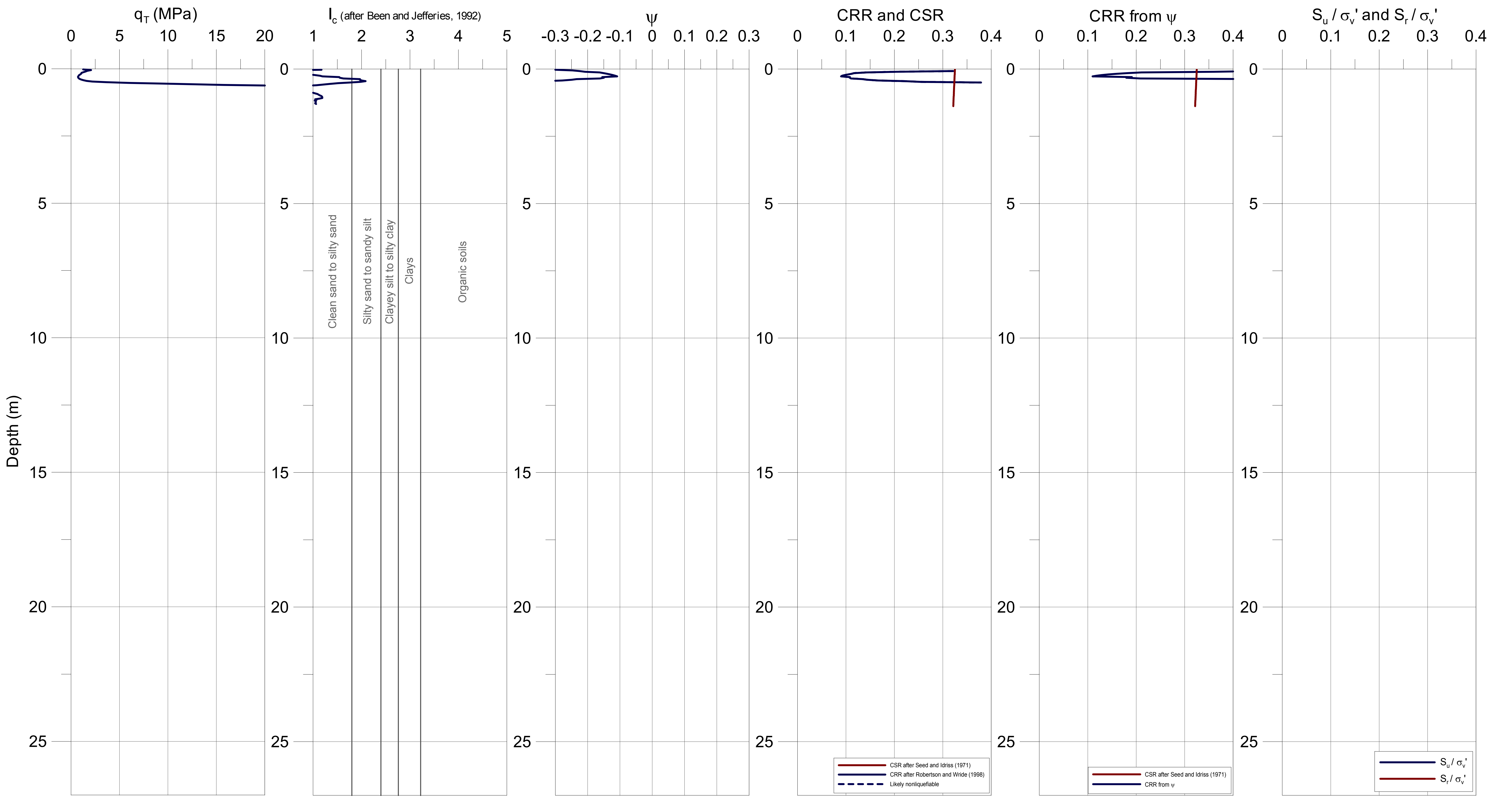
REVIEW --	--	



Notes

1. For record of CPT see Appendix III
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-11.grf

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	
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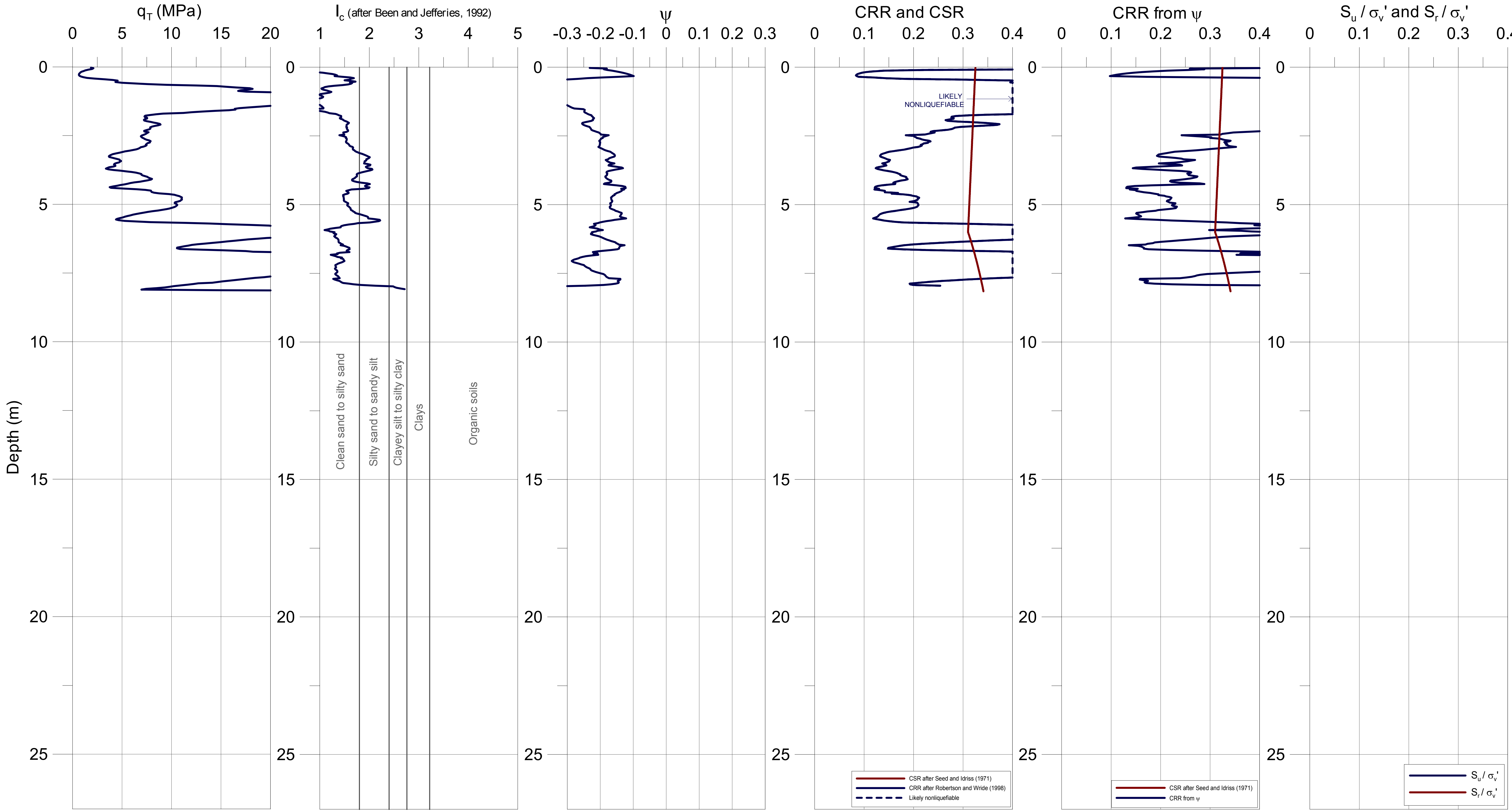
REVIEW	--	--

FILE No. plots2-CPT03-11
SCALE NTS REV.

CPT03-11



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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PROJECT	SRK CONSULTING INC. ROSE CREEK TAILINGS FACILITY ANVIL RANGE MINING COMPLEX, YUKON
TITLE	LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-11B
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Golder Associates

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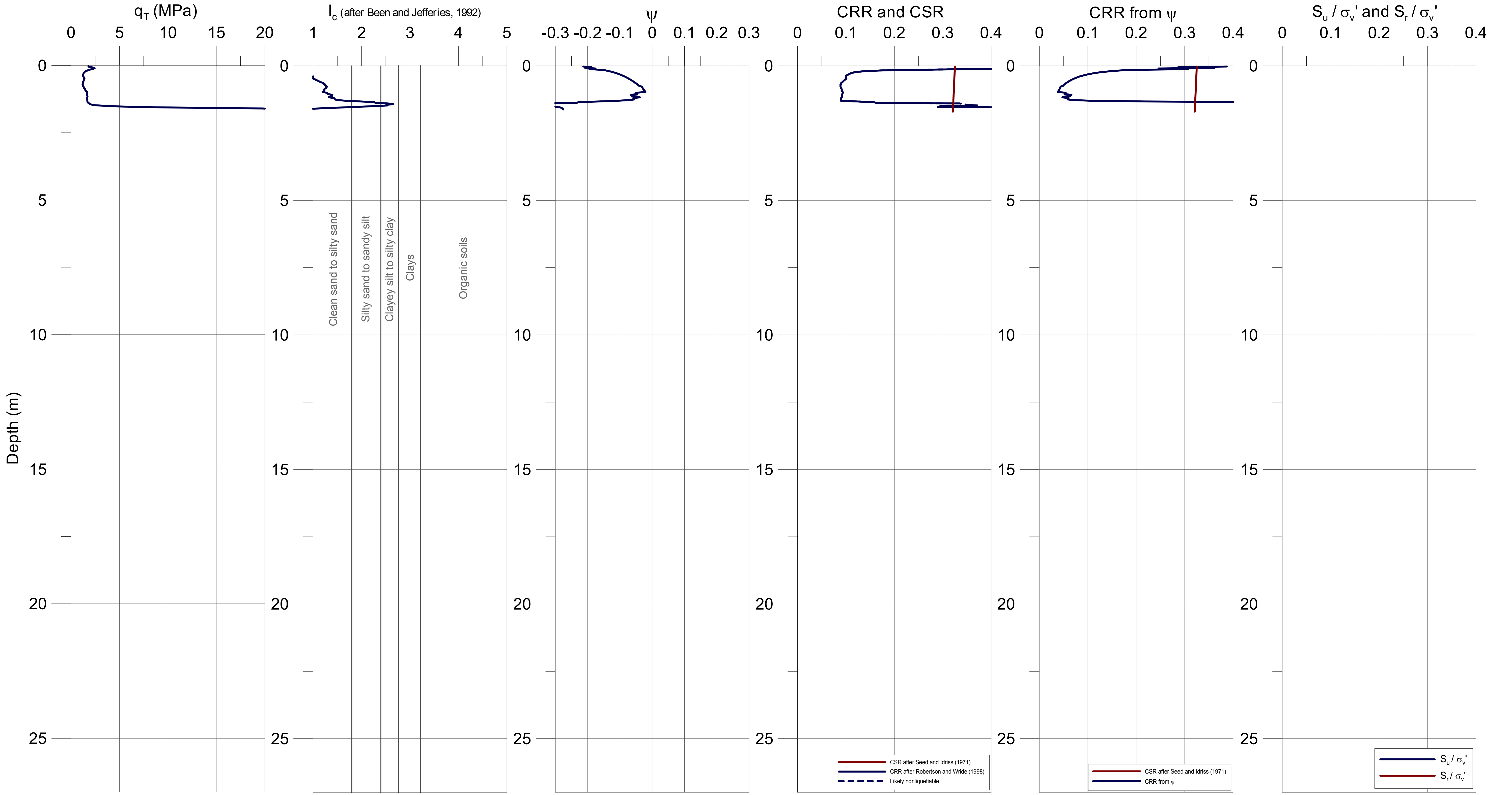

Notes

1. For record of CPT see Appendix III
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5. CRR after Robertson, P.K., and Wride (Fear), C.E. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, **35**: 442–459.
6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-12	
DESIGN	DRK	29 JAN 04	SCALE NTS REV.
CADD	DRK	29 JAN 04	**CPT03-12**
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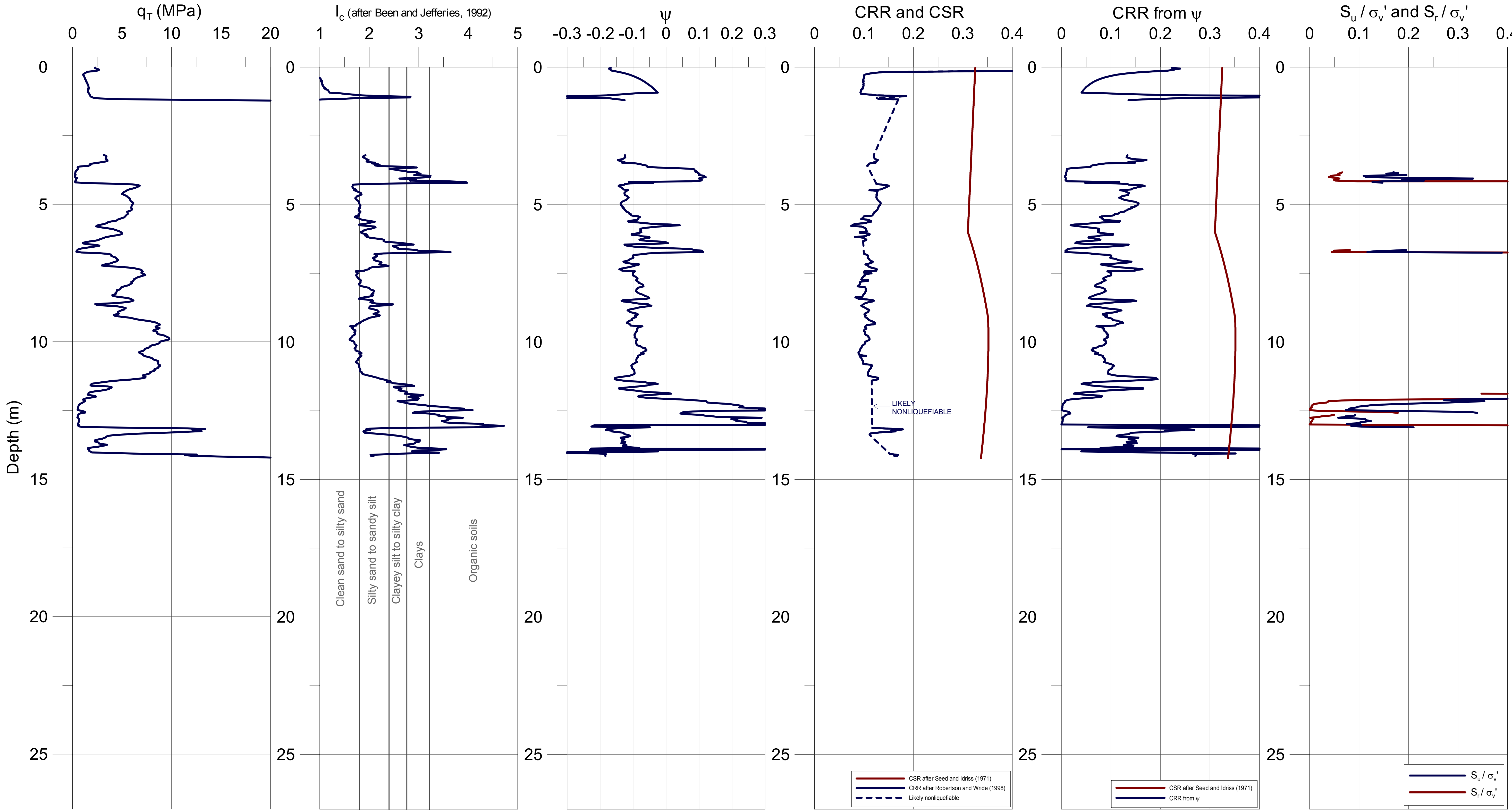
REVIEW	--	--	

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-12.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-12B.grf

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-12B	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
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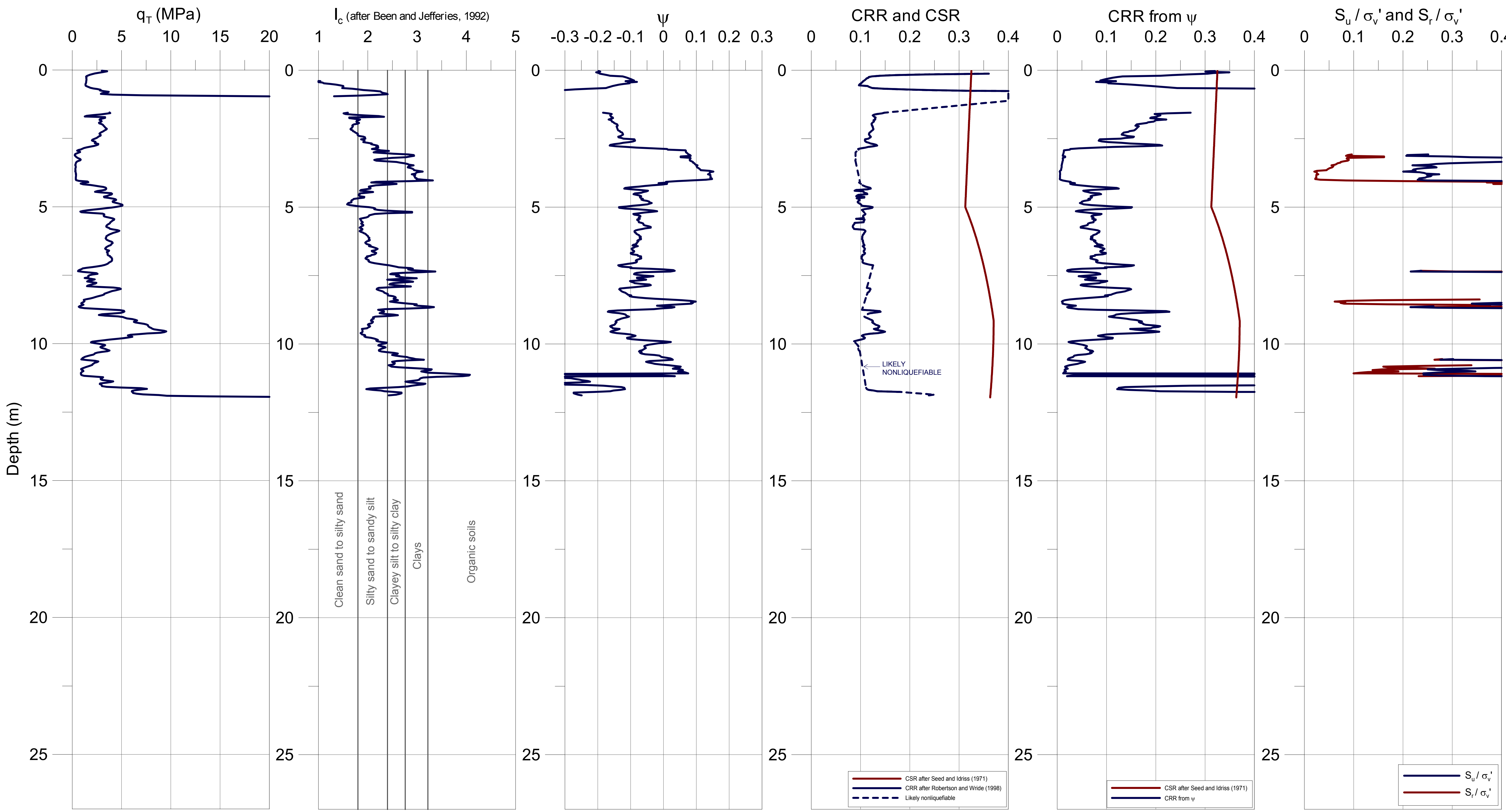

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-13.grf

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-13
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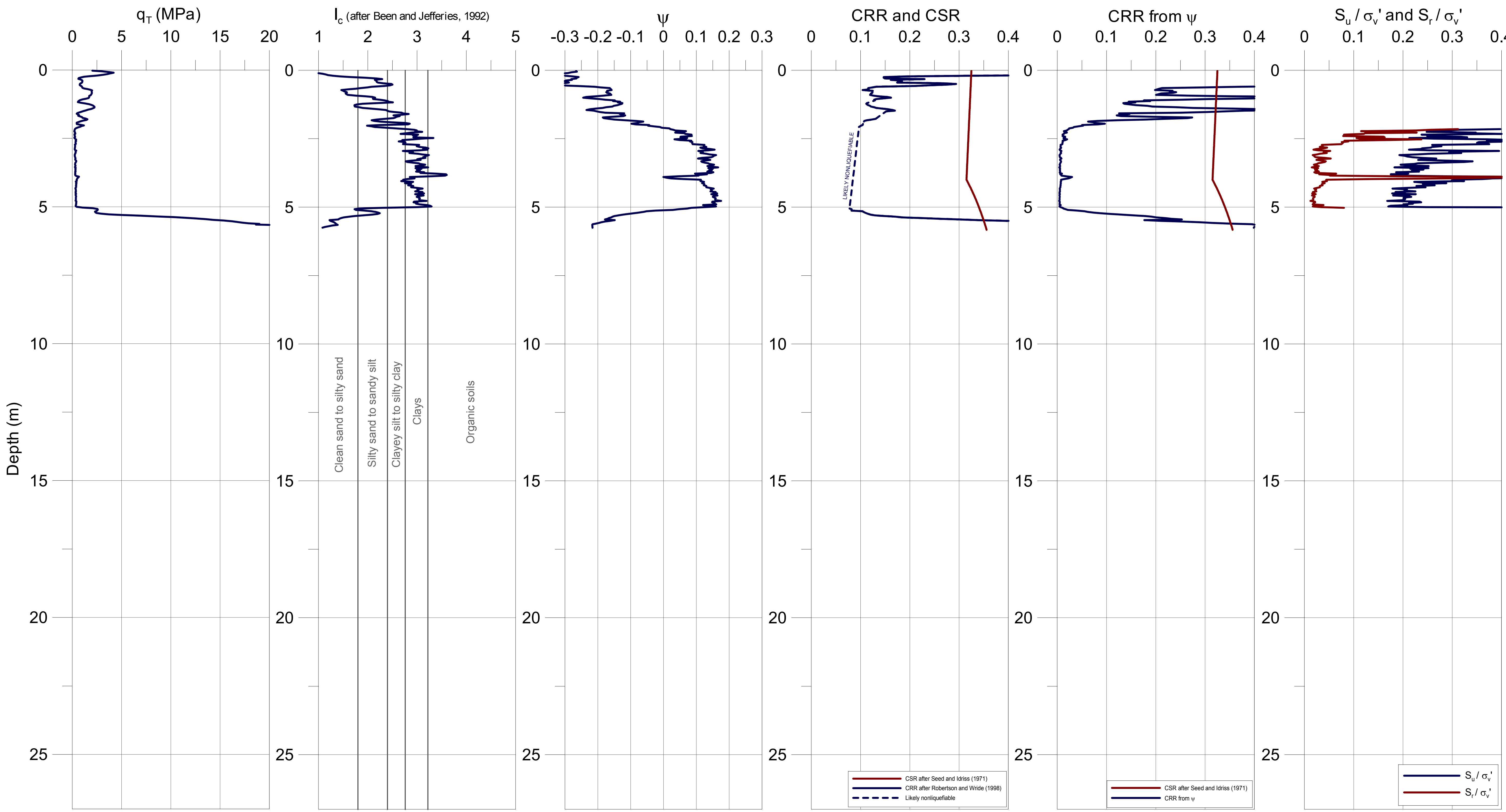
CADD	DRK	29 JAN 04	
CHECK	--	--	**CPT03-13**
REVIEW	--	--	



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-14	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04	**CPT03-14**	
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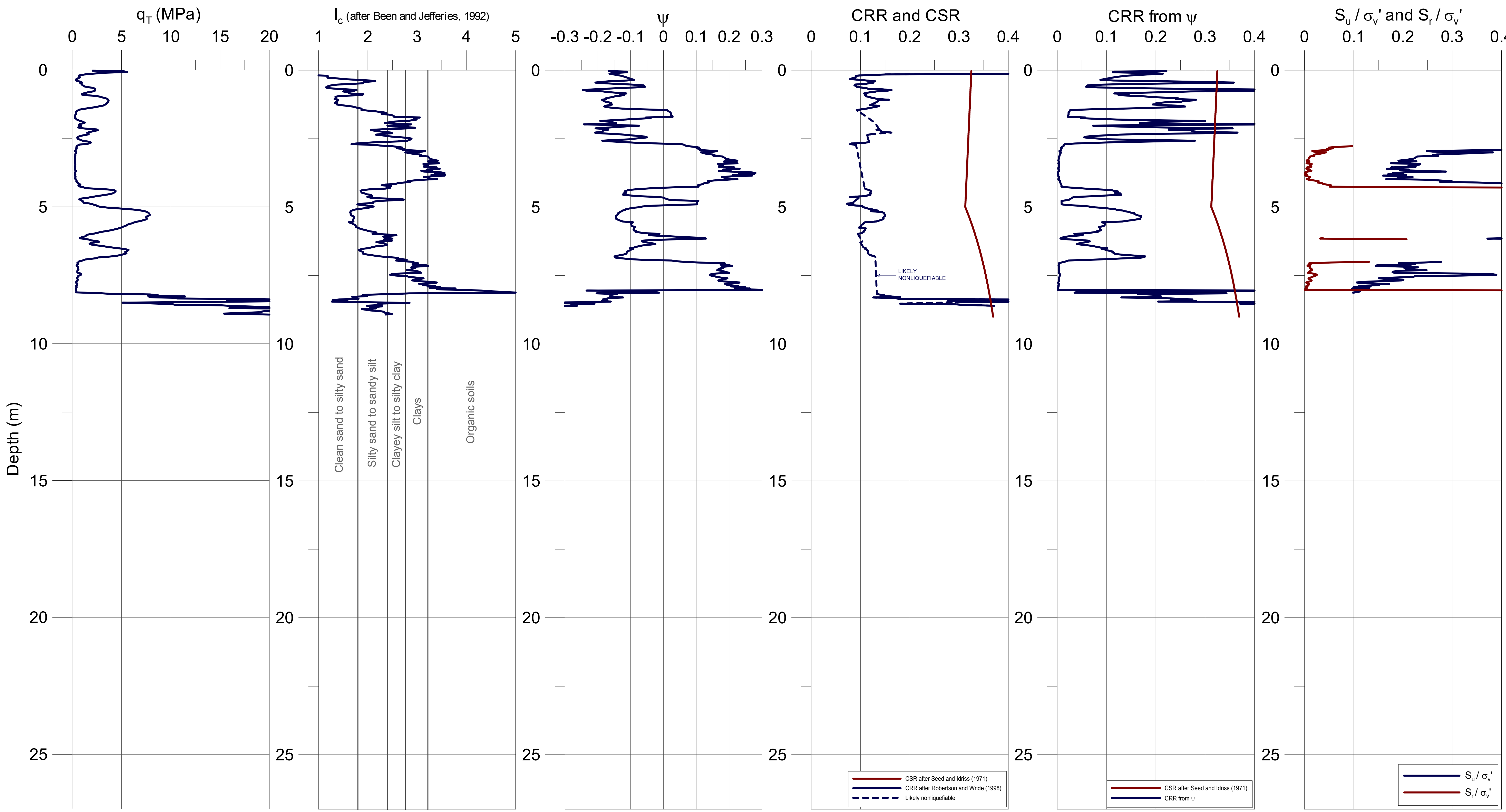
REVIEW	--	--		

REVISION DATE: 30 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-14.grf

REVISION DATE: 30 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-15.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
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ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-15
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CADD DRK	29 JAN 04	**CPT03-15**
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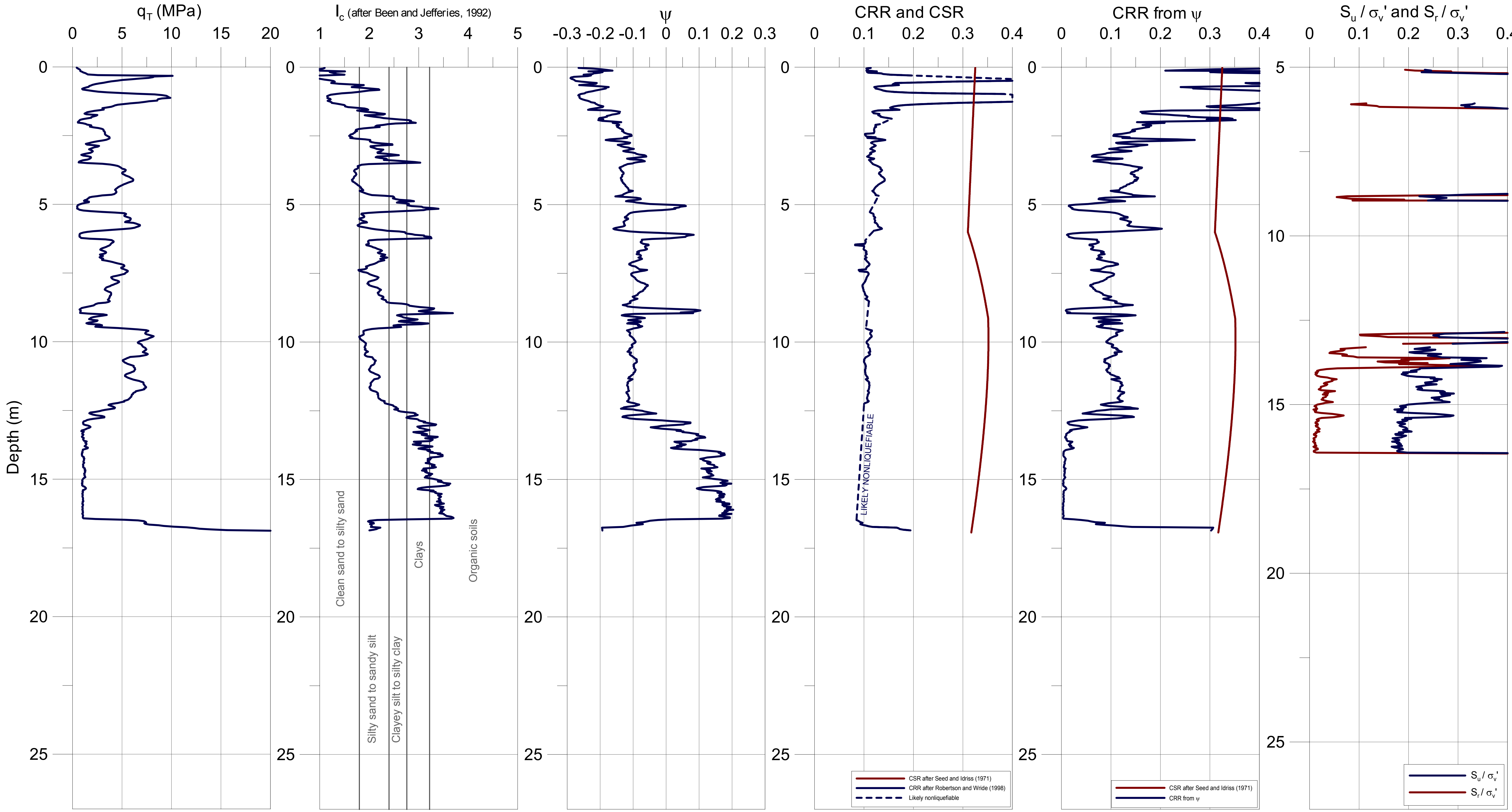
REVIEW --	--	



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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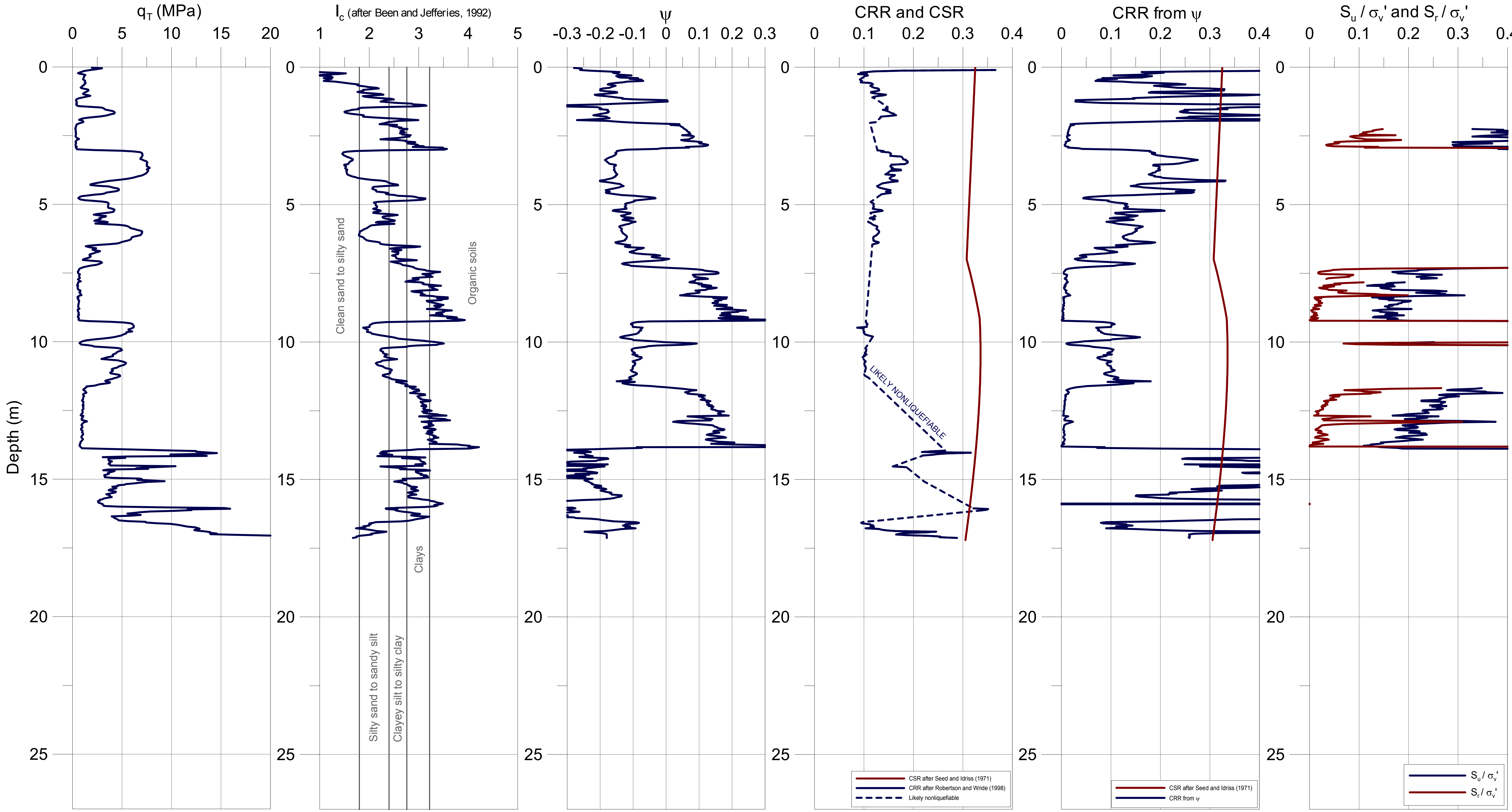
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-16
DESIGN	DRK	29 JAN 04	SCALE NTS REV.
CADD	DRK	29 JAN 04	**CPT03-16**
CHECK	--	--	
REVIEW	--	--	

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-16.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

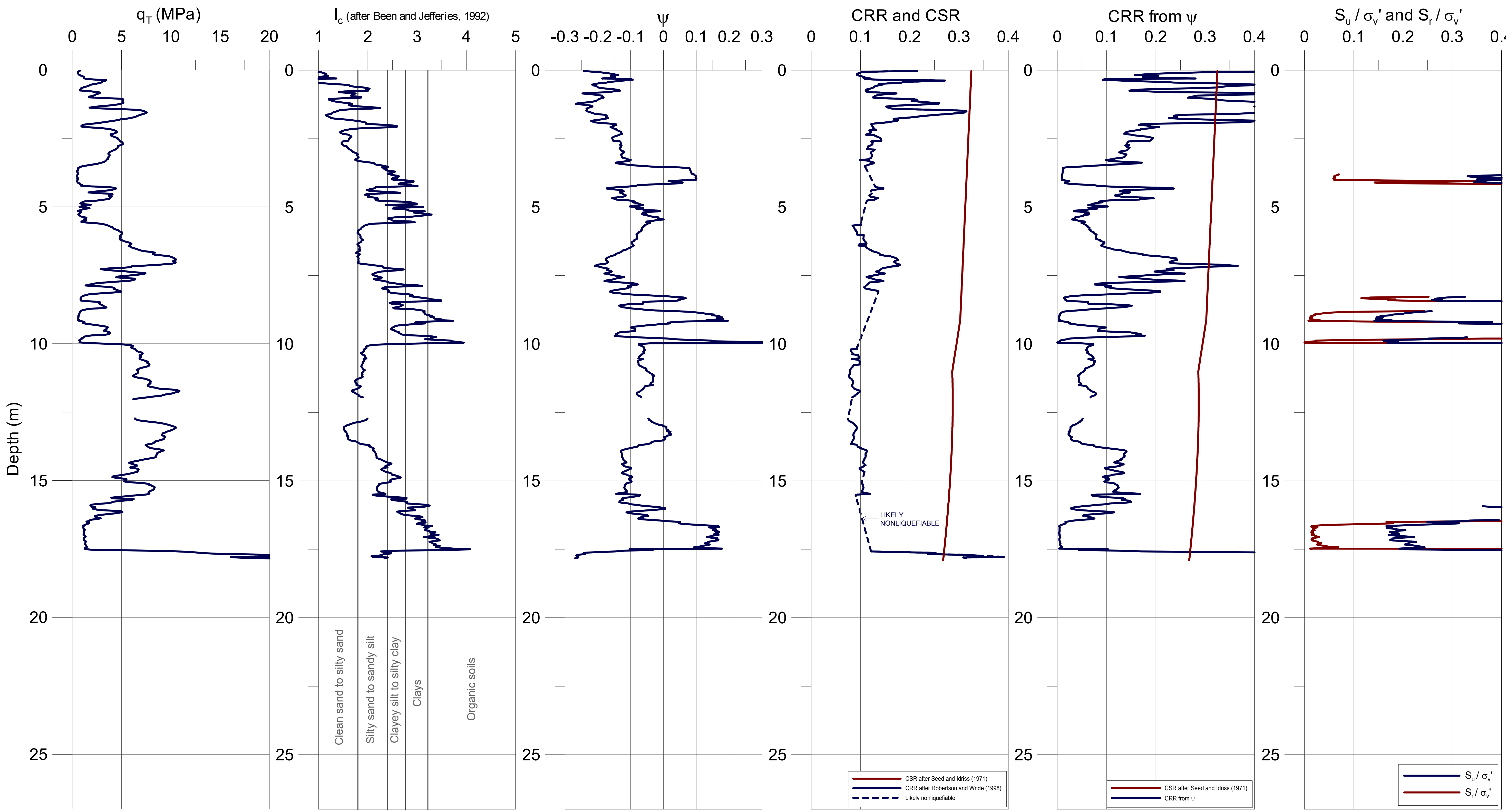
TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No.	plots2-SCPT03-17
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04		
CHECK	--	--	**SCPT03-17**	
REVIEW	--	--		

REVISION DATE: 30 Jan 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-SCPT03-17.grf

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-18.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-18	
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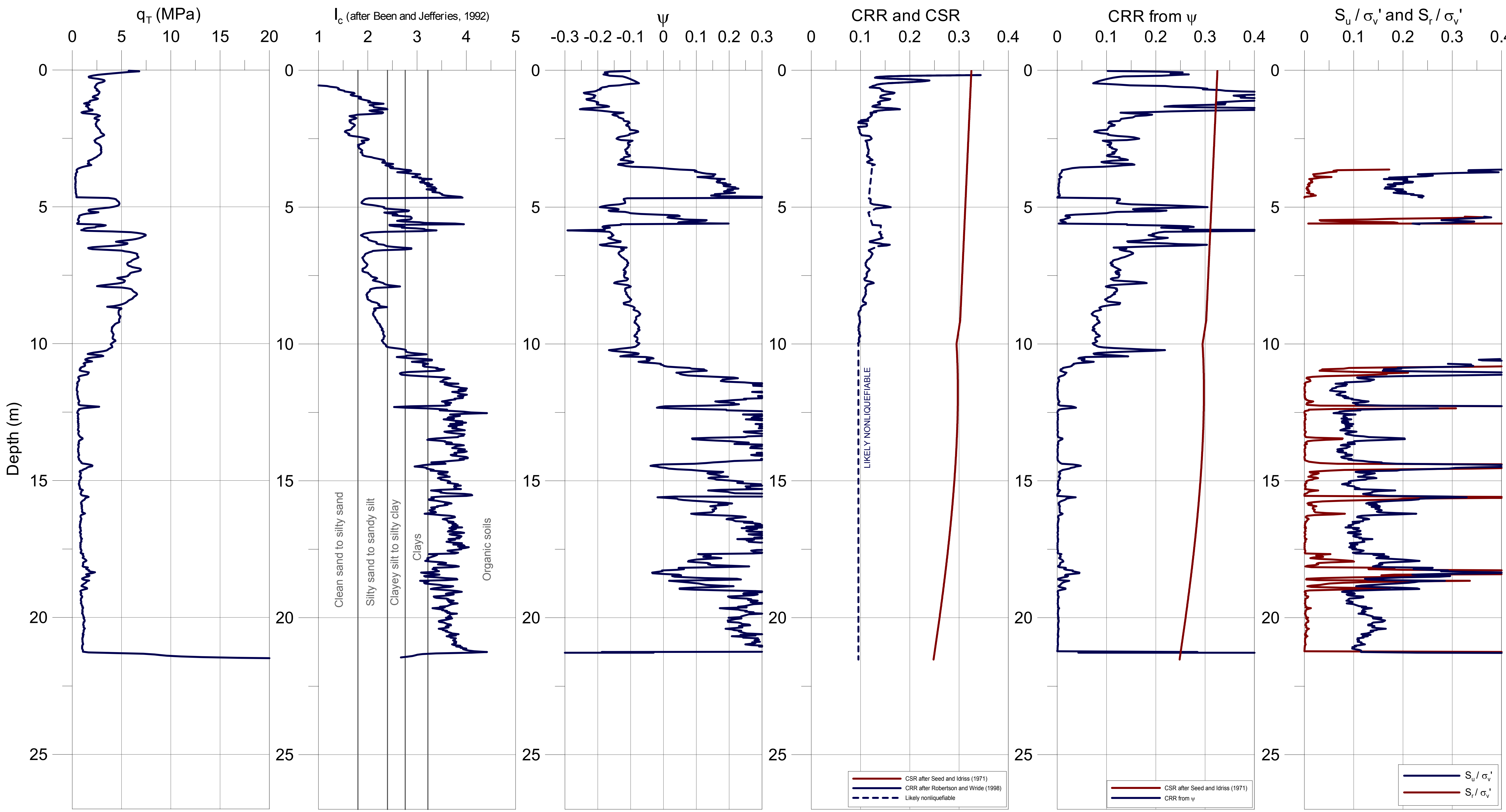

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-19
DESIGN	DRK	29 JAN 04	SCALE NTS REV.
CADD	DRK	29 JAN 04	**CPT03-19**
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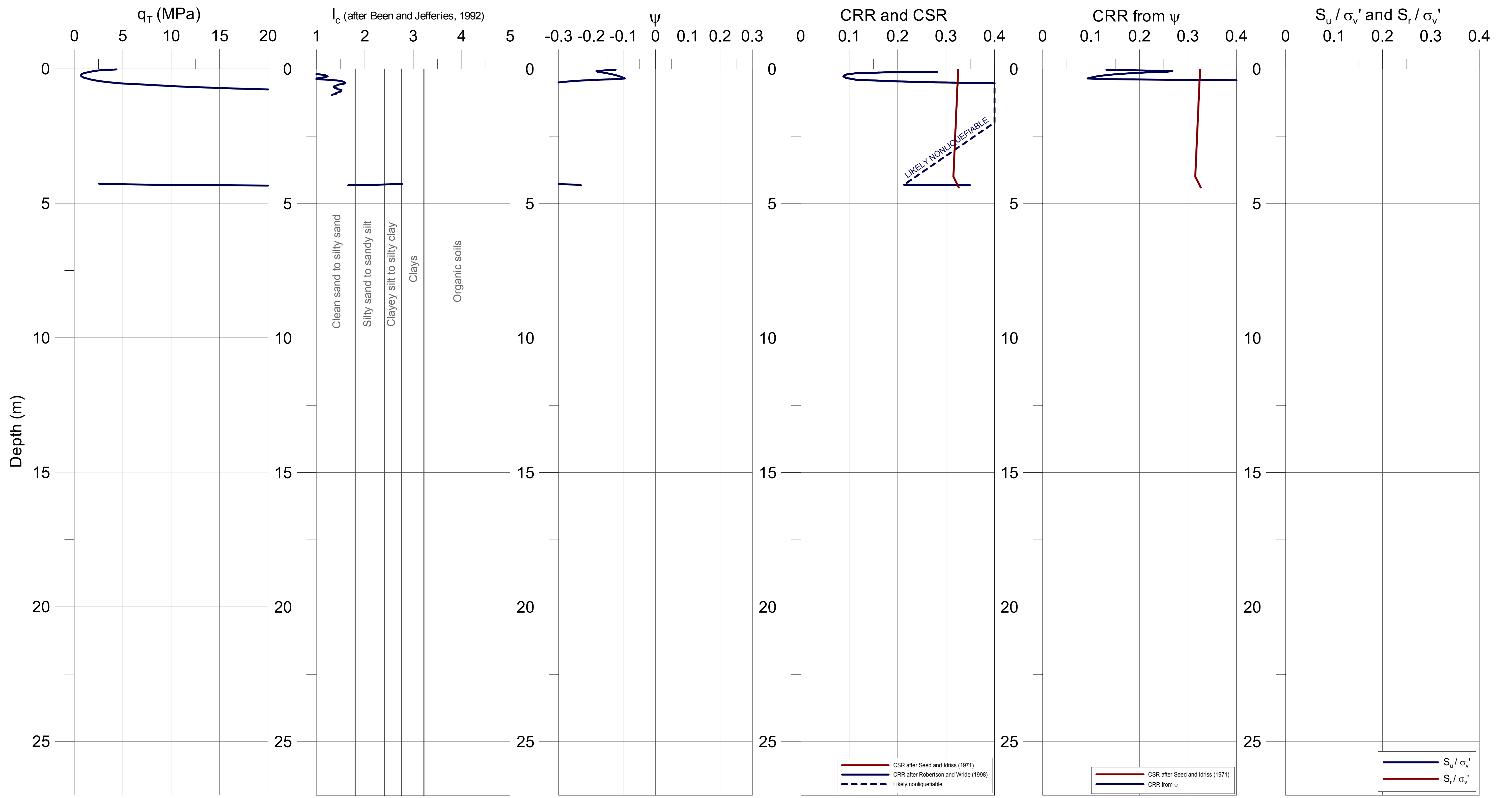
REVIEW	--	--	

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active_2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-19.grf

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-20.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-20	
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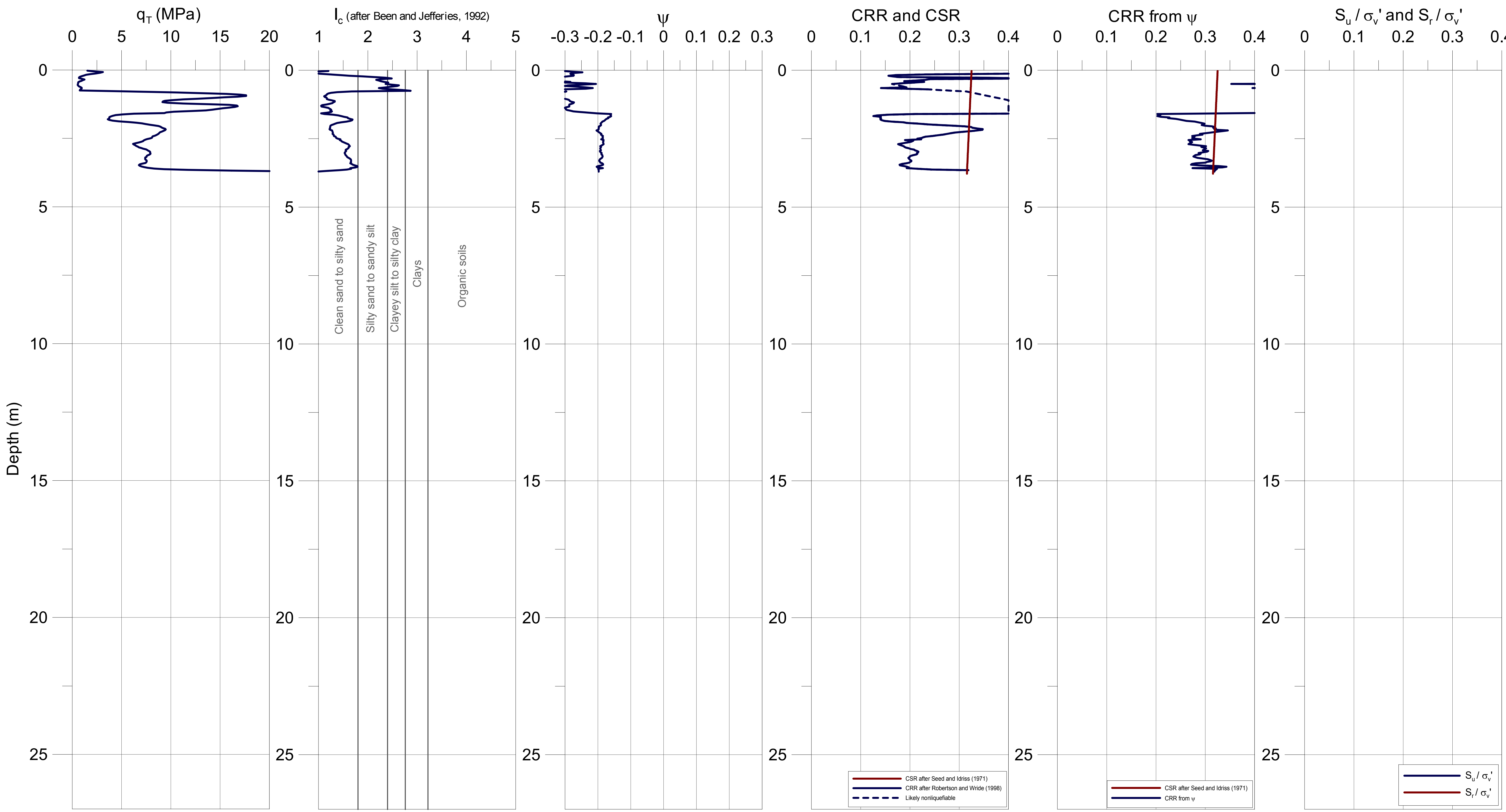
REVIEW	--	--	



Notes

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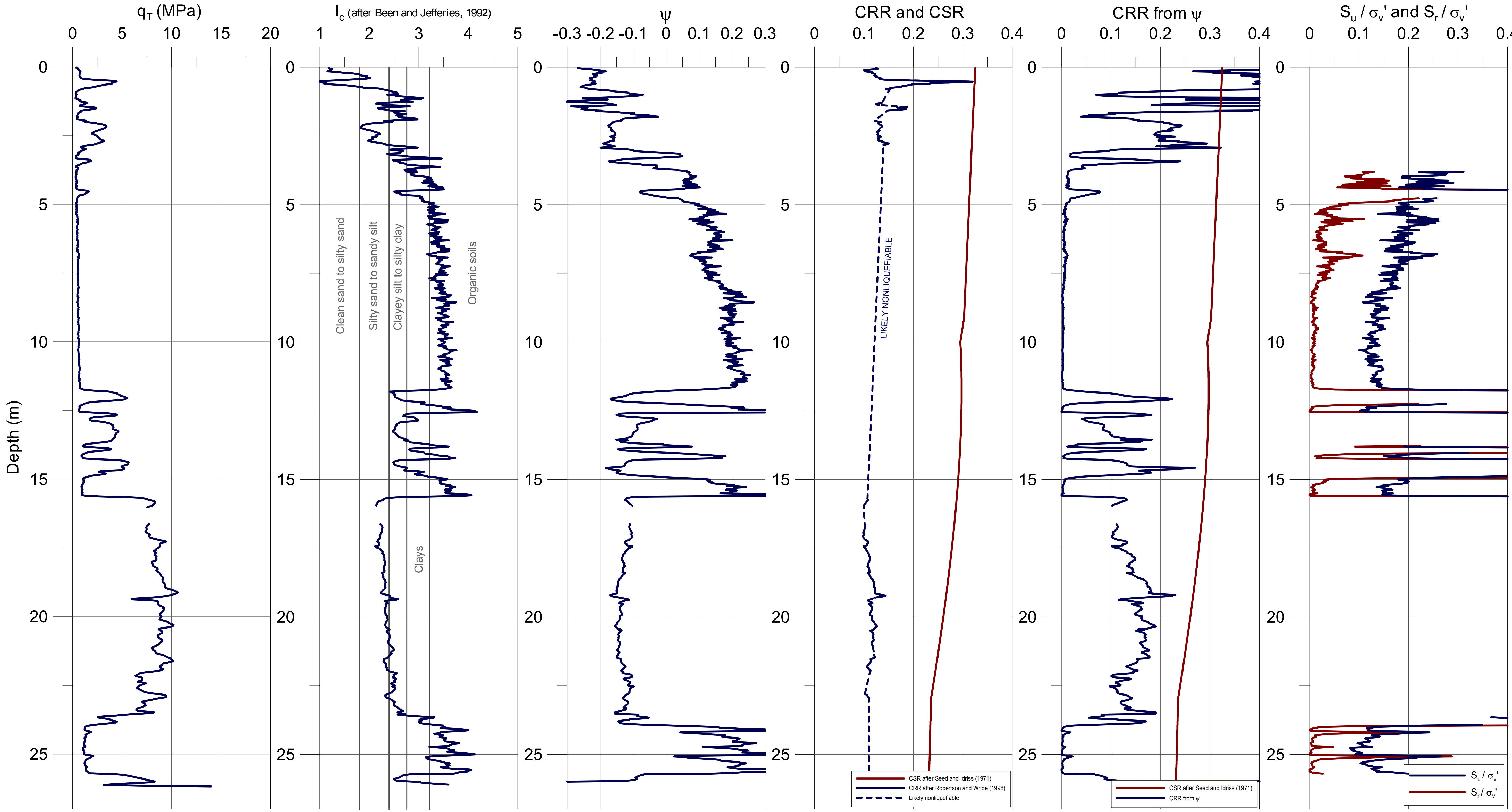
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-20X	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04	**CPT03-20X**	
CHECK	--	--		
REVIEW	--	--		

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-20X.grf



Notes

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5. CRR after Robertson, P.K., and Wride (Fear), C.E. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, **35**: 442–459.
6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-SCPT03-21.grf

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-SCPT03-21		
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CADD	DRK	29 JAN 04		
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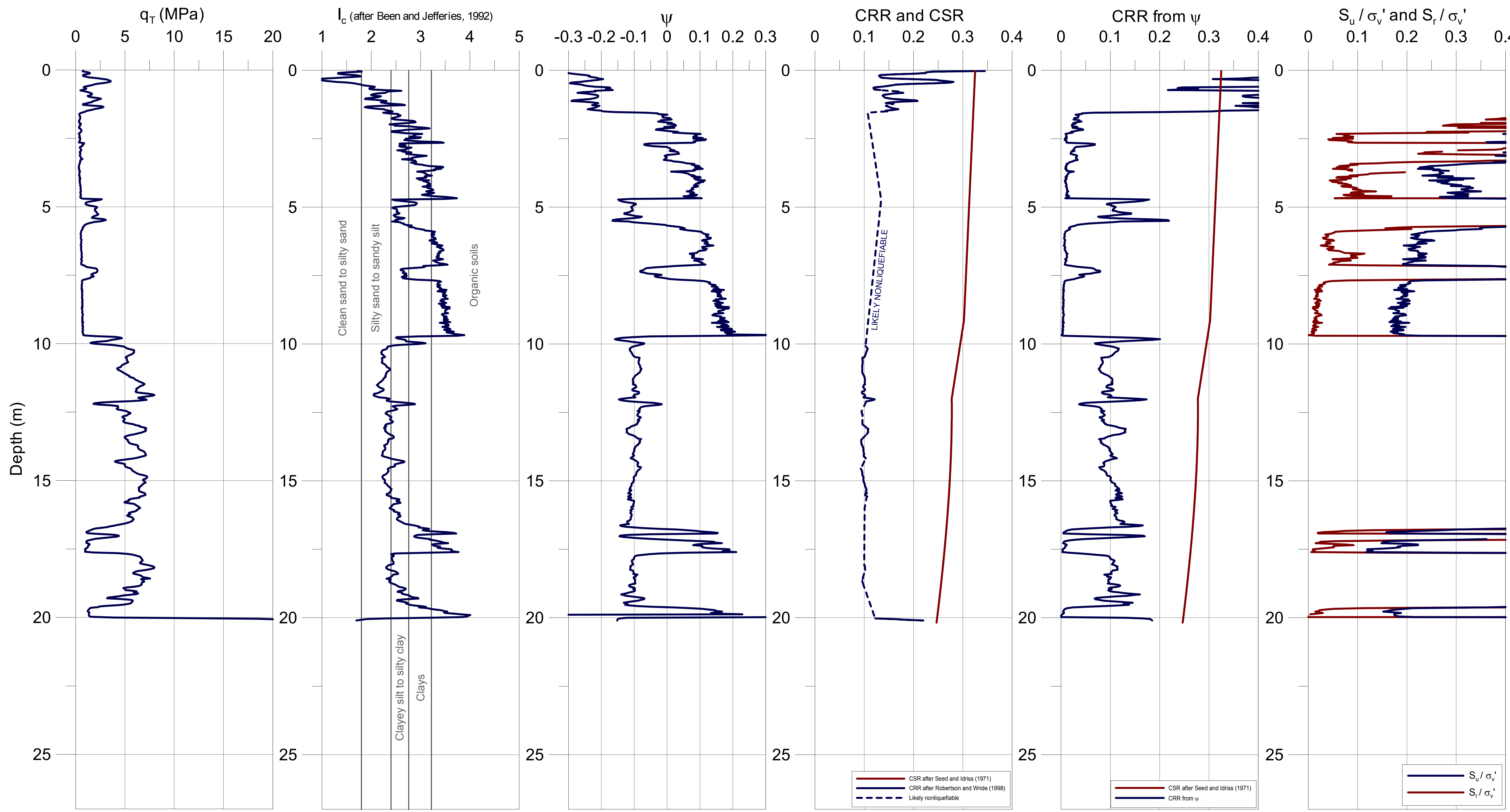
REVIEW	--	--	**SCPT03-21**	



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-22	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04	**CPT03-22**	
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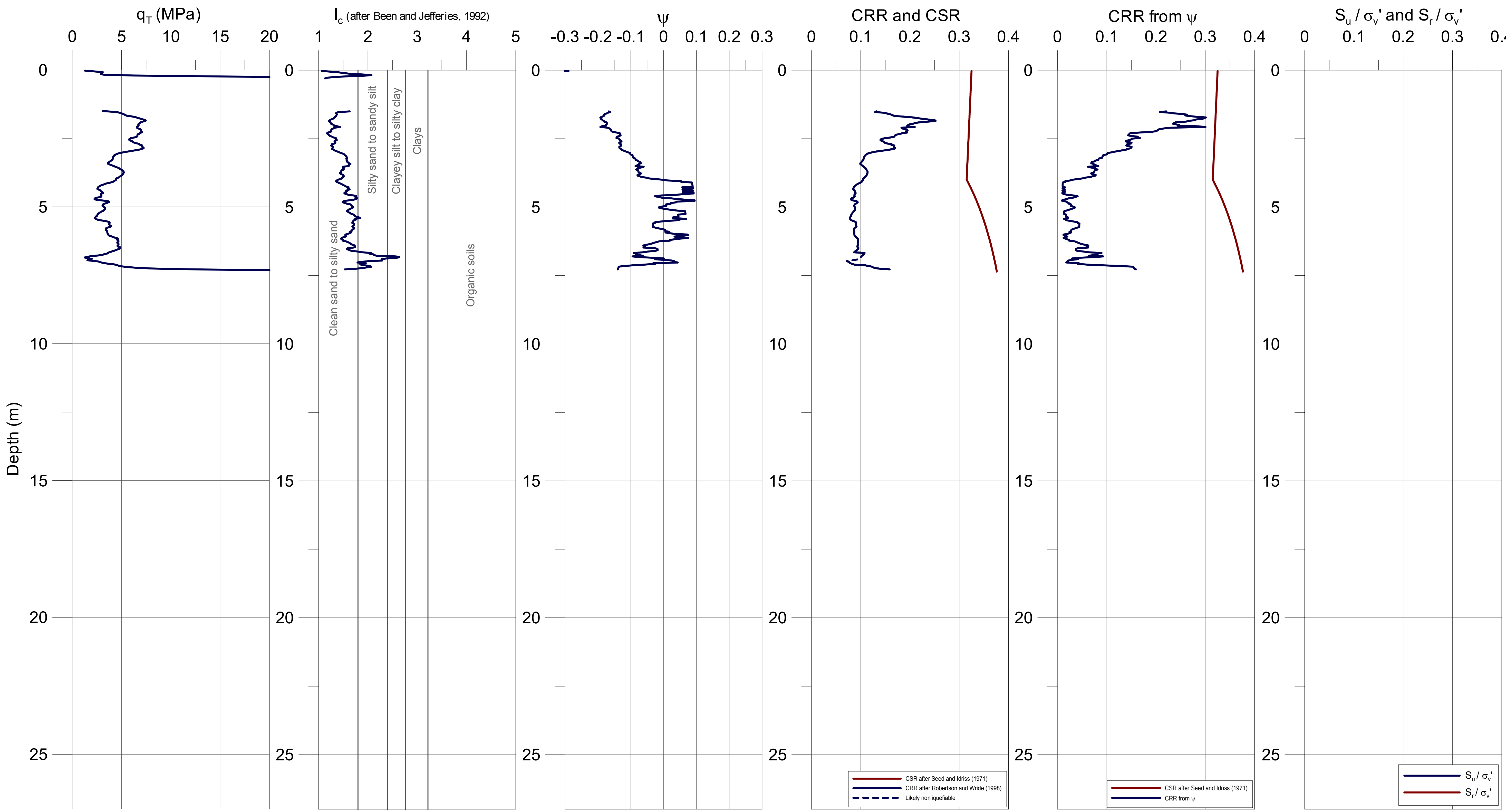
REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-22.grf



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Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-23	
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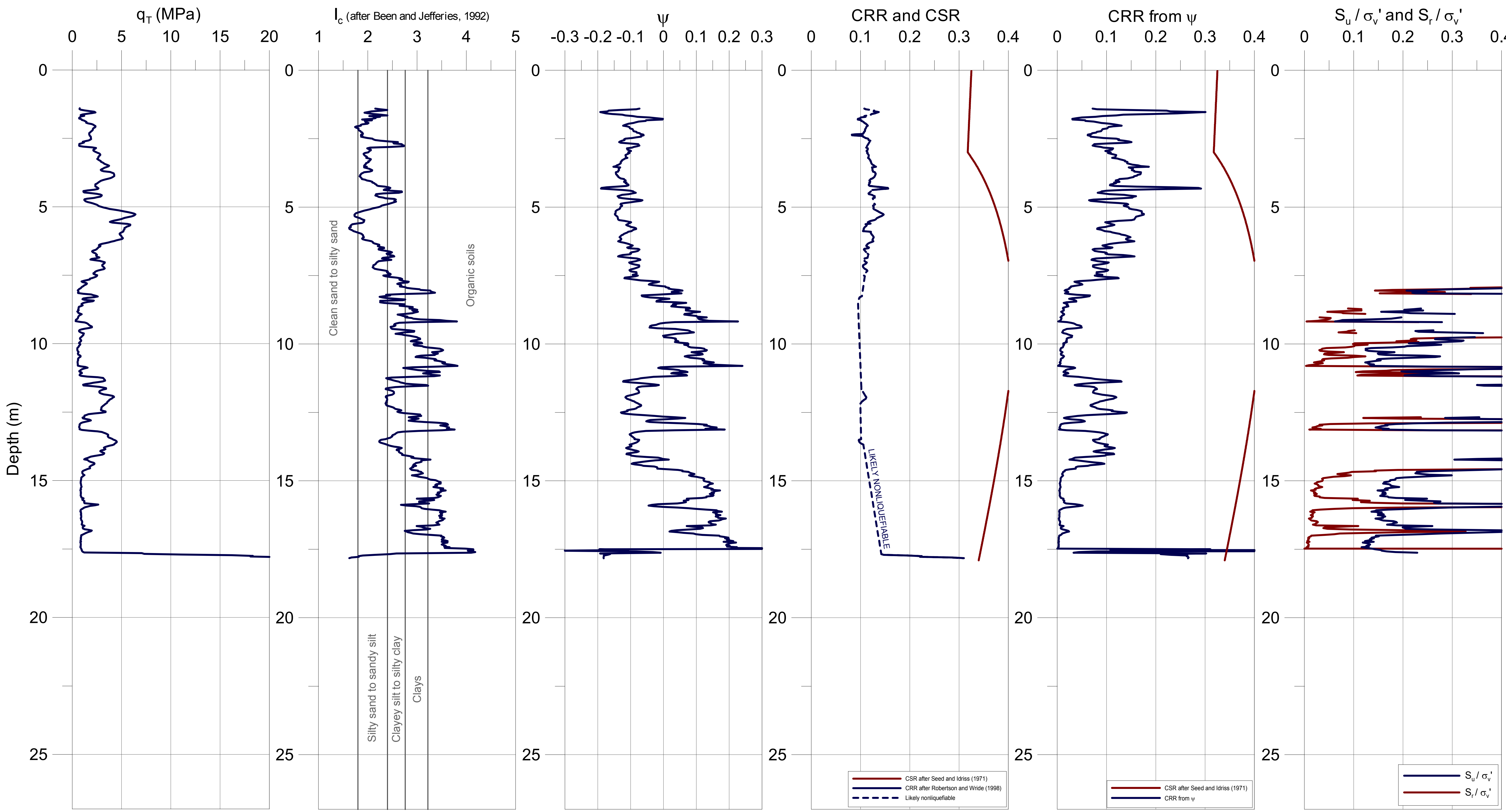

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-24	
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CADD	DRK	29 JAN 04	
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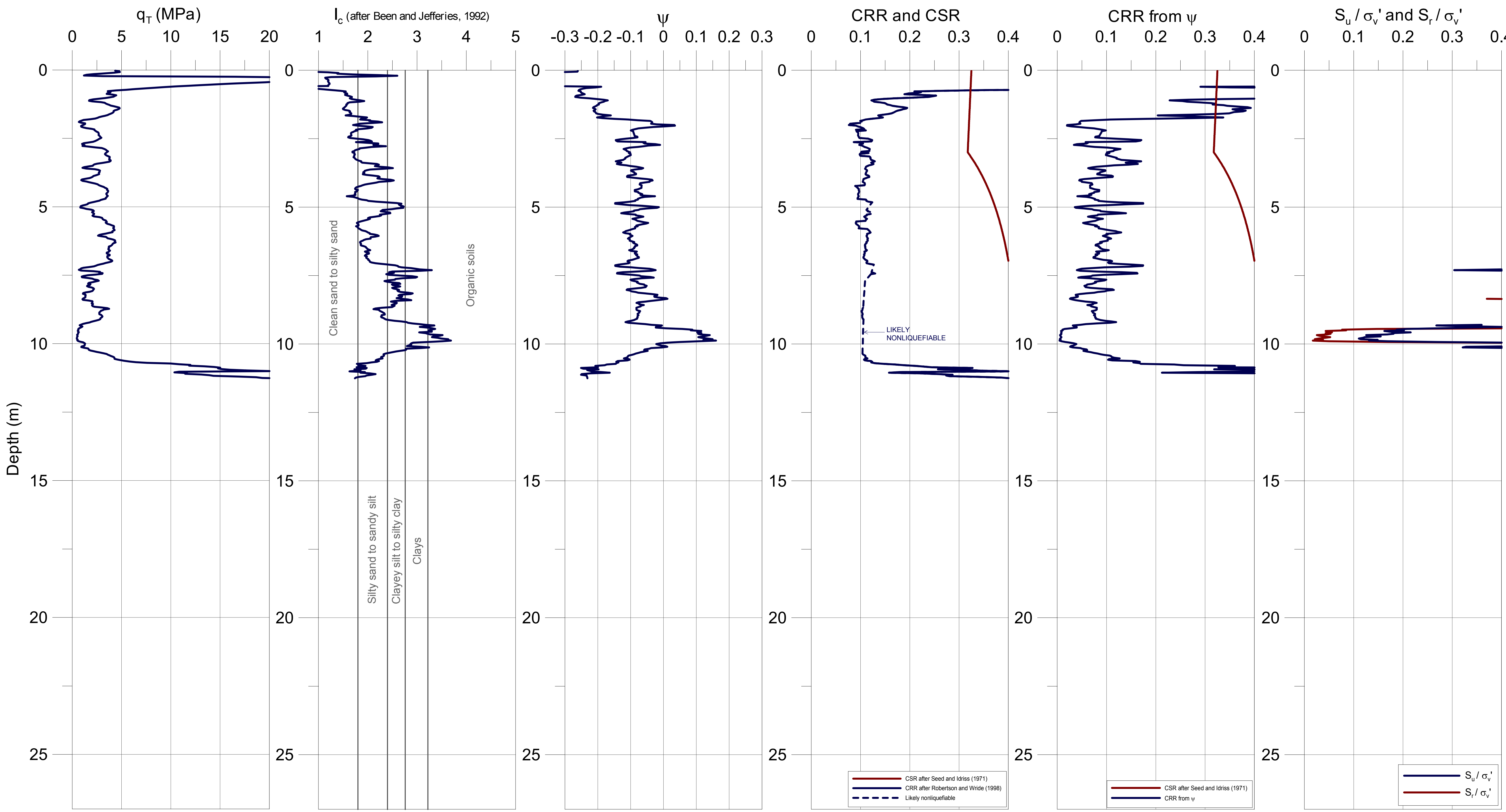
REVIEW	--	--	

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-24.grf



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Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-SCPT03-25
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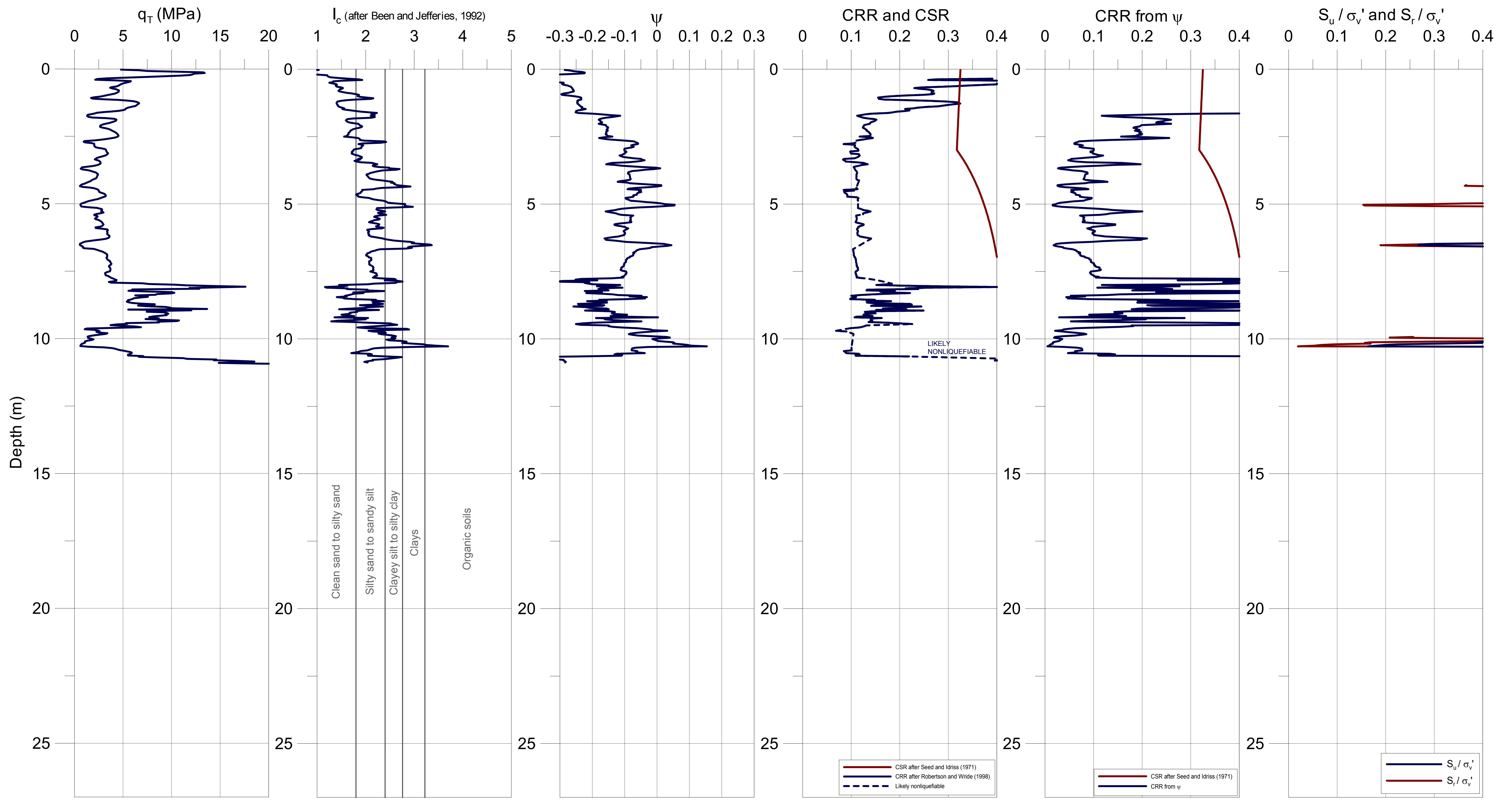

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-26	
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
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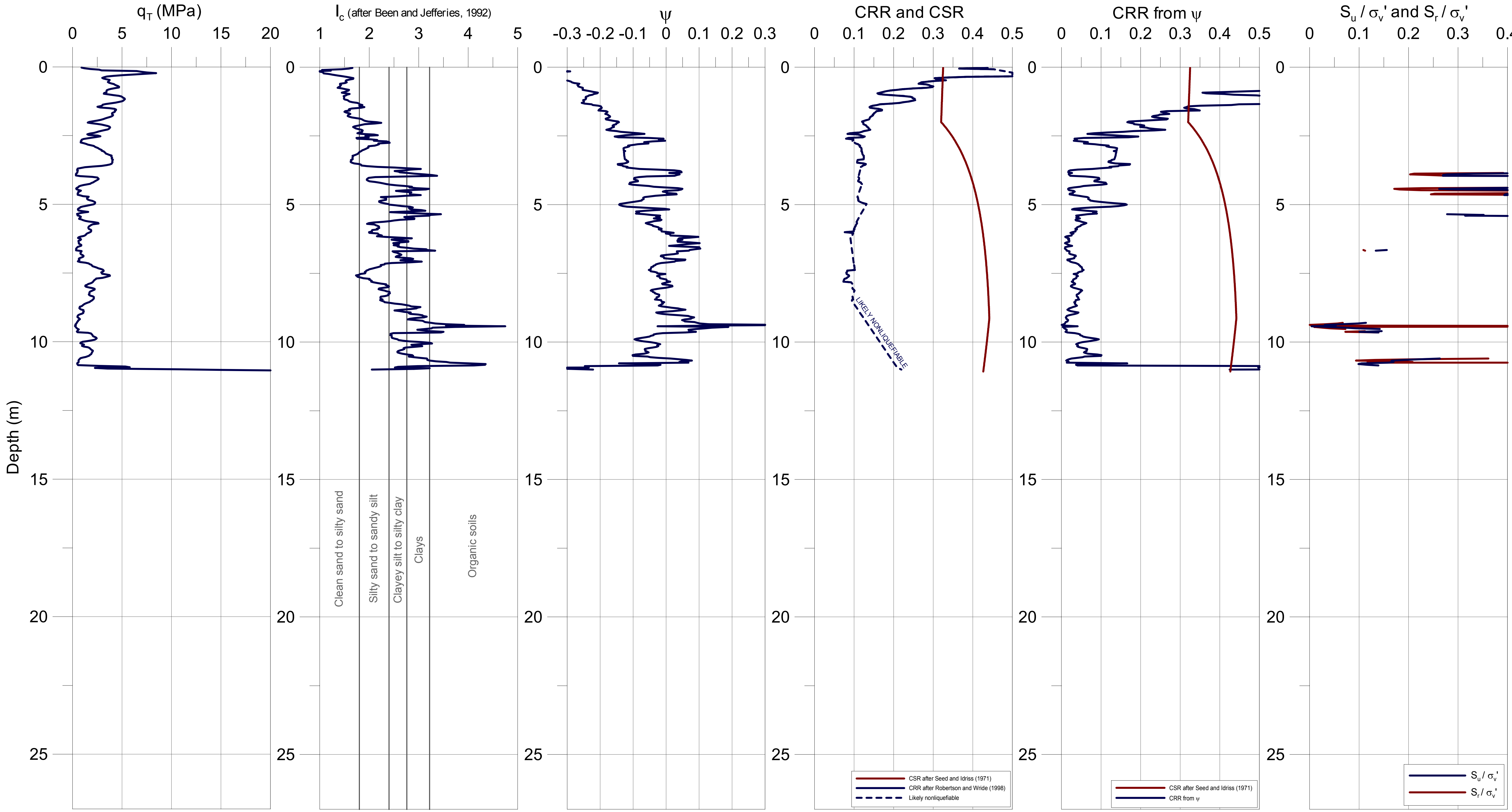
REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-26.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-27
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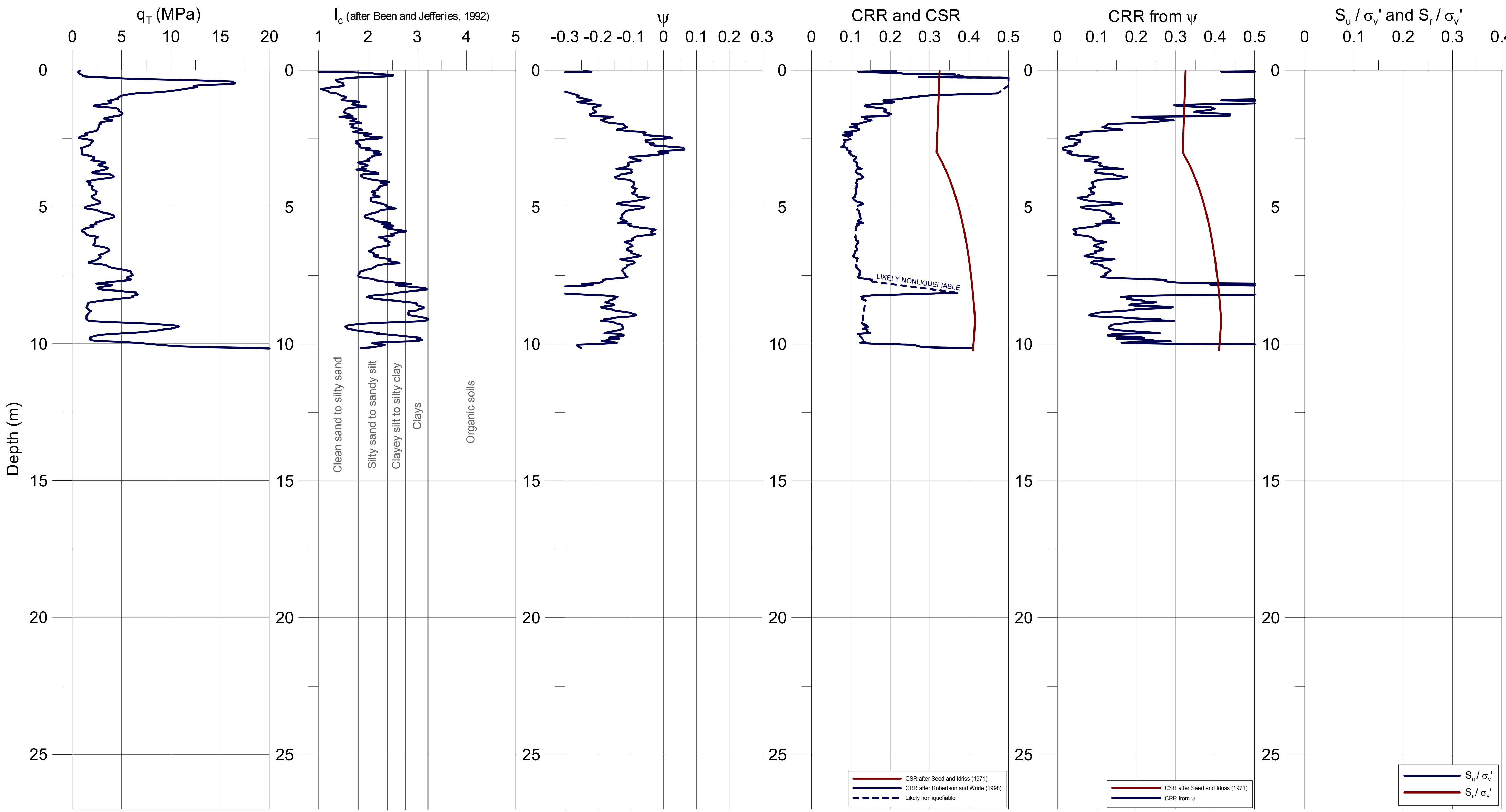

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-28.grf

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

PROJECT

SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE

LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-28	
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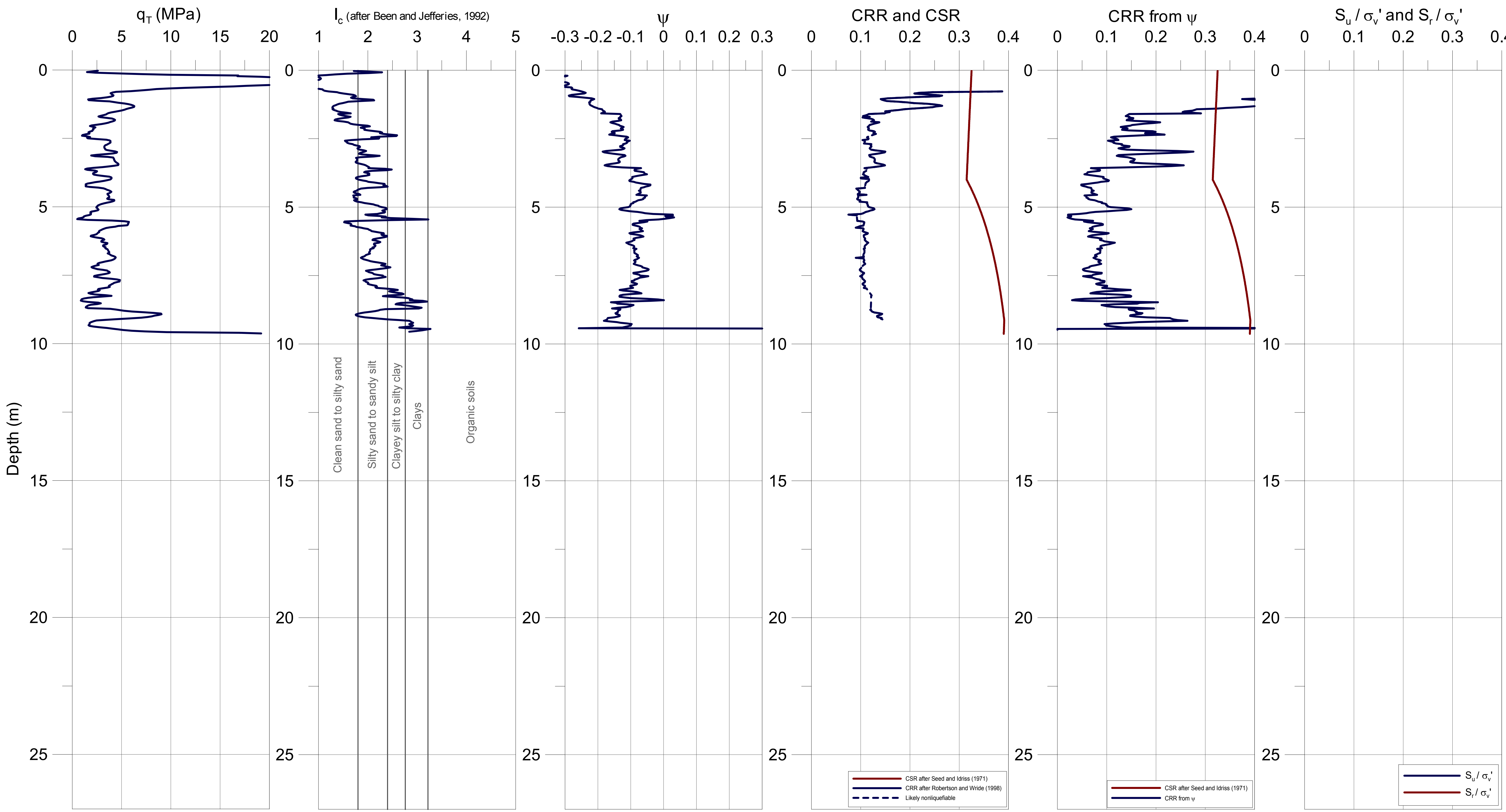

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080	FILE No. plots2-CPT03-29	
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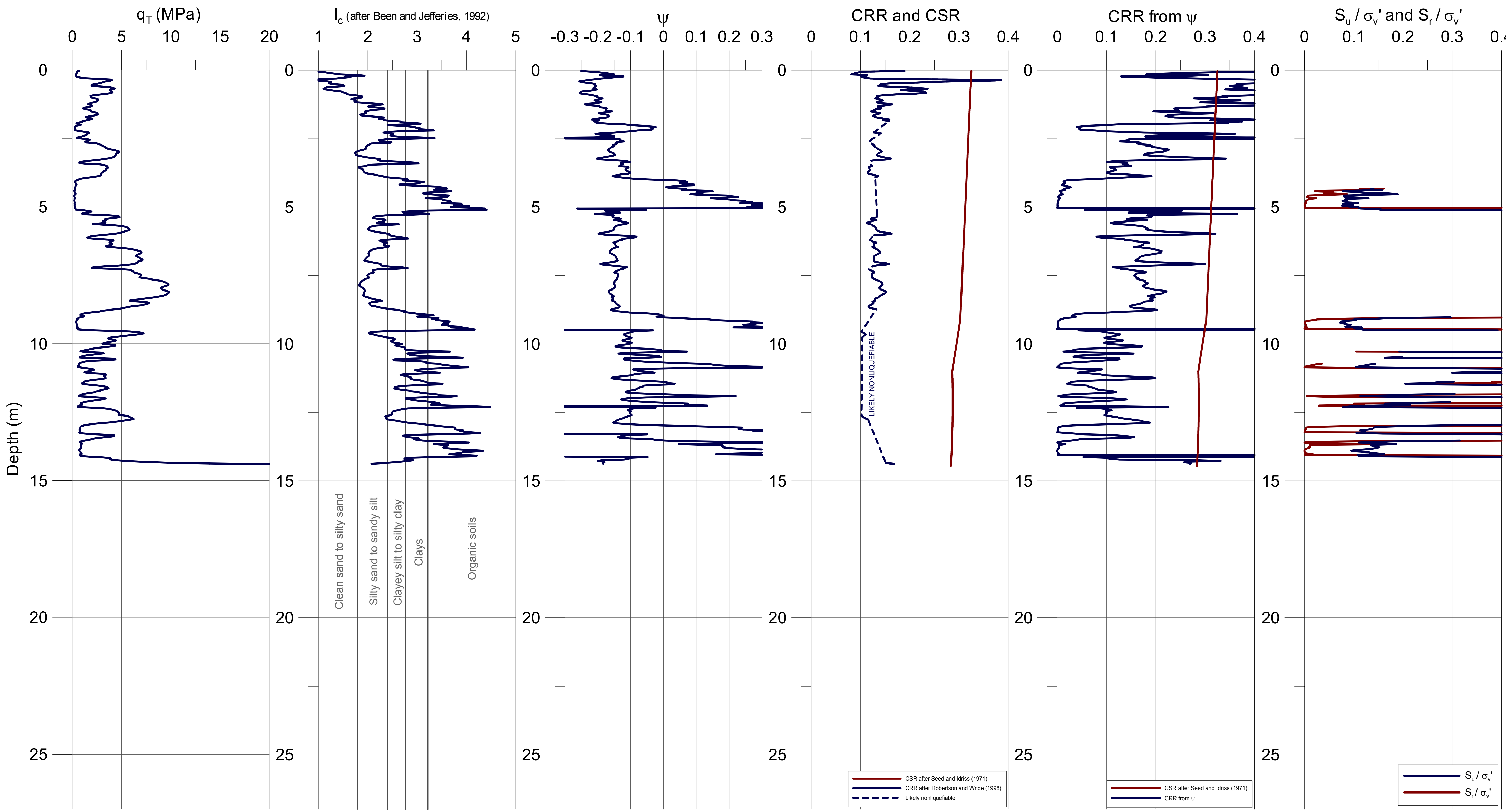
REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-29.grf



Notes

1. For record of CPT see Appendix III
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PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-30	
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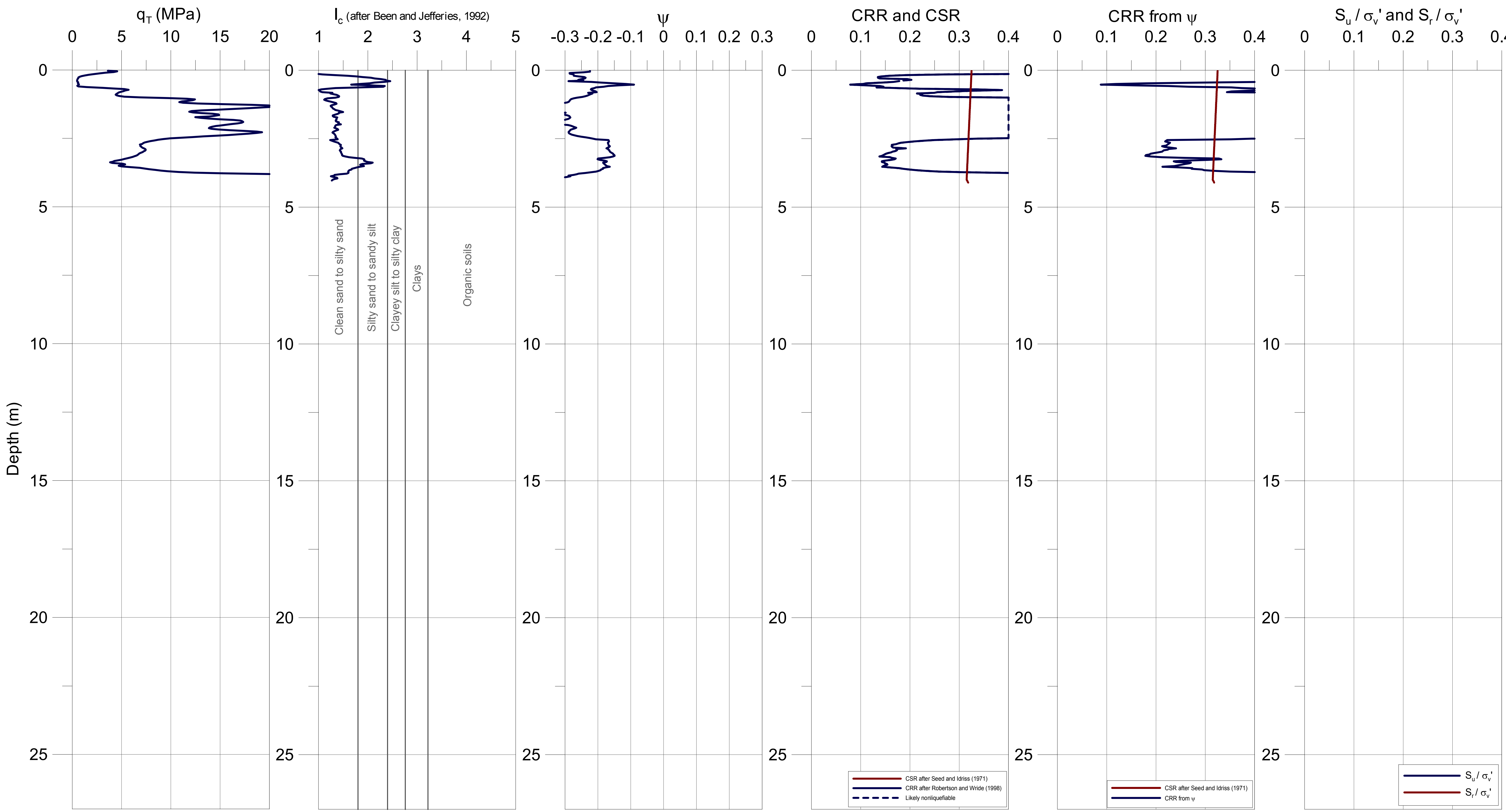
REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-30.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-31.grf

PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No. plots2-CPT03-31	
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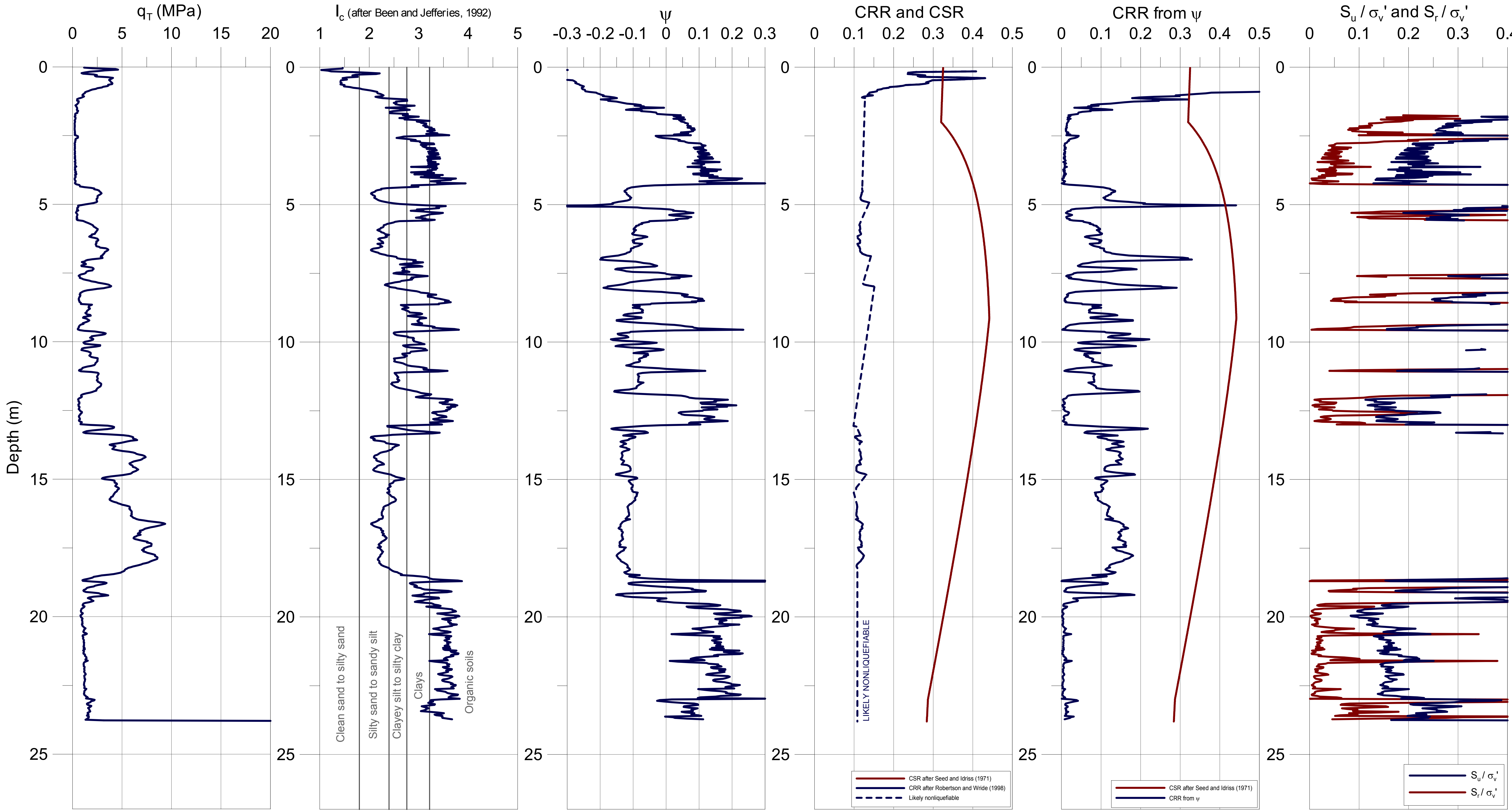

Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
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4. CSR after Seed, H.B., and Idriss, I.M. 1971. Simplified procedure for evaluating soil liquefaction potential. Journal of the Soil Mechanics and Foundations Division, ASCE, **107**(SM9): 1249-1274.
5. CRR after Robertson, P.K., and Wride (Fear), C.E. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal, **35**: 442–459.
6. CRR estimated from ψ (see text of report)
7. S_u estimated from q_T using N_{kT} = 12 (see text of report)
8. S_r estimated from ψ using λ = 0.11, M = 1.2, K_0 = 0.7 (see text of report)

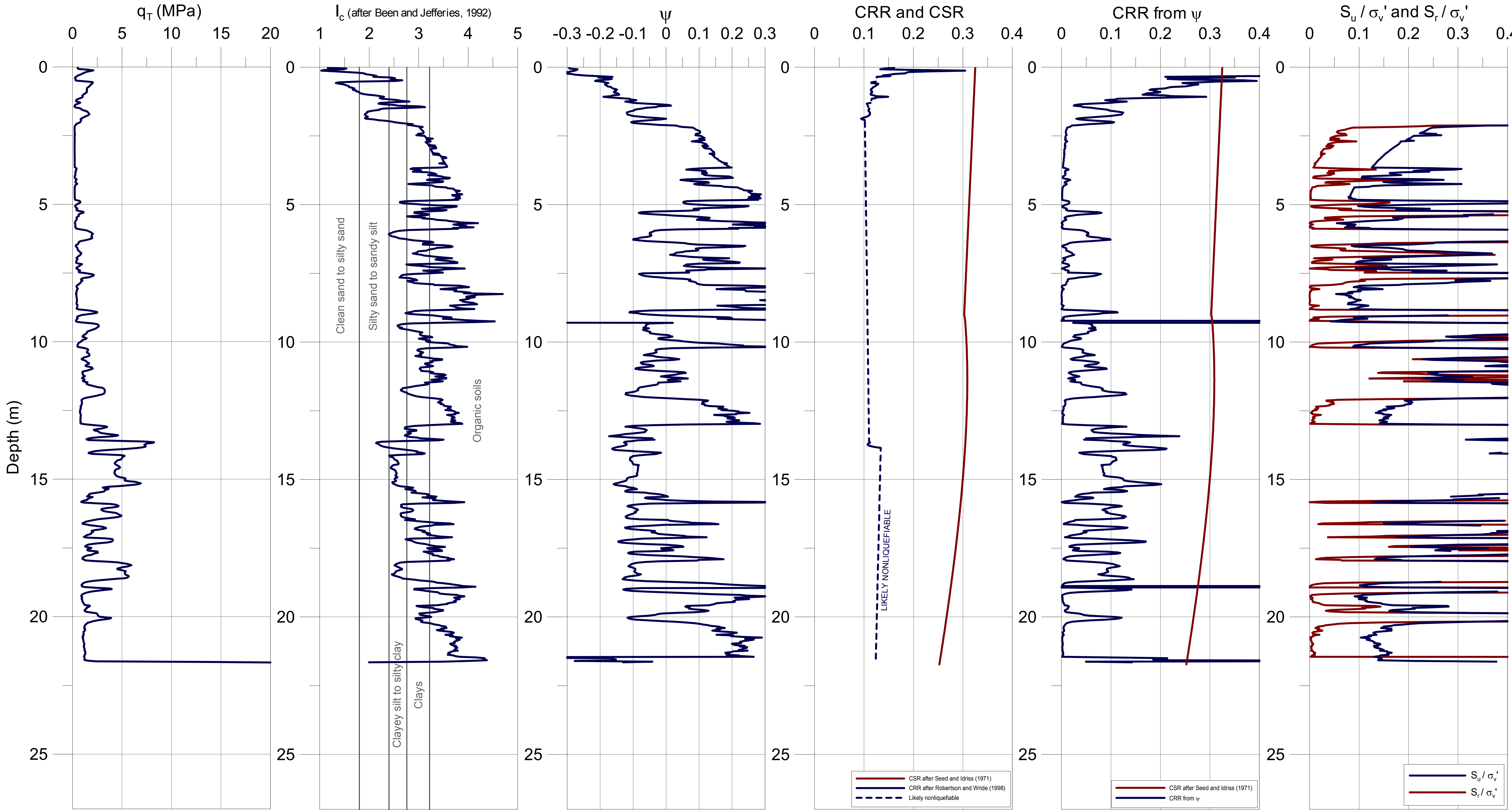
PROJECT
SRK CONSULTING INC.
ROSE CREEK TAILINGS FACILITY
ANVIL RANGE MINING COMPLEX, YUKON

TITLE
LIQUEFACTION AND RESIDUAL STRENGTHS FROM CPT DATA

Golder Associates

PROJECT No.	03-1413-080		FILE No.	plots2-SCPT03-32
DESIGN	DRK	29 JAN 04	SCALE NTS	REV.
CADD	DRK	29 JAN 04	**SCPT03-32**	
CHECK	--	--		
REVIEW	--	--		

REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-SCPT03-32.grf



Notes

1. For record of CPT see Appendix III
2. I_c values and zone boundaries after Been, K. and Jefferies, M.G. 1992. Systematic CPT interpretation. *In* Predictive Soil Mechanics: Proceedings of the Wroth Memorial Symposium, Oxford, UK, pp. 121-134
3. ψ after Plewes, H.D., Davies, M.P., and Jefferies, M.G. 1992. CPT based screening procedure for evaluating liquefaction susceptibility. *In* Proceedings of the 45th Canadian Geotechnical Conference, Toronto, Ont., pp. 4:1–4:9. Adjusted for seismic CPT data and Golder laboratory test data
4. Seismic results based on a_{max} / g = 0.5 and magnitude 7.0 (Gail Atkinson, "Draft Seismic Hazard Assessment for Faro, YK", Dec. 23, 2003)
4. CSR after Seed, H.B., and Idriss, I.M. 1971. Simplified procedure for evaluating soil liquefaction potential. Journal of the Soil Mechanics and Foundations Division, ASCE, **107**(SM9): 1249-1274.
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REVISION DATE: 2 Feb 04 BY: D Klassen GRAPHER FILE: O:\Active\#2003-4\2003\1413\03-1413-080 Faro Tailings\CPT data from ConeTec\plots2-CPT03-33.grf

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REVIEW	--	--		