ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

Intermediate and Secondary Dams

Prepared for

Deloitte & Touche Inc.

On behalf of

Faro Mine Closure Planning Office

M09237A01

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March 31, 2006

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Mr. Doug Sedgwick

Dear Mr. Sedgwick:

Anvil Range Mining Complex, Yukon
Rose Creek Tailings Impoundment
Seismic Stability Assessment

Enclosed are ten (10) copies of our 2004/2005 report on the seismic stability assessment of the Intermediate and Secondary Dams. This report was first issued for comment in November 2004 and then in May 2005. There is a separate addendum, Task 22C, to this report which documents the subsequent investigation work done in 2005 and analyses which resulted from recommendations in this report.

Yours truly,

KLOHN CRIPPEN CONSULTANTS LTD.

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AnvilLiquefactionReport2006.doc
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EXECUTIVE SUMMARY

This report evaluates the seismic stability of the Intermediate Dam and the West and East Limbs of the Secondary Dam which retain tailings in the Rose Creek Valley from the closed Anvil Range mine and mill complex near Faro, Yukon. This report has been prepared as part of the ongoing technical evaluation for the closure planning of the Faro Mine. Aspects of this report may have been superseded by subsequent technical studies. The seismic stability of the Cross Valley Dam which retains a Polishing Pond just downstream of the Intermediate Dam was evaluated in an addendum which was issued separately and after this report. The addendum also includes additional work at the Intermediate and the East Limb of the Secondary Dam.

Both the Intermediate and Secondary Dams are classified as high consequence dams according to the Canadian Dam Safety Guidelines (1999) on the basis that a release of tailings to Rose Creek and to the Pelly River would have severe environmental and economic consequences. This classification requires that the dams be designed for the highest of earthquake loadings which is the Maximum Credible Earthquake, MCE, or the 1 in 10,000 year seismic ground motions.

In 2003 Atkinson set the seismic hazard at this site and found that the median 1 in 10,000 year motions at the NEHRP C firm ground condition was 0.3 g. This was an increase of 50 % over the standard GSC result of 0.2 g which did not explicitly consider that the nearby Tintina Trench was seismically active. Seismic work available in 2003 gave sufficient evidence to show that this zone was likely active. Atkinson produced uniform seismic response spectra at this return period and provided six earthquake acceleration records which matched these response spectra.

The recent seismic stability assessments by Byrne and Seid-Karbasi (2004) and Golder Associates (2004) provide important background to this report. Byrne used the Atkinson seismic loadings and the limited penetration resistance results from the Cross Valley and

Intermediate Dams obtained in the 1980s to conclude that it was possible that the natural foundation soils which support the dams could liquefy in a major earthquake. Golder Associates (2004) did a comprehensive liquefaction evaluation of the tailings only and found that the fine tailings would liquefy during a major earthquake. Golder Associates (2004) estimated seismic displacements of the Intermediate Dam but assumed the foundation soils would not liquefy. Neither of these evaluations are conclusive because penetration resistance values for the supporting natural foundation soils were not available.

One objective of our work was to obtain penetration resistance values for the Intermediate and Secondary Dam foundation soils which could be used for liquefaction assessment. Traditionally, the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT) are used for such evaluations but the known presence of gravel and larger particle sizes in the natural foundation soils precluded the use of these two test techniques. Instead the Becker Penetration Test (BPT) which uses a double acting diesel hammer to drive a 170 mm casing into the ground was used. This test technique has been especially adapted to determine the liquefaction resistance of gravelly soils by Harder (1988) and Sy (1993).

The Harder and Sy methods are alternative techniques for converting raw Becker blowcounts to $(N_1)_{60}$ values which, in turn, are used in liquefaction assessments. There were problems with the diesel hammer misfiring during the program which made data interpretation more difficult than usual. Correlations to site specific data and similar comparisons of Harder and Sy on other sites showed that the Sy results are more reliable so the Sy conversions were used in the liquefaction assessments.

Existing test hole information, aerial photographs prior to site development, and the 14 BPTs in this program were used to develop a surficial geologic model for the Rose Creek foundation soils. The natural ground in the area of interest is now completely covered

with tailings and the dams. The aerial photographs show a glaciofluvial deposit which survived as a terrace about 8 m high primarily on the north side of the Rose Creek Channel. The terrace ends abruptly just upstream of the Cross Valley Dam. Recent Rose Creek sediments have eroded through these glaciofluvial sediments to deposit sands and silts in a meandering stream environment. Both of these units are founded on sediments which have likely been overridden by the last advance of the glaciers in this area. The two dams are entirely founded on the Recent Rose Creek sediments and the glaciofluvial terrace. The underlying sediments are dense and not liquefiable.

The seismic stability of the dams were assessed by first determining which portions of the foundation soils would liquefy and then evaluating the stability of the dams with reduced or residual strengths applied to the liquefied zones. The converted BPT penetration resistance values showed that the terrace or glaciofluvial deposits which form most of the uppermost foundation soils of the Intermediate Dam, the northern foundation of the West Limb of the Secondary Dam, and most of the foundation of the East Limb of the Secondary Dam are not liquefiable. Conversely, the Recent sediments of the Rose Creek beneath the West Limb of the Secondary Dam and the southeast corner of the East Limb are liquefiable. Based on a single Becker test hole, the Rose Creek sediments beneath the Intermediate Dam were found not to be liquefiable. Using the Golders work, we assumed that all of the tailings would liquefy during a major earthquake.

With the tailings left in place along the toe of the West Limb, the Secondary Dam will survive the design earthquake without loss of its retained tailings. With the tailings removed, the section of the West Limb over the Rose Creek channel, about 300 m long, will fail and release its retained tailings. The small section of the East Limb over Rose Creek will also fail and release its tailings. However, this section of the dam is low and a tailings release would be small and would likely flow into the diversion channel.

The Intermediate Dam was found to be stable in both the terrace and Rose Creek sections. There is only a single Becker hole through the Rose Creek section of the foundation. Again, the stability was assessed assuming that the retained tailings would liquefy and have little strength. We consider that more investigation in the Rose Creek foundation section of the Intermediate Dam is required before seismic stability can be confirmed. For planning purposes, we have estimated the cost of remediation of the Rose Creek section of the Intermediate Dam assuming that the foundation would liquefy during a major earthquake.

Conceptual remediation options were developed for the Secondary Dam with cost estimates that are conservative given the lack of drill holes in the area that needs improvement. A compacted berm/drainage alternative option for the 300 m West Limb remediation was estimated to cost $6.4 M. A vibroreplacement improvement of the East Limb would cost about $1.3 million. A berm/densification option for the Rose Creek section of the Intermediate Dam would cost about $4.3 million.

TABLE OF CONTENTS

PAGE

EXECUTIVE SUMMARY I
1. INTRODUCTION 1
1.1 General 1
1.2 Tailings Disposal History 3
1.3 Datum, Grid and Units 5
1.4 Available Information 6
2. SITE CONDITIONS 7
2.1 Seismicity 7
2.2 Last Stages of Glaciation in Rose Creek 8
2.3 Rose Creek Surficial Geology 10
2.4 Groundwater Conditions 12
2.5 Existing Dams 12
2.5.1 Intermediate Dam 12
2.5.2 Secondary Dam 13
2.6 Tailings Liquefaction Potential 14
3. SEISMIC STABILITY METHODOLOGY 16
4. PENETRATION RESISTANCE OF FOUNDATION SOILS 20
4.1 Site Investigation Program 20
4.2 Becker Penetration Tests (BPT) 21
4.3 Standard Penetration Testing (SPT) at the Cross Valley Dam 23
4.4 Laboratory Testing 23
4.5 BPT-SPT Data Interpretation 23
4.6 Converted $(N_1)_{60}$ Penetration Results 25
4.7 BPT Comparison to Penetration Resistance of Tailings 27
4.8 Summary 27
5. SEISMIC RESPONSE ANALYSES 28
5.1 General 28
5.2 Intermediate Dam 30

TABLE OF CONTENTS
(continued)

PAGE

5.3 Secondary Dam 30
6. INITIAL LIQUEFACTION ASSESSMENT 32
6.1 General 32
6.2 Intermediate Dam 32
6.3 Secondary Dam-West Limb 34
6.4 Secondary Dam-East Limb 35
7. SEISMIC STABILITY 36
7.1 General 36
7.2 Flow Liquefaction Analyses for Intermediate and Secondary Dams 36
7.3 Secondary Dam – Remediation Alternatives 39
7.4 Cost Estimate for Secondary Dam Improvements 42
7.5 Intermediate Dam – Remedial Alternatives and Cost 44
REFERENCES 47
SUPPORTING DOCUMENTS 50

TABLES

Table 4.1 Summary of Becker Testing Program 22
Table 4.2 Energy Ratios Used in the Computation of Equivalent SPT $(N_1)_{60}$ Using the Sy Method 24
Table 5.1 Summary of SHAKE Analyses 28
Table 5.2 Average Peak Surface Acceleration and Amplification Factors 29
Table 7.1 Summary of Flow Liquefaction Stability Analyses 37
Table 7.2 Stability Analyses Results with and without the Remediation 42
Table 7.3 Cost Estimate for Vibroreplacement Option 1 for the Secondary Dam West and East Limb Sections 43
Table 7.4 Cost Estimate for Remedial Option 2 with Berm and Drains for the Secondary Dam-West Limb Section 44
Table 7.5 Cost Estimate for Vibroreplacement and Berm Option for the Intermediate Dam 45

TABLE OF CONTENTS
(continued)

FIGURES

Figure 1.1 Location Plan

Figure 2.1 Test Hole Location Plan Pre-Mining Topography

Figure 2.2 Terrace Soil Gradation Curves

Figure 2.3 Intermediate Dam Foundation Stratigraphy

Figure 2.4 Secondary Dam Foundation Stratigraphy

Figure 2.5 Intermediate Dam Typical Section

Figure 2.6 Secondary Dam - West Limb Typical Section

Figure 2.7 Secondary Dam - East Limb Typical Section

Figure 4.1 Test Hole Location Plan 2003 Topography

Figure 4.2 Summary of Equivalent SPT $(N_1)_{60}$ at the Intermediate Dam by Sy and Harder Methods

Figure 4.3 Summary of Equivalent SPT $(N_1)_{60}$ at the Secondary Dam by Sy and Harder Methods

Figure 4.4 Comparison of Equivalent SPT $(N_1)_{60}$ at BPT04-10 and BH80-46 at the Intermediate Dam

Figure 4.5 Comparison of Equivalent SPT $(N_1)_{60}$ at BPT04-14 and BKS04-14 at the Cross Valley Dam

Figure 4.6 Summary of SPT $(N_1)_{60}$ at the Cross Valley Dam

Figure 4.7 Comparison of SPT $(N_1)_{60}$ in Tailings Estimated from BPT Data at BPT04-02 and CPT Data at CPT03-23

Figure 5.1 Peak Acceleration Profiles through the Dam Crest at the Intermediate Dam

Figure 5.2 Peak Acceleration Profiles through the Dam Toe at the Intermediate Dam

Figure 5.3 Peak Acceleration Profiles at the Secondary Dam Toe with Tailings

Figure 5.4 Peak Acceleration Profiles at the Secondary Dam with No Tailings at the Downstream Toe

Figure 5.5 Peak Acceleration Profiles at the Secondary Dam East Limb at BPT04-13

Figure 6.1 Liquefaction Assessment below the Intermediate Dam Centerline

TABLE OF CONTENTS
(continued)

Figure 6.2 Liquefaction Assessment below the Intermediate Dam Toe

Figure 6.3 Liquefaction Assessment at the Secondary Dam (West Limb) Toe with Tailings

Figure 6.4 Liquefaction Assessment at the Secondary Dam (West Limb) Toe without Tailings

Figure 6.5 Liquefaction Assessment at the Secondary Dam (East Limb) at BPT04-13

Figure 7.1 Conceptual Remedial Options for Secondary Dam (West Limb)

Figure 7.2 Conceptual Remedial Option for Secondary Dam (East Limb)

Figure 7.3 Conceptual Remedial Option for Intermediate Dam

APPENDICES

Appendix I Test Hole Logs

Appendix II Site Investigation Photographs

Appendix III Laboratory Test Results

Appendix IV BPT PDA Measurements

Appendix V BPT Data Interpretation

Appendix VI Site Response and Liquefaction Analyses

Appendix VII Stability Analyses and Seismic Displacements

1. INTRODUCTION

1.1 General

This report is an assessment of the seismic stability of the Intermediate and Secondary Dams which impound tailings from the closed Anvil Range lead-zinc mine and mill. The site location is shown on a composite aerial photograph in Figure 1.1 and is approximately 20 km from Faro which is 200 km NNE from Whitehorse in the Yukon Territory. The mill has been shutdown since 1998. This work is part of the closure plan for the mining lease which is managed by Deloitte Touche Inc.

A request for proposal for this assignment was issued by SRK Consulting, Inc who acted on behalf of Deloitte Touche Inc. and were the primary contact during the execution of this work. This work follows that outlined in our proposal of July 16, 2004. Deloitte Touche Inc. authorized this work in a letter dated August 10, 2004. This report was issued in draft in November, 2004 and subsequently in May, 2005. The technical literature used in this report represents that readily available to November, 2004.

The Anvil Range tailings are retained by a series of engineered earthfill dams in the Rose Creek floodplain which is south of the closed mill. The earthfill dams are founded on saturated granular recent alluvial, glaciofluvial, and Pleistocene sediments. Seismic hazard studies by researchers through conference preprints available in 2003 to Atkinson (2003) postulate that the Tintina Fault, which runs along the Pelly River near Faro, is active and capable of generating major earthquakes. These new seismic hazard results raise the possibility that the granular sediments of Rose Creek may liquefy and lead to failure of the earthfill dams after a major earthquake on the nearby Tintina Fault. Assessment of this seismic vulnerability is the objective of this report.

Following the Atkinson (2003) report, Byrne and Seid-Karbasi (2004) reviewed the liquefaction potential of the Intermediate Dam using penetration data acquired at the time

of the construction of the Down Valley Project in 1980 and in light of the Atkinson seismic loadings. Their work indicated that it was possible that the granular foundation soils might liquefy using the Atkinson seismic loadings. However, this was a screening level assessment using only existing, historic data. This current assignment follows from that work but uses new in situ data acquired to specifically address liquefaction potential for the foundation soils of the Intermediate and Secondary Dams. The Byrne and Seid-Karbasi (2004) work did not include the Secondary Dam.

Golder Associates (2004) prepared a report on the liquefaction potential of the tailings. Their work included forty-one CPT profiles in 2003 through the tailings specifically to assess the liquefaction resistance of the tailings. No penetration resistance or other data were acquired in the underlying foundation soils at any location. Their report also included an assessment of the seismic stability of the Intermediate Dam but assumed that the foundation soils of that dam would not liquefy. They recommended further work to fill the gap in foundation information. This is done herein.

The mandate of this report is to determine the seismic stability of the Intermediate Dam and the West and East Limbs of the Secondary Dam using new site investigation data acquired specifically for this purpose. If the seismic stability of the dams is deficient with respect to modern seismic design criteria, then remedial measures are to be designed to a conceptual level with approximate costs. The seismic stability assessment is also to include the Intermediate Dam without the Polishing Pond on the downstream slope and the Secondary Dam without the tailings on the downstream slope.

SRK (2004) have established that the Rose Creek Tailings Impoundments are high consequence structures. According to CDA (1999), this means that their seismic stability must be assessed at an annual exceedance probability of 1 in 10,000. As stated above, Atkinson has provided the seismic loading at this exceedance probability.

1.2 Tailings Disposal History

The Anvil Range mining complex consists of the Faro Mine site, which was in production from 1969 to 1992, and the Vangorda Plateau Mine site, which was in production from 1986 to 1998. The Faro Mine site, which includes the mill and tailings facilities, is located approximately 20 km north of the town of Faro. The Vangorda Plateau Mine site, which includes two open pits and associated mine facilities, is located approximately 9 km northeast of the town of Faro and can be reached by a 12 km haul road from the Faro Mine site. This rendition of the site history is based on the extensive reporting by others.

The Faro Mine was first started by Anvil Range Corporation and began production in 1969 at 5000 tonnes of zinc-lead-silver ore per day. The production increased to 6000 tonnes per day in 1970. In 1974, a mill expansion allowed a further increase in ore production to 9300 tonnes per day. In 1979, Cyprus Anvil purchased the mineral deposits and claims including Grum and Vangorda and then embarked on a program of expansion to bring the Vangorda Plateau deposits (Vangorda and Grum) into production to supplement the Faro Mill feed. Cyprus Anvil ceased production in 1982. The property was shut down and remained idle until the operation was acquired by Curragh Resources in November 1985. The mine facilities were reactivated in December 1985 with a production rate of 13,500 tonnes per day. The deposit was depleted of economic ore reserves in 1992.

Mining of the Vangorda Deposit began in 1990 and 5.7 million tonnes of ore were mined from 1990 to 1993 by Curragh Resources. 52,000 tonnes of ore was mined from the Grum Pit by Curragh Resources prior to the temporary mine shut down in 1993.

Anvil Range Mining Corporation assumed ownership of the mine site in November 1994 and resumed pre-production stripping at the Grum open pit and mining in the Vangorda

open pit. By early 1998, mining in the Vangorda open pit was completed, and mining in the Grum open pit was partially completed.

Anvil Range Mining Corporation entered into receivership in April 1998. The mine sites have been shut down and under the management of Deloitte & Touche Inc., acting as the court-appointed interim receiver, since that time.

From mill startup to 1992 tailings were deposited in Rose Creek Valley south of the mill as shown in Figure 1.1. From 1992 to 1996, tailings were deposited in a spent open pit. Figure 1.1 shows the major tailings retention structures in Rose Creek Valley that are described below.

Tailings are retained in the Rose Creek Valley behind three retention dykes; the Original Dyke; the Secondary Dam, and the Intermediate Dam. Downstream of the Intermediate Dam is a Polishing Pond which is retained by Cross Valley Dam. Rose Creek is diverted around the tailings by a diversion canal on the south side of the valley.

The alignment of the original tailings dyke is shown on Figure 1.1. The original starter dyke was constructed on natural ground on the north side of the Rose Creek Valley starting near the old Faro Creek confluence with Rose Creek. It appears that diversion works for Rose Creek during that stage of development were minimal. The tailings retention dyke was constructed of uncompacted rockfill and raised using upstream construction. According to Golder (2004), the original dyke was 20 m high including the starter dyke. This original dyke operated until 1973 and was subsequently inundated with tailings deposited behind the Secondary Dam. The downstream slope of the original dyke is buried by tailings with only the crest alignment visible. The original tailings dyke failed in 1975 losing tailings into the Rose Creek Valley (Gartner Lee, 2001). The seismic stability of the original dam is not an issue as it is completely inundated by tailings.

The Secondary Dam was designed by Golder Associates and construction was started in 1974. The West Limb of the Secondary Dam is defined as that curved portion which crosses the valley roughly north to south. The East Limb is that portion of the Secondary Dam whose alignment is parallel to the south valley wall. The West Limb of the Secondary Dam was raised using centerline construction and compacted fill to a maximum height of 27 m. The East Limb is a low retention dyke which was constructed of compacted fill. Tailings were deposited behind this dam from 1975 to 1982 and then for approximately 5 months in 1986. Rose Creek was diverted to a channel which ran parallel to the south valley wall and emptied into its original channel downstream of the Secondary Dam.

The Intermediate Dam was constructed as part of the Down Valley Project in 1979 to 1981 to create additional tailings storage. The Down Valley Project consists of the Intermediate Dam, the Rose Creek Diversion and outlet channels, and the Cross Valley Dam which retains the Polishing Pond. Rose Creek was diverted around the tailings area while tailings transport water and direct precipitation were conveyed to the Polishing Pond by a spillway on the north abutment. The Polishing Pond was designed for 60 day retention before release. Currently, the Intermediate Pond and Polishing Pond discharges are treated before release.

1.3 Datum, Grid and Units

As with most historic mining sites in Canada, there are several elevation datums and drawings are in both imperial and SI units. In this report we use SI units unless specifically noted and the mean sea level datum. The site UTM grid is used on all drawings in this report. In the tailings area there is the Down Valley Datum which is converted to mean sea level by subtracting 32 m or 106.0 ft. The surveyors reported borehole locations to mean sea level and the UTM grid. There is also a mine datum which is converted to mean sea level by subtracting 33.3 m or 109.2 ft.

1.4 Available Information

SRK made all information in their possession available to us for the purposes of this assignment. We viewed the information in their offices and copied excerpts of documents that were relevant to this assignment. This information is listed at the end of the report under the heading "Supporting Documents" as compared to "References" which are generally technical publications available in the public domain.

After issue of a draft of this report in November of 2004, SRK obtained from Golder Associates test hole logs which were drilled in the 1970s through the early 1980s along the Secondary Dam alignment. However, neither a complete set of historic test holes nor the as-built drawings for the Secondary Dam were obtained.

2. SITE CONDITIONS

2.1 Seismicity

Atkinson (2003) did a seismic hazard assessment of the site to estimate earthquake loading at an exceedance probability of 1 in 10,000. The starting point for any seismic hazard assessment in Canada is the Geologic Survey of Canada (GSC) per Adams and Halchuk (2003) which is an update of the National Building Code of Canada seismic hazard for design of buildings. The GSC work is directed towards annual exceedance probabilities of 1 in 2500. Klohn Crippen (2002) extrapolated their work as did Atkinson to 1 in 10000 which gave a horizontal peak ground acceleration (PGA) of 0.19 g for firm ground NEHRP C site conditions.

The GSC has two models, the H and R models. The H model is based on historical seismicity while the R model is based more on regional tectonics. Their resulting value is the higher of the two models which is referred to as the Robust model. The GSC warns that extrapolation of their seismic model to 1 in 10,000 motions may not be appropriate and that site specific studies are often required. In this case, the Tintina Trench which is about 10 km from site and which is a known tectonic boundary between accreted terrain on the west and the continental plate on the east is not specifically accounted for in the GSC model. This is because seismic activity along this tectonic boundary on which the GSC model is based was thought to be not much different from the background seismicity in the Southern Yukon Zone. However, recent work suggests that seismic activity on the Tintina Trench is greater than background and should be treated separately from the background zone for this part of the Yukon.

Atkinson refers to recent preprints of papers by Cassidy et al (2003) and Hyndman (2003) which report that there has been 450 to 1000 km of right lateral displacement along this fault over the last 65 million years which is a slip rate of 8 mm/ year. They also cite GPS data that indicates current slip rates of up to 5 mm/year. This is significant because it

indicates that the fault is active. Corroborating this is a study by Doig (1998) which suggests that there may have been two shaking events that disturbed sediments along the Tintina Trench in the last 1000 years.

Atkinson used Frisk 88 to estimate the seismic hazard at the site. Frisk 88 is a computer program based on the Cornell-McGuire method which also treats epistemic or model uncertainty. In short, Frisk 88 allows weighting of different seismic models to give, in effect, a blended result. Atkinson gave a 1/3 weight to the GSC model which does not include a separate Tintina Trench zone, 1/3 weight to a Tintina Trench treated as a zone, and 1/3 weight to the Tintina Trench treated as a fault source. All other parameters were as in the GSC model.

Atkinson presents the results as response spectra (spectral acceleration versus frequency) for the mean, median, 16^{th} and 84^{th} percentiles. At 50 Hz or peak ground acceleration, Atkinson obtained 0.3 g for the median motion which compares to 0.19 g for the unaltered, extrapolated GSC model. This means that incorporation of possible Tintina Trench activity increases the seismic hazard by 50% at the median.

Atkinson also chose acceleration records for seismic analysis which fit the target or calculated spectrum for M7 earthquakes at 10 to 20 km. Atkinson chose actual records that match the entire spectrum as well as possible, without scaling to any point on the target spectrum. This results in the use of acceleration records with PGAs ranging from 0.3 to 0.55 g. We used these records without spectral matching in the site response analyses described in Section 5.

2.2 Last Stages of Glaciation in Rose Creek

The recent glacial history of the Rose Creek area has been well summarized by Gartner Lee (2002) which will be paraphrased in the following. The writer also met with the

Yukon Geological Survey during the field campaign to get their latest published work on the area which is a 1999 surficial geology map at a scale of 1:25,000.

The last widespread glaciation of the eastern Yukon was the McConnell glaciation whose extent is documented in Bond (2001). The basal till which blanket the schist bedrock beneath the Rose Creek valley is thought to have been deposited by the McConnell Glaciation. This glacial till and, of course, the underlying bedrock are not liquefiable so deserve no further mention. As the McConnell glaciation retreated, glacial outwash sands and gravels were deposited in the Rose Creek valley on the basal till. During a brief interglacial period, some erosional down cutting of the McConnell outwash sand and gravel likely occurred. This erosional channel became blocked at some point causing a glacial lake and deposition of the channel with glaciolacustrine silt and fine sand. However, no continuous glaciolacustrine deposits were found during this or earlier investigations.

The Yukon Geological Survey considers that there was a late re-advance of the ice in this area, called the Pelly re-advance, which was more depositional than erosive. The Pelly re-advance would have deposited the glacial tills on the rock slopes above the Rose Creek floodplain but may not have eroded all of the McConnell glacial outwash sands and gravels in Rose Creek valley. As the Pelly glacier retreated, another sequence of sands and gravels were deposited. An intermediated glacial till layer may be presented in the deeper sediments well above the rock but no well-defined horizon has been found.

If the McConnell outwash sands and gravels were not completely eroded, then the Pelly ice re-advance would have compressed these deposits, perhaps causing them to be so dense so as not to be liquefiable. Finding this potential liquefaction/non-liquefaction horizon was one objective of our Becker penetration program in 2004.

Following the Pelly retreat, another outwash sequence of sands and gravels was deposited in Rose Creek Valley. These outwash sands and gravels form the dry[1] terraces on the north side of the valley which will be described in the next section. Following deglaciation, Rose Creek established itself in the valley by incising into the Pelly outwash sands and gravels forming the terraces. As the creek meandered across the valley, small oxbow ponds and backwaters were formed that contain fine-grained, organic sediments. Typically, these finer grained sediments are of limited extent but occur throughout the section.

2.3 Rose Creek Surficial Geology

Figure 2.1 shows the locations of all drill holes available to us that penetrate the foundation soils beneath the tailings and the retention dams. The Golder drill holes for the Down Valley project and the 1980 Klohn Leonoff rotary, mudded drill holes were drilled prior to tailings deposition downstream of the Secondary Dam. The Gartner Lee sonic holes and our Becker penetration holes were drilled through the tailings. Our Becker penetration logs are included in Appendix I. A description of the drilling program is given at the beginning of the next section.

Figure 2.1 is a composite 1967 aerial photograph prior to tailings deposition with superimposed, original contours and outlines of the retention dams for reference. The north terrace which straddles the valley is clearly visible on this figure. This terrace is about 8 m high and, as evident from the aerial photographs[2] to the east, extends from the confluence of the north Rose Creek to just upstream of the Cross Valley Dam which is mostly founded on recent Rose Creek sediments underlain by McConnell outwash deposits.

[1] By "dry" is meant that the water table in the terrace was at the elevation of Rose Creek prior to site development
[2] These eastern aerial photographs are not part of Figure 2.1

Note that the airstrip was located on this terrace which indicates that the terrace was relatively flat. This terrace was used as a borrow source for the sands and gravel fill of the Intermediate and Cross Valley Dams. Since these borrow sources were at the toe of the terrace, the water table would have been below this level. Gradation curves from Golder test pits in the terrace sands and gravels are given in Figure 2.2.

The Rose Creek channel prior to tailings deposition is relatively broad at the Secondary Dam with terrace sands and gravels on both sides of the west limb. The terrace deposits are present the entire length of the east limb of the Secondary Dam to the corner where BPT 04-13 was drilled. At this location, there is a tributary stream from the south valley wall which intercepts Rose Creek at exactly this location. North across the Rose Creek Valley are remnants of the terrace with continuous terrace at the north valley wall. At the Intermediate Dam the terrace gravels extend from the north across the valley to a narrow Rose Creek channel

A section along the Intermediate Dam crest is shown on Figure 2.3. A section along the toe of the Secondary Dam is shown on Figure 2.4. The Intermediate Dam section shows the original Golder geologic interpretation with a few minor changes. The geologic units shown are those based on the surficial geology in the previous section. On the right abutment are the terrace or glaciofluvial deposits to about El. 1005 which is the elevation at which the measured Becker blowcounts in BPT04-09 and BPT04-08 increase. Below this elevation are likely Pleistocene deposits which are not liquefiable.

On Figure 2.4 along the Secondary Dam the Rose Creek channel deposits cut much deeper into the section and through the terrace deposits. The terrace deposits are shown along the east limb which is based on an interpretation of the aerial photographs.

From this geologic interpretation, we consider that all soils shown as Pleistocene are not liquefiable. This is self-evident given the high blowcounts but is confirmed in a later

section using corrected blowcounts. The Rose Creek and terrace deposits may be liquefiable and this evaluation is described in the next section.

2.4 Groundwater Conditions

The groundwater pressures in the aquifer beneath the tailings are controlled by the hydraulic gradient from the return water pond elevation upstream of the tailings at about El. 1057m to the polishing water pond at El. 1031 m. This is a decrease in total head of about 26 m over 3300 m or a gradient of 0.0078. Everywhere along the tailings this hydraulic grade line is well below the surface of the tailings and likely well below the water level in the tailings. This means that there is a downward hydraulic gradient in the tailings to the aquifer. All foundation soils are saturated.

At the time of construction of the Down Valley Project, it was reported by Golder (1980) that there was scattered frozen ground in the foundation soils of both the Intermediate Dam and the Cross Valley Dam. For this report, we assume that all frozen ground has thawed.

2.5 Existing Dams

2.5.1 Intermediate Dam

The Intermediate Dam was constructed across the Rose Creek valley to contain supernatant water and tailings solids in 1981, and raised in 1988, 1989 and 1991 to a final crest elevation of 1049.4 m amsl, a maximum height of 32 m above the old Rose Creek channel. A typical section through the dam is shown on Figure 2.5. This is now the main tailings impoundment structure for the Anvil Range mining complex. There is an emergency spillway channel on the right abutment, constructed as a rock lined channel in natural overburden material

The Intermediate Dam is a zoned earthfill embankment with a central low permeability core designated as Class VI, and a downstream Class VIII random fill shell. Granular filter zones were constructed on both sides of the core (Class X and Class XI). The initial construction in 1981 raised the ultimate dam footprint to elevation 1031.7 m amsl and made use of natural terrace material present across the valley. A drainage blanket was placed over this footprint downstream of the core and filter zone, extending to the toe of the ultimate height dam. Upstream and downstream slopes are at 2H:1V. Including the 20 m wide berm adjacent to the Polishing Pond, the overall downstream slope is at 2.1H: 1V.

There is a nominal cutoff for seepage reduction through the uppermost soils beneath the core of the dam. There is no deep foundation cutoff to reduce seepage through the permeable soils. Instead, the tailings on the upstream slope and foundation were relied on to reduce seepage to an acceptable level. Note that the drains on the dam section are set to just above the design level of the Polishing Pond. With the Polishing Pond in place, the head across the dam is 16 m. If the Polishing Pond is drained, the head will double to 29 m. This would double the gradient across the dam.

2.5.2 Secondary Dam

Sections through the Secondary Dam, West Limb are shown on Figure 2.6 and sections through the East Limb are shown on Figure 2.7. The starter dam was constructed in 1974 and 1975 to El. 1051.6 m (Robertson, 1996). The sections refer to dam stationing but a plan that shows stationing was not available to us. The section shows that the starter dyke has a central impervious core, a filter on the downstream side of the core, and a blanket toe drain. Thereafter, the dyke was raised with compacted tailings sand and sand and gravel using centreline construction to El. 1060.7 m by 1980.

There is no foundation cutoff to reduce seepage through the dam. Golders designed this structure so that seepage would be reduced to acceptable values by the tailings. Also, it is standard practice to strip and proof roll dam foundations prior to fill placement which may explain the high blowcounts in our Becker test hole BKS04-03. Alternatively, this high blowcount zone could be the downstream toe of the dam.

The East Limb of the Secondary Dam was constructed of compacted tailings with a typical height of 4.3 m using downstream construction. The final lift was capped with a protective layer of sand and gravel fill, and provided with a toe berm. The downstream slope is at 2H: 1V. There is no indication of a foundation cutoff on the drawings.

2.6 Tailings Liquefaction Potential

Golder Associates (2004) describe their cone penetration testing program in the tailings and liquefaction evaluation of the tailings using the Atkinson seismic hazard results. They obtained conventional cone profiles at 36 locations for a total length of 482 m of cone soundings. At another five locations they obtained shear wave velocity measurements with a seismic cone. At 41 locations, they measured pore pressure dissipation with a pore pressure transducer in the cone tip. They assessed the initial liquefaction potential of the tailings using the state parameter approach and, also, using the standard NCEER methodology. They also assessed the flowslide potential of the tailings which they found could be classified as tailings sands or tailings silts with rapid changes between these two end members in a single borehole. Their description of the tailings deposition changes over time is the only available such account for this site which was available to us.

The NCEER methodology predicted that the tailings sands would liquefy but that the tailings silts would not liquefy. The state parameter approach predicted liquefaction in both the tailings sands and silts. Golders contend that most of the tailings sand would not

lose much strength when liquefied but the tailings slimes would lose at least 90 percent of their strength.

For the purposes of this assignment, we have assumed that the entire tailings profile upstream and downstream of the Secondary Dam and upstream of the Intermediate Dam would liquefy during a major earthquake. Based on Golders comprehensive test data, we have assigned normalized residual strength values to the tailings for use in seismic stability analysis described in later sections.

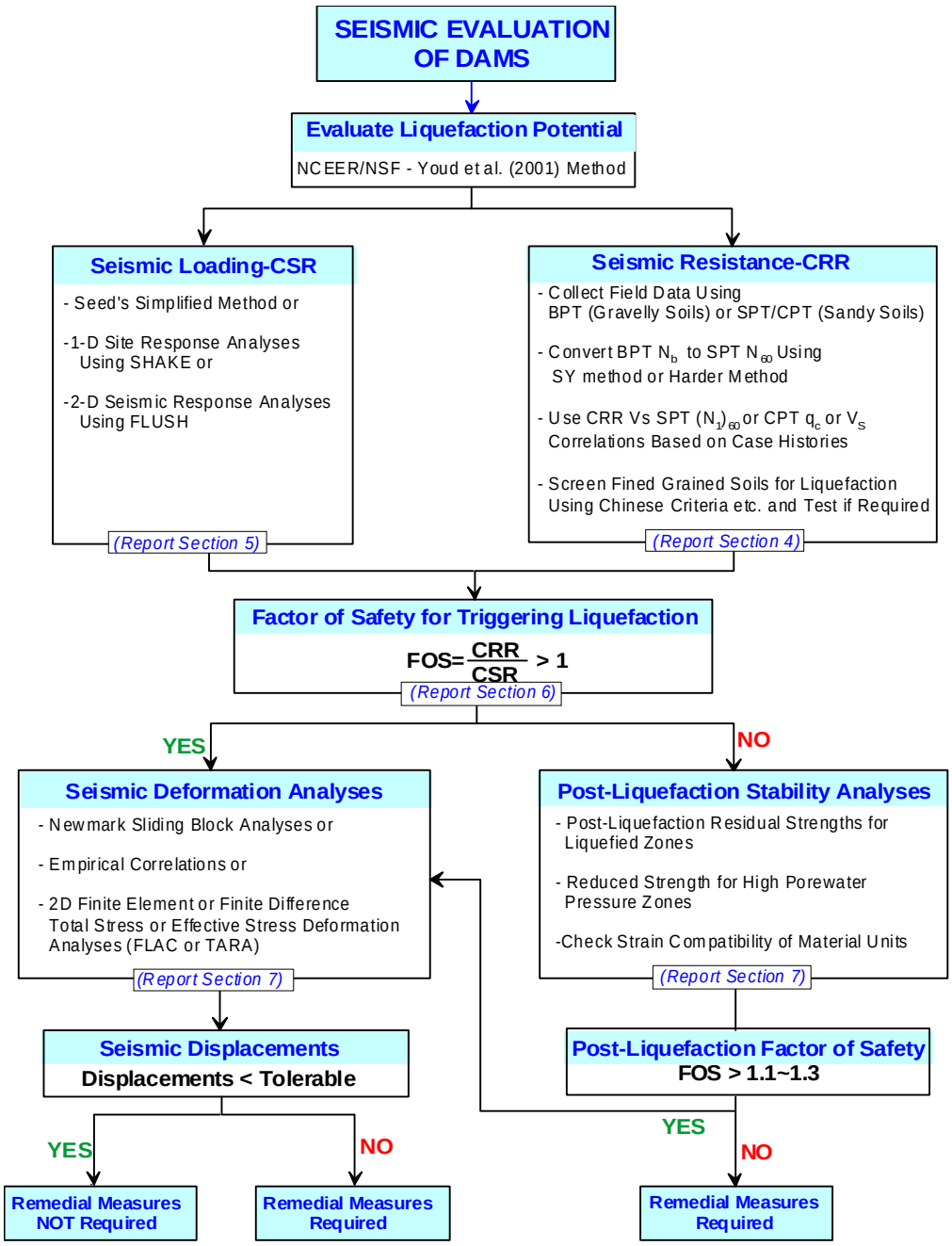
3. SEISMIC STABILITY METHODOLOGY

The flow chart on the following page shows our general analytical approach to evaluation of the seismic stability of the Intermediate and Secondary Dams. This methodology follows from Youd et al. (2001) which, in turn, is based on the work pioneered by H.B. Seed at the University of California in Berkeley. This flowchart assumes that characterization of seismic hazard and surficial geology is sufficient to proceed with analysis. The first step is to determine whether the foundation soils will liquefy under the design earthquake. By liquefy is meant reduction of the effective stress in the saturated foundation soils to zero during the earthquake. This is evaluated using Equation 3.1.

$$FOS = \frac{CRR}{CSR} \qquad 3.1$$

The CRR is the cyclic resistance ratio which is a measure of the resistance of the soil to liquefaction. The CRR is usually scaled from charts of $(N_1)_{60}$ or normalized cone penetration test (CPT) resistance versus cyclic stress ratio for sites that have and have not liquefied in past earthquakes. The Becker penetration test was used to determine the penetration resistance of the foundation soils. The Becker was chosen because existing test hole information indicated large particle sizes in the foundation soils which conventional SPT and CPT cannot penetrate. The Becker penetration test values need to be converted to $(N_1)_{60}$ values so that the standard charts in Youd et al (2001) can be used. The method proposed by Sy (1993) for converting BPT blow counts to equivalent SPT N_{60} values was used in this report. The acquisition of the Becker blowcounts at the Intermediate Dam and the Secondary Dam with their conversion to $(N_1)_{60}$ is described in Section 4.

CSR or cyclic stress ratio is a measure of the imposed seismic stresses at the site from the design earthquake. We used a simple one-dimensional analysis embodied in the industry standard SHAKE computer program to determine CSR. SHAKE is a total stress program



which models the acceleration time history using the wave equation in the frequency domain propagating through the soil profile. We have chosen the one-dimensional or column analysis instead of two-dimensional analysis because of its simplicity.

The basic inputs to SHAKE are the earthquake acceleration time histories and the dynamic properties of the soil mass. The Atkinson (2004) report provides six time histories which match the calculated response spectra at this site for the seismic design criteria of 1 in 10,000. We used the six Atkinson records without analytical matching of the records to the spectra derived from the seismic hazard analysis. In other words, we did not alter the records which is often done to better match the spectra. Calculation of the induced seismic stresses is also called response analysis and is the subject of Section 5.

In Section 6, initial liquefaction is evaluated for the foundation soils of both dams. If initial liquefaction occurs, then the strength of the soil is reduced for evaluation of stability and displacement. If initial liquefaction does not occur, then seismic displacements are estimated using the strengths operative in the soil profile for the static condition but the inertial loading of the soil due to the design earthquake is included.

The seismic stability of the dams is evaluated in Section 7 using strengths appropriate to the soil given the initial liquefaction assessment in Section 6. If there is no initial liquefaction of the soil, case records show that the dams will not fail but there will be some permanent seismic displacements. These displacements are calculated using a variation of the Newmark sliding block analysis or some more sophisticated approach if warranted.

Even if initial liquefaction occurs in the soil profile, it does not mean that the dam will fail. The stability of the dam is evaluated using conventional stability analysis with residual strengths used in the liquefied zones. This residual strength value is normally assigned to the liquefied zones from empirical correlations of blowcount and residual strength from case records. The seismic stability assessment of both dams is described in Section 7.

Finally, if the seismic stability of the dam is insufficient then improvements to the dam are necessary to meet stability and displacement criteria. In Section 7 are presented improvements to the dams where necessary to meet seismic stability and displacement criteria appropriate for tailings dams and the consequences of failure in this specific instance.

The approach outlined above is standard engineering practice for these types of tailings retention structures for these site conditions. There are more sophisticated analytical procedures available but their use is only justified if these simpler techniques do not give a clear answer to the need for remediation.

4. PENETRATION RESISTANCE OF FOUNDATION SOILS

4.1 Site Investigation Program

The existing ground information was largely sufficient to develop a simple surficial geological model as described in Section 2 but penetration values using modern techniques were absent except for the tailings work done by Golder Associates (2004). As stated above, the Becker was chosen because existing test hole information indicated large particle sizes in the foundation soils which the conventional SPT and CPT cannot penetrate. The site investigation program consisted of primarily of open-end Becker drill holes and closed-end Becker Penetration Test (BPT) holes, and was conducted between August 16 and September 4, 2004.

The BPT blowcounts were converted to SPT blowcounts using both the Harder (1988) and the Sy (1993) method. The Sy (1993) is more fundamental and provides more reliable correlations to SPT. Although reliable, the Sy method is more difficult to apply because it requires site-specific PDA (pile driver analyzer) measurements and more data reduction. Unfortunately, the Becker hammer routinely misfired during the investigation program which made data interpretation more difficult and required more site calibration than normal. This section and the accompanying appendices document the test data and interpretation in more detail.

The closed-end BPTs were used to determine the penetration resistance or density of the soil and the open-end drill holes were done mainly to determine soil stratigraphy and to retrieve samples. The open ended samples are completely disturbed and only give a basic indication of soil type. The test hole locations were positioned along the Intermediate Dam and along the East and West Limbs of the Secondary Dam. Figure 4.1 shows the Becker test hole locations and Table 4.1 summarizes the test program. BPTs on the Intermediate Dam were done from the downstream berm whereas the BPTs on the West Limb of the Secondary Dam were done at the downstream toe from the tailings surface.

A pair of open and closed-end holes (BPT04-14 and BKS04-14) was conducted at the toe of the Cross Valley Dam to calibrate the Becker penetration resistance against SPT values from an earlier investigation.

Three conventional standard penetration tests were conducted at BKS04-14 at the Cross Valley Dam. Two standpipe piezometers were installed from the crest of the Intermediate Dam to the top of the drain at BKS04-06 and at BKS04-07. The drill hole locations and elevations were surveyed by Yukon Engineering Services. Selected photographs taken during the site investigation are enclosed in Appendix II.

4.2 Becker Penetration Tests (BPT)

Foundex Explorations Ltd. of Surrey, British Columbia conducted the Becker Penetration Tests using a truck-mounted model HAV180 Becker hammer drill rig. The closed-end BPT consisted of driving a closed-end double-walled 170 mm diameter casing into the ground with an ICE 180 double-acting diesel pile hammer having a manufacturer's rated hammer energy of 11 kJ per blow. During ground penetration, the following data were recorded:

- The number of blows for every 0.3 m of casing penetration.
- Hammer bounce chamber pressures using a computerized data acquisition system. The bounce chamber pressure data are required in the procedure proposed by Harder (1988) to convert BPT blow counts to equivalent standard penetration test (SPT) N-values;
- Dynamic measurements using a pile driving analyzer (PDA) by Robert Miner Dynamic Testing, Inc. of Manchester, Washington on the first two BPTs (BPT04-01 and BPT04-02). The dynamic monitoring data (enclosed in Appendix IV) are required in the BPT-SPT N-values interpretation procedure proposed by Sy (1993); and
- Casing friction measurements from pull-up tests. During each casing add-on (2.4 m and 3 m long casings used), the casing was pulled up and the pull-up tension force was measured with a load cell connected to the top of

the casing. The pull-up force is a measure of the skin friction developed on the outside of the Becker casing during the penetration test. The casing friction data are required in the Sy (1993) method to estimate equivalent SPT N-values.

Table 4.1 Summary of Becker Testing Program

BPT Hole	Open/Closed-End Becker	Dam	Max. Test Depth (m)	Surface Elevation amsl (m)	Remarks
BPT04-01	Closed-end Becker	Intermediate	7.9	1031.1	Could not penetrate fill beyond 7.9 m
BKS04-01	Open-end Becker	Intermediate	36.9	1031.1	-
BPT04-02	Closed-end Becker	Secondary	24.2	1051.7	-
BKS04-02	Open-end Becker	Secondary	27.3	1051.5	-
BPT04-03	Closed-end Becker	Secondary	43.0	1049.9	-
BKS04-03	Open-end Becker	Secondary	43.3	1049.9	Piezometer installed
BPT04-04	Closed-end Becker	Secondary	37.6	1051.0	-
BKS04-04	Open-end Becker	Secondary	40.2	1050.9	-
BPT04-05	Closed-end Becker	Intermediate	24.0	1031.0	-
BKS04-06	Open-end Becker	Intermediate	13.1	1049.1	Piezometer installed
BKS04-07	Open-end Becker	Intermediate	11.9	1049.2	Piezometer installed
BPT04-08A	Closed-end Becker	Intermediate	39.9	1031.0	Adjacent to BPT04-05
BPT04-08B	Closed-end Becker	Intermediate	8.8	1031.0	Adjacent to BPT04-08A
BPT04-09A	Closed-end Becker	Intermediate	33.1	1031.7	-
BPT04-09B	Closed-end Becker	Intermediate	5.8	1031.0	Adjacent to BPT04-09A
BKS04-09	Open-end Becker	Intermediate	8.8	1031.7	-
BPT04-10	Closed-end Becker	Intermediate	39.1	1031.1	Adjacent to BPT04-01
BPT04-11	Closed-end Becker	Secondary	13.0	1050.2	-
BKS04-11	Open-end Becker	Secondary	12.8	1050.2	-
BPT04-12	Closed-end Becker	Secondary	14.6	1055.5	-
BKS04-12	Open-end Becker	Secondary	14.6	1055.5	-
BPT04-13	Closed-end Becker	Secondary	12.5	1057.5	-
BKS04-13	Open-end Becker	Secondary	15.7	1057.5	-
BPT04-14	Closed-end Becker	Cross Valley	27.7	1016.0	-
BKS04-14	Open-end Becker	Cross Valley	9.0	1016.0	-

The open-end Becker sampling holes were carried out adjacent to most of the closed-end BPT holes except where there was already drill hole information. In these sampling holes, the casing was driven open-ended with compressed air forced down the annulus of the casing to flush the cuttings up the centre of the inner pipe to the ground surface. Continuous cuttings (soil particles) were collected at the ground surface via a cyclone.

Disturbed soil samples were obtained and the subsoil stratigraphy was logged by a Klohn Crippen field engineer. The BPT test hole logs are given in Appendix I.

4.3 Standard Penetration Testing (SPT) at the Cross Valley Dam

Three conventional SPT tests were conducted at test hole BKS04-14. The SPT consisted of driving a split spoon sampler (51 mm O.D. by 38 mm I.D.) 0.45 m into the ground using an automatic trip hammer. The weight of the hammer was 63.5 kg (140 lbs) and the drop height was 0.76 m (30 inches). The SPT was carried out in accordance with ASTM D1586-99 test method. The field blow counts for the sampler penetration of 0.30 m, after an initial seating penetration of 0.15 m, were recorded as the measured N-value.

4.4 Laboratory Testing

Soil samples obtained from the BPT sampling drill holes were returned to our Vancouver laboratory for further testing. Moisture content tests were conducted on the recovered samples and the results are included in the test hole logs. Since fines content can influence liquefaction susceptibility, washed sieves on the No. 200 screen was carried out on seventeen samples from BKS04-03, BKS04-04, BKS04-09 and BKS04-13. Sieve analyses, hydrometer and Atterberg limits tests were conducted on the silt samples recovered at BKS04-09 to compare against index property-based liquefaction indicators for fine grained soils. The laboratory test results are included in Appendix III.

4.5 BPT-SPT Data Interpretation

As described above, equivalent SPT N_{60} values were estimated from BPT blow counts following two procedures: the Harder (1988) method and the Sy (1993) method. For the Harder method, the measured peak bounce chamber pressures (BCP) were used in the BPT data interpretation. For the Sy method, the measured transferred energies from the

PDA, together with the load cell measured casing friction from pull-up tests, were used in the BPT data interpretation.

Since PDA measurements were performed only in BPT04-01 and BPT04-02, the transferred energy required to interpret the other BPTs following the Sy procedure was estimated based on correlations of the measured energy data and the measured peak bounce chamber pressure at BPT04-01 and BPT04-02. The assumed transferred energy values used for BPT interpretation are listed in Table 4.2. The energy ratios used were 25% for eight BPTs, 20% and 30% for two BPTs each. The energy ratio in the tailings at BPT04-02 where no PDA energy measurement were made was assumed as 20%.

Table 4.2 Energy Ratios Used in the Computation of Equivalent SPT $(N_1)_{60}$ Using the Sy Method

BPT Hole	Dam	Estimated Energy Ratio (%)	Remarks
BPT04-01	Intermediate	-	Measured energy using PDA was used
BPT04-02	Secondary	-	Measured energy using PDA was used except in the tailings
BPT04-03	Secondary	20	-
BPT04-04	Secondary	20	-
BPT04-05	Intermediate	25	-
BPT04-08A	Intermediate	25	-
BPT04-08B	Intermediate	25	-
BPT04-09A	Intermediate	25	-
BPT04-09B	Intermediate	25	-
BPT04-10	Intermediate	30	-
BPT04-11	Secondary	30	-
BPT04-12	Secondary	25	-
BPT04-13	Secondary	25	-
BPT04-14	Cross-Valley	25	-

Casing friction data were obtained from casing pull-up tests at each BPT. The casing friction computed using CAPWAP analyses of the PDA recorded wave traces at BPT04-02 confirmed that the measured friction from pull-up tests was acceptable (See Appendix

IV for CAPWAP analyses results). Additional details of the BPT data interpretation using the Harder and the Sy methods are given in Appendix V.

4.6 Converted $(N_1)_{60}$ Penetration Results

The Harder and Sy methods were applied to the fourteen BPTs conducted in the Intermediate Dam, Secondary Dam and the Cross Valley Dam, and the results for each BPT are presented in Appendix V. Summary of $(N_1)_{60}$ profiles by Harder and Sy methods are shown in Figures 4.2 to 4.6. Details are as follows:

- Figure 4.2 shows the summary of equivalent $(N_1)_{60}$ profiles obtained from the measured BPT blow counts using the Sy and Harder methods for BPT holes at the Intermediate Dam. Figure 4.3 shows the same plots for the BPT holes at the Secondary Dam.

- Figure 4.4 shows the equivalent $(N_1)_{60}$ profiles by Sy and Harder methods at BPT04-10 located in the Intermediate Dam. Previous "SPTs" were conducted at a location BH80-46 (See Figure 4.1 for the location) close to BPT04-10 by Golder Associates in a mud rotary drill hole using a 76 mm OD sampler. A 63.5 kg hammer and a nominal drop height of 0.76 m (Golder Associates, 1980) were apparently used during testing. The 76 mm OD sampler N-values were converted by Klohn Crippen to the standard SPT N-values (SPTs are normally conducted using 51 mm sampler) using the procedure proposed by Daniel et al. (2003) assuming an energy ratio of 60%. These values are shown in Figure 4.4 together with the values reported by SRK (2004) that were not corrected for the size of the sampler.

- Figure 4.5 shows the equivalent $(N_1)_{60}$ profiles by Sy and Harder methods at BPT04-14 located in the Cross Valley Dam. It also shows the three SPTs conducted in the adjacent open Becker hole BKS04-14. An energy ratio of 80% was assumed by Klohn Crippen for the automatic trip hammer used to conduct the SPTs.

- Figure 4.6 shows the comparison of equivalent $(N_1)_{60}$ by Sy and Harder methods at BPT04-14 and the SPTs by Golder Associates (1980) at the Cross-Valley Dam. The locations of the boreholes conducted by Golder Associates are shown in Figure 4.1. The $(N_1)_{60}$ values as reported by SRK (2004) are shown on Figure 4.6a. Some of the SPTs were reportedly

conducted using a 76 mm OD sampler by Golder Associates. These values were converted to equivalent SPT values following the procedure proposed by Daniel et al. (2003). The converted SPT $(N_1)_{60}$ values are shown in Figure 4.6b.

Comparison of the equivalent SPT N_{60} values from the two approaches indicates that the Sy approach generally gives higher values than those from Harder's method. The difference, as much as 2 to 3 times at certain depths, is mainly attributed to the effects of casing friction that are not explicitly accounted for in the Harder method. As illustrated in Sy (1997), the Harder method has significant casing friction embedded in their empirical BPT-SPT correlation, which was developed from sand sites; the magnitude of the casing friction increases with increasing blow counts. Sy (1997) compiled measured casing friction values from various sites, and showed that average casing friction in gravely sites is generally less than that at sand sites. As shown in Figure V-3, the casing friction measured at Rose Creek is relatively low, even lower than the average "gravel" trend line compiled by Sy (1997). Using the Harder approach in gravely soils with such low casing friction will result in unreasonably low equivalent SPT N_{60} values.

In early 2005 after all correlations with existing SPT information available from the beginning of this assignment in mid-2004 were completed, historic drill holes from the Golder Associates investigation along the alignment of the Secondary Dam became available. These are labelled BH-2 through BH-6 on Figure 4.1. The SPT values from these historic Golder holes are plotted on Figure 6.3 which shows $(N_1)_{60}$ values using the Sy approach. The Golder SPT values are plotted on the nearest Becker penetration hole.

The $(N_1)_{60}$ comparisons on Figure 6.1 between the Becker data and the historic Golder SPT data are quite good. No more than qualitative comparisons can be made because the test holes were not drilled at the same locations and were drilled under different confining stress conditions; i.e. with and without tailings.

Better correlation of measured $(N_1)_{60}$ values with Sy was noted in several case histories documented in Sy et al (1995) and Yan and Sy (2002). Based on these case records, we consider that the equivalent SPT N_{60} values estimated by the Sy procedure are more reasonable than the Harder's values.

4.7 BPT Comparison to Penetration Resistance of Tailings

BPT readings were taken in tailings at BPT04-02 which provided an opportunity to compare the nearest Golder CPT results to converted Becker penetration results from both the Harder and Sy methods. Figure 4.7 shows the equivalent SPT N_{60} profiles within the tailings derived using the Harder and Sy methods. CPT03-23 conducted in the tailings by Golder Associates (2003) is located close to BPT04-02. The tip resistance, q_c data from this CPT was converted to equivalent SPT N_{60} using the relationships proposed by Jefferies and Davies (1993) and Robertson and Campanella (1985) between q_c/N_{60} and D_{50}. As it appears that the tailings at these holes was relatively coarse, the D_{50} of the tailings in these holes were taken as 0.1 mm based on the data provided in Golder Associated (2003). Figure 4.7 also shows the equivalent N_{60} profiles derived from the CPT tip resistance using the correlation by Jefferies and Davies (1993) and Robertson and Campanella (1985). These CPT based profiles are slightly greater than that derived using the Sy method based on the BPT data. However, the N_{60} profile derived using the Harder method shows much lower values within the tailings.

4.8 Summary

Given the comparisons with $(N_1)_{60}$ results from past investigations and this investigation in the foundation soils, the comparison with the single CPT penetration resistance, the inappropriate friction correction in the Harder method, and multiple similar evaluations at other sites, we consider that the $(N_1)_{60}$ values derived from the Sy method are more reliable than Harder-derived values.

5. SEISMIC RESPONSE ANALYSES

5.1 General

One dimensional total stress site response analyses, using the SHAKE computer program (SHAKE91 version by Idriss and Sun, 1992), were carried out at three locations on the Intermediate Dam and three locations each on the West and East Limbs of the Secondary Dam. The objective of the site response analyses was to determine the variations of peak horizontal ground accelerations and cyclic shear stresses with depth for initial liquefaction assessment. The locations and conditions under which the analyses were carried out are listed in Table 5.1.

Table 5.1 Summary of SHAKE Analyses

Dam	Location[a]	Analysis Condition	Remarks
Intermediate Dam	BPT04-10	With and without the dam	Rose Creek –Maximum Section
	BPT04-08	With and without the dam	Terrace
	BPT04-09	With and without the dam	Terrace
Secondary Dam- West Limb	BPT04-03	With and without D/S Tailings	Rose Creek –Maximum Section
	BPT04-04	With and without D/S Tailings	Terrace
	BPT04-02	With and without D/S Tailings	Terrace
Secondary Dam- East Limb	BPT04-11	As drilled condition	Terrace
	BPT04-12	As drilled condition	Terrace
	BPT04-13	With and without the dam	Rose Creek/Tributary

[a]See Figure 4.1 for the location of BPT holes

The soil stratigraphic data from the adjacent open-end Becker test holes were used to estimate soil properties at all locations. The equivalent SPT $(N_1)_{60}$ profiles obtained using the Sy method from the measured BPT at each hole were used to obtain the shear wave velocity profiles required in the SHAKE analyses.

All six natural earthquake records recommended by Atkinson (2003) were used as input motions in the site response analyses. These records included four from the 1989 Loma Prieta earthquake and two from the 1994 Northridge earthquake. The peak accelerations of these six records varied between 0.37 g and 0.54 g and this range is roughly within the

median (0.3 g) and mean (0.5 g) peak ground accelerations (PGAs) for the 10,000 year return period design earthquake motions recommended by Atkinson (2003) for the project site. The acceleration spectra of these unscaled natural records generally match the 10,000 year return period design spectrum in the median (50^{th} percentile) to mean range. The input acceleration time histories, acceleration spectra, derivation of shear wave velocity profiles from the equivalent SPT $(N_1)_{60}$, soil models and properties and the results of the site response analyses are described in more in Appendix VI.

The average of peak surface accelerations from SHAKE analyses for all six earthquake records are listed in Table 5.2 for the Intermediate and the Secondary Dam. The amplification factor between the input motion and the surface acceleration for each individual record was computed and the average of all six records for each location is listed in Table 5.2.

Table 5.2 Average Peak Surface Acceleration and Amplification Factors

Dam	Location	Condition	Average Peak Ground Acceleration(g)	Average Amplification Factor
Intermediate Dam	BPT04-08	Below Dam Crest	0.52	1.2
		Below Dam Toe	0.67	1.5
	BPT04-09	Below Dam Crest	0.31	0.7
		Below Dam Toe	0.46	1.1
	BPT04-10	Below Dam Crest	0.54	1.2
		Below Dam Toe	0.90	2.1
Secondary Dam-West Limb	BPT04-02	With Tailings	0.67	1.5
		Without Tailings	0.88	2.0
	BPT04-03	With Tailings	0.56	1.3
		Without Tailings	0.75	1.7
	BPT04-04	With Tailings	0.57	1.3
		Without Tailings	0.75	1.7
Secondary Dam-East Limb	BPT04-11	As-Drilled	0.82	1.9
	BPT04-12	As-Drilled	0.85	1.9
	BPT04-13	Below Dam Crest	0.54	1.2
		As-Drilled	0.69	1.6

5.2 Intermediate Dam

Figure 5.1 shows the peak acceleration profiles through the dam crest from SHAKE analyses at BPT04-08, BPT04-09 and BPT04-10. Each of these three profiles uses the ground conditions at the respective Becker holes but corrected for the weight of the dam fill. The crest accelerations at BPT04-10 and BPT04-08 are amplified, while the crest accelerations at BPT04-09, are de-amplified.

Figure 5.2 shows the acceleration profiles at the toe of the dam from SHAKE at the same BPT locations. The acceleration at the surface is generally amplified but the amplification factors are much higher than those for the response at the dam crest (See Figure 5.1). Also, the amplification factor for the SHAKE column at the maximum section at Rose Creek (at BPT04-10) is higher than those at the terrace.

Amplification occurs when the natural period of the ground is close to the predominate frequencies of the input seismic ground motions. It is important to note that the amplification of ground motions is greater than the increase in firm ground motion from the GSC seismic model to the Atkinson model.

5.3 Secondary Dam

Figure 5.3 shows the peak acceleration profiles through the toe of the Secondary Dam with the tailings in place at BPT04-02, BPT04-03 and BPT04-04. The input motions are generally amplified at the tailings surface with amplification highest at BPT04-02 where the glacial till/bedrock was at shallow depth.

Figure 5.4 shows the peak accelerations for ground condition without the tailings. Input motions are highly amplified at the surface at all three locations and the amplification is much higher than that with the tailings. Thus, the removal of tailings at the downstream of the dam will amplify seismic motions in the foundation soils.

At BPT04-02, the surface acceleration exceeded 1 g for the Gilroy #3 record with a PGA of 0.54 g. Note that the SHAKE analyses use an equivalent linear total stress approach. The yielding in soils that may occur when the induced stresses exceed the soil strength and when development of excess pore water pressures leads to loss in the strength and stiffness of the soils are not modeled in SHAKE analyses. Thus, the predicted accelerations and shear stresses are generally too high or conservative.

Figure 5.5 shows the peak acceleration response at BPT04-13 on the east limb of the Secondary Dam for as-drilled condition and for a condition through the dam crest. The input motions are generally amplified and the amplification was higher for as-drilled condition.

The SHAKE results indicate that amplification generally occurs in the foundation sand and gravel overlying the till/bedrock. Presence of tailings and dam fill tend to reduce amplification.

6. INITIAL LIQUEFACTION ASSESSMENT

6.1 General

This section uses Equation 3.1 to calculate the Factor of Safety against triggering of liquefaction. Equation 3.1 compares the $(N_1)_{60}$ converted from the field Becker blowcounts in Section 4 to the $(N_1)_{60}$ required to prevent triggering of liquefaction from the site response analysis in Section 5. The acceleration profiles in Section 5 have been converted to $(N_1)_{60}$ values as explained in Appendix VI.

In this report we will follow the various definitions of liquefaction as given by Robertson et al. (2001) for the Canadian Liquefaction Experiment (CANLEX) research effort. In their definition, initial liquefaction or triggering of liquefaction is also called cyclic liquefaction. Cyclic liquefaction is possible to blowcounts of 30. However, for blowcounts over 20 the deformations are small and stop after the earthquake is over. For blowcounts less than 20, large deformations are possible if the static shear stresses exceed the residual strength of the liquefied soils. These large deformations are called flow liquefaction and can occur after the earthquake shaking is over. It is these large deformations that are of concern in this evaluation as only deformations of large magnitude could lead to loss of tailings to the downstream environment.

The comparison plots in this section (Figures 6.1 to 6.5) use the $(N_1)_{60}$ values that are calculated directly from the response analysis to a blowcount of 30. However, an $(N_1)_{60}$ of 20 will be used to evaluate whether the ground will liquefy and potentially flow.

6.2 Intermediate Dam

Figures 6.1 shows the liquefaction evaluation in the foundation soils beneath the dam crest and Figure 6.2 shows the same at the dam toe. The liquefied zones within the granular soils are shaded in yellow.

Below the dam crest, the upper bound of required $(N_1)_{60}$ (from all six earthquake records) to prevent triggering of liquefaction in the granular soils in the foundation at all three locations is near the limiting value of 30. At the toe, the limiting value of 30 is reached everywhere because of the large amplification at the toe.

The Becker holes through the terrace soils, BPT04-05 and 08, show initial liquefaction triggering between about El. 1010 m to 1019 m but the $(N_1)_{60}$ values do not anywhere fall below 20. Therefore these soils cannot flow during the design earthquake.

At BPT04-09 between about El. 1024 and 1029 m there is a silt layer with low equivalent $(N_1)_{60}$ values. The liquefaction potential of this fine-grained layer was evaluated using the Chinese criteria and more recent criteria proposed by Andrews and Martin (2000) and Bray et al. (2004). Samples from this silt layer recovered in the open-Becker hole BKS04-09 were tested in the laboratory for water content, Atterberg limits and grain size. These data were used in the screening. The Chinese and Andrews and Martin criteria for liquefaction suggest that this silt layer is not liquefiable due to its high clay content. However, the Bray et al. criteria suggested the silt layer is liquefiable. Details of the evaluation are presented in Appendix VI.

All of the drilling information at this dam or in the remainder of the Rose Creek valley suggests that all such silt layers are limited in extent. The downstream slope of Intermediate Dam is at 2H: 1V over this silt layer and has performed well. This again suggests that the silt is limited in extent. Given this, we consider that, even if the silt liquefied, it would not lead to failure of the dam so this seismic failure mode is not considered further.

Liquefaction was not predicted at BPT04-10 in the former Rose Creek either beneath the dam or at the toe of the dam. This finding, which is based on a single Becker hole, is at

odds with the data at the Secondary Dam which suggests that the Recent Rose Creek soils are liquefiable.

6.3 Secondary Dam-West Limb

Figure 6.3 shows the liquefaction evaluation for existing conditions with the tailings left in place and Figure 6.4 shows the same for condition when the tailings is removed. A liquefaction evaluation was not done beneath the centerline of the Secondary Dam as we expect the toe conditions govern.

With the tailings in place, amplification of ground motions required a limiting value of $(N_1)_{60}$ of 30 to prevent initial liquefaction everywhere at the toe. Without the tailings, the amplification was greater so the limiting value also governed the liquefaction evaluation.

At BPT04-02 in the foundation soils on the terrace, liquefaction triggering was not predicted. The upper 8 m of soil at this Becker hole is tailings which are shown to liquefy. Liquefaction triggering is predicted at BPT04-04, also on terrace soils, between El. 1016 and 1038 m in intermittent layers of about 2-3 m thick. However, the average equivalent $(N_1)_{60}$ of these layers was about 22. Again, because these blowcounts are above 20, flow liquefaction is not possible through these soils.

At BPT04-03 in the former Rose Creek channel, liquefaction will be triggered between El. 1023 and 1030 m where the lower bound equivalent $(N_1)_{60}$ from the measured BPT values was only about 10. Below El. 1023 m, marginal liquefaction was predicted at BPT04-03. However, the equivalent $(N_1)_{60}$ was generally greater than 22.

In summary, only the soils between El. 1023 m and 1030 m in the former Rose Creek channel can liquefy and lose strength at the Secondary Dam – West Limb.

6.4 Secondary Dam-East Limb

At BPT04-11 and BPT04-12 on the east limb of the Secondary Dam, the equivalent $(N_1)_{60}$ from the measured BPT were higher than the limiting value for triggering liquefaction indicating that the foundation soils in these locations are too dense to liquefy. These soils are also part of the remnant terrace on the south side of Rose Creek channel.

Figure 6.5 shows the liquefaction evaluation for as drilled conditions and through the dam crest at BPT04-13 located at the southeast corner of the dam at the foot of a tributary creek to Rose Creek. The required $(N_1)_{60}$ to prevent liquefaction triggering was about 30 for both conditions. Liquefaction will be triggered between El. 1049.5 and 1053.0 m as the lower bound $(N_1)_{60}$ of this layer is about 10. The former Rose Creek channel is narrow in this area but the foundation soils within this narrow channel appear to be liquefiable.

7. SEISMIC STABILITY

7.1 General

This section presents the results of seismic stability analyses at the Secondary and Intermediate Dams. At the Secondary Dam we used liquefied strengths in the foundation soils and the retained tailings as appropriate. At the Intermediate Dam, we also used liquefied strengths in the tailings but not in the foundation soils because the limited data shows these foundation soils will not liquefy.

This section includes conceptual designs and associated cost estimates for Secondary Dam remedial works. A conceptual design and cost estimate are also presented for the Intermediate Dam over the Rose Creek section assuming that the Rose Creek sediments are liquefiable during a major earthquake. Although ground investigations to date do not indicate such foundation liquefaction, this work was done to estimate costs for remediation in the event that further field investigations give lower penetration values.

7.2 Flow Liquefaction Analyses for Intermediate and Secondary Dams

Flow liquefaction stability analyses were carried out on the critical sections of the Intermediate and Secondary Dam to determine the potential for flow sliding and the factor of safety (FOS). Although the foundation soils of the Intermediate Dam will not liquefy, we still evaluate seismic stability because liquefied tailings on the upstream slope could potentially de-stabilize the dam during a major earthquake.

Details of the analyses are presented in Appendix VII. The computer program SLOPE/W (Geo-Slope, 2003) and the Morgenstern-Price method were used in the analyses. Analyses were performed for the following conditions:

- Intermediate Dam with and without a downstream polishing pond; and
- Secondary Dam with and without downstream tailings.

The loose granular soils layers that were found to trigger liquefaction under the design earthquake motions beneath the Secondary Dam in the former Rose Creek channel were assigned post-liquefaction residual strengths. The residual strength of layers with an equivalent $(N_1)_{60}$ of 10 was set to 10% of the initial effective overburden stress as suggested by Idriss (2004) (i.e. $S_{ur}/\sigma_{vo}'=0.1$). The strength of denser layers with $(N_1)_{60}$ greater than about 22 where cyclic liquefaction was triggered were assigned a friction angle of 35 degrees with no cohesion. The liquefied tailings retained by both the Intermediate and the Secondary Dam were assigned a post-liquefaction residual strength of 5% of initial effective overburden stress ($S_{ur}/\sigma_{vo}'=0.05$).

Table 7.1 summarizes the Factor of Safety against flow liquefaction for the Intermediate and Secondary Dams. The stability sections are in Appendix VII.

Table 7.1 Summary of Flow Liquefaction Stability Analyses

Dam	Section[a]	Liquefaction Potential	Condition	Post-Liquefaction Factor of Safety (Morgenstern-Price)
Intermediate Dam	Through BPT04-01&10 (through Rose Creek)	Cyclic liquefaction in the foundation sand and gravel with $22<(N_1)_{60}<30$	With D/S pond	1.47[b]
			Without D/S pond	1.76[c]
Secondary Dam- West Limb	Through BPT04-03 (through Rose Creek)	Liquefaction in foundation sand and gravel with $(N_1)_{60}$ ~10 and cyclic liquefaction in foundation sand and gravel with $22<(N_1)_{60}<30$	With D/S tailings	1.56
			Without D/S tailings	0.62
Secondary Dam- East Limb	Through BPT04-13 (at the foot of a creek?)	Liquefaction in foundation sand with $(N_1)_{60}$ ~10 and cyclic in foundation sand with $22<(N_1)_{60}<30$	No D/S Tailings	0.78

[a]See Figure 4.1 for the approximate locations of sections.
[b]FOS>1.5 for or slip surface through the crest and foundation of the dam.
[c]FOS>1.8 for or slip surface through the crest and foundation of the dam.

The Intermediate Dam can retain the liquefied tailings with and without the Polishing Pond on the downstream slope. The West Limb of the Secondary Dam on the terrace

soils is stable with and without the tailings at the toe. However, the West Limb of the Secondary Dam where it is over the sediments of the former Rose Creek Channel is not stable if the tailings are removed. This is because, although the tailings have a very low residual strength, they have dead weight which helps to stabilize the toe. The East Limb of the Secondary dam is stable everywhere during the design earthquake except at the southeast corner (at BPT04-13) where the FOS is about 0.8.

Where the FOS against flow sliding is greater than 1.1 in Table 7.1 we estimated the permanent seismic displacements using the Newmark sliding block. The charts proposed by Newmark (1965), Makdisi and Seed (1978), and Hynes and Franklin (1984) for the estimation of displacements were used. The yield acceleration, which is required to bring the post-liquefaction factor of safety to unity, was estimated using the computer program SLOPE/W (Geo-Slope, 2003) and the Morgenstern-Price method under post-liquefaction conditions. Details of analyses are presented in Appendix VII. The estimated maximum displacements at the Intermediate Dam and the Secondary Dam were less than 1 metre in both cases which is acceptable.

This deficiency in the post-earthquake stability of the Secondary Dam means that, if the tailings are removed as part of the closure options, the Rose Creek section of the West Limb will have to be remediated. Irrespective of the removal of tailings, the southeast corner will have to be remediated. However, both conclusions are based on single boreholes through the weak Rose Creek foundation areas. Additional boreholes at the southeast corner would be relatively easy to complete while boreholes on the West Limb would be difficult as this portion of the tailings was not accessible during the 2004 site investigation.

Also note that the Secondary Dam is low at the southeast section. A dyke failure would not lead to a massive release of tailings as would occur if the Intermediate Dam failed. Also there is no pond which tends to reduce tailings release. If this low dyke moves and

breaches during an earthquake, tailings would be released into the diversion channel and carried down the channel to Rose Creek.

Although evaluation of the Cross Valley Dam seismic stability is not part of this evaluation, review of available ground conditions indicates that most of the downstream slope of the dam is founded on Recent Rose Creek sediments which are also likely to liquefy under the design earthquake. This needs to be confirmed but some of the lowest blowcounts are beneath this dam.

This leaves the apparently non-liquefiable Recent Rose Creek foundation beneath the Intermediate Dam as somewhat of an anomaly. Again, the predicted good performance during the design earthquake is based on a single Becker penetration hole.

More Becker penetration test holes in the Rose Creek sections of all three dams are required to confirm the conclusions in this report and better define the areas which require remediation.

The next section describes alternatives to improve the stability of the Secondary Dam at the Rose Creek section of the West Limb and East Limb. These remedial alternatives are designed to the concept level which means that the alternatives are feasible and that dimensions are approximate to the extent that rough costs can be estimated.

7.3 Secondary Dam – Remediation Alternatives

There are now many techniques to improve the seismic stability of dam foundations. At this conceptual stage in the design of such measures, it is best to cost measures that are known to have worked in similar situations using conservative costing assumptions. In this case, the major unknown is the characteristics of the ground along the 300 m section of the West Limb of the Secondary Dam given that there is only one test hole available

there. The most common remedial measure is to densify or drain the potentially liquefiable soil with a toe berm that is usually needed as a working surface during construction.

Normally, the cheapest in situ means of densifying foundation soils is explosive compaction. However, the cost depends largely on the cost to install the shot holes which can vary widely depending on the ground conditions. Another technique which is gaining favour is drainage of the foundation during the earthquake using wick drains or gravel drains. Again, the cost of this option depends largely on the drilling conditions.

Dynamic compaction is widely used where the depth of the liquefiable soils is less than 10 m. If the liquefied zone extends below this depth then a second technique such as blasting is used in combination with dynamic compaction. In this case, the zone is much deeper than the limit of dynamic compaction so this technique was not considered further at this stage.

Techniques such jet grouting and soil mixing where the foundation soils are replaced are always more expensive and are only used for specific applications where the less expensive methods are not feasible. These techniques were not considered further.

The most common technique in western Canada for densifying soils is the vibroreplacement where a vibroflot suspended from a crane is used to create a hole and densify gravel backfill. This technique again depends on ground conditions. It is not feasible in soils with significant cobbles or boulders.

All indications are that the recent Rose Creek sediments are sands and silts but the ground just below the tailings in BPT04-03 is dense. This could be natural foundation soils that were compacted by construction traffic or could be the toe of the dam fill. There is not enough information to make a determination at this time.

Given these uncertainties, two remedial options were considered to stabilize the Secondary Dam west limb section at the Rose Creek and one option to stabilize the east limb section at the southeast corner.

Option 1: In situ densification to improve the foundation soils using vibroreplacement.

Option 2: Compacted downstream berm with drains in the potentially liquefiable foundation soils. It is assumed that the installation technique will not improve the density significantly.

Figure 7.1 show the plan and elevation view of the conceptual remedial Options 1 and 2 for the Secondary Dam (West Limb) without the downstream tailings at BPT04-03. Option 1 remediation comprises a 45 m wide densified zone in the loose foundation sand and gravel with $(N_1)_{60cs}$ of about 10. The Option 2 consists of a 50 m wide and 15 m high downstream berm and 70 m wide zone in the foundation with earthquake drains.

Figures 7.2 shows the remedial concept Option 1 for the Secondary Dam (East Limb) section at BPT04-13. In this case, the remediation Option comprises 10 m wide densified zone in the loose foundation sand with $(N_1)_{60cs}$ of about 10.

To obtain the dimensions of the remedial options, seismic stability analyses were done with higher strengths in the improved zones. In Option 1 (in-situ densification), it was assumed that the densified soils would not trigger liquefaction and have a friction angle of 38 degrees. In Option 2, the friction angle of the compacted berm is taken as 38 degrees. The friction angle of loose granular soils with drains was taken as 30 degrees and an excess porewater pressure ratio of 50% was assigned to this zone in the post-liquefaction stability analyses.

Table 6.2 summarizes the results of the post-liquefaction stability analyses. Details of the analyses are presented in Appendix VII.

Table 7.2 Stability Analyses Results with and without the Remediation

Secondary Dam	Section	Post-Liquefaction Factor of Safety (Morgenstern-Price)		
		Unremediated Section	Remediated Section with Option 1	Remediated Section with Option 2
West Limb without D/S Tailings	Through BPT04-03 (through old Rose Creek?)	0.62	1.21	1.13
East Limb	Through BPT04-13 (at the foot of an old creek?)	0.78	1.42	-

7.4 Cost Estimate for Secondary Dam Improvements

The cost estimate for the vibroreplacement option is given in Table 7.3. This cost estimate includes both the West Limb and East Limb improvements. We have used a unit cost of $25 per cubic metre which is higher than the unit costs normally charged in southwestern British Columbia.

The East Limb improvement is a small percentage of the total cost. If only the East Limb improvement were required, vibroreplacement would be an expensive option and more work would be required to find a more satisfactory solution.

The vibroreplacement costs include backfill of stone columns through the dam fill, densification of the dense zone in BPT04-03 and the loose zone beneath this dense zone. It is not known whether this dense zone is an anomaly or the downstream toe of the dam. The cost includes densification of this zone which adds about $2.3 M to the cost of this option. We have also included a contingency of 30 % and a project monitoring fee of 12%.

Table 7.3 Cost Estimate for Vibroreplacement Option 1 for the Secondary Dam West and East Limb Sections

Item No.	Description of Work	Unit	Qty.	Unit Cost	Total Cost ($ CAD)
1	Mob and demob of equipment and crew	ls	1	180,000	180,000
	Secondary Dam-West Limb				
2	Construction of Workpad	m^3	85,000	5	425,000
3	Stone Columns-Zone 1 (~El 1023-1030m)	m^3	91,350	25	2,283,750
4	Stone Columns-Zone 2 (~El 1030-1037m)	m^3	91,350	25	2,283,750
5	Stone Columns-Zone 3 Through Workpad	m^3	101,500	25	2,537,500
6	Verification BPT/SPT Testing	ls	1	50,000	50,000
	Secondary Dam-East Limb				
7	Stone Columns	m^3	12,000	25	300,000
8	Stone Columns – Zone 2 Through Road Fill	m^3	15,000	25	375,000
8	Verification BPT/SPT Testing	ls	1	50,000	50,000
9	Environmental Protection Measures	ls	1	50,000	50,000
				Subtotal	**8,535,000**
10	Design and Project/Construction Management @ 12%	ls	1	1,024,200	1,024,200
				Subtotal	**9,559,200**
11	Contingency (minimum 30% recommended)	allow	1	2,867,760	2,867,760
				Total	**12,426,960**

The cost for Option 2 is shown in Table 7.4 which includes a compacted berm. The cost for this option is very sensitive to the drilling costs. However, the installation costs should be less than that for vibroreplacement. The total cost for this option is $6.4 M which is less than the vibroreplacement cost but is subject to more uncertainty. The East Limb vibro cost for the Secondary Dam is $1.3 million so the total costs would be $7.7 million.

Table 7.4 Cost Estimate for Remedial Option 2 with Berm and Drains for the Secondary Dam-West Limb Section

Item No.	Description of Work	Unit	Qty.	Unit Cost	Total Cost ($ CAD)
	Compacted Berm				
1	Construction of compacted berm	m^3	246,750	5	1,233,750
	Installation of Gravel Drains				
2	Mob and demob of equipment and crew	ls	1	50,000	50,000
3	Gravel drains (8" dia at 6 ft spacing)	ls	1	3,000,000	3,000,000
4	Pre-improvement field program	ls	1	50,000	50,000
4	Environmental protection measures	ls	1	50,000	50,000
				Subtotal	**4,383,750**
5	Design and project/construction management @ 12%	ls	1	526,050	526,050
				Subtotal	**4,909,800**
6	Contingency (minimum 30% recommended)	allow	1	1,472,940	1,472,940
				Total	**6,382,740**

Note that all cost estimates in this section are based on information available to date and are only our opinion of probable cost for budgetary purposes. Actual cost depends on many factors which can change with time, including site and market conditions at the time of construction.

7.5 Intermediate Dam – Remedial Alternatives and Cost

Figure 7.3 shows a remedial concept for the section of the Intermediate Dam over the Rose Creek section of the foundation. The remedial concept consists of a downstream compacted berm approximately 13 m high and densification using vibroreplacement of the uppermost 12 m of the foundation soils beneath the first 31 m upstream of the downstream toe. This remedial concept has been designed so that the Factor of Safety is greater than 1.1 during and after a major earthquake. We have assumed an $S_{ur}/\sigma_{vo}'=0.1$ as per our assumptions for the Rose Creek sediments at the Secondary Dam after liquefaction.

The estimated cost for this remediation is $4.3 M as detailed in Table 7.5.

Table 7.5 Cost Estimate for Vibroreplacement and Berm Option for the Intermediate Dam

Item No.	Description of Work	Unit	Qty.	Unit Cost	Total Cost ($ CAD)
	Compacted Berm				
1	Construction of containment dyke around the footprint of compacted berm to prevent sludge movement	ls	1	130,000	130,000
2	Sludge/loose soils removal from the footprint of compacted berm	m^3	37,300	10	373,000
3	Construction of compacted berm	m^3	137,400	5	687,000
	Stone Columns				
4	Mob and demob of equipment and crew	ls	1	180,000	180,000
5	Stone Columns-Zone 1 (Foundation soil)	m^3	27,900	25	697,500
6	Stone Columns-Zone 2 Through Berm	m^3	30,690	25	767,250
7	Verification BPT/SPT Testing	ls	1	50,000	50,000
8	Environmental Protection Measures	ls	1	75,000	75,000
				Subtotal	**2,959,750**
9	Design and Project/Construction Management @ 12%	ls	1	355,170.00	355,170
				Subtotal	**3,314,920**
10	Contingency (minimum 30% recommended)	allow	1	994,476.00	994,476
				Total	**4,309,396**

[a]an average thickness of 2m assumed for the sludge/loose soils.

The remedial concept shown on Figure 7.3 only meets design criteria if the polishing pond is permanently removed. If the polishing pond remains, then the densified zone will have to extend farther upstream and downstream. We have also assumed that the polishing pond can be dewatered prior to construction. A low containment dyke would have to constructed out and over the sludge in the polishing pond to create a working area for the downstream berm. The dyke could be constructed by displacing the sludge working from the toe of the dam. The mudwave in front of the containment dyke and the sludge within the containment dyke would have to be excavated and wasted to the tailings on the upstream side of the Intermediate Dam.

After completion of the containment dyke and sludge removal, the downstream toe berm could be placed and compacted to finished grade. We have assumed that this fill could be found in the right abutment area. If the Cross Valley Dam were removed, it would also be a source of fill. The vibroreplacement would be done from the berm surface.

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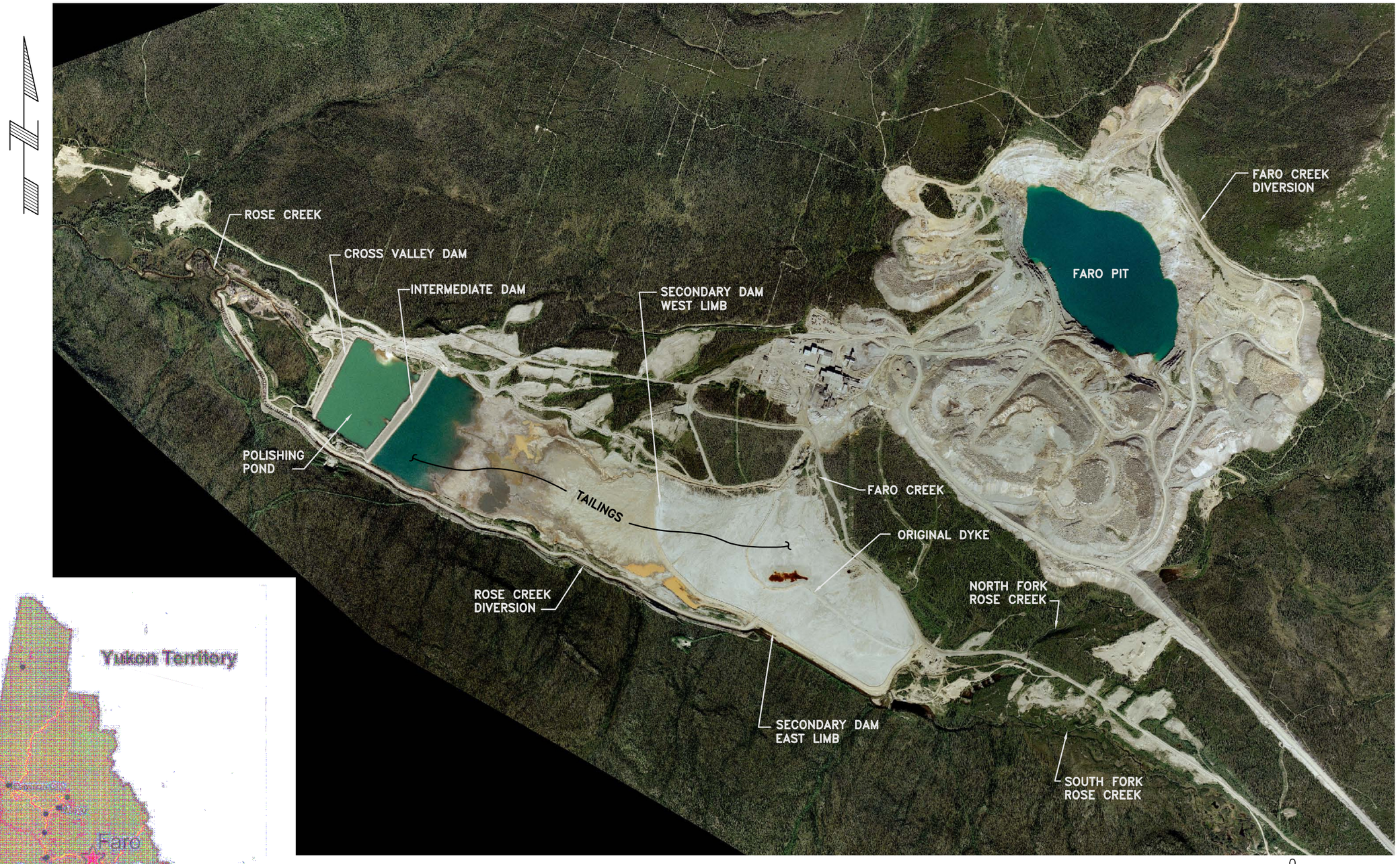
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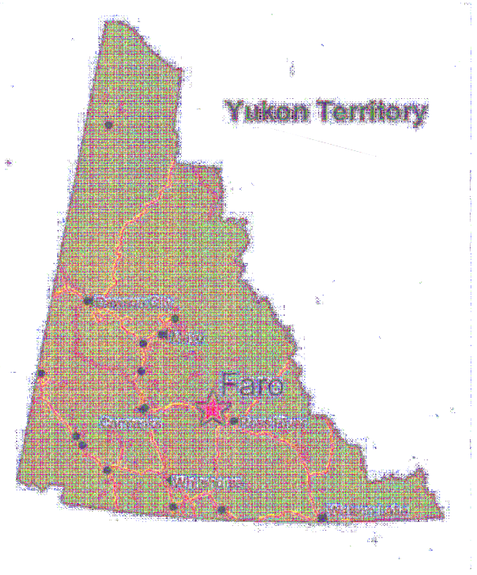
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FIGURES





KEY PLAN

NOT FOR CONSTRUCTION

0 1000 m

SCALE

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA,STATEMENTS,CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

CLIENT
DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

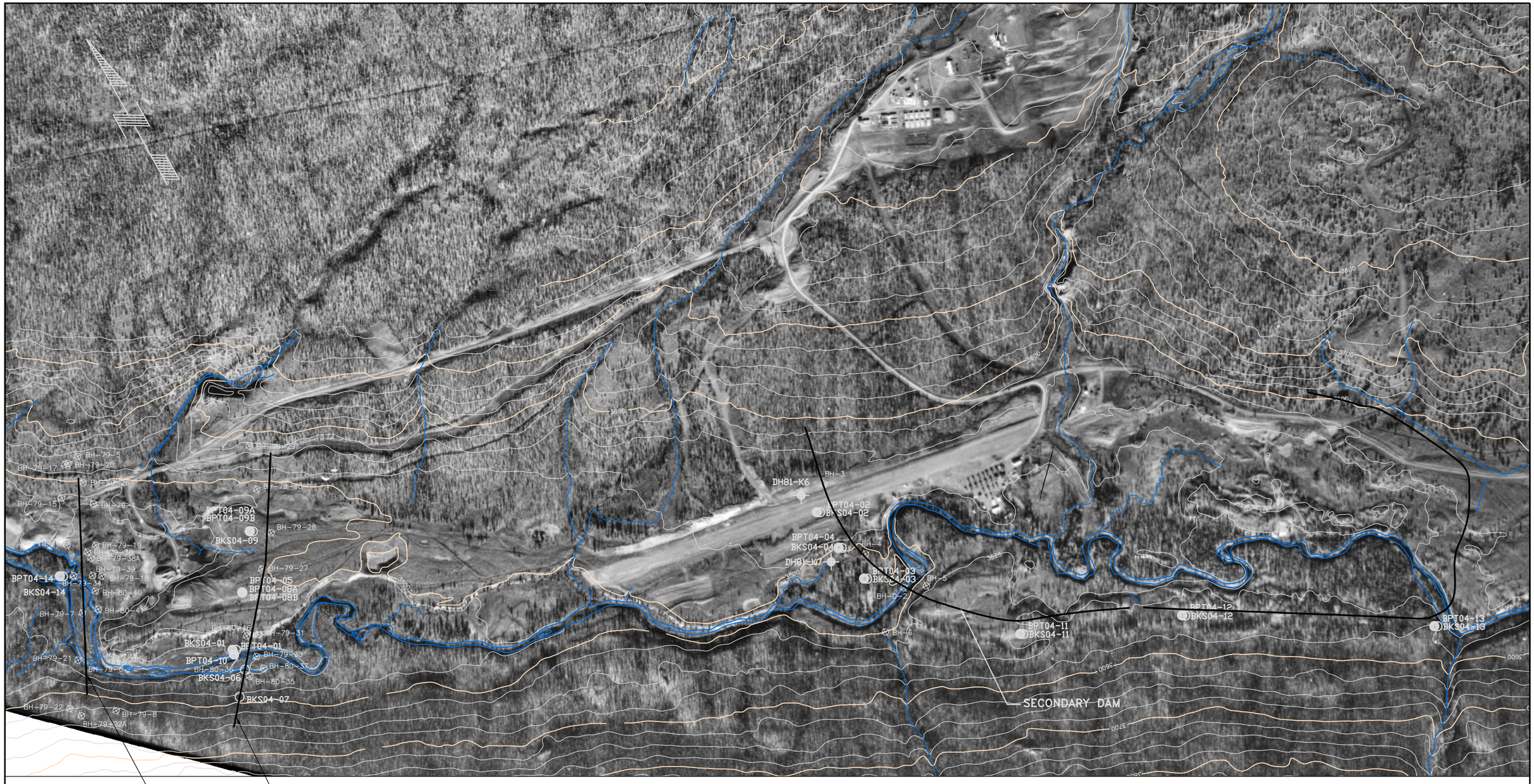
TITLE
LOCATION PLAN

PROJECT No. M09237A01

FIG. No. 1.1

KCR-BM

Date: 11/3/2004
Scale: 1=1(PS)
Drawing File: M:\M09237A01 - Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\figure 1.1.dwg (cwong)



LEGEND

BH-80-36	GOLDER TEST HOLE
BKS04-06	KLOHN CRIPPEN OPEN BECKER TEST
BPT04-10	KLOHN CRIPPEN CLOSED BECKER TEST
DH81-K7	KLOHN CRIPPEN TEST HOLE

NOTES:

1. CONTOURS ARE SHOWN FOR ILLUSTRATION ONLY AND WERE TAKEN FROM PRE-MINING_TOPO DRAWING FROM SRK.
2. CONTOUR INTERVAL IS 25 FT RELATIVE TO MINE DATUM.
3. BH3 TO BH6 AND BH-D-2 LOCATIONS ARE APPROXIMATE.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

SCALE 0 500 m

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CLIENT
DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

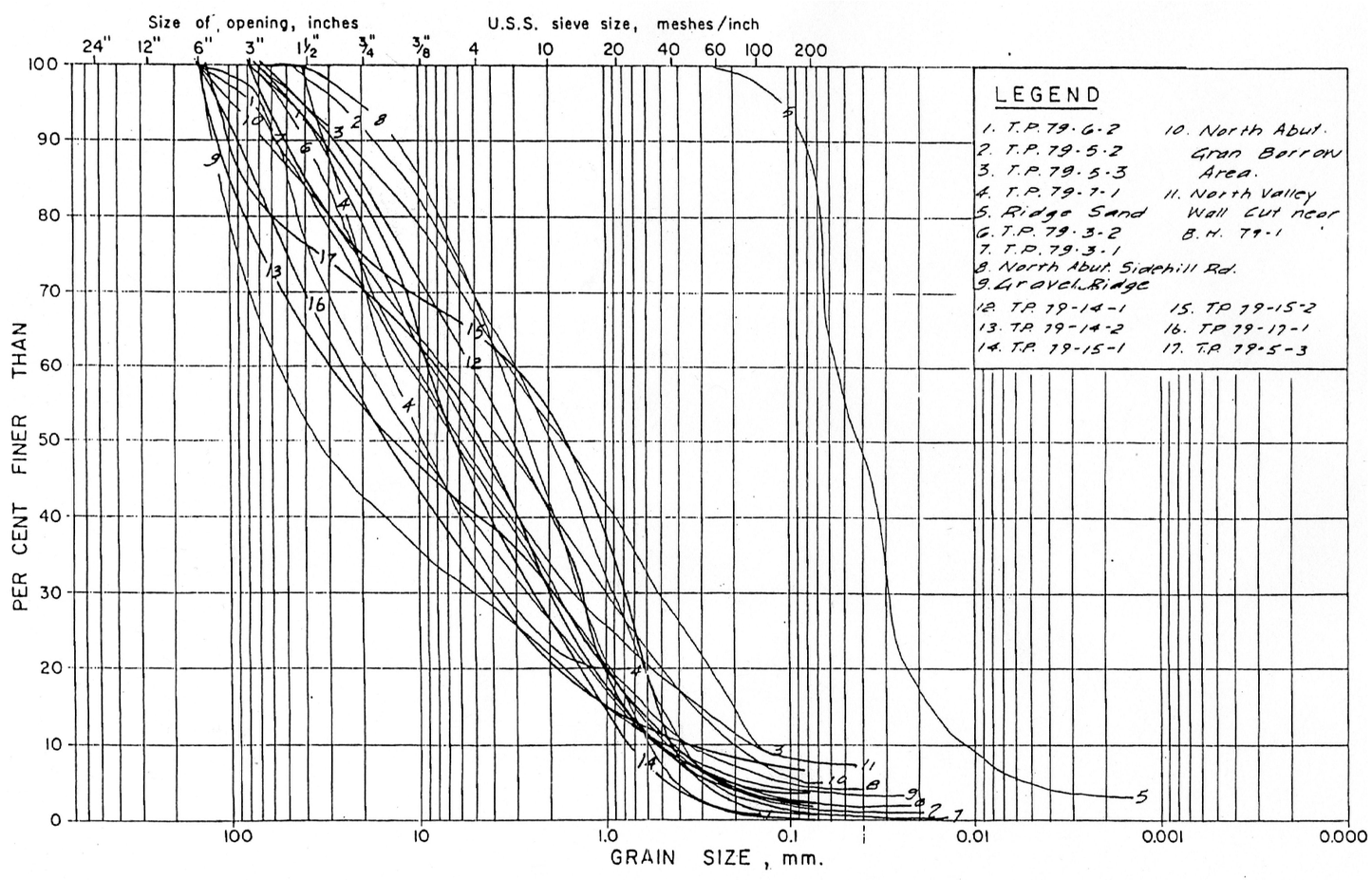
TITLE
TEST HOLE LOCATION PLAN
PRE-MINING TOPOGRAPHY

PROJECT No. M09237A01

FIG. No. 2.1

KCR-BM

Scale: 1:1(PS)
Drawing File: M:\M09237A01 - Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\figure 2.1.dwg (cwong)
Xrefs: TESTHOLE, Prelimning_topo



BOULDER SIZE	COBBLE SIZE	GRAVEL SIZE			SAND SIZE			SILT SIZE	CLAY SIZE
		coarse	medium	fine	coarse	medium	fine	fine grained	

NOTE:

GRADATION CURVES FROM GOLDER ASSOCIATES (1980) FROM BORROW AREAS IN TERRACE USED TO CONSTRUCT INTERMEDIATE DAM.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____ SCALE

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CLIENT: DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

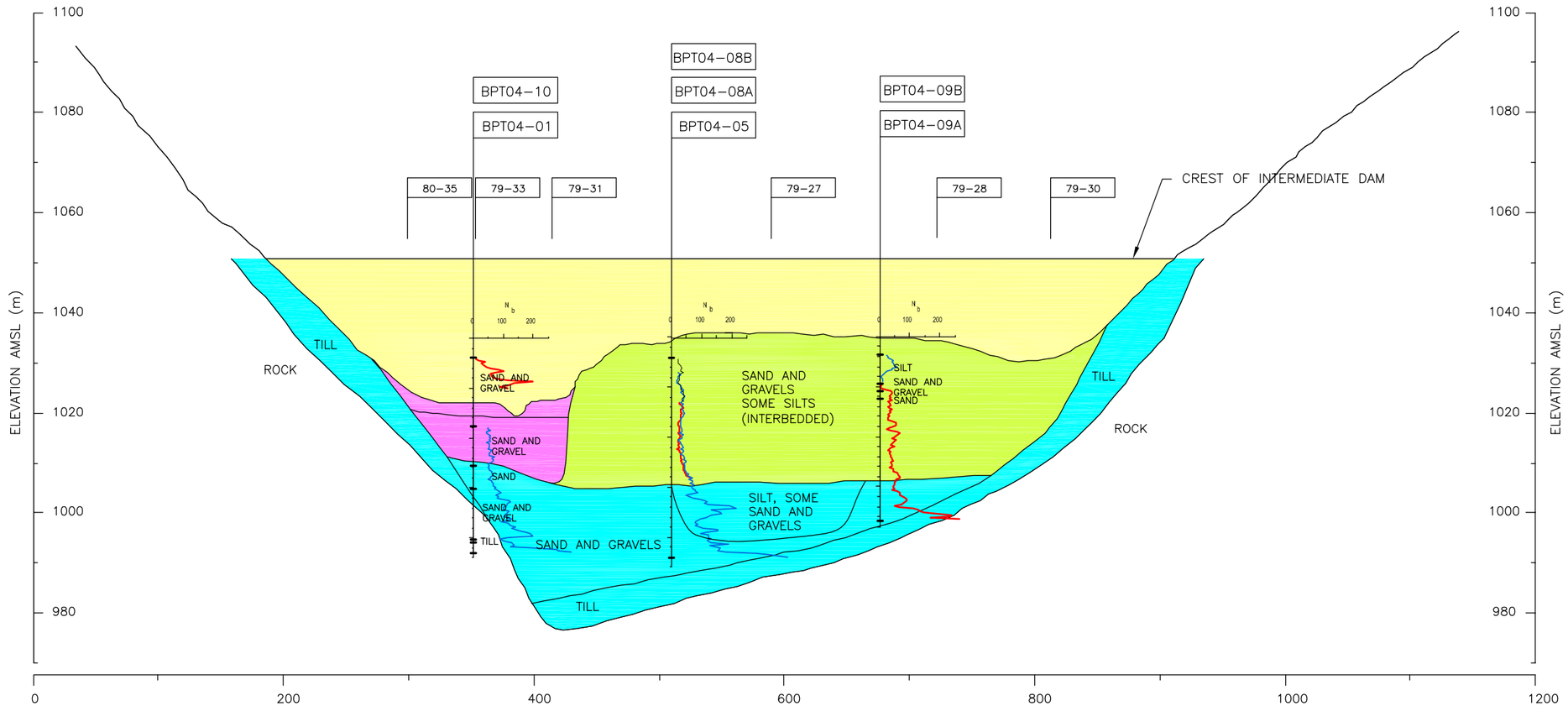
TITLE: TERRACE SOIL GRADATION CURVES

PROJECT No. M09237A01

FIG. No. 2.2

KCR-BM

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Drawing File: M:\M09237A01 - Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\figure 3.1.dwg (cwong)
Xrefs: TESTBOX



SECTION A 4.1

LEGEND

- FILL
- ROSE CREEK SEDIMENTS
- GLACIO–FLUVIAL SEDIMENTS–TERRACE
- PLEISTOCENE SEDIMENTS

NOTE:

1. SECTION IS TAKEN FROM KLOHN LEONOFF (1981) DRAWING FOR GROUND CONDITIONS BENEATH CENTERLINE OF INTERMEDIATE DAM. KC BECKER HOLE INFORMATION IS SUPERIMPOSED AND WAS OBTAINED ON DOWNSTREAM BERM.

VERTICAL SCALE 0 50 m

HORIZONTAL SCALE 0 200 m

SCALE

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

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CLIENT

DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT

ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE

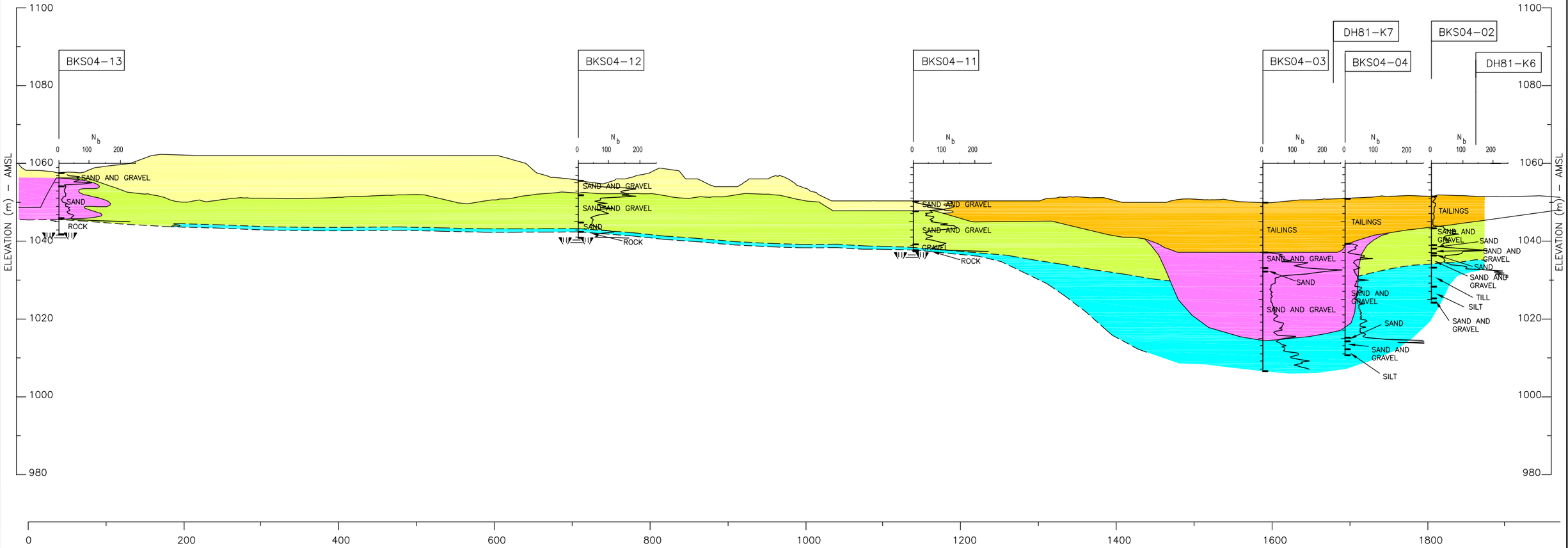
INTERMEDIATE DAM
FOUNDATION STRATIGRAPHY

PROJECT No. M09237A01

FIG. No. 2.3

KCR–BM

Scale: 1"=1'(PS)
Drawing File: M:\M09237A01 – Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\SECTION.dwg (cwong)



SECTION B
4.1

LEGEND

- FILL
- TAILINGS
- ROSE CREEK SEDIMENTS
- GLACIO-FLUVIAL SEDIMENTS-TERRACE
- PLEISTOCENE SEDIMENTS

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

VERTICAL SCALE 0 50 m

HORIZONTAL SCALE 0 200 m

SCALE

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CLIENT
DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

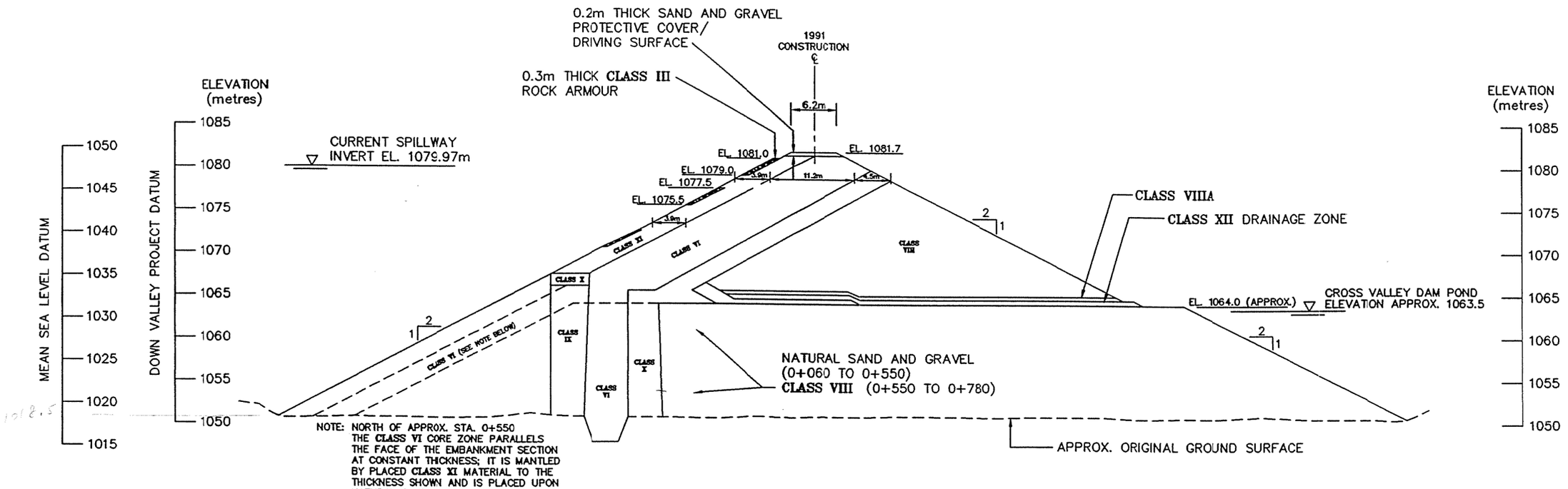
TITLE
SECONDARY DAM
FOUNDATION STRATIGRAPHY

PROJECT No. M09237A01

FIG. No. 2.4

KCR-BM

Date: 10/22/2004
Scale: 1=25(PS)
Drawing File: M:\M09237A01 - Rose Creek YK Tailings\400 Site Work and Design Calculations\410 Drawings\SECTION-SecDam.dwg (Cwong)
Xrefs: TESTHOLE



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)

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

0 5 10 15 20 25 metres

SCALE 1:500

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CLIENT: DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

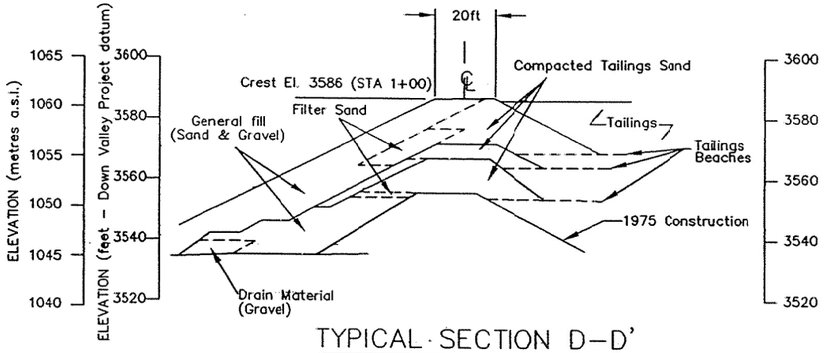
TITLE: INTERMEDIATE DAM
TYPICAL SECTION

PROJECT No. M09237A01

FIG. No. 2.5

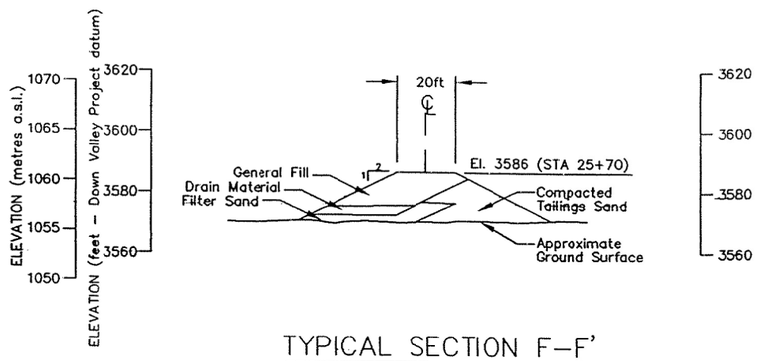
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Date: 11/3/2004
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Drawing File: M:\M09237A01 - Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\Figure2.5_7.dwg (cwong)



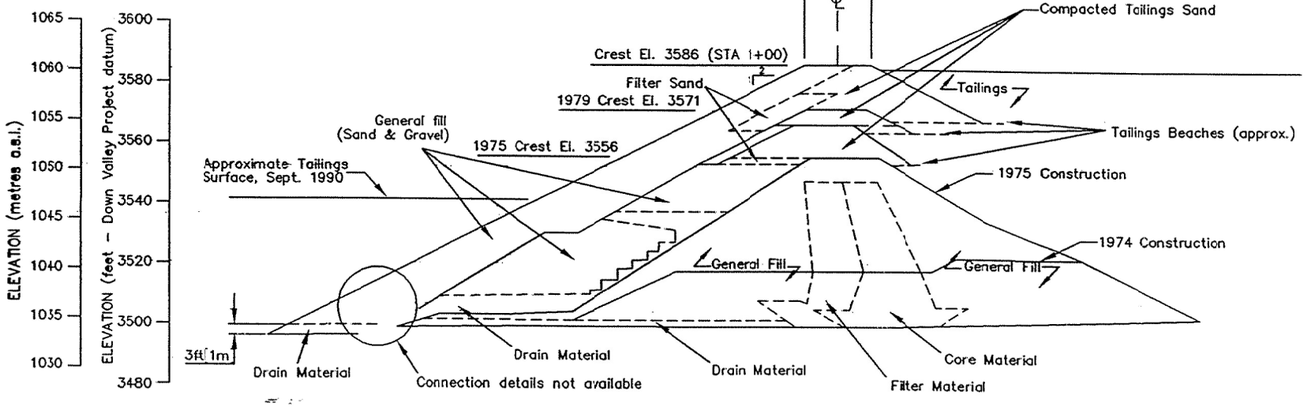
TYPICAL SECTION D–D'

(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.

0 20 40 60 80 feet

Scale 1"= 50'

NOT FOR CONSTRUCTION

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SCALE

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CLIENT

DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT

ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

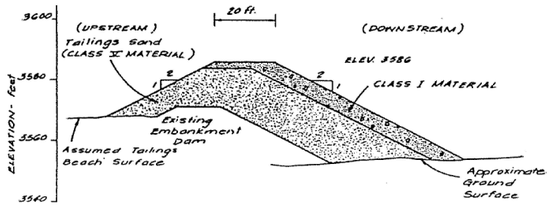
TITLE

SECONDARY DAM – WEST LIMB
TYPICAL SECTIONS

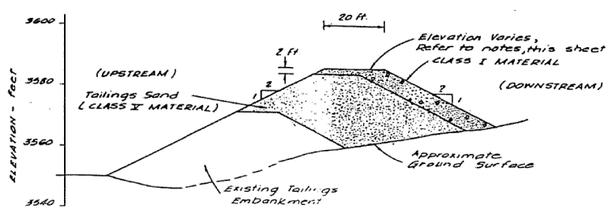
PROJECT No. M09237A01

FIG. No. 2.6

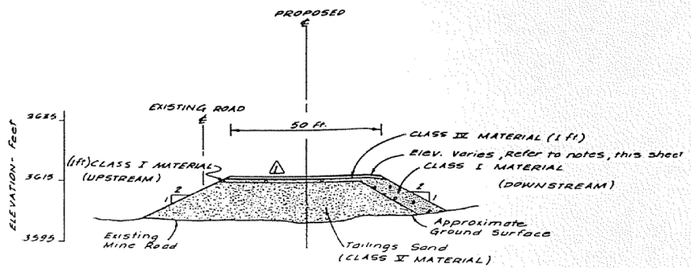
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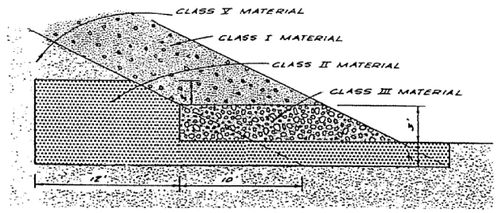
TRANSITION ZONE E-E'
TYPICAL SECTION
(REPRESENTS STATIONS 0+00 to 6+30)



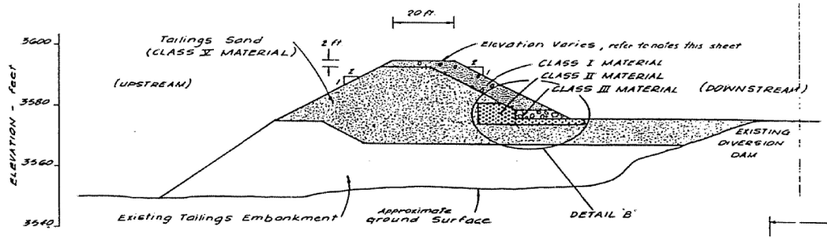
TYPICAL SECTION F-F'
(REPRESENTS STATIONS 6+30 to 28+00
AND 48+00 to 51+00)



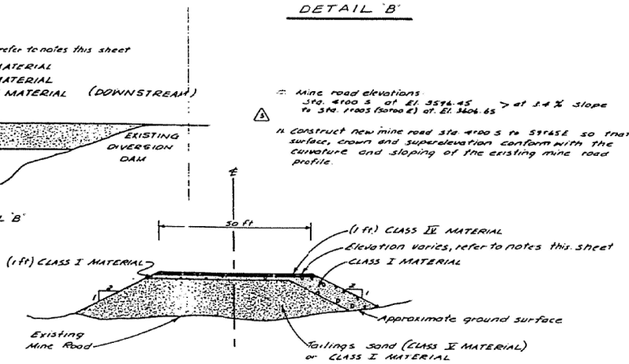
TYPICAL SECTION H-H'
(REPRESENTS STATIONS 51+00 to 61+30)



DETAIL B'



TYPICAL SECTION G-G'
(REPRESENTS STATIONS 28+00 to 48+00)



⚠ TYPICAL SECTION I-I'
(Represents stations 1+37 S (49+60 E) to 4+00 S on existing mine road toward Faro)

⚠ Mine road elevations
Sta. 4+00 S at El. 3574.45
to Sta. 1+00 S (same E) at El. 3604.65 > at 3.4 % slope

⚠ Construct new mine road Sta 4+00 S to 5S+65 E so that surface, crown and superelevation conform with the curvature and sloping of the existing mine road profile.

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. 3582 feet > at 0.40% slope
   to Station 30+00 at El. 3590 feet > at 0.67% slope ⚠2
   to Station 45+00 at El. 3600 feet > at 1.33% slope
   to End of Construction at El. 3620 feet
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)
20+00 E	50	36+00 E	43
31+00 E	49	37+00 E	43
32+00 E	48	39+00 E	42
33+00 E	47	39+00 S	41
34+00 E	46	39+65 E	40.35
35+00 E	45		

DRAWING TAKEN FROM:

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

Golder Associates
Golder Geotechnical Consultants Ltd.
Sheet 5 of 5

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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.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____ SCALE

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CLIENT
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ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

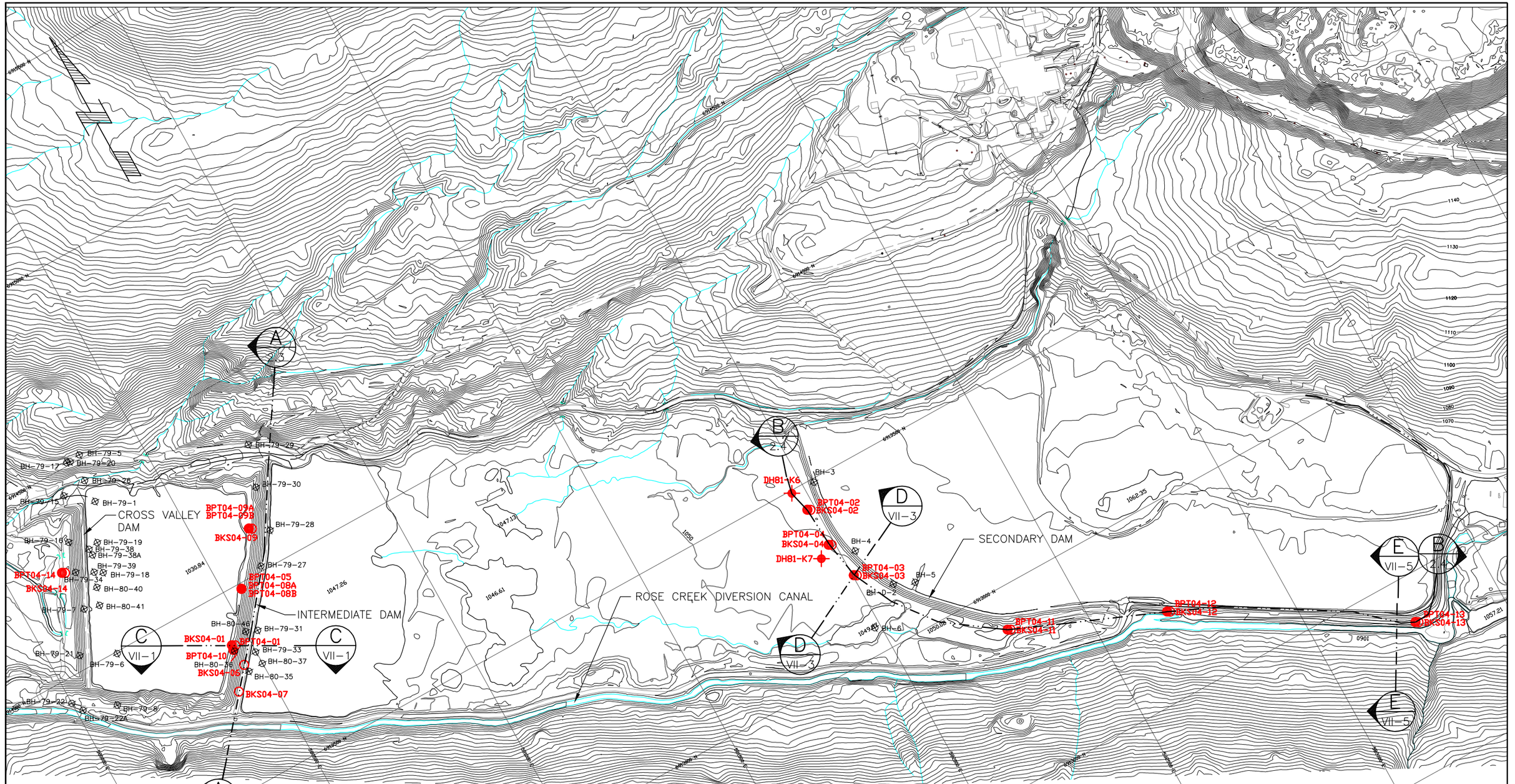
PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
SECONDARY DAM – EAST LIMB
TYPICAL SECTION

PROJECT No. M09237A01

FIG. No. 2.7

Date: 11/5/2004
Scale: 1=1(PS)
Drawing File: M:\M09237A01 – Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\Figure2.5_7.dwg (cwong)



LEGEND

BH-80-36	GOLDER TEST HOLE
BKS04-06	KLOHN CRIPPEN OPEN BECKER TEST
BPT04-10	KLOHN CRIPPEN CLOSED BECKER TEST
DH81-K7	KLOHN CRIPPEN TEST HOLE

NOTES:

1. BASE PLAN TAKEN FROM 2003 FARO SITE PLAN1_Ver2000.DWG PROVIDED BY SRK.
2. CONTOUR INTERVAL IS 2m RELATIVE TO MEAN SEA LEVEL.
3. BH3 TO BH6 AND BH-D-2 LOCATIONS ARE APPROXIMATE.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

SCALE 0 500 m

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CLIENT: DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

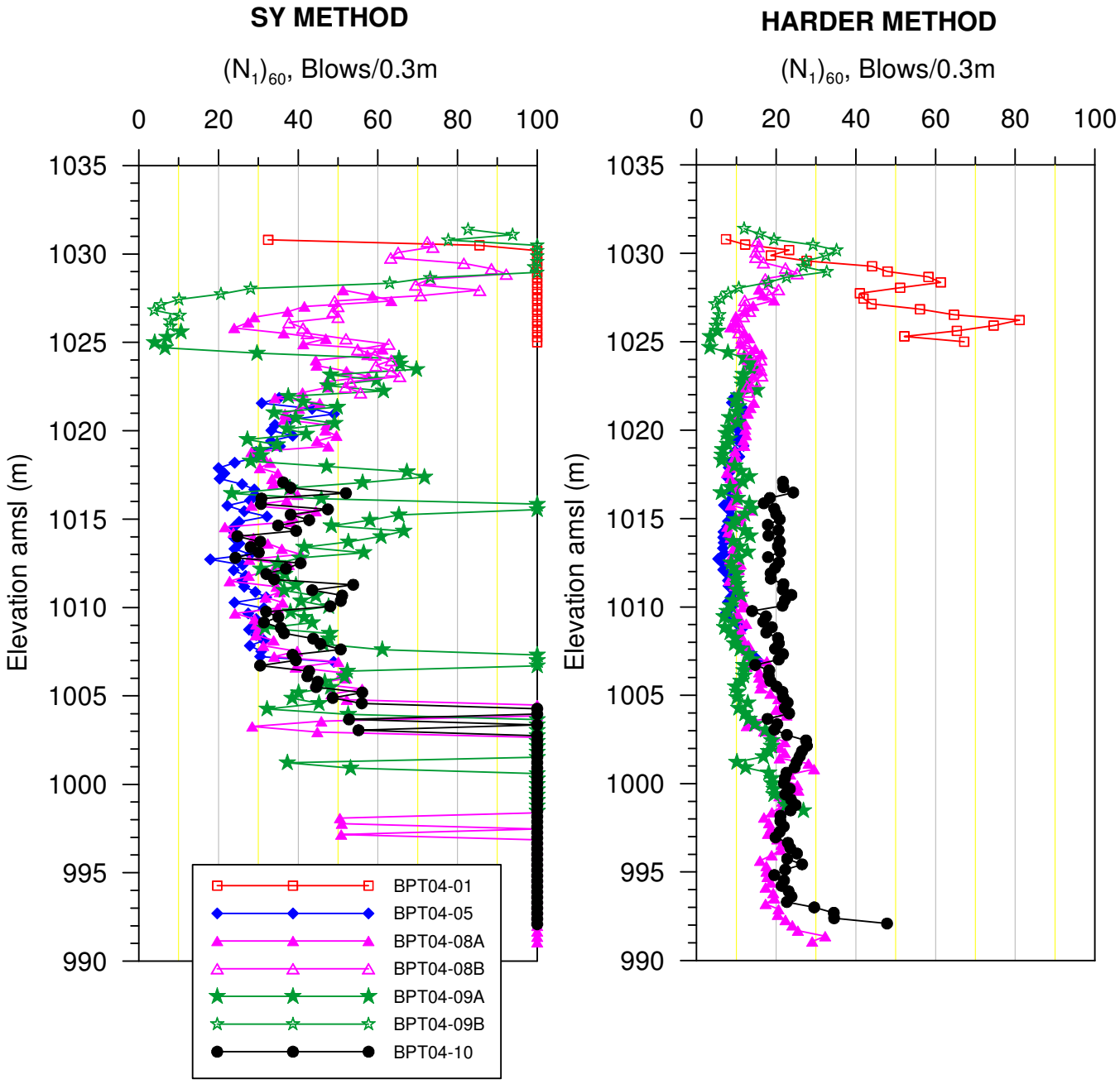
TITLE: TEST HOLE LOCATION PLAN

PROJECT No. M09237A01

FIG. No. 4.1

KCR-BM

Date: 11/8/2004
Scale: 1=5(PS)
Drawing File: M\M09237A01 - Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\figure 4.1.dwg (cwong)
Xrefs: TESTHOLE



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TO BE READ WITH KLOHN-CRIPPEN REPORT DATED MAY 05, 2005

SCALE

KLOHN CRIPPEN

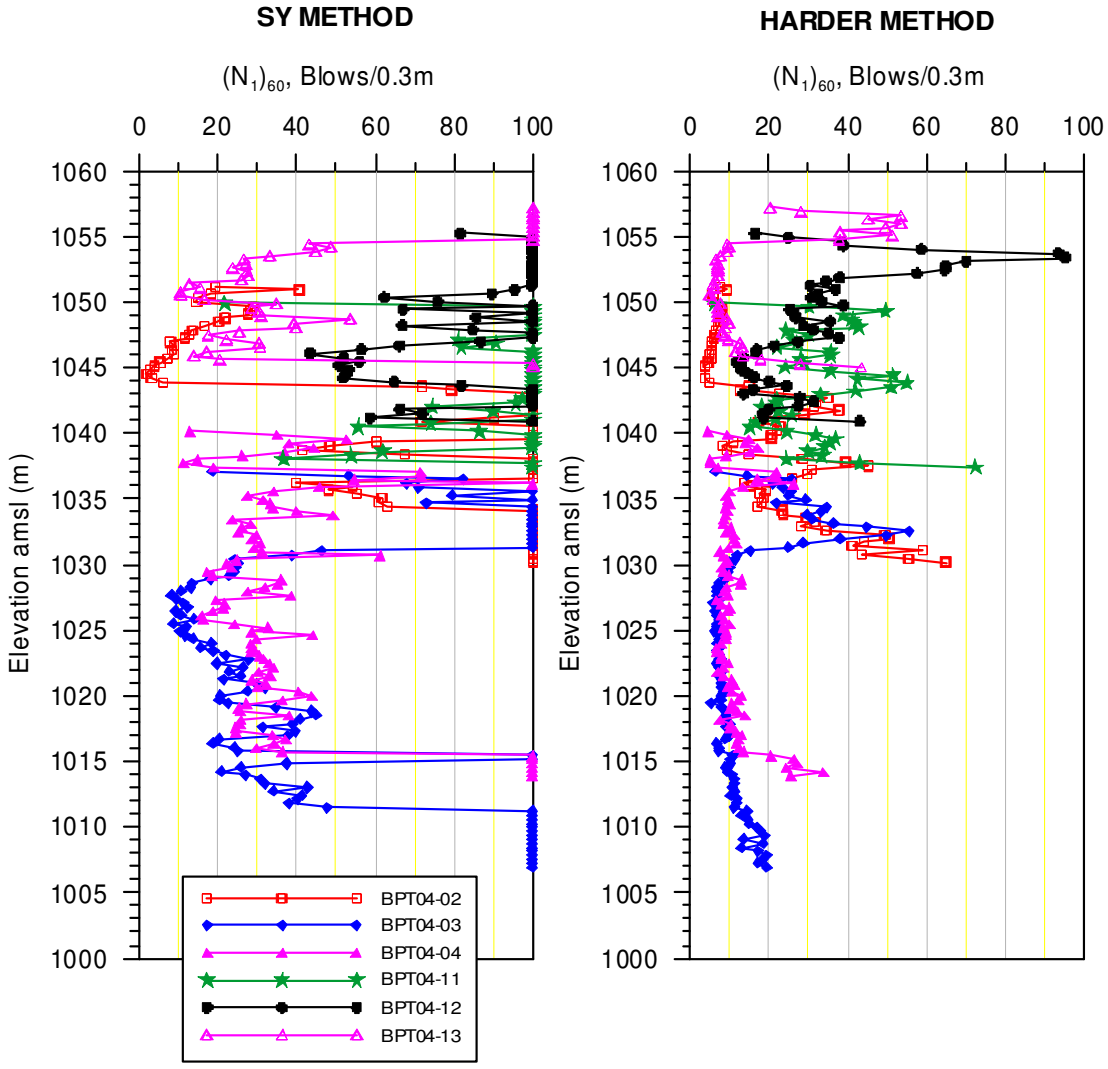
PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
SUMMARY OF EQUIVALENT SPT $(N_1)_{60}$ AT THE INTERMEDIATE DAM BY SY AND HARDER METHODS

CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M 09237 A01 01	4.2	

APPROVAL



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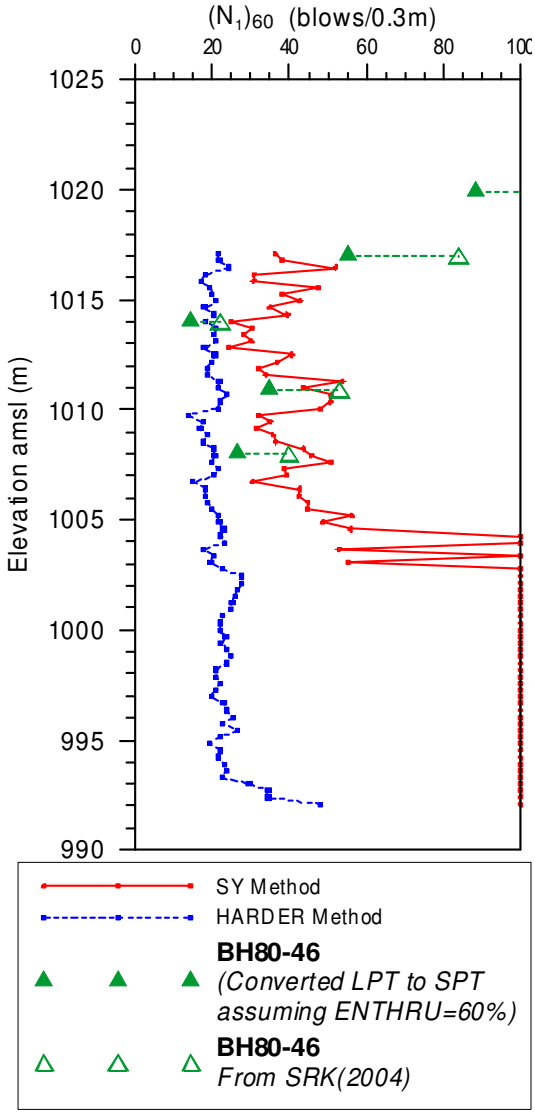
PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
SUMMARY OF EQUIVALENT SPT $(N_1)_{60}$ AT THE SECONDARY DAM BY SY AND HARDER METHODS

CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	4.3	

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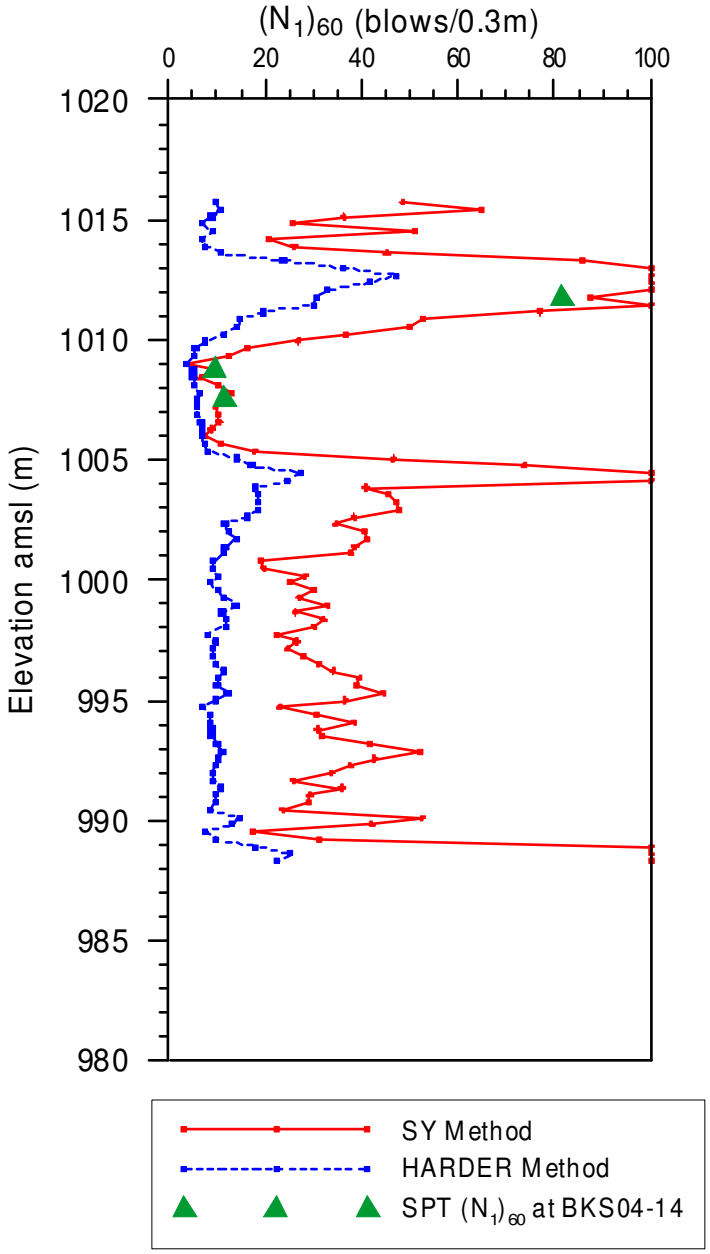
PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
COMPARISON OF EQUIVALENT SPT $(N_1)_{60}$ AT BPT04-10 AND BH80-46 AT THE INTERMEDIATE DAM

CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	4.4	

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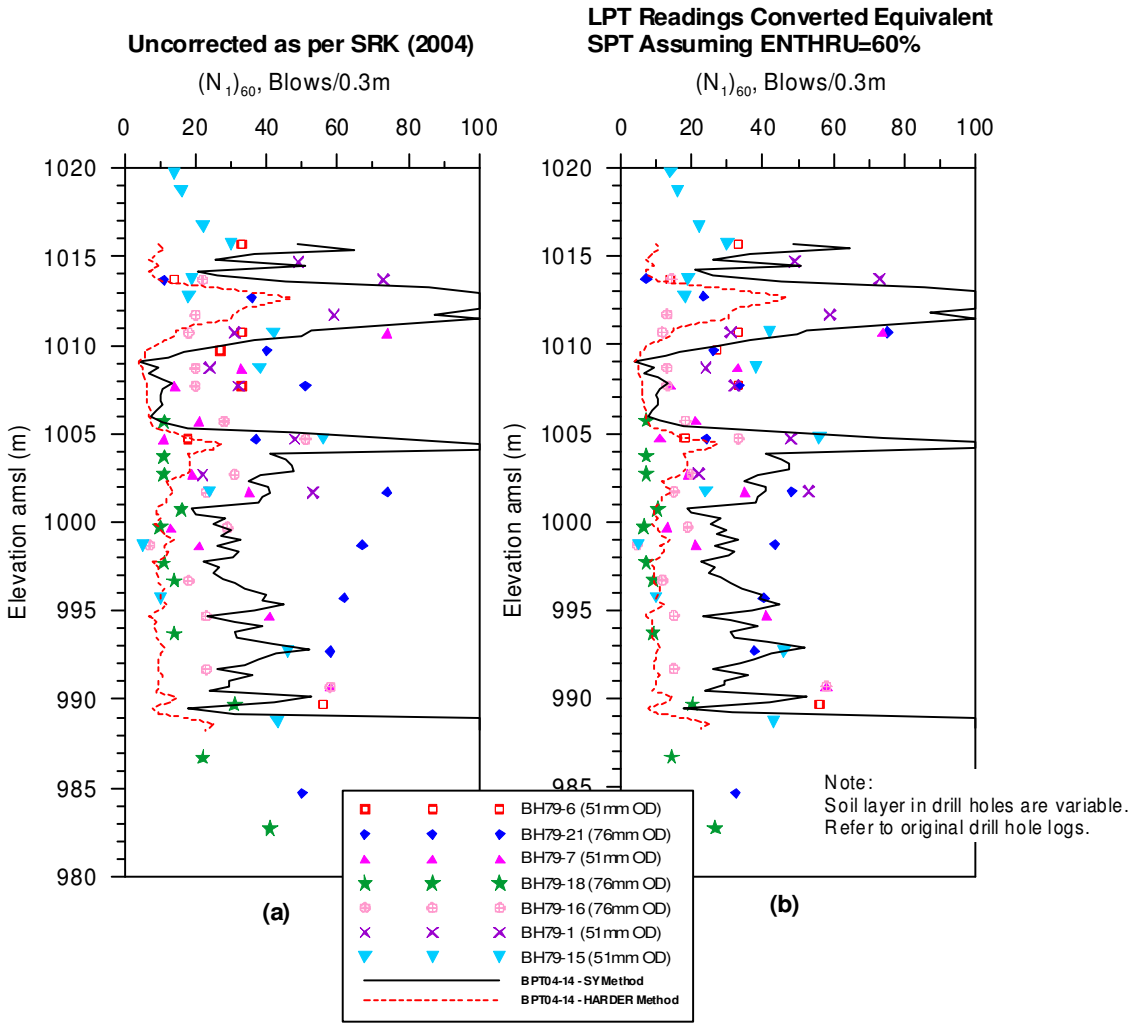
PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE: COMPARISON OF EQUIVALENT SPT $(N_1)_{60}$ AT BPT04-14 AND BKS04-14 AT THE CROSS VALLEY DAM

CLIENT: **DELOTTE & TOUCHE INC.**
ANVIL MINING RANGE COMPLEX

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	4.5	

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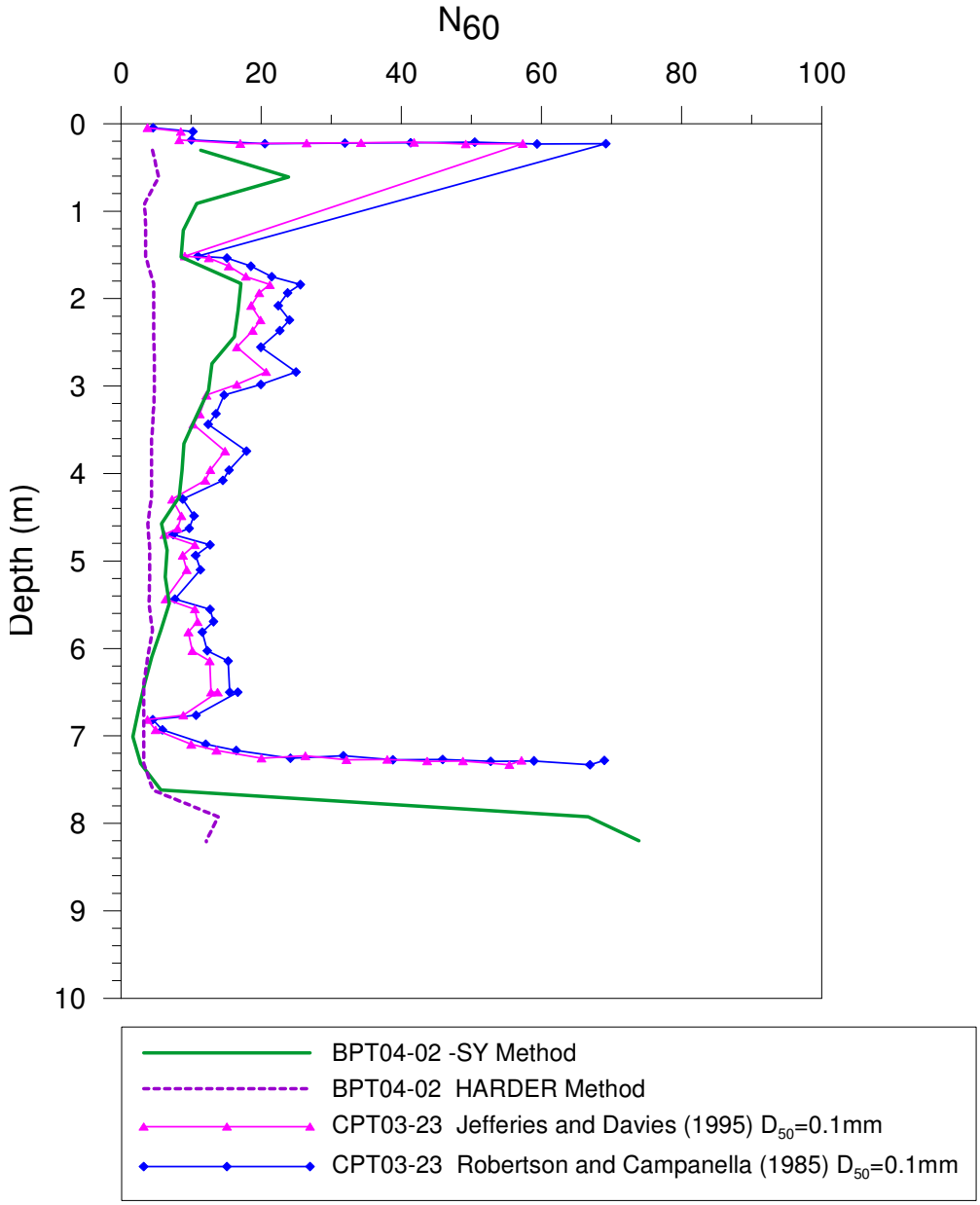
PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: SUMMARY OF SPT $(N_1)_{60}$ AT THE CROSS VALLEY DAM

CLIENT: **DELOTTE & TOUCHE INC.** ANVIL MINING RANGE COMPLEX

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	4.6	

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SCALE

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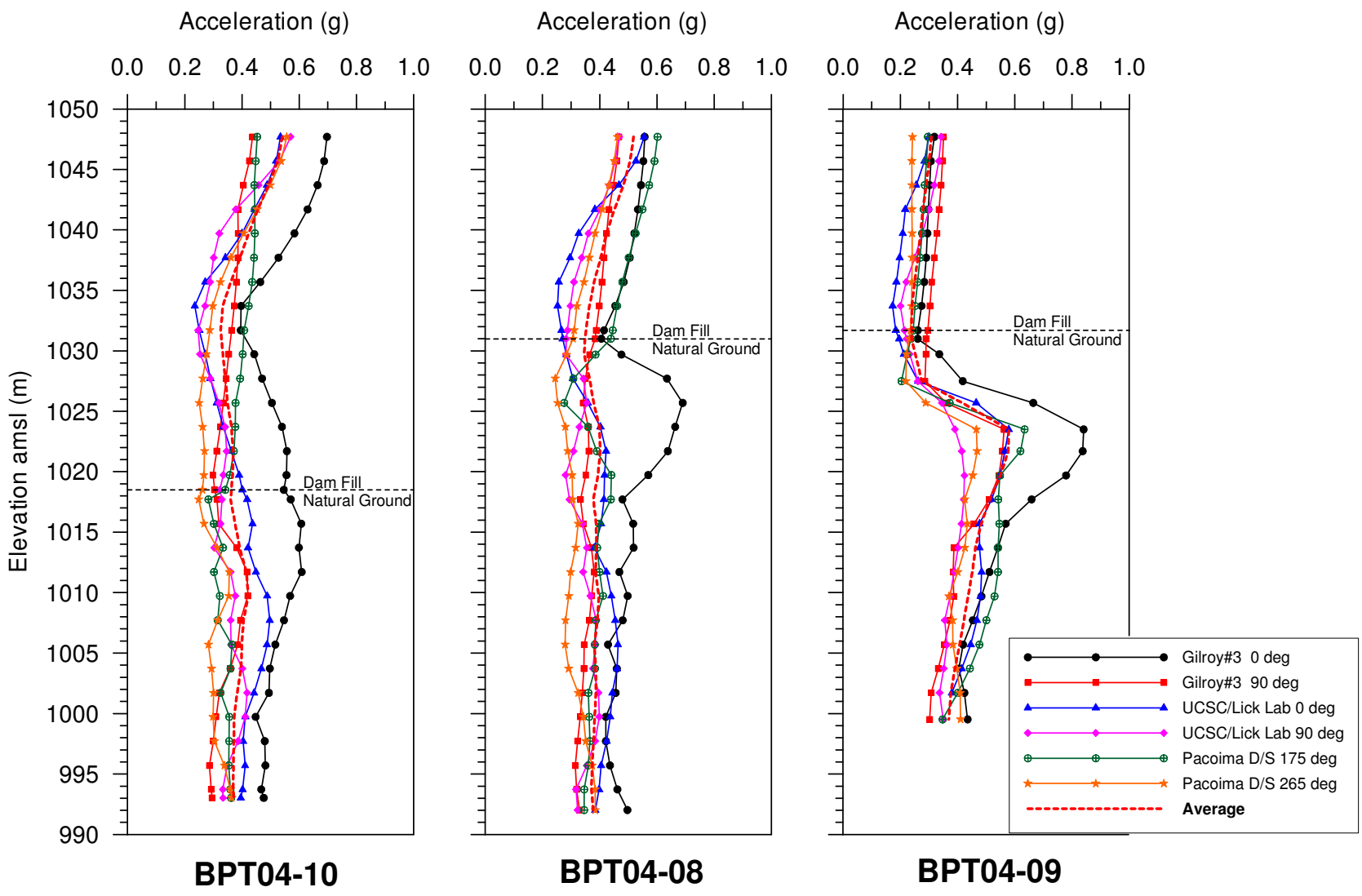
PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE: COMPARISON OF SPT N_{60} IN TAILINGS ESTIMATED FROM BPT DATA AT BPT04-02 AND CPT DATA AT CPT03-23

CLIENT: **DELOTTE & TOUCHE INC.**
ANVIL MINING RANGE COMPLEX

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	4.7	

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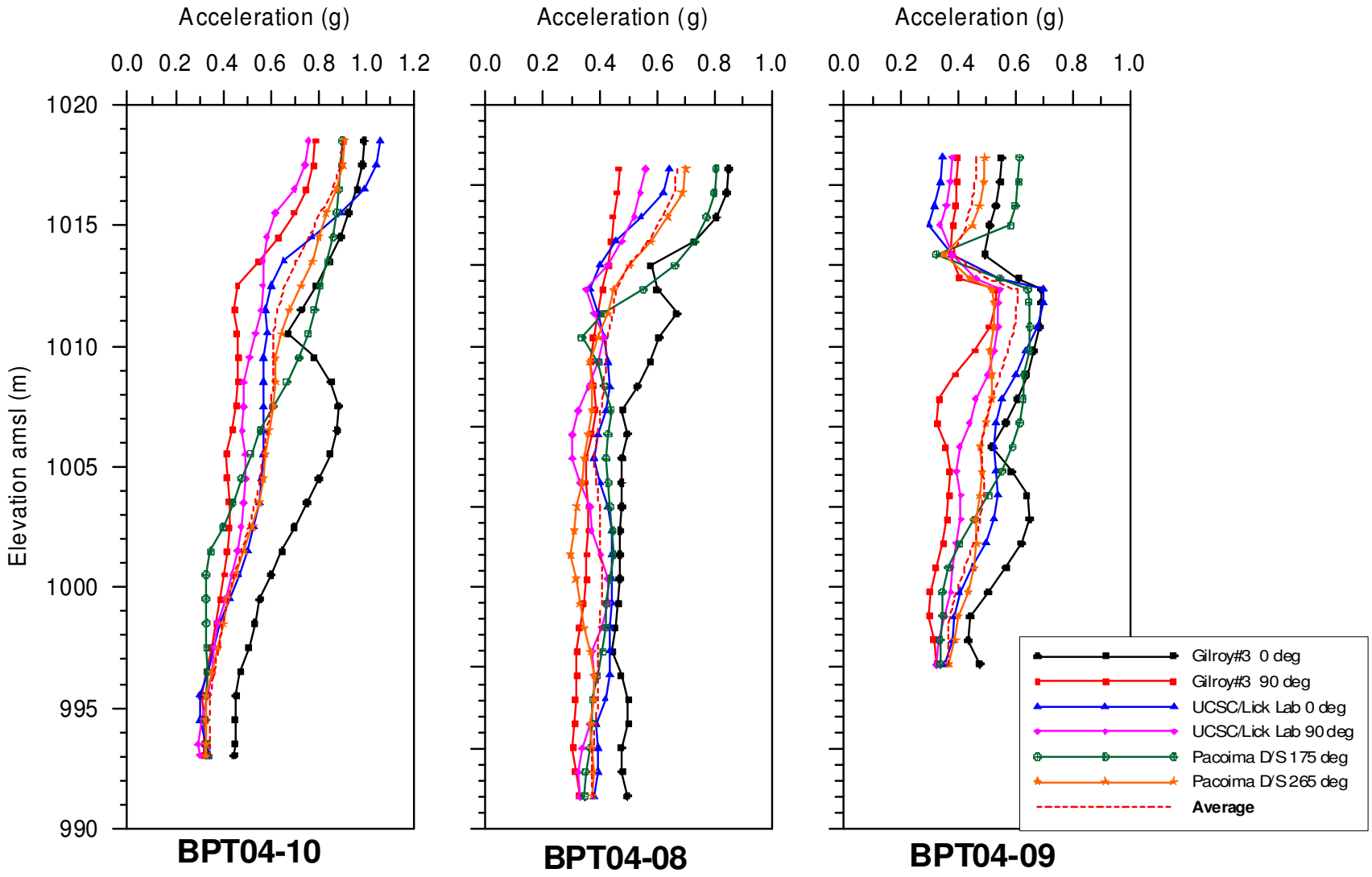
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
PEAK ACCELERATION PROFILES THROUGH THE DAM CREST AT THE INTERMEDIATE DAM

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	5.1	

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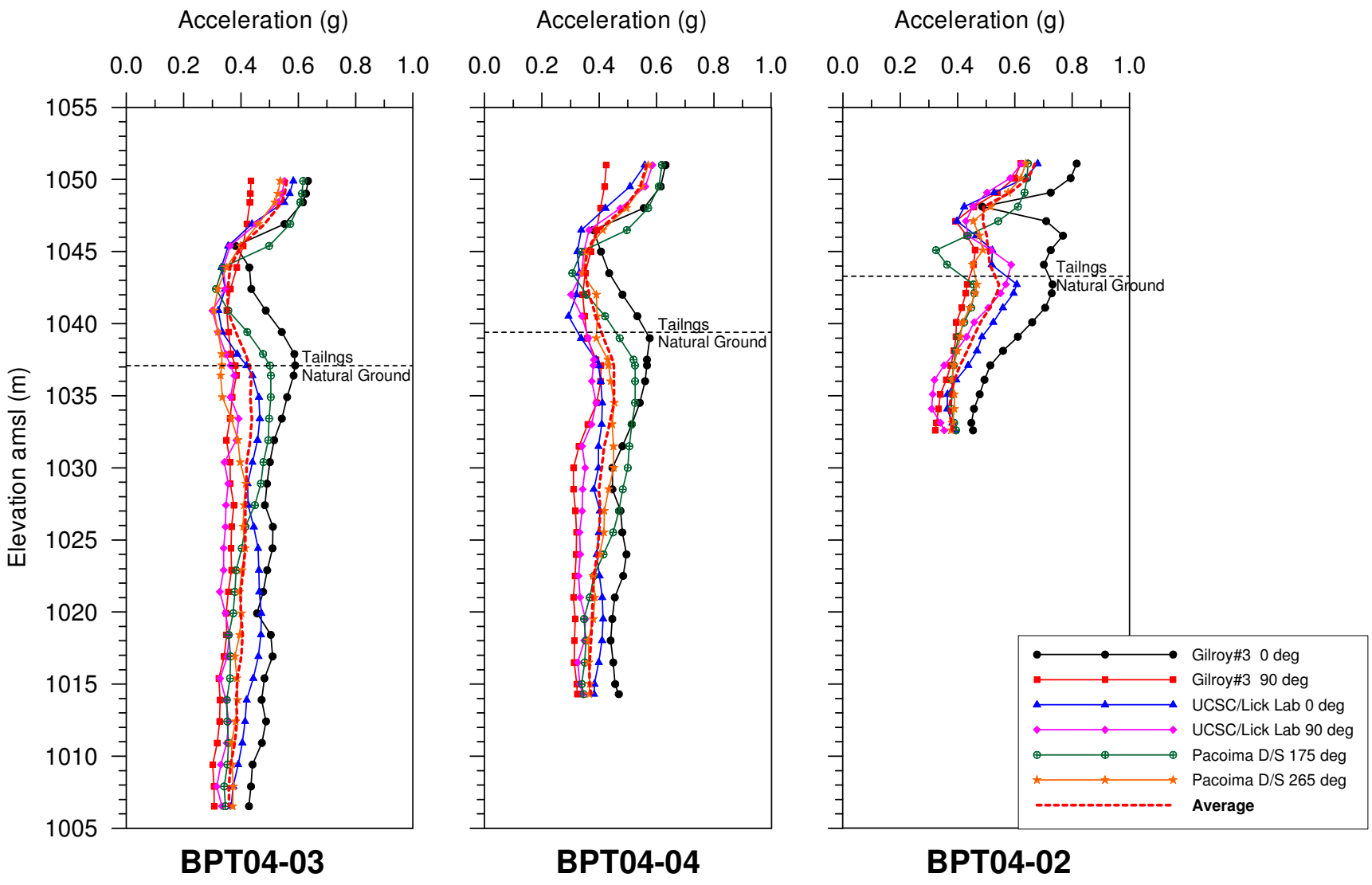
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
PEAK ACCELERATION PROFILES THROUGH THE DAM TOE
AT THE INTERMEDIATE DAM

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	5.2	

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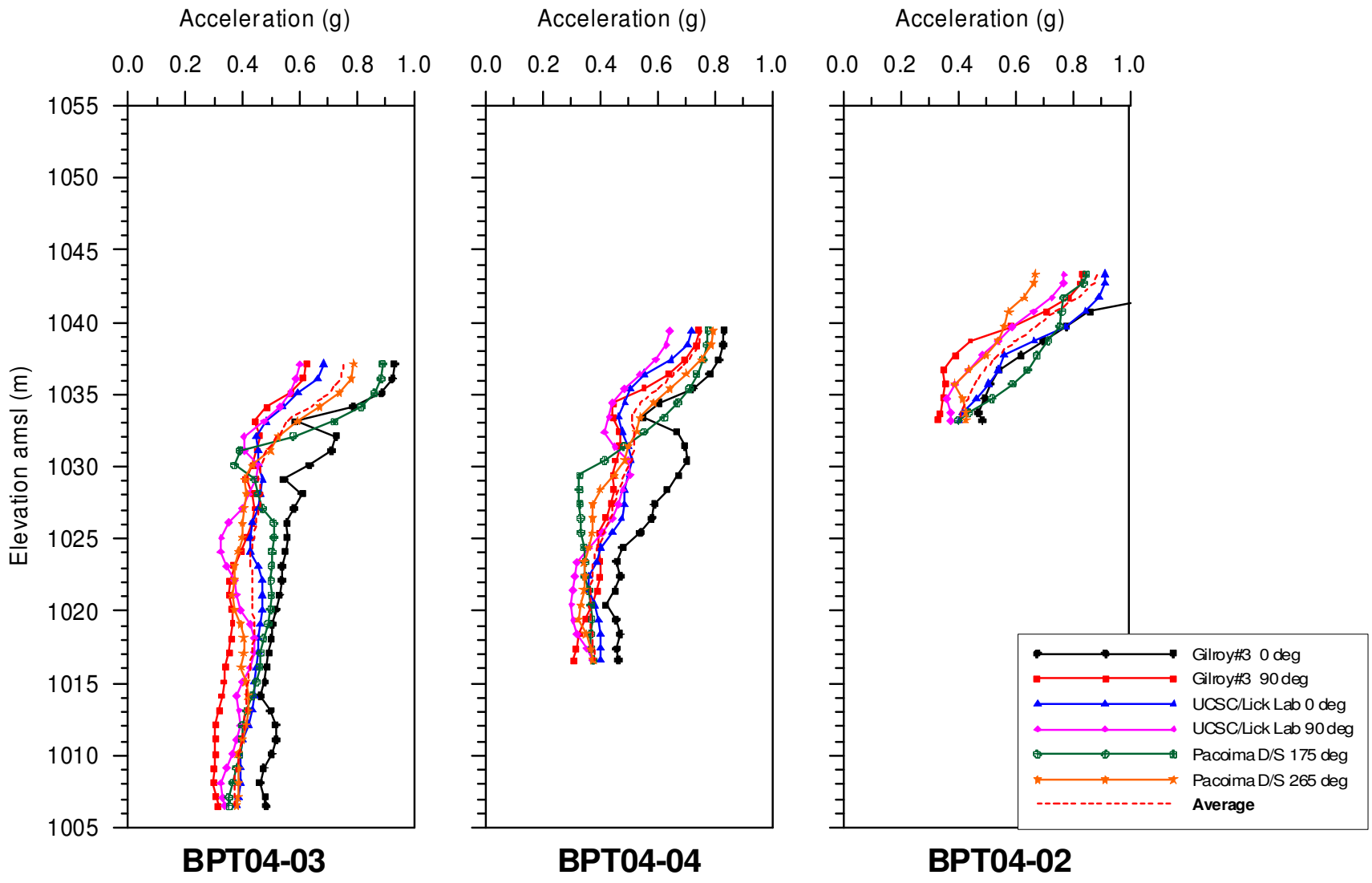
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: PEAK ACCELERATION PROFILES AT THE SECONDARY DAM TOE WITH TAILINGS

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	5.3	

APPROVAL



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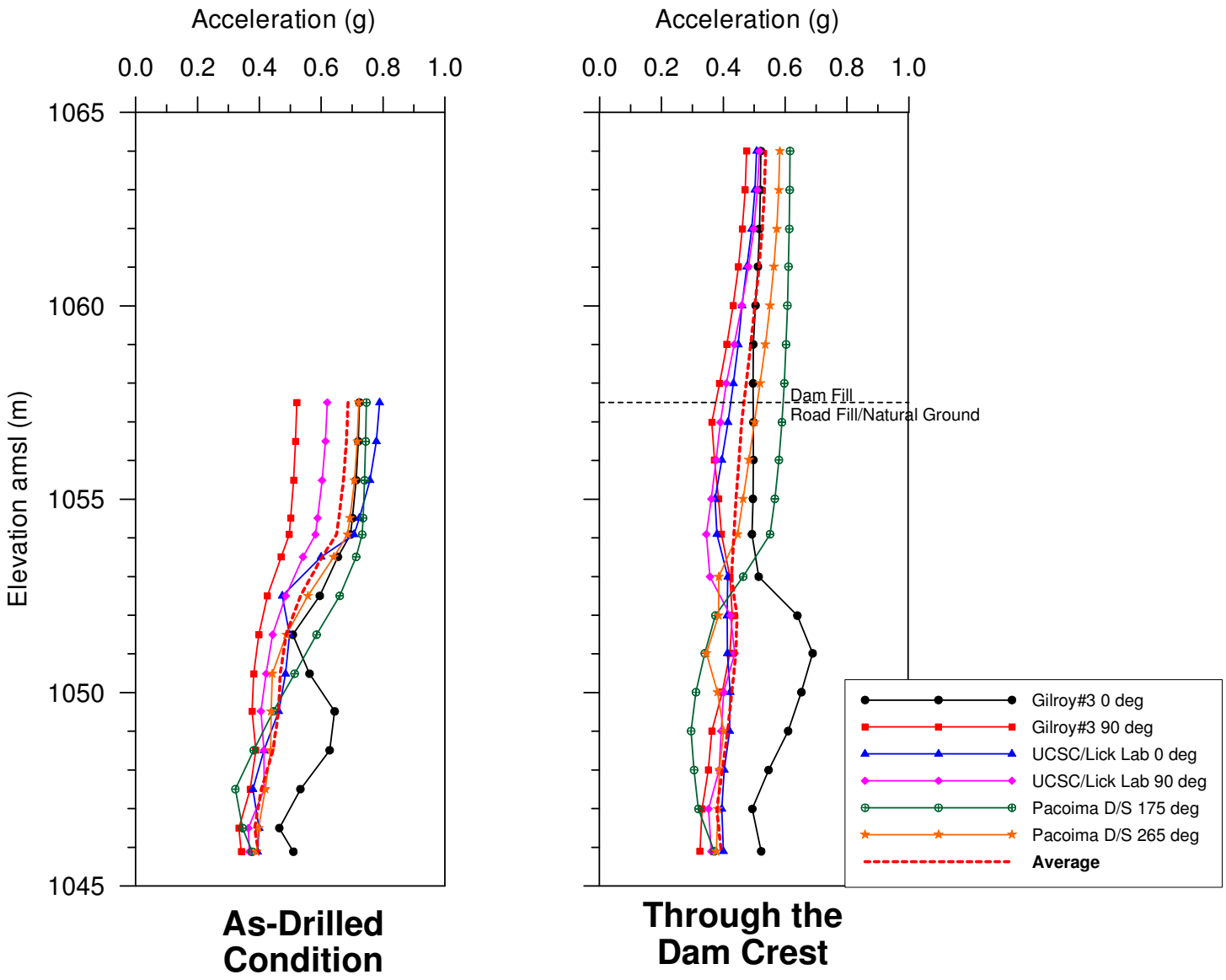
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: PEAK ACCELERATION PROFILES AT THE SECONDARY DAM WITH NO TAILINGS AT THE DOWNSTREAM TOE

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	5.4	

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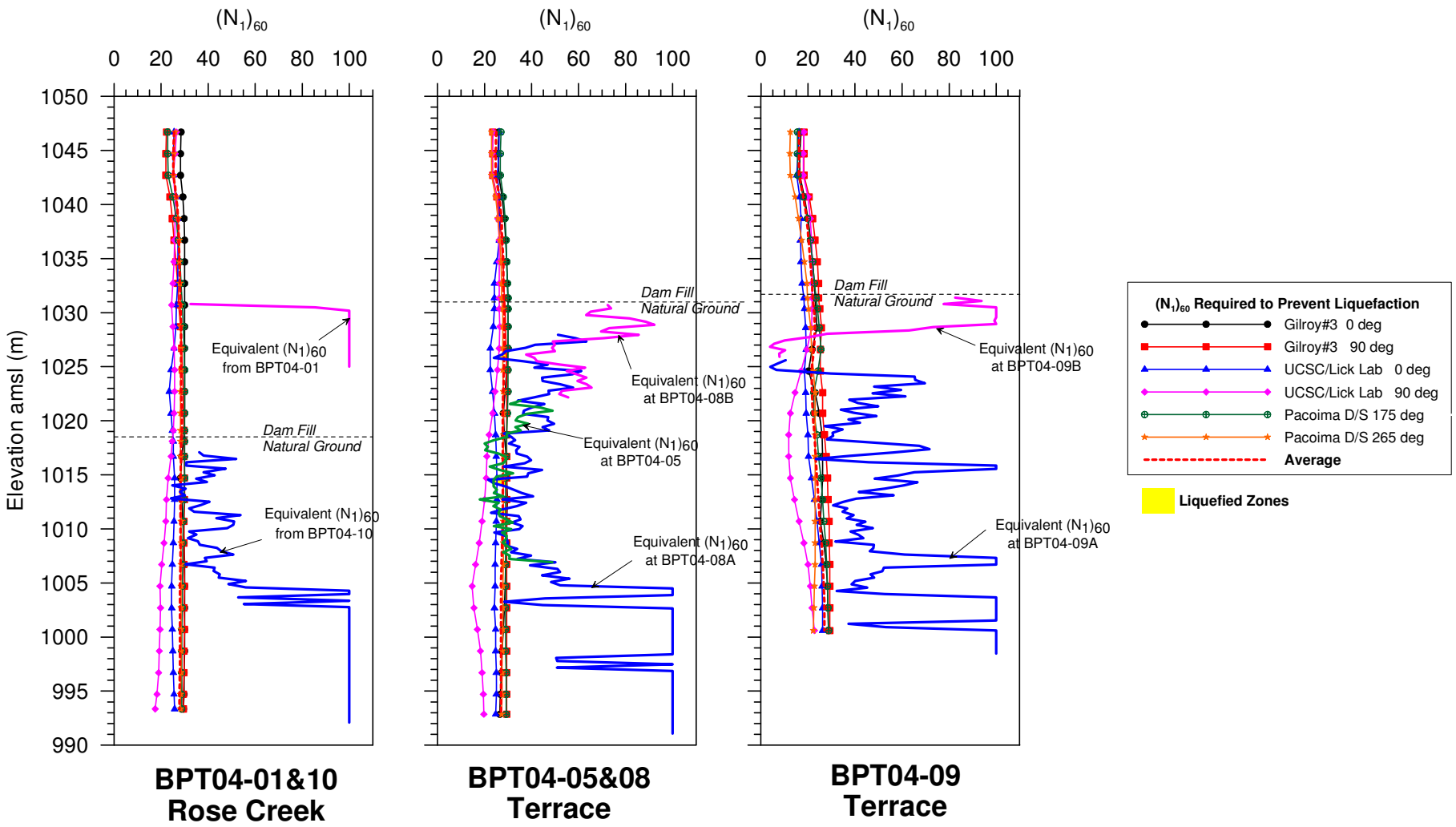
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: PEAK ACCELERATION PROFILES AT THE SECONDARY DAM EAST LIMB AT BPT04-13

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	5.5	

APPROVAL



Notes:
1. Measured BPT blow counts converted to equivalent N_{60} using the Sy (1993) method.
2. Required $(N_1)_{60}$ computed from EQ induced CSR following Youd et al. (2001).
2. Equivalent $(N_1)_{60}$ greater than 100 plotted as 100 for illustration only.
3. Dam fill and natural ground interface estimated from BPT tests.

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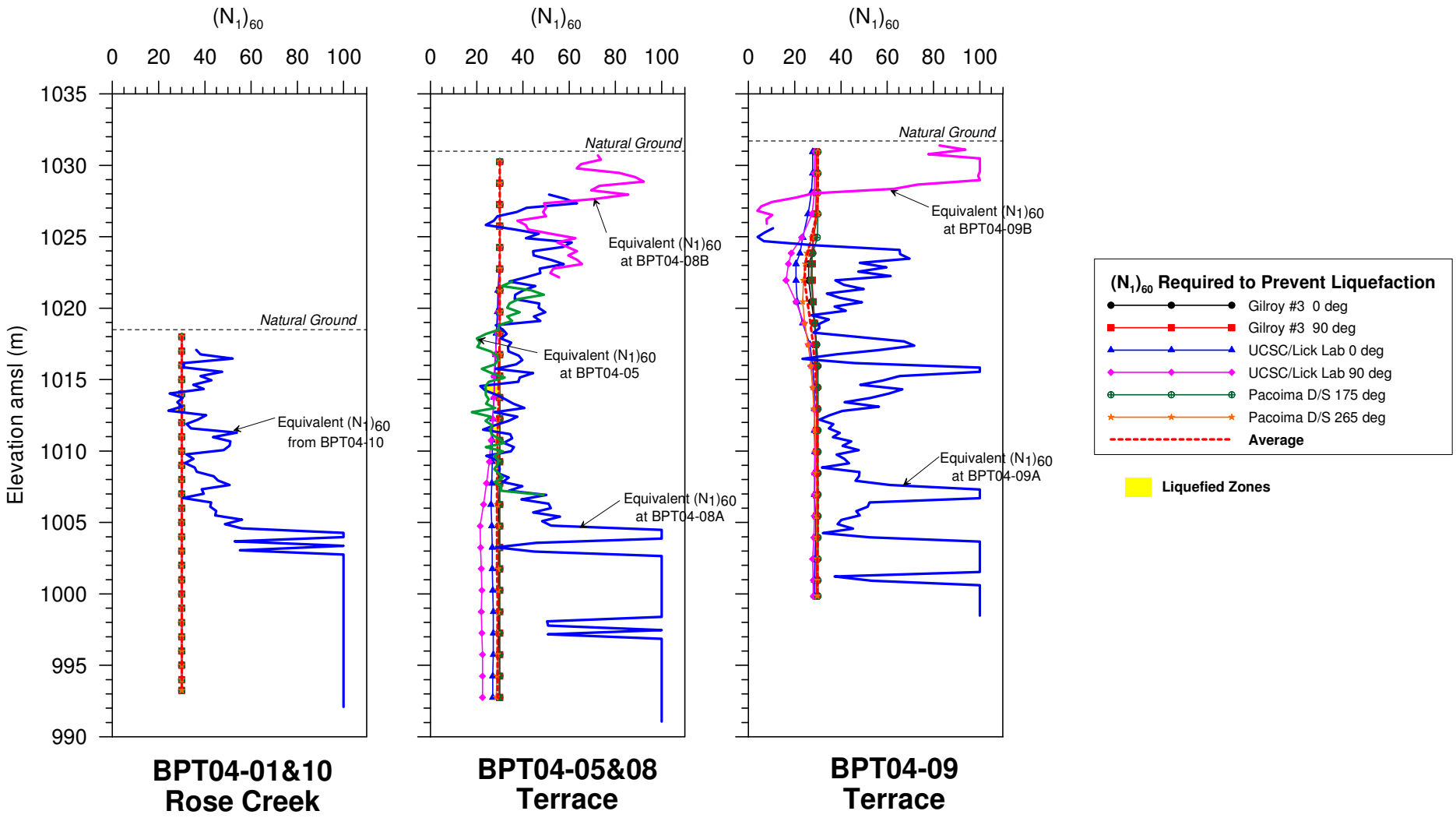
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: LIQUEFACTION ASSESSMENT BELOW THE INTERMEDIATE DAM CENTERLINE

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	6.1	

APPROVAL



Notes:
1. Measured BPT blow counts converted to equivalent N_{60} using the Sy (1993) method.
2. Required $(N_1)_{60}$ computed from EQ induced CSR following Youd et al. (2001).
2. Equivalent $(N_1)_{60}$ greater than 100 plotted as 100 for illustration only.
3. Dam fill and natural ground interface estimated from BPT tests.

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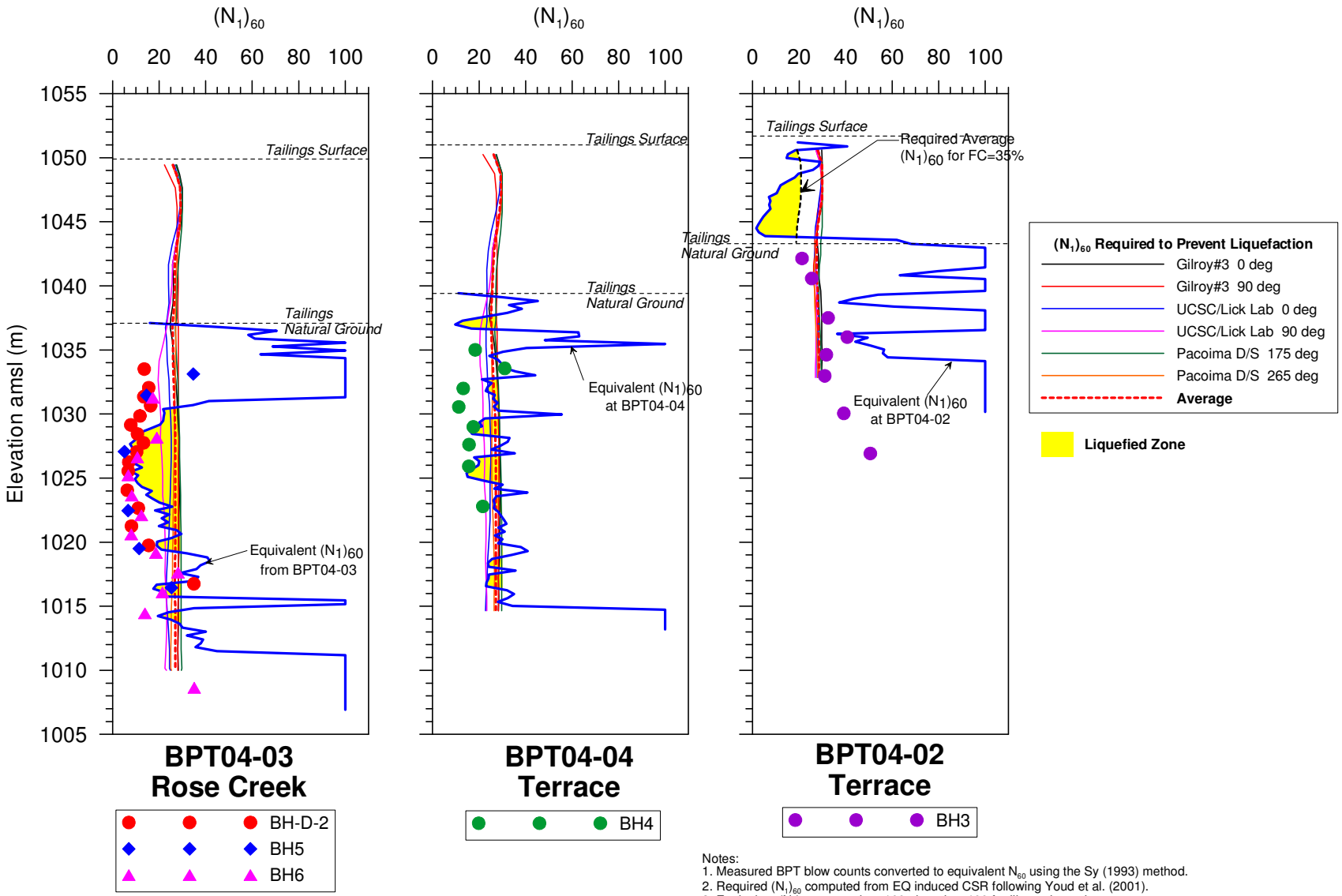
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: LIQUEFACTION ASSESSMENT AT THE INTERMEDIATE DAM TOE

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	6.2	

APPROVAL



Notes:
1. Measured BPT blow counts converted to equivalent N_{60} using the Sy (1993) method.
2. Required $(N_1)_{60}$ computed from EQ induced CSR following Youd et al. (2001).
2. Equivalent $(N_1)_{60}$ greater than 100 plotted as 100 for illustration only.
3. Tailings and natural ground interface estimated from BPT tests.
4. SPT blow counts at BH3, BH4, BH5 and BH6 were taken from Section A-A and B-B shown on Figure 5 of Golder Associates(??) Original borehole logs were not available to KC. Figure 4.1 shows the approximate location of these boreholes.
5. SPTs at BH2, BH3, BH4, BH5 and BH6 were assumed as conventional SPTs with ENTHRU of 60%. They were corrected for effective overburden stress effect assuming the watertable was same as that atRose Creek at the time of the testing.
6. ENTHRU of SPTs at BH-D-2 were assumed as 60%. Figure 4.1 shows the approximate location of this borehole.

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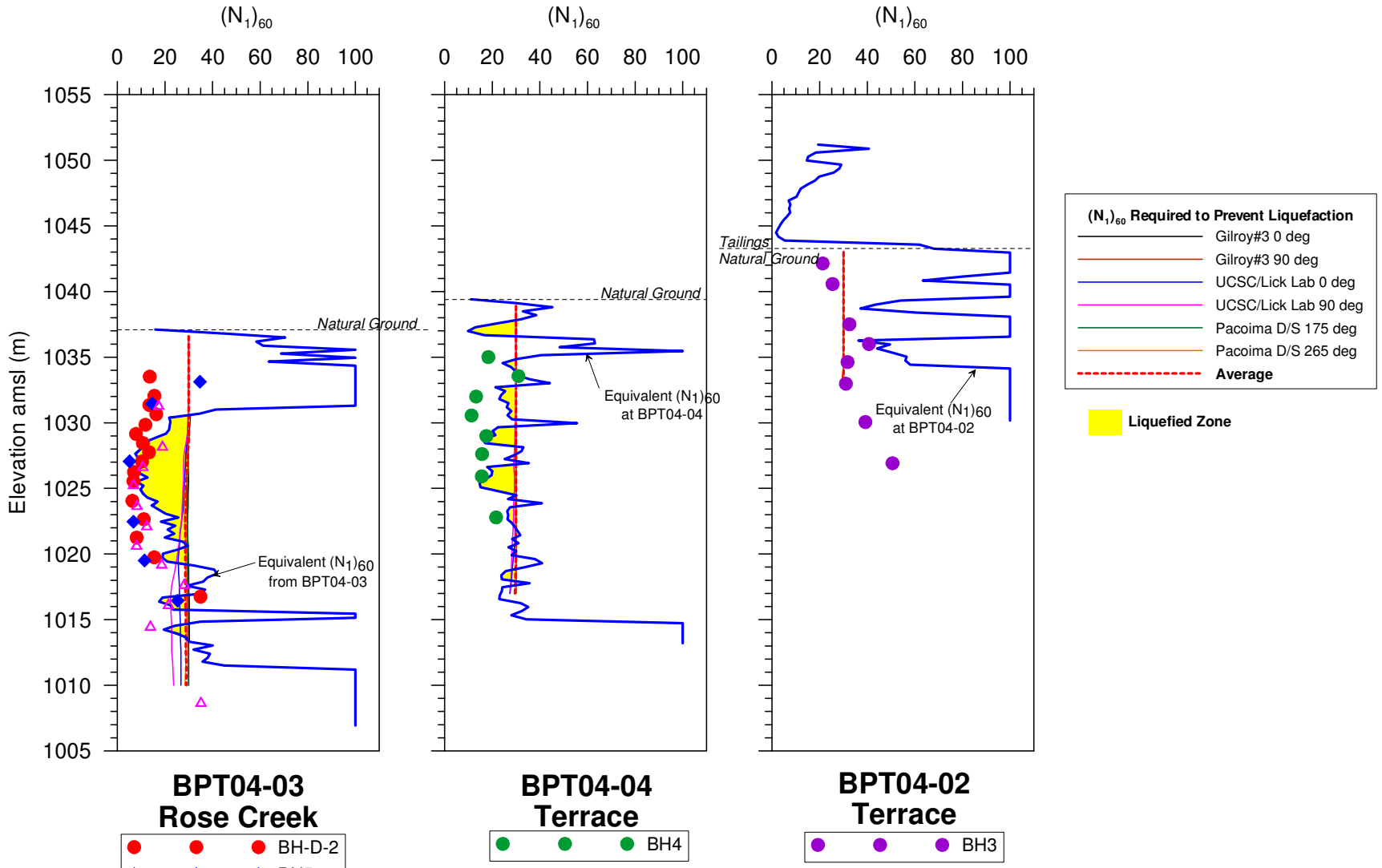
KLOHN CRIPPEN

CLIENT: **DELOTTE & TOUCHE INC.**
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: LIQUEFACTION ASSESSMENT AT THE SECONDARY DAM (WEST LIMB) TOE WITH TAILINGS

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
APPROVAL	M09237A01 01	6.3	



Notes:
1. Measured BPT blow counts converted to equivalent N_{60} using the Sy (1993) method.
2. Required $(N_1)_{60}$ computed from EQ induced CSR following Youd et al. (2001).
2. Equivalent $(N_1)_{60}$ greater than 100 plotted as 100 for illustration only.
3. Tailings and natural ground interface estimated from BPT tests.
4. SPT blow counts at BH3, BH4, BH5 and BH6 were taken from Section A-A and B-B shown on Figure 5 of Golder Associates(??) Original borehole logs were not available to KC. Figure 4.1 shows the approximate location of these boreholes.
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6. ENTHRU of SPTs at BH-D-2 were assumed as 60%. Figure 4.1 shows the approximate location of this borehole.

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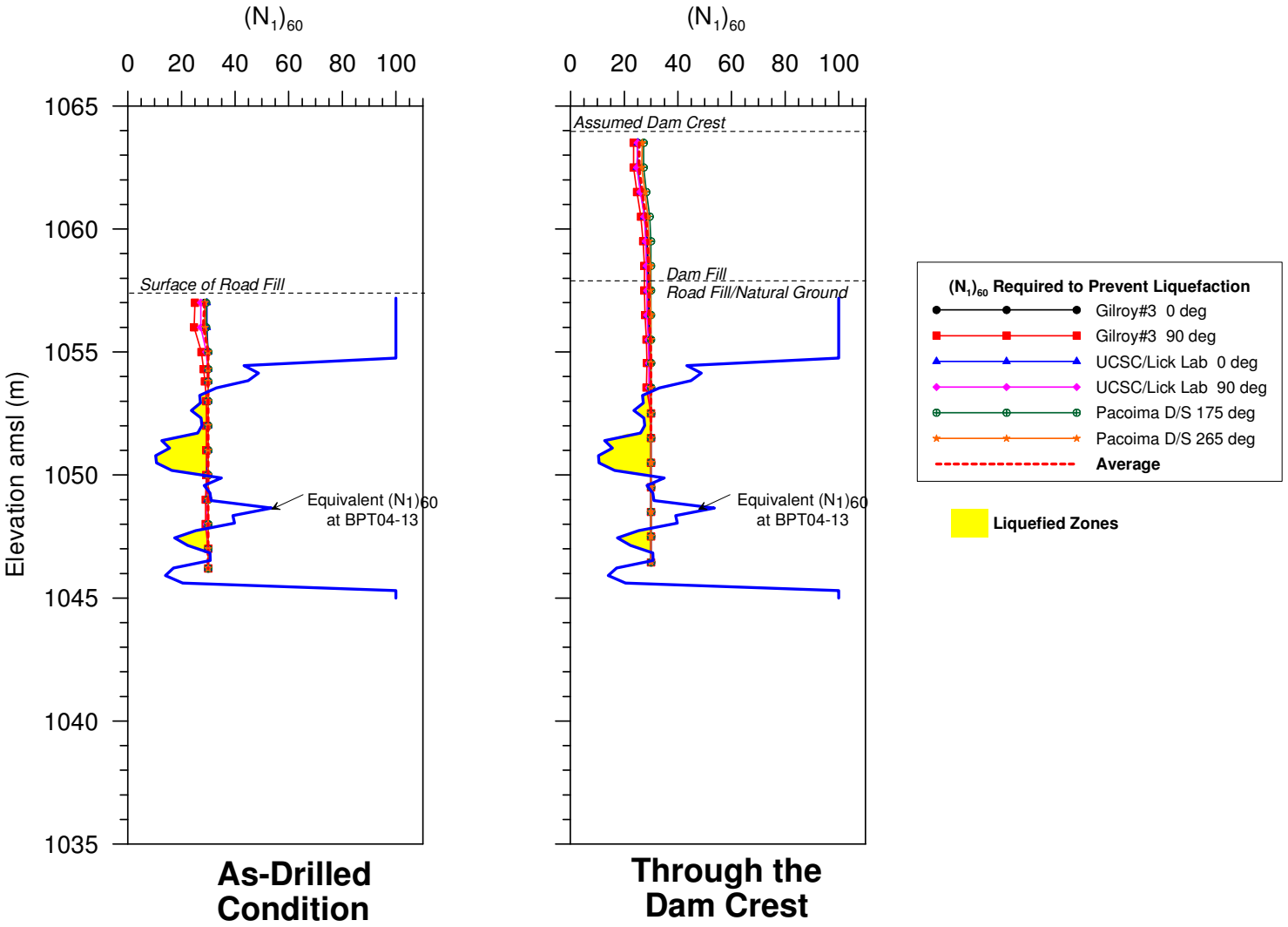
KLOHN CRIPPEN

CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

TITLE: LIQUEFACTION ASSESSMENT AT THE SECONDARY DAM (WEST LIMB) TOE WITHOUT TAILINGS

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
APPROVAL	M09237A01 01	6.4	



Notes:
1. Measured BPT blow counts converted to equivalent N_{60} using the Sy (1993) method.
2. Required $(N_1)_{60}$ computed from EQ induced CSR following Youd et al. (2001).
2. Equivalent $(N_1)_{60}$ greater than 100 plotted as 100 for illustration only.

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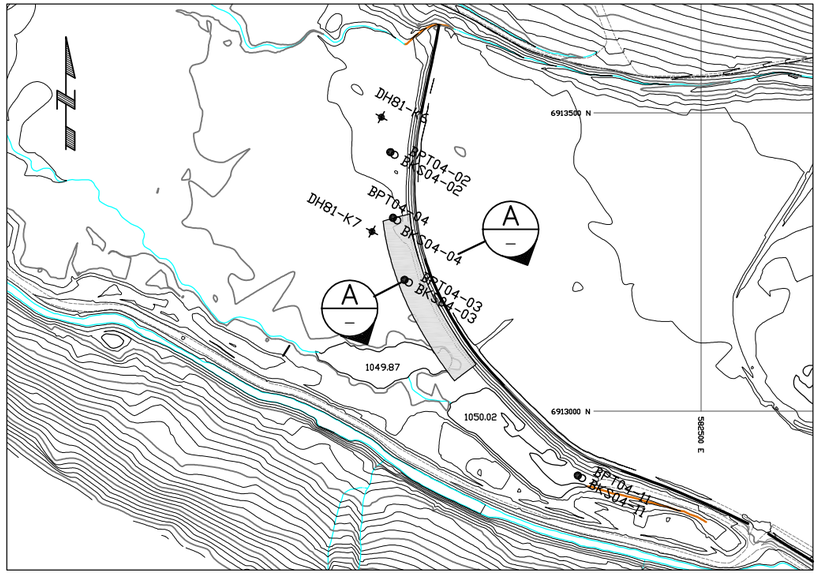
CLIENT:
DELOTTE & TOUCHE INC.
ANVIL MINING RANGE COMPLEX

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT SEISMIC STABILITY ASSESSMENT

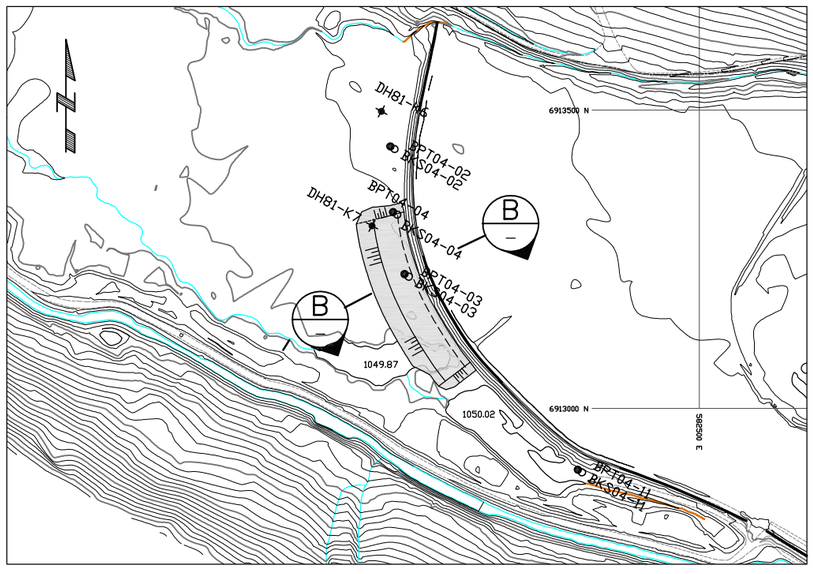
TITLE: LIQUEFACTION ASSESSMENT AT THE SECONDARY DAM (EAST LIMB) AT BPT04-13

DATE OF ISSUE	PROJECT No.	FIGURE. No.	REV.
NOV. 08, 2004	M09237A01 01	6.5	

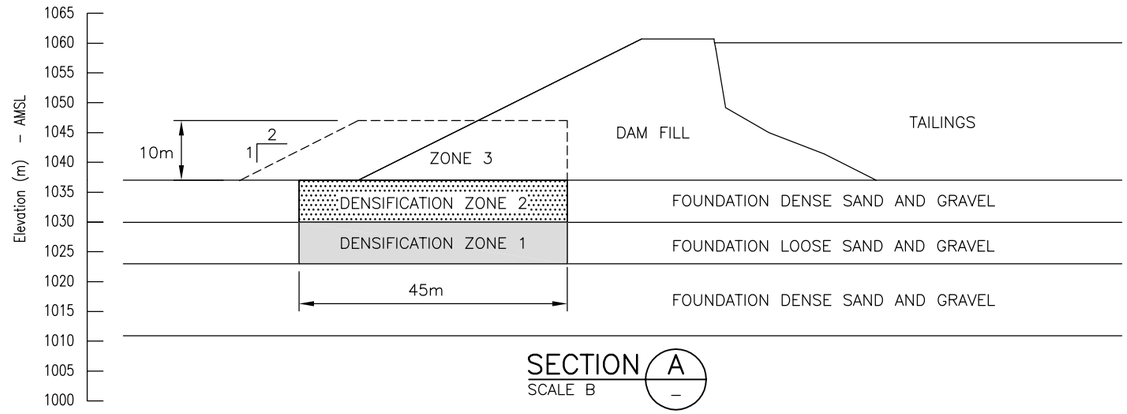
APPROVAL



PLAN 1
SCALE A

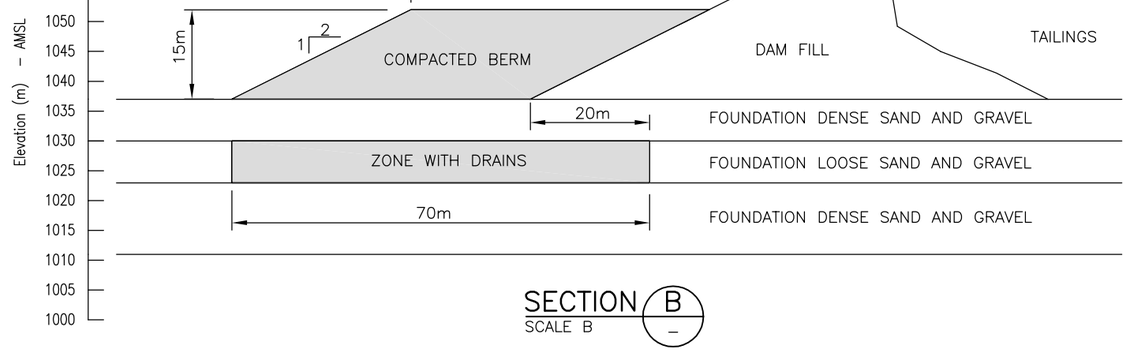


PLAN 2
SCALE A



SECTION A
SCALE B

CONCEPTUAL REMEDIATION OPTION 1: GROUND Ddensification



SECTION B
SCALE B

CONCEPTUAL REMEDIATION OPTION 2: COMPACTED BERM AND DRAINS

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

SCALE B 0 50 m

SCALE A 0 500 m

SCALE

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CLIENT
DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

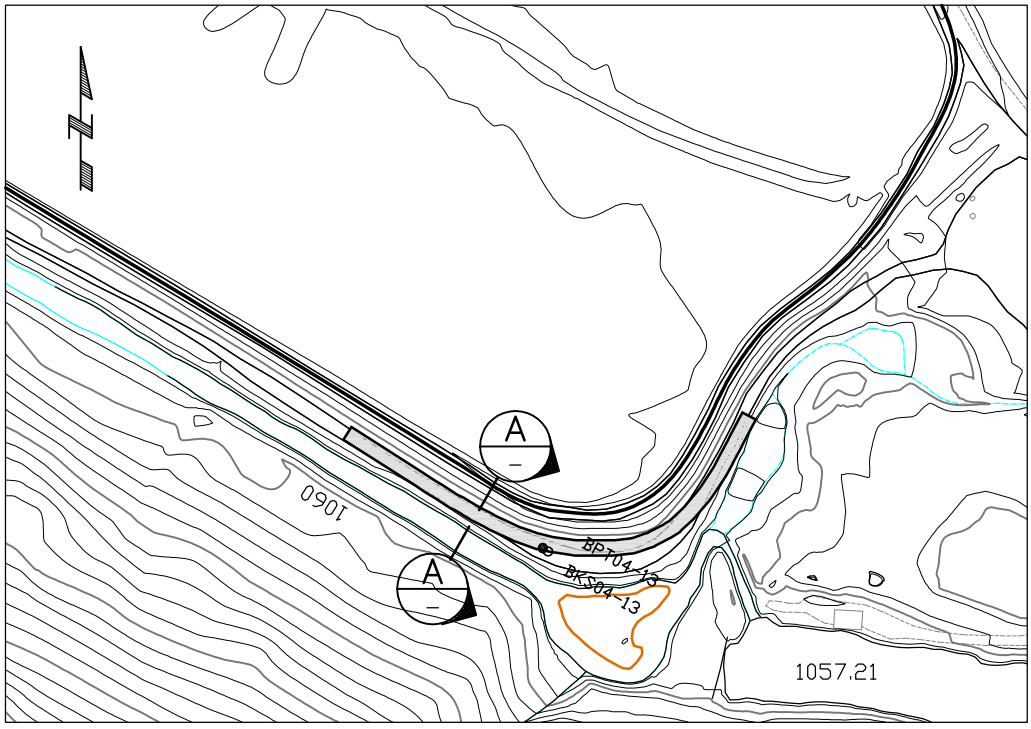
TITLE
CONCEPTUAL REMEDIAL OPTIONS
FOR SECONDARY DAM (WEST LIMB)

PROJECT No. M09237A01

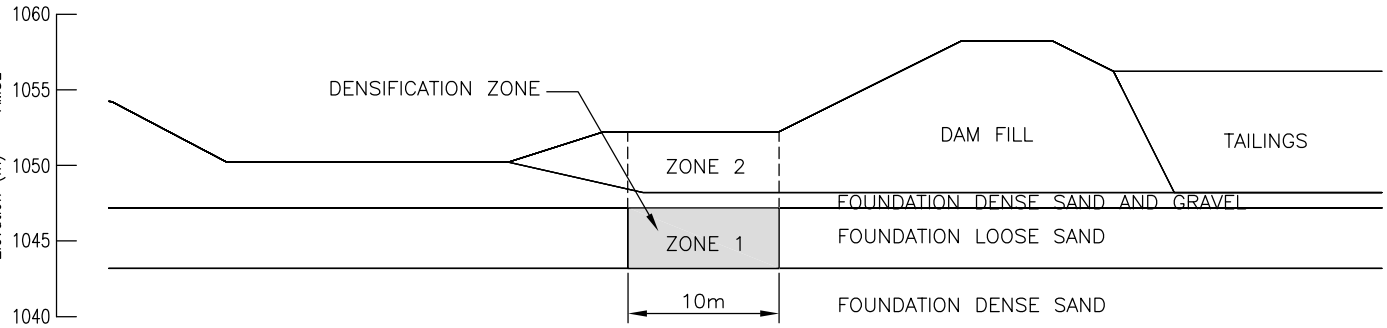
FIG. No. 7.1

KCR-BM

Drawing File: M:\M09237A01 – Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\figure 7.1.dwg (cwong)
Scale: 1=1(PS)
Date: 11/8/2004



PLAN 1
SCALE A



SECTION A
SCALE B

CONCEPTUAL REMEDIATION OPTION 1: GROUND Ddensification

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN–CRIPPEN REPORT DATED _____

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND\OR PUBLICATION OF DATA, STATEMENTS, CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

SCALE

SCALE B 0 25 m

SCALE A 0 250 m

CLIENT
DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
CONCEPTUAL REMEDIAL OPTION
FOR SECONDARY DAM (EAST LIMB)

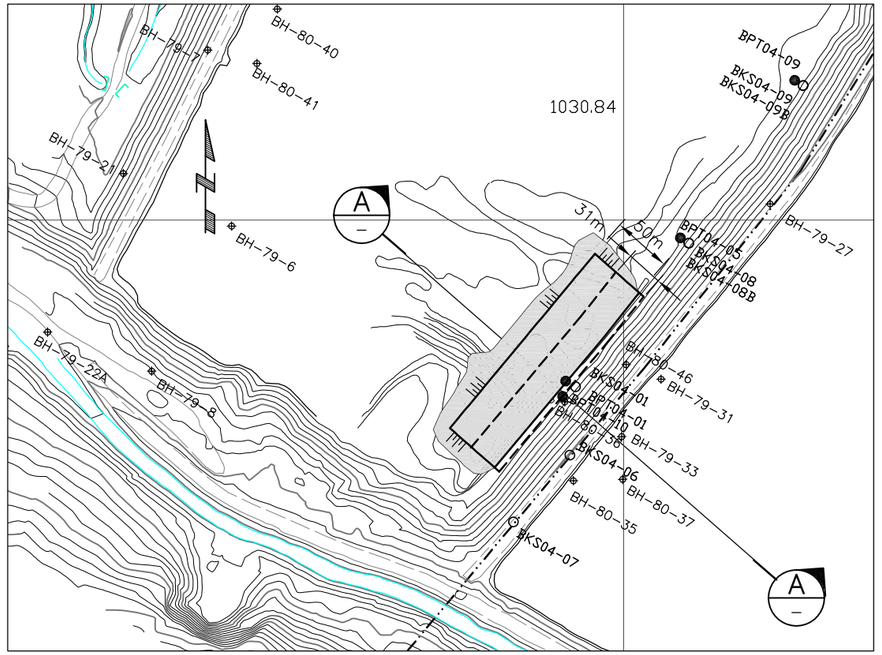
PROJECT No. M09237A01

FIG. No. 7.2

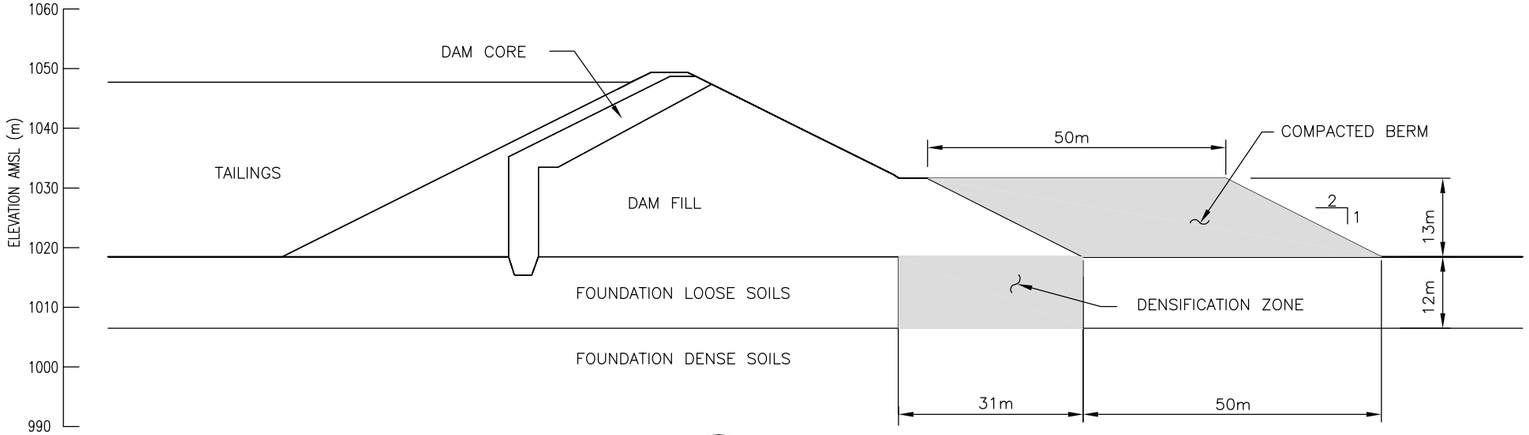
KCRA–M

CANCEL PRINTS BEARING PREVIOUS

Time: 9:21:28
Date: 11/8/2004
Scale: 1=1(PS)
Drawing File: M:\M09237A01 – Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\figure7.2.dwg (cwong)



PLAN 1
SCALE A



SECTION A
SCALE B

CONCEPTUAL REMEDIATION OPTION: BERM AND GROUND Ddensification

NOTE:
SECTION LOCATION IS APPROXIMATE AND THE SECTION IS IDEALIZED.

NOT FOR CONSTRUCTION

TO BE READ WITH KLOHN CRIPPEN REPORT DATED _____

SCALE B 0 50 m

SCALE A 0 250 m

SCALE

AS A MUTUAL PROTECTION TO OUR CLIENT, THE PUBLIC AND OURSELVES, ALL REPORTS AND DRAWINGS ARE SUBMITTED FOR THE CONFIDENTIAL INFORMATION OF OUR CLIENT FOR A SPECIFIC PROJECT AND AUTHORIZATION FOR USE AND/OR PUBLICATION OF DATA,STATEMENTS,CONCLUSIONS OR ABSTRACTS FROM OR REGARDING OUR REPORTS AND DRAWINGS IS RESERVED PENDING OUR WRITTEN APPROVAL.

CLIENT
DELOITTE & TOUCHE INC.
ANVIL RANGE MINING COMPLEX

KLOHN CRIPPEN

PROJECT
ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

TITLE
CONCEPTUAL REMEDIAL
OPTION FOR INTERMEDIATE DAM

PROJECT No. M09237A01

FIG. No. 7.3

KCR-BM

Plotted: [illegible]
Date: 1/7/2005
Scale: 1=4(PS)
Drawing File: M:\M09237A01 - Rose Creek YK Tailings Liquefaction\400 Site Work and Design Calculations\410 Drawings\7.3.dwg (cwong)

APPENDIX I

Test Hole Logs

KLOHN-CRIPPEN

SYMBOLS AND TERMS
FOR SOIL DESCRIPTION AND TEST HOLE LOGS

BASIC SYMBOLS



SYMBOL VARIATIONS - EXAMPLES[1]



CLASSIFICATION BY PARTICLE SIZE				
		Size Range		
			U.S. Standard Sieve Size	
Name		(mm) [3]	Retained	Passing
Boulders		> 200	8 inch	-
Cobbles		75 - 200	3 inch	8 inch
Gravel:	coarse	19 - 75	0.75 inch	3 inch
	fine	5 - 19	No. 4	0.75 inch
Sand:	coarse	2 - 5	No. 10	No. 4
	medium	0.4 - 2	No. 40	No. 10
	fine	0.075 - 0.4	No. 200	No. 40
Fines (Silt or Clay) [4]		< 0.075	-	No. 200

PROPORTION OF MINOR COMPONENTS BY WEIGHT[2]	
and	35 - 50%
y/ey	20 - 35%
some	10 - 20%
trace	0 - 10%

PARTICLE SHAPE	
Flat	width/thickness > 3
Elongated	length/width > 3

DENSITY OF GRANULAR SOILS		
Description	SPT N [5]	SPT $(N_1)_{60}$ [6]
Very Loose	0 - 4	0 - 3
Loose	4 - 10	3 - 8
Compact	10 - 30	8 - 25
Dense	30 - 50	25 - 42
Very Dense	> 50	> 42

CONSISTENCY OF COHESIVE SOILS			
Description	S_u [7]		SPT N [9]
	(kPa) [3]	(ksf) [8]	
Very Soft	< 12	< 0.25	< 2
Soft	12 - 25	0.25 - 0.5	2 - 4
Firm	25 - 50	0.5 - 1	4 - 8
Stiff	50 - 100	1 - 2	8 - 15
Very Stiff	100 - 200	2 - 4	15 - 30
Hard	> 200	> 4	> 30

(1) Only selected examples of the possible variations or combinations of the basic symbols are illustrated.
(2) Example: SAND, silty, trace of gravel = sand with 20% to 35% silt and up to 10% gravel, by weight.
(3) Approximate metric conversion.
(4) Fines are classified as silt or clay on the basis of Atterberg limits (refer to Plasticity Chart).
(5) Standard Penetration Test (SPT) blow count (uncorrected), after Terzaghi and Peck, 1948.
(6) Standard Penetration Test blow count, based on above N value corrected to 60% hammer efficiency and 96 kPa (1.0 ton/ft²) effective overburden pressure, after Skempton, 1986.
(7) Undrained shear strength can be estimated by vane (gives S_u), pocket penetrometer (gives unconfined compressive strength, i.e., 2 S_u), or unconfined compression test (gives 2 S_u).
(8) ksf = 1000 pounds per square foot = 0.5 tsf (ton/ft²) = approximately 0.5 kg/cm².
(9) Very approximate correlation with Standard Penetration Test blow counts, after Terzaghi and Peck, 1948.

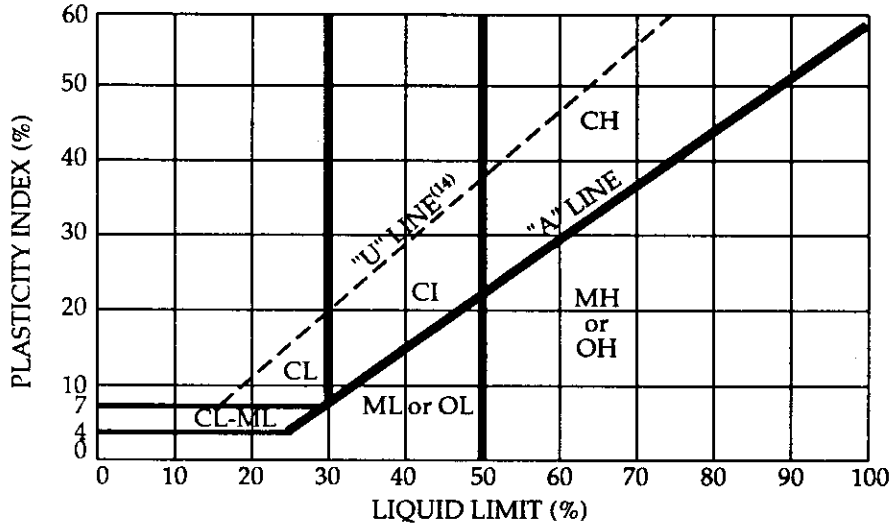
KLOHN-CRIPPEN

SYMBOLS AND TERMS
FOR SOIL DESCRIPTION AND TEST HOLE LOGS

PLASTICITY OF COHESIVE SOILS[10]		
Description	Silt	Clay
High	W_L[11] > 50	W_L > 50
Medium	–	30 < W_L < 50
Low	W_L < 50	W_L < 30
Non-Plastic	NP[12]	–

SENSITIVITY OF COHESIVE SOILS	
Description	Undisturbed Strength[13] / Remoulded Strength
High	> 8
Medium	4 to 8
Low	< 4

PLASTICITY CHART FOR SOILS PASSING NO. 40 SIEVE[10]



CLASSIFICATION OF GROUND ICE[15]			
GROUP		SUBGROUP	
Symbol	Description	Symbol	Description
N	Ice not visible by unaided eye	Nf	Poorly bonded or friable
		Nbn	Well bonded, no excess ice
		Nbe	Well bonded, excess ice
V	Visible ice less than 25 mm thick	Vx	Individual ice crystals or inclusions
		Vc	Ice coatings on soil particles
		Vr	Random or irregularly oriented ice
		Vs	Stratified or distinctly oriented ice
ICE	Visible ice greater than 25 mm thick	ICE + (soil type)	Ice with soil inclusions
		ICE	Ice without soil inclusions

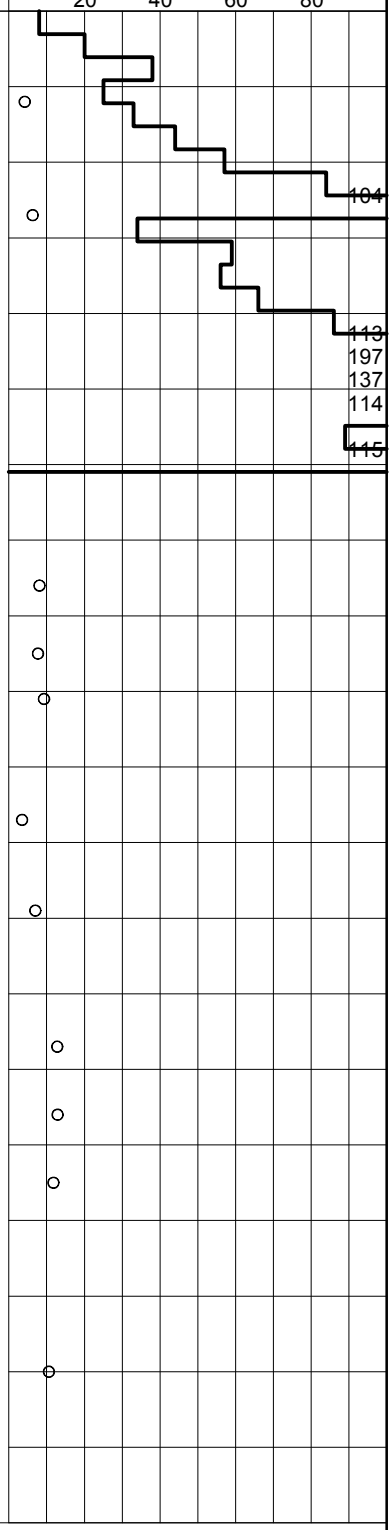
(10) This plasticity classification conforms to the Unified Soil Classification System (USCS) and the ASTM D-2487 plasticity chart, except for the addition of an intermediate category for clay, where the liquid limit is between 30% and 50% (CI). Under ASTM and USCS, all clays with a liquid limit less than 50% are classified as low plasticity (CL).
(11) W_L = Liquid Limit (%).
(12) NP = Non Plastic (silts only).
(13) Dimensionless ratio.
(14) "U" Line marks typical upper limit. "A" Line divides clays from silts and organic soils.
(15) For soil descriptions, estimate percentage of ground ice based on volume, after National Research Council of Canada, 1963.

BECKER TEST HOLE LOG

STARTED: Aug. 16, 2004 **FINISHED:** Aug. 17, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.1
COORDINATES (m): N 6913865 E 580451.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					SAND AND GRAVEL (SW-GW), fine to coarse, some cobbles, well graded, sub-rounded to sub-angular, brown, moist. [FILL]	
1					- rock in bit from 0.914m to 2.13m.	
2		Grab	1			
3					- water recovery at 2.74m. ▽	
4		Grab	2			
5						
6					- trace silt at 6.10m.	
7						
					7.6	
8		Grab	3		1023.4 SAND, fine to coarse, silty, some gravel, very wet. [FILL]	
					8.5	
9		Grab	4		1022.5 SAND AND GRAVEL, fine to coarse, some cobbles, trace silt, sub-rounded, brown, wet. [FILL]	
					9.1	
		Grab	5		1021.9 SAND, fine to coarse, silty, some gravel, brown, very wet. [FILL]	
10						
					10.7	
11		Grab	6		1020.4 SAND AND GRAVEL, fine to coarse, some cobbles, trace silt, sub-rounded to sub-angular, brown, wet. [FILL]	
12		Grab	7			
13						
					13.7 - high water yield at 13.72m.	
14		Grab	8		1017.3 SAND AND GRAVEL, medium to coarse, trace cobbles, brown, wet.	
15		Grab	9			
16		Grab	10			
17						
18						
19		Grab	11		- rust coloured water at 18.90m.	
20					Continued Next Page	



KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 3 **HOLE NO.:** BK04-01

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 16, 2004 **FINISHED:** Aug. 17, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.1
COORDINATES (m): N 6913865 E 580451.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
21		Grab	12			
					21.6 / 1009.4 SAND, fine to medium, silty, trace gravel, gap-graded, angular, brown, wet, gravel layers.	
22		Grab	13			
23		Grab	14			
24						
25						
26					26.2 / 1004.9 SAND AND GRAVEL (SW-GW), fine to coarse, some cobbles, trace silt, well graded, sub-rounded, grey, wet.	
27		Grab	15			
28						
29						
30		Grab	16			
31						
32						
33		Grab	17			
34		Grab	18			
35						
36					36.3	
		Grab	19		994.8 GRAVEL AND SAND, some silt, v. dense. [TILL]	
37					36.9 / 994.2 End of Becker Drill Hole at 36.9 m	
38						
39						
40						

NOTES:
1. Original ground at 13.7m.
2. Too dense below 6.1m for closed-Becker to penetrate.
3. Closed-Becker located 1.5m away from open-Becker.
4. See BPT04-10 for closed-Becker data.

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 3 **HOLE NO.:** BK04-01

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 16, 2004 **FINISHED:** Aug. 17, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.1
COORDINATES (m): N 6913865 E 580451.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% × — W% ○ — W_L% ×
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
41					to 39.0m depth.	
42						
43						
44						
45						
46						
47						
48						
49						
50						
51						
52						
53						
54						
55						
56						
57						
58						
59						
60						

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 3 **OF** 3 **HOLE NO.:** BK04-01

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 18, 2004 **FINISHED:** Aug. 19, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1051.5
COORDINATES (m): N 6913434 E 581979.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					[TAILINGS]	
8.2 / 1043.3					SAND AND GRAVEL(SW-GW), fine to coarse, cobbly, well graded, sub-angular to sub-rounded, grey, wet.	
9		Grab	1			
11		Grab	2		- rock in bit from 10.97 to 12.5m, poor recovery.	
12.5 / 1039.0					SAND, fine to coarse, some silt, brown, wet.	
13.4 / 1038.1					-too wet to recover sample.	
14		Grab	3		- loose zone from 12.5 to 13.41m. SAND AND GRAVEL, fine to coarse, some cobbles.	
14.6 / 1036.9		Grab	4		- loose zone at 14.63m.	
15.2					SAND, fine to coarse, some silt, brown, wet.	
1036.3					SAND AND GRAVEL (SW-GW), fine to coarse, some cobbles, well graded, sub-rounded to sub-angular, grey, wet.	
16		Grab	5			
18.3 / 1033.2		Grab	6		GRAVEL, sandy, very stiff. [TILL?]	
19					- no water return at 18.9m, material moist and brown.	
20					Continued Next Page	

135
179
101
114
110
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233
208

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** BK04-02A

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 18, 2004 **FINISHED:** Aug. 19, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1051.5
COORDINATES (m): N 6913434 E 581979.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% × — W% ○ — W_L% ×
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS	BECKER PENETRATION TEST
21		Grab	7				269 239 296 319
		Grab	8		- high water yield, silty matrix at 21.3m		
22							
23					23.2 / 1028.4 SILT, banded grey and black layers.		
		Grab	9				
24							
25		Grab	10		- 38 blows with Open Crowd-In Bit. - 42 blows with Open Crowd-In Bit. - 53 blows with Open Crowd-In Bit.		
26					- some gravel at 25.9m.		
		Grab	11		26.2 / 1025.3 SAND AND GRAVEL, fine to coarse, some silt, sub-rounded, grey, wet. -very hard at 26.8m.		
27					27.3 / 1024.2 -from 27.0 to 27.3m were 650 to 700 blows, refusal. End of Becker Drill Hole at 27.3 m		
28							
29					NOTES: 1. PDA conducted during closed-Becker. 2. Original ground at 7.92m. 3. Closed-Becker stopped at 21.4m due to boulder or large cobble. 4. See BPT04-2B for closed-Becker data from 23.5 to 24.4m.		
30							
31							
32							
33							
34							
35							
36							
37							
38							
39							
40							

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** BK04-02A

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 19, 2004 **FINISHED:** Aug. 19, 2004

DRILL RIG MODEL: Becker HAV-180

GROUND ELEV. (m): 1051.5

COORDINATES (m): N 6913434 E 581979.4

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)

Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)

● SPT N — BOUNCE PRESS. (kPa)

W_P% × — — — o — — — × W_L%

20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
1					Open hole only to 23.5m with BPT from 23.5 to 24.4m. Purpose of hole was to determine whether refusal in BK04-2A was due to a boulder or dense soil.	
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						
18						
19						
20					Continued Next Page	

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam

PROJECT: Rose Creek Site Investigation

LOCATION: Anvil Range Mine, Yukon

LOGGED BY: RF **CHECKED BY:**

SHEET 1 **OF** 2 **HOLE NO.:** BPT04-02B

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 19, 2004 **FINISHED:** Aug. 19, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1051.5
COORDINATES (m): N 6913434 E 581979.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% × — W% ○ — W_L% ×
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
21						
22						
23						
24					24.4 (1027.1) End of Becker Drill Hole at 24.4 m	
25						
26						
27						
28						
29						
30						
31						
32						
33						
34						
35						
36						
37						
38						
39						
40						

244
247
342

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** BPT04-02B

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 22, 2004 **FINISHED:** Aug. 23, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1049.9
COORDINATES (m): N 6913220 E 582003.2

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					[TAILINGS]	
					- No BPT data collected from 0 to 12.8m.	
12.8						
1037.1					SAND AND GRAVEL, fine to coarse, some cobbles, trace silt, sub-angular to angular, grey, wet.	
14		Grab	1			
15						
16		Grab	2		- boulder (Gneiss) from 15.8 to 16.8m.	
16.8						
1033.1					SAND, fine to medium, silty, some cobbles, angular, brown, wet.	
17.7						
1032.2		Grab	3		SAND AND GRAVEL, fine to coarse, some cobbles, trace silt, sub-rounded, grey, wet.	
					- high water yield. - silty from 18.9 to 19.8m.	
20					Continued Next Page	

144
135
104
120
139
175
255
216
162
141

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 3 **HOLE NO.:** BK04-03(PZ)

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 22, 2004 **FINISHED:** Aug. 23, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1049.9
COORDINATES (m): N 6913220 E 582003.2

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT	DETAILS
20		Grab	4		- grey, loose, high water yield.		
21							
22					- silty from 21.9 to 22.6m.		
23							
24					- silty from 23.5 to 23.8m.		
25		Grab	5				
26							
27							
28							
29		Grab	6		- rust coloured water at 29m.		
30							
31							
32		Grab	7				
33					- compacted material, low water yield from 32.6 to 34.1m.		
34							
35					- loose, grey, high water yeild at 34.4m.		
36							
37					- flat, elongated gravel from 36 to 37.8m.		
38		Grab	8				
39							
40		Grab	9		- boulder from 39.3 to 39.6m. - some silt, broken rock fragments		

Continued Next Page

107
112
117
117

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 3 **HOLE NO.:** BK04-03(PZ)

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 22, 2004 **FINISHED:** Aug. 23, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1049.9
COORDINATES (m): N 6913220 E 582003.2

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT	DETAILS	
41					from 39.6 to 40.8m.			112 127 150
42		Grab	10		- cobbly from 41.8 to 43m.			125 129 103 115
43					43.3 / 1006.6 End of Becker Drill Hole at 43.3 m			123 140 152
44					NOTES:			
45					1. No closed-Becker data from 0 to 12.8m. 2. Original ground at 12.8m. 3. Closed-Becker located 1.5m away from open-Becker.			
46					Piezometer Details for BK04-03(PZ)			
47					Cave In/Cuttings 0 - 38.7m Bentonite 38.7 - 41.1m Filter Sand 41.1 - 43m Screen 43 - 43.3m			
48					- Casagrande Tip Standpipe			
49								
50								
51								
52								
53								
54								
55								
56								
57								
58								
59								
60								

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 3 **OF** 3 **HOLE NO.:** BK04-03(PZ)

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 24, 2004 **FINISHED:** Aug. 25, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1050.9
COORDINATES (m): N 6913324 E 581983.8

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					[TAILINGS]	
1					- No BPT data collected from 0 to 11.3m	
2						
3						
4						
5						
6						
7						
8						
9						
10						
11						
11.6 / 1039.3					SAND AND GRAVEL, sand (fine to coarse), gravel (fine to coarse, angular to sub-angular), some cobbles (sub-rounded to sub-angular), trace silt, brown, wet.	
12		Grab	1		- high water yield.	
13						
14		Grab	2		- some silt at 13.7m.	
15					- gravel, sub-rounded, trace silt.	
16		Grab	3			
17						
18					- some silt from 18.3 to 18.9m.	
19						
20		Grab	4			

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 3 **HOLE NO.:** BK04-04

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 24, 2004 **FINISHED:** Aug. 25, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1050.9
COORDINATES (m): N 6913324 E 581983.8

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
21						
22						
23		Grab	5		- some silt from 22.6 to 23.2m.	
24						
25						
26		Grab	6			
27					- some silt from 26.8 to 28m.	
28						
29		Grab	7			
30						
31		Grab	8		- some silt from 30.2 to 31.1m.	
32					- bit pushing boulder at 31.5m. - dense at 31.7m - cobbly from 32 to 32.6m.	
33		Grab	9		- rock jammed in bit at 33.2m.	
34		Grab	10		- some silt from 34.1 to 35m.	
35		Grab	11		- cobbly, sub-angular from 34.7 to 35.7m.	
					35.7	
36		Grab	12		1015.2 SAND, fine to medium, some silt, some gravel, gap-graded, brown, wet.	
					36.6	
37		Grab	13		1014.3 SAND AND GRAVEL, some cobbles, trace to some silt, sub-angular to angular, grey, wet. [TILL?]	
38		Grab	14			
					38.7	
39		Grab	15		1012.2 SILT.	
40					Continued Next Page	

139
265
284
169
324
257

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 3 **HOLE NO.:** BK04-04

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 24, 2004 **FINISHED:** Aug. 25, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1050.9
COORDINATES (m): N 6913324 E 581983.8

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
1010.7					40.2 End of Becker Drill Hole at 40.2 m	

NOTES:

1. Original ground at 11.6m.
2. No BPT data from 0 to 11.3m.
3. Open-Becker located 1.5m away from closed-Becker.

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 3 **OF** 3 **HOLE NO.:** BK04-04

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 26, 2004 **FINISHED:** Aug. 26, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.0
COORDINATES (m): N 6913985 E 580547.5

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)

W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
1–20					No open-Becker was conducted. Penetration data for this hole is suspect due to hammer misfiring. This hole was replaced by BPT04-08A,B.	

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** BPT04-05

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 26, 2004 **FINISHED:** Aug. 26, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.0
COORDINATES (m): N 6913985 E 580547.5

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
24					24.1 / 1006.9 End of Becker Drill Hole at 24.1 m	

NOTES:

1. Drilling stopped at 24.1m due to mechanical problems.

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** BPT04-05

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 26, 2004 **FINISHED:** Aug. 26, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1049.1
COORDINATES (m): N 6913803 E 580454.9

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					SAND AND GRAVEL, sand (fine to coarse), gravel (fine to coarse, sub-angular), some cobbles, trace silt, brown, dry. [FILL]	
5					- moist at 5.18m.	
13					13.1 / 1036.0 End of Becker Drill Hole at 13.1 m	

NOTES:

Piezometer Details for BKS04-06(PZ)

Concrete	0 - 0.61m
Cuttings	0.61 - 0.91m
Bentonite Chips	0.91 - 2.4m
Cuttings	2.4 - 10.2m
Bentonite Chips	10.2 - 11.8m
Sand	11.8 - 13.1m
Screen	12.8 - 13.1m

- Casagrande Tip Standpipe

Water Level Depth Below Ground Surface

BK04-06(PZ) Dry

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** BKS04-06(PZ)

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 27, 2004 **FINISHED:** Aug. 27, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1049.2
COORDINATES (m): N 6913747 E 580407.6

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT	DETAILS
0					SAND AND GRAVEL, sand (fine to coarse), gravel (fine to coarse, sub-angular to sub-rounded) some cobbles, trace silt, brown, dry. [FILL]		
5					- moist at 4.57m.		
11.9 (1037.3)					End of Becker Drill Hole at 11.9 m		

NOTES:

Piezometer Details for BKS04-07(PZ)

Concrete	0 - 0.61m
Bentonite Chips	0.61 - 2.24m
Cuttings	2.24 - 8.69m
Bentonite Chips	8.69 - 10.11m
Sand	10.11 - 11.81m
Screen	11.5 - 11.81m

- Casagrande Tip Standpipe

Water Level Depth Below Ground Surface

BK04-07(PZ) Dry

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** BKS04-07(PZ)

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 28, 2004 **FINISHED:** Aug. 28, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.0
COORDINATES (m): N 6913985 E 580547.5

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)

W_P% × — W% ○ — W_L% ×
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
1–20					No open-Becker conducted.	

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 1 **OF** 3 **HOLE NO.:** BPT04-08A

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 28, 2004 **FINISHED:** Aug. 28, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.0
COORDINATES (m): N 6913985 E 580547.5

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
21						
22						
23						
24						
25						
26						
27						
28						
29						
30						
31						
32						
33						
34						
35						
36						
37						
38						
39						
40						

100
119
111
121
191
213
133
147
160
165
131
114
100
155
153
113
122
124
132
117
123
130
188
128
161
158
152
179
280
329
386

NOTES:

1. BPT data from 0 to 8.8m was collected during BPT04-08B.
2. An alternate set of BPT data for this location is shown on BPT04-05.
3. BPT04-05, BPT04-08, and BPT04-08B are within 3.0m of each other.

39.9

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 3 **HOLE NO.:** BPT04-08A

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 28, 2004 **FINISHED:** Aug. 28, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.0
COORDINATES (m): N 6913985 E 580547.5

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% × — W% ○ — W_L% ×
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					991.1 End of Becker Drill Hole at 39.9 m	

41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 3 **OF** 3 **HOLE NO.:** BPT04-08A

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

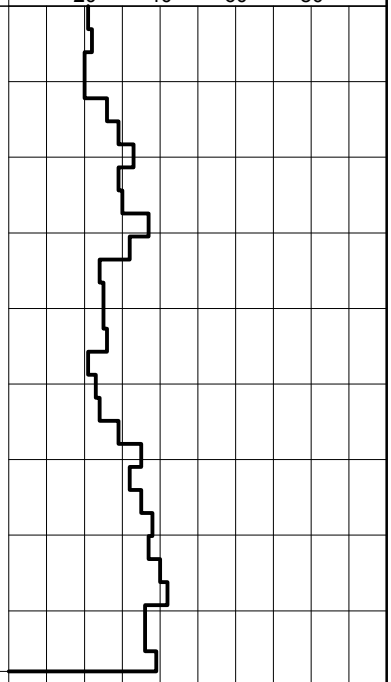
BECKER TEST HOLE LOG

STARTED: Aug. 31, 2004 **FINISHED:** Aug. 31, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.0
COORDINATES (m): N 6913985 E 580547.5

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% × — W% ○ — W_L% ×
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
1–8					No open-Becker conducted.	
8.8 / 1022.2					End of Becker Drill Hole at 8.8 m	



KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: FL **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** BPT04-08B

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 29, 2004 **FINISHED:** Aug. 30, 2004

DRILL RIG MODEL: Becker HAV-180

GROUND ELEV. (m): 1031.7

COORDINATES (m): N 6914117 E 580643.3

DESCRIPTION OF MATERIALS

DEPTH (m) | SPT BLOWS PER 0.15m | SAMPLE TYPE | SAMPLE No. | SYMBOL | INSTRUMENT DETAILS

See BK04-09B for soil description and BPT data for 0 to 8.8m.

Continued Next Page

BECKER PENETRATION TEST

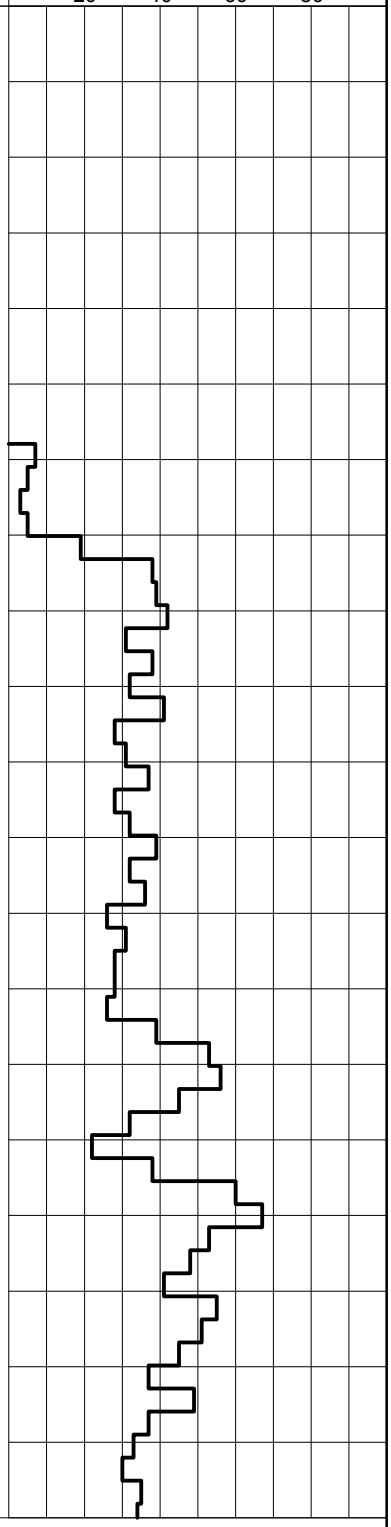
Casing O.D.: CLOSE ENDED (blows/0.3m)

Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)

● SPT N — BOUNCE PRESS. (kPa)

W_P% — W% — W_L%

20 40 60 80



KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam

PROJECT: Rose Creek Site Investigation

LOCATION: Anvil Range Mine, Yukon

LOGGED BY: RF **CHECKED BY:**

SHEET 1 **OF** 2 **HOLE NO.:** BPT04-09A

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 29, 2004 **FINISHED:** Aug. 30, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.7
COORDINATES (m): N 6914117 E 580643.3

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
33.2 / 998.5					End of Becker Drill Hole at 33.2 m	

NOTES:

1. No BPT data from 0 to 5.8m.
2. See BK04-09B for lithology from 0 to 8.8m, and BPT data from 0 to 5.8m.

110
131
133
147
239
235
169
265

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: RF **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** BPT04-09A

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 31, 2004 **FINISHED:** Aug. 31, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.7
COORDINATES (m): N 6914117 E 580643.3

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					SAND AND GRAVEL, some cobbles, trace silt, brown, damp.	
3.4 / 1028.3					- moist at 3.05 - water yield. SILT, trace to some sand (fine to medium, angular), brown, moist to wet.	
4		Grab	1			
5		Grab	2			
5.8 / 1025.9					SAND AND GRAVEL, sand (fine to coarse, sub-angular), gravel (fine to coarse, sub-angular), some cobbles (fine, sub-angular to sub-rounded).	
7		Grab	3			
7.3 / 1024.4					SAND, medium to coarse, sub-angular to rounded, gravelly (fine, sub-rounded), highly permeable.	
8		Grab	4			
8.8 / 1022.9					End of Becker Drill Hole at 8.8 m	

NOTES:

1. Original ground at 3.4m.
2. No BPT data from 5.8 to 8.8m.
3. See BPT04-09 for BPT from 5.8 to 33.2m depth.

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: FL **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** BK04-09B

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 30, 2004 **FINISHED:** Aug. 31, 2004

DRILL RIG MODEL: Becker HAV-180

GROUND ELEV. (m): 1031.1

COORDINATES (m): N 6913865 E 580451.4

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)

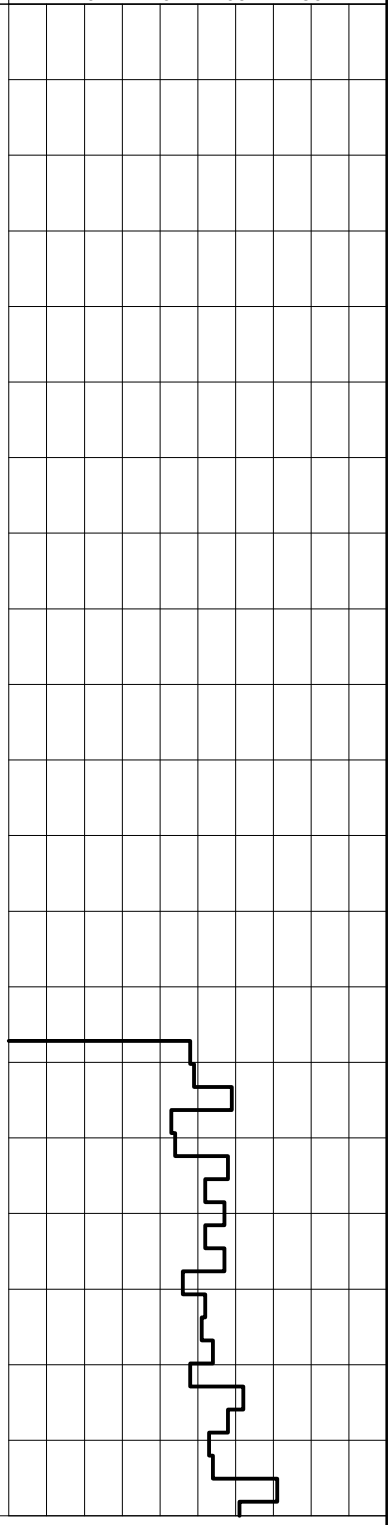
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)

● SPT N — BOUNCE PRESS. (kPa)

W_P% W% W_L%

20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
1–20					This hole was drilled to obtain BPT data below 14m orthe base of compacted fill to supplement BK04-01.	



Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam

PROJECT: Rose Creek Site Investigation

LOCATION: Anvil Range Mine, Yukon

LOGGED BY: FL **CHECKED BY:**

SHEET 1 **OF** 2 **HOLE NO.:** BPT04-10

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Aug. 30, 2004 **FINISHED:** Aug. 31, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1031.1
COORDINATES (m): N 6913865 E 580451.4

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS	BECKER PENETRATION TEST
21							
22							
23							
24							
25							
26							
27							
28							119
29							122 113 109 104 101
30							118
31							116 111 101
32							116 129 117
33							113 125
34							143 126
35							143 153 178 191
36							200 114
37							107 127
38							136 124 209 279
39					39.0 992.1 End of Becker Drill Hole at 39.0 m		282 324
40							

NOTES:

1. See BK04-01 for soil description.
2. BPT04-10 located 1.5m away from BK04-01.

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Intermediate Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: FL **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** BPT04-10

KC_BECKER-SI INTERMEDIATE DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Sep. 1, 2004 **FINISHED:** Sep. 1, 2004

DRILL RIG MODEL: Becker HAV-180

GROUND ELEV. (m): 1050.2

COORDINATES (m): N 6912892 E 582293.6

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)

Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)

● SPT N — BOUNCE PRESS. (kPa)

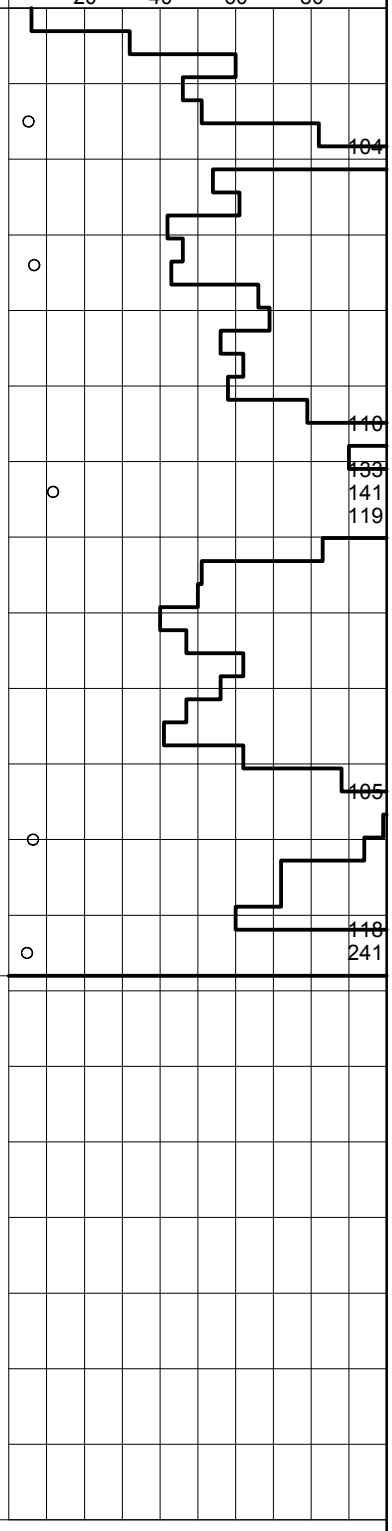
W_P% W% W_L%

20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					SAND AND GRAVEL, sand (fine to coarse, angular), gravel (fine to coarse, sub-angular to sub-rounded), trace to some silt. [FILL]	
1					- soft zones appear to be mainly sand.	
2		Grab	1		- boulder at 1.83m.	
2.5					1047.7 - water yield at 2.44m; silty, greyish brown water. ▽	
3					SAND AND GRAVEL, sand (fine to coarse), gravel (fine to coarse, sub-rounded), cobbly (fine to coarse, sub-rounded), trace silt, brown.	
4		Grab	2		- soft zone at 3.35m.	
5					- cobbles or boulder at 4.27m.	
6						
7		Grab	3			
8						
9						
10						
11.0					1039.2	
11		Grab	4		GRAVEL (fine to coarse, angular), sandy (angular), cobbly (sub-angular), some silt.	
12						
12.5					1037.7	
12.8		Grab	5		ROCK, flaky, smooth, grey, easy to break, no water yield.	
13					1037.4 End of Becker Drill Hole at 12.8 m	
14						
15						
16						
17						
18						
19						
20						

NOTES:

1. Original ground at 2.5m.
2. Open-Becker located 1.5m away from closed-Becker.



KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam

PROJECT: Rose Creek Site Investigation

LOCATION: Anvil Range Mine, Yukon

LOGGED BY: FL **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** BK04-11

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Sep. 1, 2004 **FINISHED:** Sep. 2, 2004

DRILL RIG MODEL: Becker HAV-180

GROUND ELEV. (m): 1055.5

COORDINATES (m): N 6912727 E 582691.7

BECKER PENETRATION TEST

Casing O.D.: CLOSE ENDED (blows/0.3m)

Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)

● SPT N — BOUNCE PRESS. (kPa)

W_P% W% W_L%

20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
					SAND AND GRAVEL, some silt, light brown, dry. [FILL]	
1					- boulder at 1.37m.	
2					- boulder and cobbles from 1.98 to 2.74m.	
3					- boulder at 3.05m.	
					3.7 - water yield, brown at 3.35m. ▽	
4		Grab	1		1051.8 SAND AND GRAVEL, sand (medium to coarse, angular to sub-angular), gravel (fine to coarse, angular to sub-rounded), trace silt, highly permeable, brown.	
5						
6						
7		Grab	2			
8					- boulder from 7.62 to 8.23m.	
9					- very tight, dark brown water; high yield at 8.23m. - bit pushing big rock at 8.53m.	
10					- brown water at 8.84m.	
					- pushing big rock at 9.45m.	
					10.7 - silty water at 10.36m.	
11		Grab	3		1044.8 SAND, coarse, angular, gravelly, fine, highly permeable.	
12					- trace silt at 12.2m.	
13					13.1	
		Grab	4		1042.4 ROCK, angular, hard pieces, some silt, low water yield.	
14					14.6	
15					1040.9 End of Becker Drill Hole at 14.6 m	
16					NOTES: 1. Original ground at 3.7m. 2. Open-Becker located 1.5m away from closed-Becker.	
17						
18						
19						
20						

157 186 142 137 151 164 140 185 109 102 114 163

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam

PROJECT: Rose Creek Site Investigation

LOCATION: Anvil Range Mine, Yukon

LOGGED BY: FL **CHECKED BY:**

SHEET 1 **OF** 1 **HOLE NO.:** BK04-12

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Sep. 2, 2004 **FINISHED:** Sep. 2, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1057.5
COORDINATES (m): N 6912379 E 583261.3

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					SAND AND GRAVEL, sand (fine to medium), gravel (fine to coarse, angular), some cobbles, trace to some silt, brown, dry. [FILL]	
1						
2		Grab	1			
3						
3.4						
1054.1					SAND, medium to coarse, some gravel to gravelly (fine, sub-angular to sub-rounded), some cobbles. ▽	
4		Grab	2		- water yield, brown at 3.96m.	
5						
6		Grab	3		- reddish brown water at 5.79m.	
7						
8						
9						
10						
11		Grab	4			
11.6						
1045.9					ROCK, weathered, flaky pieces, some sand, some silt, brownish grey, no water yield.	
12						
13					- bedrock at 15.7m.	
14						
15		Grab	5			
15.7						
1041.8					End of Becker Drill Hole at 15.7 m	
16						
17					NOTES:	
18					1. Original ground at 3.4m. 2. Open-Becker located 1.5m away from closed-Becker. 3. Closed-Becker stopped at 12.5m due to v. dense ground.	
19						
20						

106
134
134
229

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Secondary Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: FL **CHECKED BY:**
SHEET 1 **OF** 1 **HOLE NO.:** BK04-13

KC_BECKER-SI SECONDARY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

STARTED: Sep. 3, 2004 **FINISHED:** Sep. 3, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1016.0
COORDINATES (m): N 6914256.3 E 580146.7

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)
W_P% W% W_L%
20 40 60 80

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
0					SAND AND GRAVEL. [FILL]	
1						
2					- water yield at 1.52m.	
					2.1 - lost water at 1.83m	
					1013.9 SAND AND GRAVEL, sand (fine to coarse, sub-rounded), gravel (fine to coarse, sub-rounded), trace to some silt.	
3						
4		Grab	1		- water yield at 3.7m.	
	24,27,27	SPT	2			
5						
					5.8	
6					1010.3 SAND, fine, grey.	
7						
					7.3	
	2,3,4	SPT	3		1008.7 SILT AND SAND, fine sand, low plasticity, grey.	
8						
	2,4,5	SPT	4			
9					9.0	
					1007.0 No open-Becker beyond 9m.	
10						
11						
12						
13						
14						
15						
16						
17						
18						
19						
20						

Continued Next Page

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Cross Valley Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: FL **CHECKED BY:**
SHEET 1 **OF** 2 **HOLE NO.:** BK04-14

KC_BECKER-SI CROSS VALLEY DAM.GPJ KC_DATA.GDT 11/8/04

BECKER TEST HOLE LOG

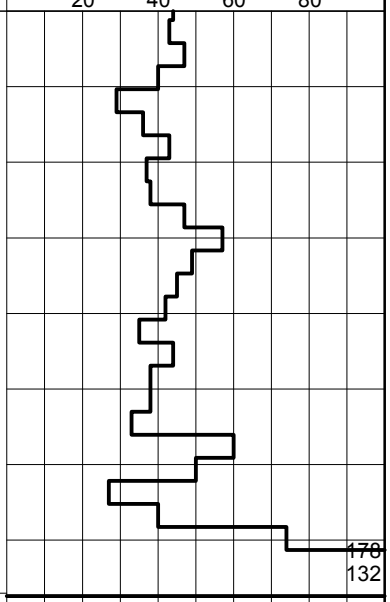
STARTED: Sep. 3, 2004 **FINISHED:** Sep. 3, 2004
DRILL RIG MODEL: Becker HAV-180
GROUND ELEV. (m): 1016.0
COORDINATES (m): N 6914256.3 E 580146.7

BECKER PENETRATION TEST
Casing O.D.: CLOSE ENDED (blows/0.3m)
Hammer Energy: 11 kJ max. rated — OPEN ENDED (blows/0.3m)
● SPT N — BOUNCE PRESS. (kPa)

DEPTH (m)	SPT BLOWS PER 0.15m	SAMPLE TYPE	SAMPLE No.	SYMBOL	DESCRIPTION OF MATERIALS	INSTRUMENT DETAILS
27.7 (988.3)					End of Becker Drill Hole at 27.7 m	

NOTES:

1. Original ground at 2.1m.
2. Open-Becker located 1.5m away from closed-Becker.
3. No open-Becker beyond depth of 9m.



KLOHN CRIPPEN

PROJECT NO.: M09237 A01 - Cross Valley Dam
PROJECT: Rose Creek Site Investigation
LOCATION: Anvil Range Mine, Yukon
LOGGED BY: FL **CHECKED BY:**
SHEET 2 **OF** 2 **HOLE NO.:** BK04-14

KC_BECKER-SI CROSS VALLEY DAM.GPJ KC_DATA.GDT 11/8/04

APPENDIX II

Site Investigation Photographs



Photo II-1 BPT at BPT04-01 Using HAV180 Drill Rig and Instrumented Casing



Photo II-2 BPT at BPT04-01 on the Intermediate Dam D/S Berm



Photo II-3 HAV 180 Drill rig and the Support Truck at BPT04-01



Photo II-4 HAV 180 Drill Rig and the Support Truck at BPT04-01



Photo II-5 Instrumented Casing Used in PDA Measurements



Photo II-6 PDA Data Acquisition System and Monitoring Equipments



Photo II-7 Drill Rig being Set up at the Secondary Dam D/S Tailings



Photo II-8 HAV 180 Drill Rig and the Support Truck at BPT04-02 on Secondary Dam D/S Tailings



Photo II-9 Slotted PVC Used in the Piezometer at BKS04-06



Photo II-10 Pea Gravel Used to Backfill Becker Open Holes



Photo II-11 BPT at BPT04-11 at the East Limb of the Secondary Dam



Photo II-12 BPT at BPT04-12 at the East Limb of the Secondary Dam



Photo II-13 BPT at BPT04-13 at the SE Corner of the Secondary Dam East Limb



Photo II-14 SPT Sample 1 Taken at BKS04-14 at the Cross-Valley Dam



Photo II-15 SPT Sample 2 Taken at BKS04-14 at the Cross-Valley Dam



Photo II-16 SPT Sample 3 Taken at BKS04-14 at the Cross-Valley Dam

APPENDIX III

Laboratory Test Results

FINES CONTENT OF SOIL

Hole No.	BKS04-3	BKS04-3	BKS04-3	BKS04-3	BKS04-4	BKS04-4	BKS04-4
Depth (m)	19.5-21.0	28.0-30.2	30.2-33.2	38.1-40.5	11.9-13.4	13.7-14.9	18.0-21.0
Sample No.	4	6	7	9	1	2	4
Dry Weight + Tare (g)	468.20	431.07	522.13	416.65	468.23	486.82	600.18
Wash Weight + Tare (g)	455.94	419.78	511.90	402.97	442.83	458.93	587.73
Tare (g)	177.65	181.89	195.25	157.07	154.81	172.56	156.16
Fines (g)	12.26	11.29	10.23	13.68	25.40	27.89	12.45
Total Dry Weight (g)	290.55	249.18	326.88	259.58	313.42	314.26	444.02
Fines Content (%)	4.2	4.5	3.1	5.3	8.1	8.9	2.8

Hole No.	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-9
Depth (m)	21.0-24.1	24.1-27.1	30.2-31.7	32.3-33.2	34.4-35.7	35.7-36.6	4.0-5.2
Sample No.	5	6	8	9	11	12	1
Dry Weight + Tare (g)	411.51	383.63	438.48	855.72	560.69	480.83	272.15
Wash Weight + Tare (g)	403.89	376.27	430.03	821.76	529.17	409.69	179.00
Tare (g)	142.03	147.06	159.40	148.78	166.73	146.89	156.97
Fines (g)	7.62	7.36	8.45	33.96	31.52	71.14	93.15
Total Dry Weight (g)	269.48	236.57	279.08	706.94	393.96	333.94	115.18
Fines Content (%)	2.8	3.1	3.0	4.8	8.0	21.3	80.9

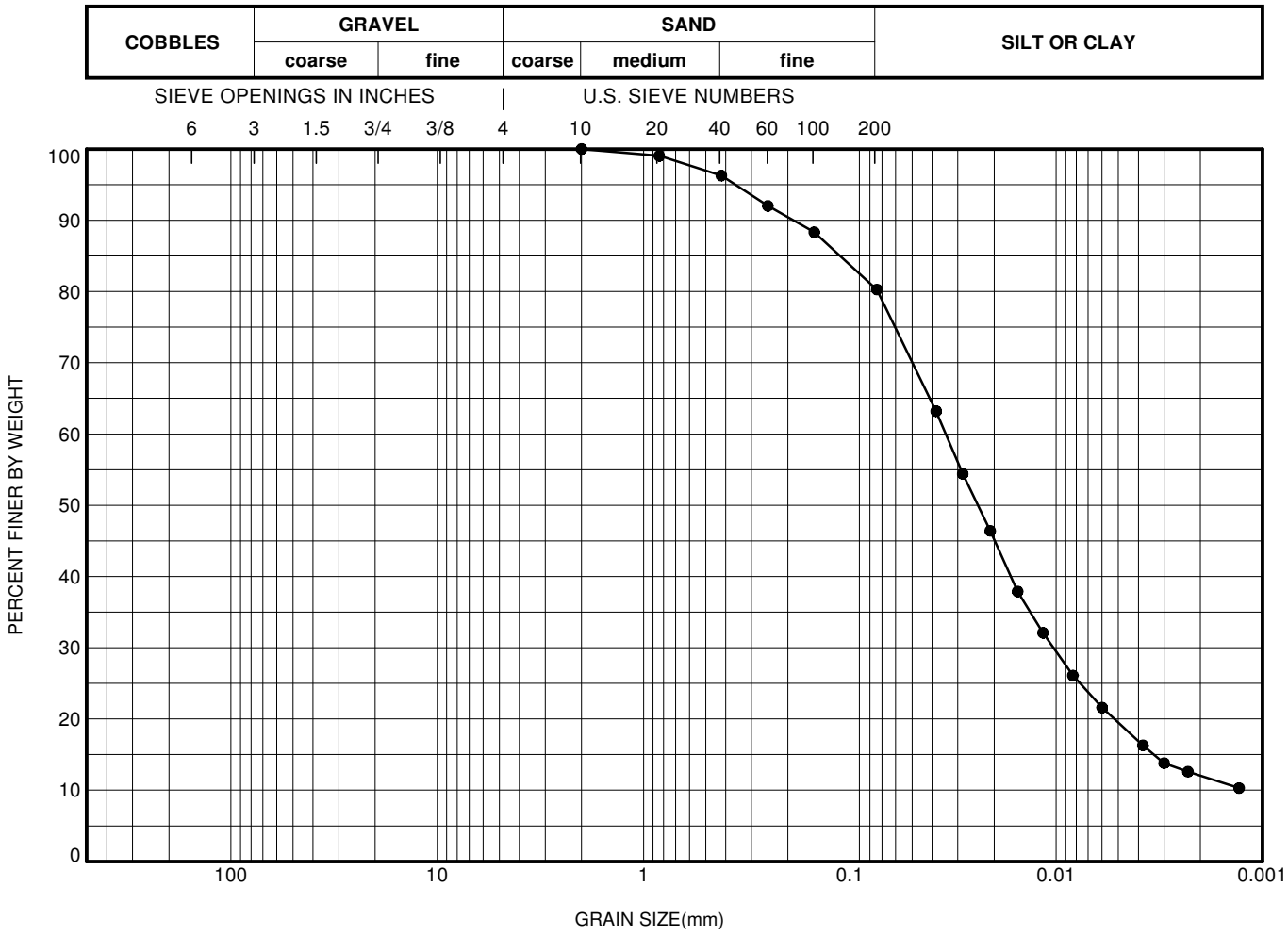
Hole No.	BKS04-9	BKS04-13	BKS04-13				
Depth (m)	5.2-5.5	3.7-4.3	6.1-6.4				
Sample No.	2	2	3				
Dry Weight + Tare (g)	289.25	459.32	485.54				
Wash Weight + Tare (g)	166.77	446.92	474.34				
Tare (g)	157.41	174.55	168.44				
Fines (g)	122.48	12.40	11.20				
Total Dry Weight (g)	131.84	284.77	317.10				
Fines Content (%)	92.9	4.4	3.5				

Hole No.							
Depth (m)							
Sample No.							
Dry Weight + Tare (g)							
Wash Weight + Tare (g)							
Tare (g)							
Fines (g)							
Total Dry Weight (g)							
Fines Content (%)							

KLOHN CRIPPEN

JOB NO.:	M09237 A01 01	
PROJECT:	ROSE CREEK TAILINGS IMPOUNDMENT	
LOCATION:	Faro, Yukon	
DATE:	October 15, 2004	
TESTED BY:	Juan	CHECKED BY: Ganan

GRAIN SIZE DISTRIBUTION



	HOLE	DEPTH (m)	D85	D60	D50	D15	D10	CU	%GRAVEL	%SAND	%FINES
●	BKS04 - 9	4.00	0.112						0.0	19.6	80.4

	HOLE	SAMPLE	DEPTH (m)	W%	W_L	W_P	PI	REMARKS / SAMPLE DESCRIPTION
●	BKS04 - 9	1	4.00		28	20	7	Depth = 4.0 - 5.2 m

CU = COEFFICIENT OF UNIFORMITY = D60/D10 **PARTICLE SIZES, e.g. D85, in mm** **Tested by Wet Sieving Method (ASTM D1140 & D422)**

KLOHN CRIPPEN

PROJECT NO.: M09237 A01 01

PROJECT: ROSE CREEK TAILINGS IMPOUNDMENT

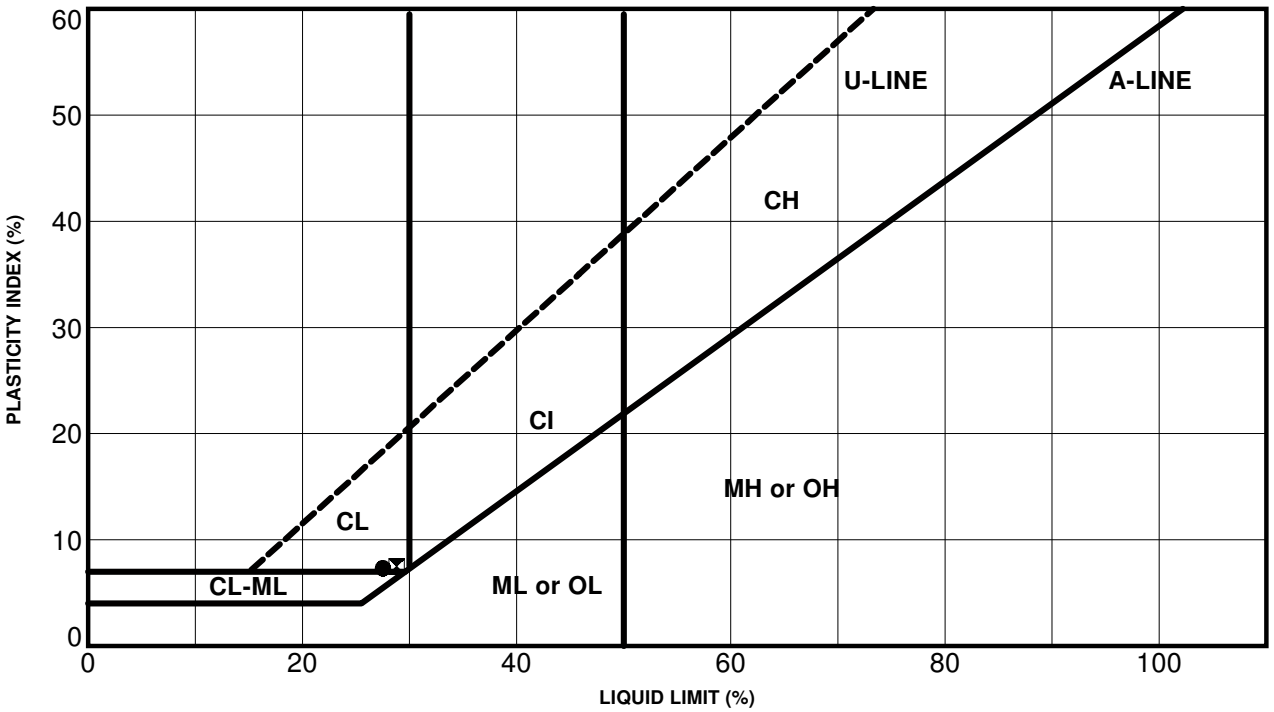
LOCATION: Faro, Yukon

FIGURE:

DRAWN BY: Juan **CHECKED BY:**

KC_GRAIN_SIZE-SI M03792A01 ROSE CREEK.GPJ SIEVE.GDT 11/6/04

PLASTICITY CHART



	HOLE	SAMPLE	DEPTH (m)	W_L	W_P	PI	% FINES	REMARKS/SAMPLE DESCRIPTION
●	BKS04 - 9	1	4.0	28	20	7		Depth = 4.0 - 5.2 m
⊠	BKS04 - 9	2	5.2	29	21	8		Depth = 5.2 - 5.5 m

KLOHN CRIPPEN

PROJECT NO.:	M09237 A01 01
PROJECT:	ROSE CREEK TAILINGS IMPOUNDMENT
LOCATION:	Faro, Yukon
FIGURE:	
DRAWN BY:	Juan **CHECKED BY:**

KC_ATTERBERG-SI M03792A01 ROSE CREEK.GPJ SIEVE.GDT 11/6/04

KLOHN CRIPPEN

Project No.: M09237 A01 01
Project: ROSE CREEK TAILINGS IMPOUNDMENT

Test Method: Torvane

Tested by: Juan Garfias
Checked by: Ganan Gananathan
Test Date: October 20, 2004

Hole #	Sample #	Depth (m)	Su (kg/cm2)	Su (kPa)
BKS04-9	1	4.0-5.2	0.11	10.8
	2	5.2-5.5	0.38	37.3
BKS04-14	SPT	7.3-7.6	0.28	27.5
	SPT	8.5-8.8	0.21	20.6

WATER CONTENT OF SOIL
(ASTM D2216)

Hole No.	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1
Depth (m)	1.2-2.7	2.7-5.8	7.6-8.2	8.5-9.1	9.1-9.8	10.7-11.9	11.9-13.1
Sample No.	1	2	3	4	5	6	7
Wet Weight + Tare (g)	1240.23	1289.72	854.02	1062.62	627.61	1225.13	1103.95
Dry Weight + Tare (g)	1196.84	1221.31	802.76	998.81	588.43	1188.88	1043.09
Tare (g)	171.14	137.92	167.03	165.71	169.32	159.33	170.38
Water Weight (g)	43.39	68.41	51.26	63.81	39.18	36.25	60.86
Total Dry Weight (g)	1025.70	1083.39	635.73	833.10	419.11	1029.55	872.71
Water Content (%)	4.2	6.3	8.1	7.7	9.3	3.5	7.0

Hole No.	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1
Depth (m)	13.7-14.6	14.6-15.5	15.5-16.5	18.0-19.2	20.4-21.6	21.6-22.9	22.9-24.1
Sample No.	8	9	10	11	12	13	14
Wet Weight + Tare (g)	939.45	689.46	1217.55	744.16	686.96	635.16	636.15
Dry Weight + Tare (g)	852.41	630.40	1106.88	687.62	603.72	569.38	575.64
Tare (g)	173.04	174.22	169.81	155.51	162.94	167.35	163.95
Water Weight (g)	87.04	59.06	110.67	56.54	83.24	65.78	60.51
Total Dry Weight (g)	679.37	456.18	937.07	532.11	440.78	402.03	411.69
Water Content (%)	12.8	12.9	11.8	10.6	18.9	16.4	14.7

Hole No.	BKS04-1	BKS04-1	BKS04-1	BKS04-1	BKS04-1		
Depth (m)	26.2-27.7	29.3-30.8	32.6-33.8	33.8-35.1	36.3-36.9		
Sample No.	15	16	17	18	19		
Wet Weight + Tare (g)	1580.12	765.19	959.75	1442.56	1441.72		
Dry Weight + Tare (g)	1450.70	685.76	861.58	1379.48	1340.68		
Tare (g)	171.02	136.83	139.12	152.66	173.87		
Water Weight (g)	129.42	79.43	98.17	63.08	101.04		
Total Dry Weight (g)	1279.68	548.93	722.46	1226.82	1166.81		
Water Content (%)	10.1	14.5	13.6	5.1	8.7		

Hole No.	BKS04-2	BKS04-2	BKS04-2	BKS04-2	BKS04-2	BKS04-2	BKS04-2
Depth (m)	8.8-10.4	10.4-11.9	13.4-14.6	14.6-14.9	15.2-17.1	18.3-18.9	18.9-21.0
Sample No.	1	2	3	4	5	6	7
Wet Weight + Tare (g)	1160.27	713.25	1105.60	723.34	616.79	729.01	1190.24
Dry Weight + Tare (g)	1104.38	645.04	1045.33	647.06	553.10	665.73	1143.10
Tare (g)	151.61	149.33	143.63	170.56	175.20	155.78	153.10
Water Weight (g)	55.89	68.21	60.27	76.28	63.69	63.28	47.14
Total Dry Weight (g)	952.77	495.71	901.70	476.50	377.90	509.95	990.00
Water Content (%)	5.9	13.8	6.7	16.0	16.9	12.4	4.8

KLOHN CRIPPEN

JOB NO.:	M09237 A01 01		
PROJECT:	ROSE CREEK TAILINGS IMPOUNDMENT		
LOCATION:	Faro, Yukon		
DATE:	October 15, 2004		
TESTED BY:	Juan	CHECKED BY:	Ganan

WATER CONTENT OF SOIL
(ASTM D2216)

Hole No.	BKS04-2	BKS04-2	BKS04-2	BKS04-2			
Depth (m)	21.0-22.3	23.2-24.1	24.4-25.6	26.5-26.8			
Sample No.	8	9	10	11			
Wet Weight + Tare (g)	1025.57	580.12	517.19	712.58			
Dry Weight + Tare (g)	955.27	514.85	454.83	655.94			
Tare (g)	166.81	155.09	164.35	162.89			
Water Weight (g)	70.30	65.27	62.36	56.64			
Total Dry Weight (g)	788.46	359.76	290.48	493.05			
Water Content (%)	8.9	18.1	21.5	11.5			

Hole No.	BKS04-3	BKS04-3	BKS04-3	BKS04-3	BKS04-3	BKS04-3	BKS04-3
Depth (m)	13.7-14.9	14.9-17.1	17.1-18.3	19.5-21.0	24.7-26.2	28.0-30.2	30.2-33.2
Sample No.	1	2	3	4	5	6	7
Wet Weight + Tare (g)	961.41	1140.01	842.97	951.00	921.16	911.23	976.99
Dry Weight + Tare (g)	908.99	1096.44	803.99	903.40	885.50	854.47	929.83
Tare (g)	180.09	163.25	157.29	158.22	175.80	167.14	149.78
Water Weight (g)	52.42	43.57	38.98	47.60	35.66	56.76	47.16
Total Dry Weight (g)	728.90	933.19	646.70	745.18	709.70	687.33	780.05
Water Content (%)	7.2	4.7	6.0	6.4	5.0	8.3	6.0

Hole No.	BKS04-3	BKS04-3	BKS04-3				
Depth (m)	35.7-38.1	38.1-40.5	41.8-43.0				
Sample No.	8	9	10				
Wet Weight + Tare (g)	809.90	809.21	1600.42				
Dry Weight + Tare (g)	794.91	772.97	1573.34				
Tare (g)	139.24	146.82	180.68				
Water Weight (g)	14.99	36.24	27.08				
Total Dry Weight (g)	655.67	626.15	1392.66				
Water Content (%)	2.3	5.8	1.9				

Hole No.	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4
Depth (m)	11.9-13.4	13.7-14.9	14.9-18.0	18.0-21.0	21.0-24.1	24.1-27.1	27.1-30.2
Sample No.	1	2	3	4	5	6	7
Wet Weight + Tare (g)	1111.05	904.01	977.37	1127.13	797.18	714.39	1198.14
Dry Weight + Tare (g)	1017.13	821.76	934.03	1044.12	756.44	681.71	1120.06
Tare (g)	167.19	172.14	160.51	133.68	170.47	154.88	179.40
Water Weight (g)	93.92	82.25	43.34	83.01	40.74	32.68	78.08
Total Dry Weight (g)	849.94	649.62	773.52	910.44	585.97	526.83	940.66
Water Content (%)	11.1	12.7	5.6	9.1	7.0	6.2	8.3

KLOHN CRIPPEN

JOB NO.:	M09237 A01 01		
PROJECT:	ROSE CREEK TAILINGS IMPOUNDMENT		
LOCATION:	Faro, Yukon		
DATE:	October 15, 2004		
TESTED BY:	Juan	CHECKED BY:	Ganan

WATER CONTENT OF SOIL
(ASTM D2216)

Hole No.	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4	BKS04-4
Depth (m)	30.2-31.7	32.3-33.2	33.5-34.1	34.4-35.7	35.7-36.6	36.6-38.1	38.1-38.7
Sample No.	8	9	10	11	12	13	14
Wet Weight + Tare (g)	886.67	1536.04	1387.45	1606.65	1017.61	1142.27	1422.21
Dry Weight + Tare (g)	819.73	1472.63	1311.31	1541.03	915.00	1052.53	1338.52
Tare (g)	170.24	170.18	143.65	144.91	169.89	190.36	178.01
Water Weight (g)	66.94	63.41	76.14	65.62	102.61	89.74	83.69
Total Dry Weight (g)	649.49	1302.45	1167.66	1396.12	745.11	862.17	1160.51
Water Content (%)	10.3	4.9	6.5	4.7	13.8	10.4	7.2

Hole No.	BKS04-4						
Depth (m)	38.7-40.2						
Sample No.	15						
Wet Weight + Tare (g)	961.78						
Dry Weight + Tare (g)	820.68						
Tare (g)	155.48						
Water Weight (g)	141.10						
Total Dry Weight (g)	665.20						
Water Content (%)	21.2						

Hole No.	BKS04-9	BKS04-9	BKS04-9	BKS04-9			
Depth (m)	4.0-5.2	5.2-5.5	6.7-7.6	7.6-8.8			
Sample No.	1	2	3	4			
Wet Weight + Tare (g)	560.58	468.94	1653.49	1169.13			
Dry Weight + Tare (g)	471.57	394.53	1589.58	1065.48			
Tare (g)	163.27	145.60	158.53	160.53			
Water Weight (g)	89.01	74.41	63.91	103.65			
Total Dry Weight (g)	308.30	248.93	1431.05	904.95			
Water Content (%)	28.9	29.9	4.5	11.5			

Hole No.	BKS04-11	BKS04-11	BKS04-11	BKS04-11	BKS04-11		
Depth (m)	1.5-1.8	3.4-4.0	6.4-7.0	11.0-11.3	12.8		
Sample No.	1	2	3	4	5		
Wet Weight + Tare (g)	1115.32	1675.24	1273.45	1248.64	658.41		
Dry Weight + Tare (g)	1068.61	1580.21	1155.78	1183.92	634.45		
Tare (g)	171.17	158.09	153.90	165.80	134.61		
Water Weight (g)	46.71	95.03	117.67	64.72	23.96		
Total Dry Weight (g)	897.44	1422.12	1001.88	1018.12	499.84		
Water Content (%)	5.2	6.7	11.7	6.4	4.8		

KLOHN CRIPPEN

JOB NO.:	M09237 A01 01		
PROJECT:	ROSE CREEK TAILINGS IMPOUNDMENT		
LOCATION:	Faro, Yukon		
DATE:	October 15, 2004		
TESTED BY:	Juan	CHECKED BY:	Ganan

WATER CONTENT OF SOIL
(ASTM D2216)

Hole No.	BKS04-12	BKS04-12	BKS04-12	BKS04-12			
Depth (m)	4.0-4.3	6.4-6.7	11.0-11.3	13.4-13.7			
Sample No.	1	2	3	4			
Wet Weight + Tare (g)	757.05	1221.58	634.90	696.88			
Dry Weight + Tare (g)	727.93	1163.91	591.05	666.51			
Tare (g)	160.35	157.68	162.85	153.81			
Water Weight (g)	29.12	57.67	43.85	30.37			
Total Dry Weight (g)	567.58	1006.23	428.20	512.70			
Water Content (%)	5.1	5.7	10.2	5.9			

Hole No.	BKS04-13	BKS04-13	BKS04-13	BKS04-13	BKS04-13		
Depth (m)	1.5-2.1	3.7-4.3	6.1-6.4	10.4-10.7	14.3-14.7		
Sample No.	1	2	3	4	5		
Wet Weight + Tare (g)	1190.46	897.00	908.02	813.75	1023.00		
Dry Weight + Tare (g)	1171.02	822.11	831.52	760.47	979.18		
Tare (g)	157.94	163.70	158.75	169.01	193.48		
Water Weight (g)	19.44	74.89	76.50	53.28	43.82		
Total Dry Weight (g)	1013.08	658.41	672.77	591.46	785.70		
Water Content (%)	1.9	11.4	11.4	9.0	5.6		

Hole No.	BKS04-14	BKS04-14	BKS04-14	BKS04-14			
Depth (m)	3.7-4.3	4.3-4.7	7.3-7.6	8.5-8.8			
Sample No.	1	SPT	SPT	SPT			
Wet Weight + Tare (g)	1637.11	445.33	310.25	293.68			
Dry Weight + Tare (g)	1589.82	417.83	279.05	266.92			
Tare (g)	156.17	173.06	166.47	164.36			
Water Weight (g)	47.29	27.50	31.20	26.76			
Total Dry Weight (g)	1433.65	244.77	112.58	102.56			
Water Content (%)	3.3	11.2	27.7	26.1			

Hole No.							
Depth (m)							
Sample No.							
Wet Weight + Tare (g)							
Dry Weight + Tare (g)							
Tare (g)							
Water Weight (g)							
Total Dry Weight (g)							
Water Content (%)							

KLOHN CRIPPEN

JOB NO.:	M09237 A01 01		
PROJECT:	ROSE CREEK TAILINGS IMPOUNDMENT		
LOCATION:	Faro, Yukon		
DATE:	October 15, 2004		
TESTED BY:	Juan	CHECKED BY:	Ganan

APPENDIX IV

BPT PDA Measurements

Robert Miner Dynamic Testing, Inc.

Consulting, Dynamic Measurements and Analyses for Deep Foundations

October 11, 2004

Mr. Alex Sy, P. Eng.
Klohn Crippen Consultants Ltd.
500-2955 Virtual Way
Vancouver, British Columbia, V5M 4X6

Re: Dynamic Measurements of Becker Hammer Drilling
M09237A01 Rose Creek Dam Seismic Upgrade
BPT 04-01, BPT 04-02, August 17 & 18, 2004
Faro, Yukon
RMDT Job No. 04F33

Dear Mr. Sy;

This final report presents results from dynamic measurements made during operation of a Becker hammer drill for two test holes at the project referenced above. The dynamic measurements were made by Robert Miner Dynamic Testing, Inc. (RMDT) at your request, and in association with the requirements of Klohn Crippen Consultants, Ltd. Field results were provided to Klohn Crippen shortly after the testing and results of CAPWAP analyses were provided to Klohn Crippen on October 1, 2004.

Results from RMDT's monitoring and analysis include data on energy transfer from the hammer to the drill casing and soil resistance acting on the casing. These results are based on Case Method and CAPWAP analyses. Appendix A provides descriptions of our test equipment, the Case Method and CAPWAP analysis. Appendix B contains a summary of the field Case Method results. Appendix C contains measured force and velocity records for representative hammer blows from various penetrations in each monitored Becker test hole.

The Becker hammer drill uses an ICE 180 double acting diesel hammer with a maximum energy rating of 8.1 kip-ft and a 1.73 kip ram. Attached figures provide a graphical summaries of the energy transfer efficiency data for each test hole.

Following the testing, RMDT completed CAPWAP analyses of individual hammer blows at depths selected by Klohn Crippen Consultants, Ltd. Results from these analyses are summarized below and Appendix D contains detailed CAPWAP output. The CAPWAP results include computed soil resistance along the casing shaft and at the toe.

Test	Depth	Blow Count	CAPWAP Soil Resistance, kips		
	ft	Blow/ft	Total	Shaft	Toe
BPT 04-02	41 to 42	29	50	20	30
BPT 04-02	51 to 52	40	74	32	42

Mailing Address: P.O. Box 340, Manchester, WA, 98353, USA **Phone:** 360-871-5480
Location: 2288 Colchester Dr. E., Ste A, Manchester, WA, 98353 **Fax:** 360-871-5483

It was a pleasure to assist you on this project. Please do not hesitate to contact us if you have any questions regarding this report or our participation on the project.

Sincerely Yours,

Robert Miner Dynamic Testing, Inc.

Robert F. Miner

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by Goble Rausche Likins and Associates, Inc. and may only be copied with its written permission.

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing.*

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- Bearing capacity at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- Dynamic pile stresses, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- Pile integrity assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- Hammer performance parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

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Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a sufficient waiting time following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- Bearing capacity i.e. the mobilized capacity present at the time of testing
- Resistance distribution including shaft resistance and end bearing components
- Stresses in pile or shaft calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.
- Shaft impedance vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile
- Dynamic soil parameters for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

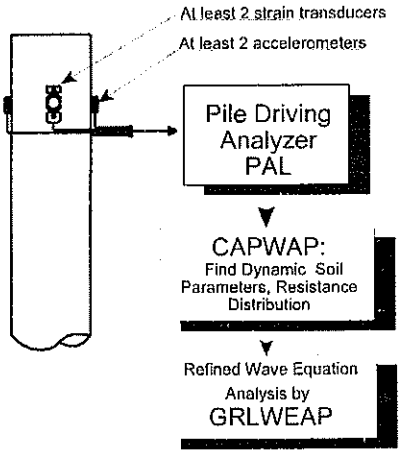
The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or "Pulse-Echo Method" of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

ANALYTICAL SOLUTIONS
BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \quad (1)$$

where

t = a point in time after impact
t_2 = time t + 2L/c
L = pile length below gages
c = $(E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\, c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \quad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \quad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2

therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = ½[F(t) - Zv(t)] \quad (4)$$

$$W_d = ½[F(t) + Zv(t)] \quad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\,\rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The

magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \quad (6)$$

with

$$\alpha_i = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling wave due to the damage reflection.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = {}_0\int^t F(t)v(t)\,dt \quad (8a)$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \quad (8b)$$

where

E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \quad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \quad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \qquad (11)$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \qquad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \qquad (12b)$$

or strain

$$\varepsilon = v / c \qquad (12c)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile

diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(B) Relaxation

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90% of yield strength for steel piles

85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression

100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Wave equation analysis results

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

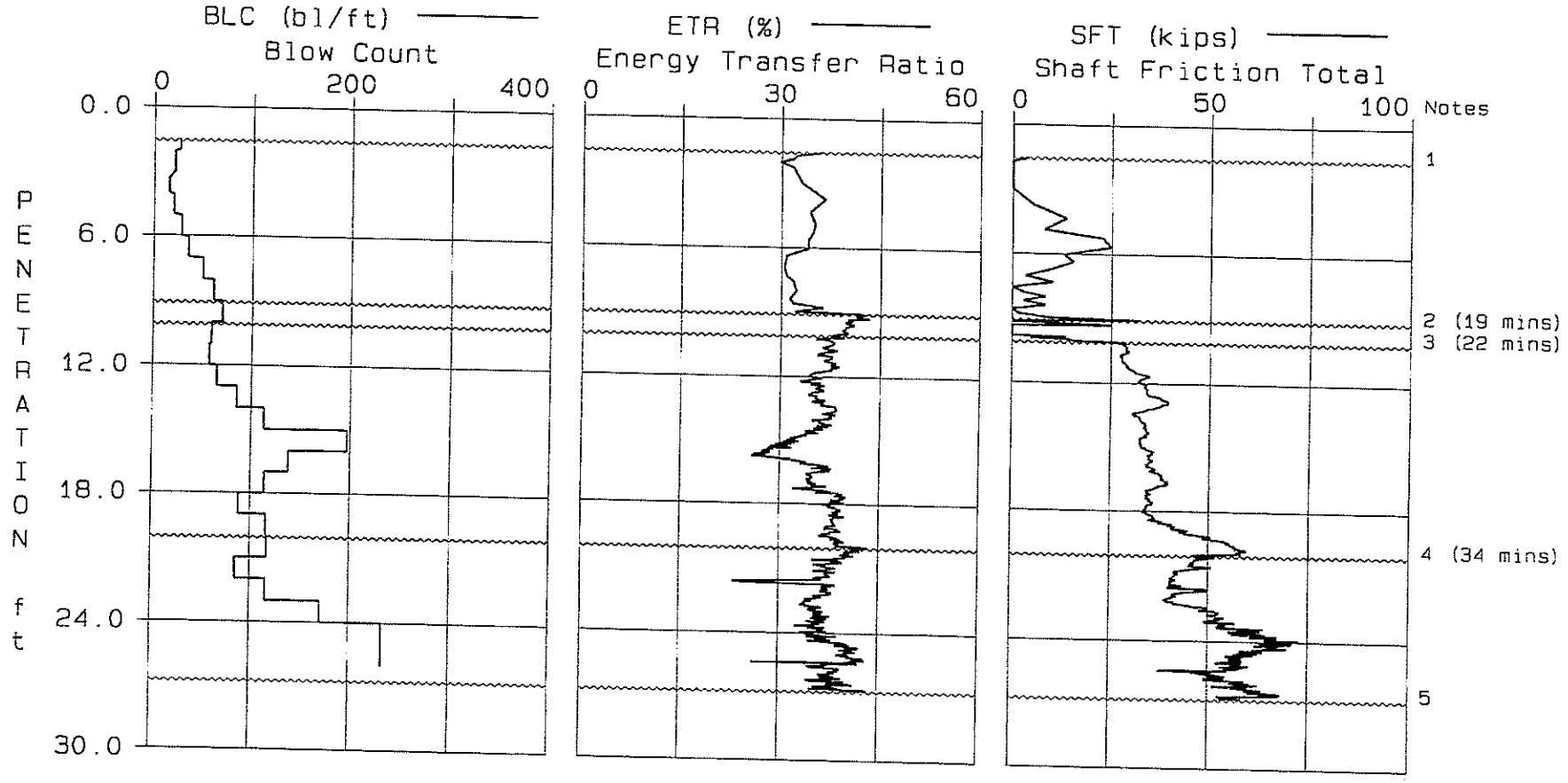
Appendix B

Case Method Results

R Miner Dynamic Test

2004-Aug-17

KCC, ROSE CREEK DAM, BPT 04-01, HAV 180, 6-5/8" CLOSED



Notes

1. BEGIN TESTING, BPT 04-01, 11:40 17AUG2004.
2. PAUSE AT 9 FT.
3. STOP AT 10 FT, PULL TEST, ADD 10 FT ROD, CONTINUE.
4. STOP AT 20 FT, PULL TEST, ADD 10 FT ROD, CONTINUE.
5. END DRIVING AND TESTING, BPT 04-01, 26.7 FT, 13:51 17AUG2004.

Robert Miner Dynamic Testing, Inc.

Pile: BPT 04-01
Info: HAV 180, 6-5/8" CLOSED
AR: 12.1 in^2
LE: 12.0 ft

Proj: KCC, ROSE CREEK DAM
SP: 0.493 k/ft^3
WS: 16800 ft/s
EM: 30004 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio
SFT: Shaft Friction Total

BPM: Blows Per Minute
FMX: Max Measured Force
RA2: Capacity - RA2
DMX: Max Meas'd Displacement

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
1	26	1.50	AV	8.01	12.31	2.7	33	0	0.0	61	63	0.54
2	26	1.54	AV	12.64	17.60	3.3	40	10	44.6	69	67	0.57

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	RMX kips	RA2 kips	DMX inch
14	26	2.00	AV	11.73	16.06	2.7	32	0	90.9	83	84	0.50
			SD	0.36	0.50	0.2	2	1	0.2	2	4	0.16
			MX	12.14	16.90	3.1	37	5	91.2	86	90	0.86
35	21	3.00	AV	11.05	17.04	2.6	32	0	76.1	72	67	0.52
			SD	1.28	1.22	0.1	2	0	22.3	11	9	0.09
			MX	12.31	18.76	2.8	34	0	91.4	85	79	0.62
50	15	4.00	AV	11.05	14.61	3.0	36	4	83.1	63	58	0.74
			SD	0.90	1.16	0.1	2	6	20.4	2	1	0.10
			MX	11.98	16.44	3.1	38	14	92.2	64	58	0.88
70	20	5.00	AV	12.35	15.15	2.9	35	11	91.2	77	76	0.50
			SD	0.51	0.34	0.1	1	5	0.3	5	5	0.06
			MX	12.97	15.61	2.9	35	16	91.6	81	80	0.64
98	28	6.00	AV	14.55	16.32	2.8	34	20	90.5	91	90	0.47
			SD	0.87	1.27	0.2	2	8	0.2	4	9	0.06
			MX	15.78	18.01	3.1	38	26	90.8	97	105	0.58
133	35	7.00	AV	14.55	18.53	2.5	31	13	90.4	101	111	0.36
			SD	0.55	0.65	0.2	2	11	0.2	6	3	0.05
			MX	15.49	19.58	2.8	34	26	90.8	109	117	0.46
183	50	8.00	AV	14.78	22.63	2.6	32	5	87.6	118	117	0.39
			SD	0.51	2.66	0.1	1	9	11.0	2	5	0.11
			MX	15.45	26.69	2.7	33	27	90.4	121	126	0.66
243	61	8.98	AV	13.76	25.52	2.8	34	4	90.2	129	131	0.39
			SD	0.28	1.03	0.3	4	7	0.1	6	9	0.15
			MX	14.29	27.68	3.3	40	22	90.4	139	146	0.94

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
244	61	9.00	AV	9.50	14.71	0.9	11	0	0.0	115	105	0.15
314	70	10.00	AV	17.17	24.75	3.3	41	6	90.2	208	147	0.47
			SD	0.86	0.93	0.2	2	13	0.4	10	5	0.20
			MX	20.16	26.94	4.0	49	40	91.8	244	157	1.14

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
373	59	11.00	AV	17.51	18.99	3.0	37	28	90.7	212	120	0.42
			SD	0.51	1.44	0.1	2	2	0.2	6	7	0.13
			MX	18.67	21.86	3.3	40	31	91.1	226	135	1.04
430	57	12.00	AV	17.62	20.02	3.0	37	31	90.7	213	119	0.37
			SD	0.55	1.22	0.1	2	2	0.2	7	8	0.01
			MX	18.59	21.90	3.3	40	36	91.1	225	140	0.39
495	65	13.00	AV	18.07	21.46	2.9	35	36	90.3	219	123	0.33
			SD	0.65	0.50	0.1	1	2	0.1	8	7	0.01
			MX	19.17	22.64	3.1	38	40	90.6	232	137	0.37
580	85	14.00	AV	19.47	20.88	3.0	37	33	90.1	236	141	0.32
			SD	0.40	0.89	0.1	1	2	0.1	5	9	0.01
			MX	20.24	22.39	3.2	39	38	90.3	245	163	0.34
693	113	15.00	AV	19.10	21.20	2.9	35	33	90.1	231	159	0.29
			SD	0.57	0.59	0.2	2	1	0.1	7	4	0.01
			MX	20.49	22.64	3.3	40	35	90.3	248	168	0.33
890	197	16.00	AV	18.05	19.69	2.4	30	35	90.2	218	161	0.25
			SD	0.57	0.46	0.2	2	1	0.1	7	4	0.01
			MX	19.58	20.90	2.8	34	36	90.6	237	169	0.28
1028	138	17.00	AV	19.94	22.48	2.9	35	37	90.3	241	164	0.29
			SD	0.62	1.15	0.1	2	1	0.1	7	7	0.01
			MX	21.73	24.21	3.3	40	41	90.4	263	180	0.31
1142	114	18.00	AV	19.62	24.91	3.1	37	35	90.3	237	154	0.31
			SD	0.61	1.01	0.2	2	1	0.1	7	11	0.02
			MX	20.90	27.35	3.4	41	36	90.6	253	176	0.35
1230	88	19.00	AV	18.96	27.95	3.1	38	38	90.3	230	131	0.33
			SD	0.32	1.33	0.1	1	4	0.1	4	4	0.01
			MX	19.91	30.74	3.4	41	46	90.6	241	145	0.36
1346	116	20.00	AV	17.79	33.51	3.1	38	54	90.3	215	139	0.31
			SD	0.42	2.16	0.1	1	5	0.0	5	3	0.01
			MX	18.67	38.26	3.4	41	61	90.4	226	147	0.33
1463	117	21.00	AV	19.55	22.68	3.2	39	46	89.7	237	150	0.34
			SD	0.66	1.39	0.2	2	3	8.4	8	9	0.01
			MX	21.32	25.20	3.7	45	55	91.1	258	172	0.38
1548	85	22.00	AV	18.53	21.95	2.9	35	42	90.6	224	136	0.34
			SD	1.17	1.74	0.4	4	4	0.5	14	9	0.03
			MX	20.16	26.03	3.3	40	54	92.0	244	151	0.37
1664	116	23.00	AV	18.75	27.63	2.9	35	47	90.3	227	154	0.33
			SD	0.37	0.83	0.1	2	6	0.1	4	13	0.01
			MX	19.66	30.00	3.2	39	62	90.6	238	171	0.36

Info: HAV 180, 6-5/8" CLOSED

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
1835	171	24.00	AV	18.99	29.62	2.9	36	58	90.4	230	158	0.33
			SD	0.44	0.94	0.1	2	7	0.1	5	8	0.01
			MX	20.08	31.98	3.3	40	76	90.9	243	177	0.35
2068	233	25.00	AV	19.38	33.19	3.2	38	61	90.4	235	169	0.33
			SD	0.58	1.70	0.2	3	6	0.1	7	8	0.01
			MX	20.90	36.77	3.7	45	77	90.7	253	187	0.38
2301	233	26.00	AV	19.72	30.89	3.2	39	54	90.4	239	193	0.31
			SD	0.87	7.67	0.3	3	5	0.2	11	10	0.03
			MX	20.82	39.33	3.6	44	64	91.7	252	216	0.35
2739	625	26.70	AV	20.56	23.30	3.1	38	60	90.6	249	216	0.28
			SD	0.67	0.82	0.2	2	5	0.2	8	12	0.01
			MX	22.47	25.20	3.7	45	71	92.3	272	247	0.31

BL#	COMMENTS
1	BEGIN TESTING, BPT 04-01, 11:40 17AUG2004.
243	PAUSE AT 9 FT.
314	STOP AT 10 FT, PULL TEST, ADD 10 FT ROD, CONTINUE.
1346	STOP AT 20 FT, PULL TEST, ADD 10 FT ROD, CONTINUE.
2739	END DRIVING AND TESTING, BPT 04-01, 26.7 FT, 13:51 17AUG2004.

DRIVE TIME SUMMARY (2004-Aug-17 : BPT0401.MDF)			DRIVE ----- minutes	WAIT ----
BN 1 -> 243,	START 11:40:51 -> 11:45:13	STOP,	4.37	
				19.45
BN 244 -> 314,	START 12:04:40 -> 12:05:27	STOP,	0.78	
				22.62
BN 315 -> 1346,	START 12:28:04 -> 13:02:07	STOP,	34.05	
				34.08
BN 1347 -> 2739,	START 13:36:12 -> 13:51:36	STOP,	15.40	
Total Elapsed time 130.75 minutes		Total Time	54.60 minutes	76.15

Info: HAV 180, 6-5/8" CLOSED

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio
SFT: Shaft Friction Total

BPM: Blows Per Minute
FMX: Max Measured Force
RA2: Capacity - RA2
DMX: Max Meas'd Displacement

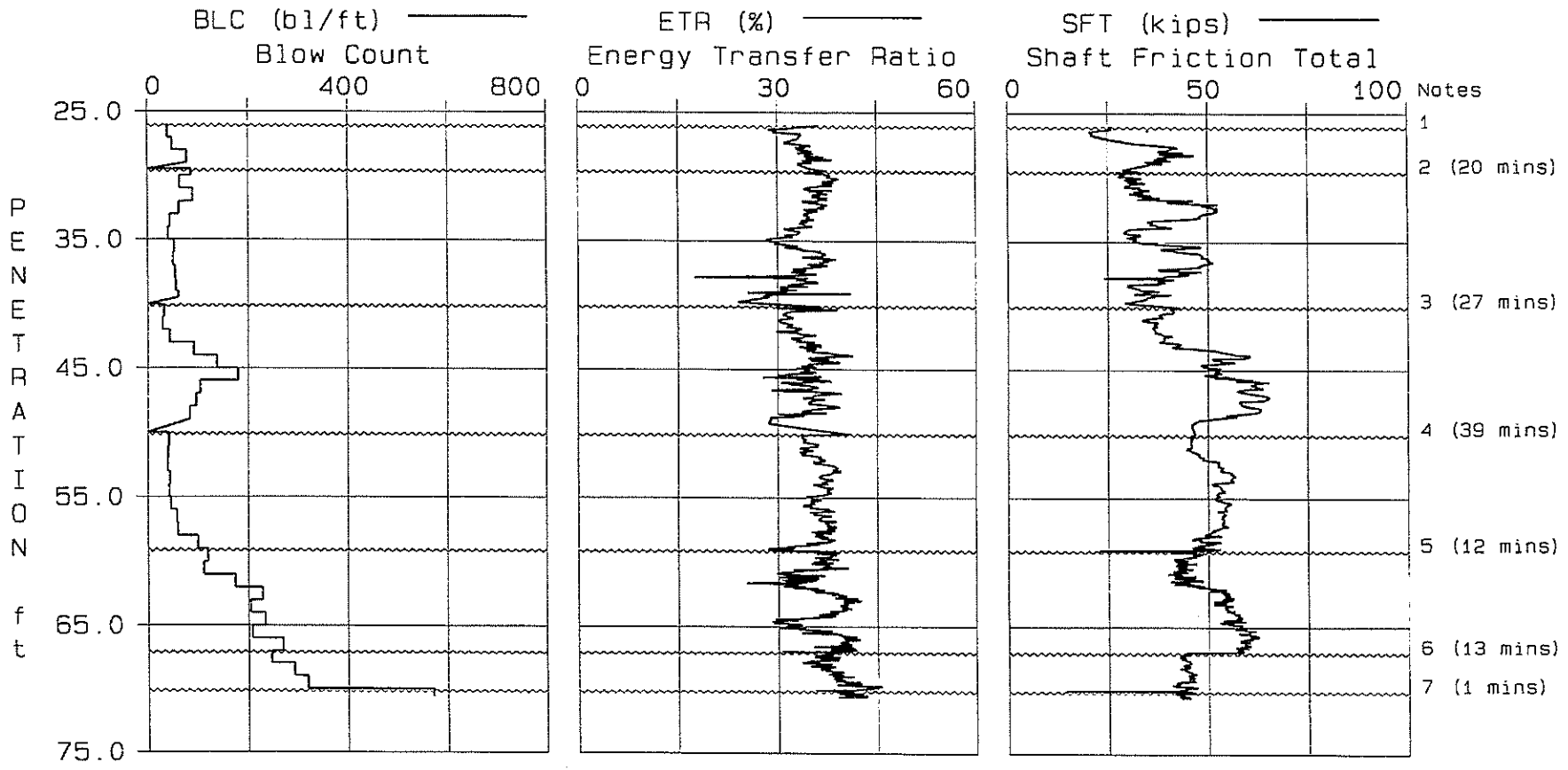
BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
1	26	1.50	AV	8.01	12.31	2.7	33	0	0.0	61	63	0.54
2	26	1.54	AV	12.64	17.60	3.3	40	10	44.6	69	67	0.57
BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	RMX kips	RA2 kips	DMX inch
14	26	2.00	AV	11.73	16.06	2.7	32	0	90.9	83	84	0.50
35	21	3.00	AV	11.05	17.04	2.6	32	0	76.1	72	67	0.52
50	15	4.00	AV	11.05	14.61	3.0	36	4	83.1	63	58	0.74
70	20	5.00	AV	12.35	15.15	2.9	35	11	91.2	77	76	0.50
98	28	6.00	AV	14.55	16.32	2.8	34	20	90.5	91	90	0.47
133	35	7.00	AV	14.55	18.53	2.5	31	13	90.4	101	111	0.36
183	50	8.00	AV	14.78	22.63	2.6	32	5	87.6	118	117	0.39
243	61	8.98	AV	13.76	25.52	2.8	34	4	90.2	129	131	0.39
BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
244	61	9.00	AV	9.50	14.71	0.9	11	0	0.0	115	105	0.15
314	70	10.00	AV	17.17	24.75	3.3	41	6	90.2	208	147	0.47
373	59	11.00	AV	17.51	18.99	3.0	37	28	90.7	212	120	0.42
430	57	12.00	AV	17.62	20.02	3.0	37	31	90.7	213	119	0.37
495	65	13.00	AV	18.07	21.46	2.9	35	36	90.3	219	123	0.33
580	85	14.00	AV	19.47	20.88	3.0	37	33	90.1	236	141	0.32
693	113	15.00	AV	19.10	21.20	2.9	35	33	90.1	231	159	0.29
890	197	16.00	AV	18.05	19.69	2.4	30	35	90.2	218	161	0.25
1028	138	17.00	AV	19.94	22.48	2.9	35	37	90.3	241	164	0.29
1142	114	18.00	AV	19.62	24.91	3.1	37	35	90.3	237	154	0.31
1230	88	19.00	AV	18.96	27.95	3.1	38	38	90.3	230	131	0.33
1346	116	20.00	AV	17.79	33.51	3.1	38	54	90.3	215	139	0.31
1463	117	21.00	AV	19.55	22.68	3.2	39	46	89.7	237	150	0.34
1548	85	22.00	AV	18.53	21.95	2.9	35	42	90.6	224	136	0.34
1664	116	23.00	AV	18.75	27.63	2.9	35	47	90.3	227	154	0.33
1835	171	24.00	AV	18.99	29.62	2.9	36	58	90.4	230	158	0.33
2068	233	25.00	AV	19.38	33.19	3.2	38	61	90.4	235	169	0.33
2301	233	26.00	AV	19.72	30.89	3.2	39	54	90.4	239	193	0.31
2739	625	26.70	AV	20.56	23.30	3.1	38	60	90.6	249	216	0.28

BL#	COMMENTS
1	BEGIN TESTING, BPT 04-01, 11:40 17AUG2004.
243	PAUSE AT 9 FT.
314	STOP AT 10 FT, PULL TEST, ADD 10 FT ROD, CONTINUE.
1346	STOP AT 20 FT, PULL TEST, ADD 10 FT ROD, CONTINUE.
2739	END DRIVING AND TESTING, BPT 04-01, 26.7 FT, 13:51 17AUG2004.

DRIVEN (2004-Aug-17 : BPT0401.MDF)

R Miner Dynamic Test 2004-Aug-18

KCC, ROSE CREEK DAM, BPT 04-02, HAV 180, 6-5/8" CLOSED



Notes
1. BEGIN PDA TEST, BPT 04-02, 12:00 18AUG04
2. PAUSE @ 29FT. DRIVE TO 29.5FT WITHOUT PDA.
3. PAUSE AT 39.5FT, DRIVE TO 40FT WITHOUT PDA, PULL TEST, ADD 10 FT.
4. PAUSE AT 49 FT, DRIVE TO 50 FT WITHOUT PDA, PULL TEST, ADD 10 FT.
5. STOP AT 59 FT, PULL TEST, ADD 8 FT ROD, CONTINUE.

Notes
6. STOP AT 67 FT, PULL TEST, ADD 8 FT ROD, CONTINUE.
7. DRILLER PULLS CASING BACK, THEN CONTINUE.

Robert Miner Dynamic Testing, Inc.

Pile: BPT 04-02
Info: HAV 180
AR: 12.1 in^2
LE: 32.0 ft

Proj: KCC
SP: 0.493 k/ft^3
WS: 16800 ft/s
EM: 30004 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio
SFT: Shaft Friction Total

BPM: Blows Per Minute
RMX: Capacity - RMX
RA2: Capacity - RA2
DMX: Max Meas'd Displacement

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	RMX kips	RA2 kips	DMX inch
1	40	26.00	AV	8.76	10.08	2.3	28	24	0.0	71	71	0.44
41	40	27.00	AV	12.71	13.80	2.6	32	23	92.5	73	69	0.47
			SD	1.45	1.48	0.3	4	3	1.0	3	3	0.04
			MX	18.26	19.50	4.0	49	31	93.7	81	77	0.63
65	49	27.49	AV	14.14	16.03	2.6	32	31	92.0	96	84	0.38
			SD	0.44	1.06	0.2	2	5	0.3	8	9	0.03
			MX	14.62	17.02	2.8	34	38	92.5	105	97	0.42

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
90	49	28.00	AV	15.45	18.35	2.8	34	40	91.3	187	107	0.37
			SD	0.55	0.59	0.2	2	3	0.3	7	4	0.02
			MX	16.52	19.33	3.1	38	46	92.0	200	114	0.41
168	78	29.00	AV	15.79	18.27	2.9	35	39	91.0	191	112	0.37
			SD	0.59	0.74	0.2	2	4	0.3	7	4	0.01
			MX	17.27	20.16	3.3	40	49	91.7	209	124	0.40
169	2	29.50	AV	10.57	13.22	1.9	23	28	0.0	128	85	0.33
212	86	30.00	AV	16.31	18.44	3.0	37	29	91.0	197	97	0.42
			SD	0.82	0.94	0.2	3	4	0.9	10	4	0.03
			MX	20.90	23.38	4.4	54	45	95.8	253	109	0.57
276	64	31.00	AV	16.64	17.86	3.0	37	32	90.8	201	103	0.42
			SD	0.38	0.44	0.2	2	4	0.2	5	5	0.02
			MX	17.68	18.84	3.3	40	45	91.1	214	111	0.46
365	89	32.00	AV	16.56	18.12	2.9	36	37	90.7	200	97	0.42
			SD	0.49	0.68	0.1	2	6	0.2	6	7	0.01
			MX	17.93	19.42	3.3	40	54	91.0	217	113	0.44
427	62	33.00	AV	16.35	17.18	2.9	36	50	90.7	198	84	0.43
			SD	0.63	0.79	0.1	2	3	0.3	8	9	0.01
			MX	17.35	18.59	3.2	39	58	91.3	210	109	0.46
471	44	34.00	AV	14.71	15.48	2.8	34	39	91.5	178	72	0.47
			SD	0.67	0.73	0.1	1	5	0.4	8	5	0.02
			MX	15.86	17.76	3.1	38	52	92.1	192	83	0.51
511	40	35.00	AV	13.26	14.21	2.6	31	31	91.8	160	64	0.46
			SD	0.72	1.06	0.2	2	2	0.5	9	3	0.03
			MX	14.71	16.61	2.9	35	43	92.8	178	70	0.50

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
563	52	36.00	AV	14.91	15.62	2.7	33	42	91.5	181	89	0.42
			SD	1.09	1.33	0.2	3	5	0.4	13	11	0.02
			MX	16.61	17.60	3.1	38	51	92.4	201	110	0.46
613	50	37.00	AV	16.44	17.53	3.0	37	49	91.3	199	87	0.44
			SD	0.54	0.71	0.1	2	2	0.3	6	3	0.02
			MX	17.35	19.17	3.2	39	53	92.3	210	91	0.46
667	54	38.00	AV	14.53	16.76	2.7	32	41	91.8	176	97	0.42
			SD	2.08	2.50	0.5	6	8	1.1	25	14	0.05
			MX	16.28	20.57	3.2	39	50	97.0	197	119	0.47
723	56	39.00	AV	13.63	19.15	2.7	32	34	91.9	165	97	0.44
			SD	1.04	1.19	0.2	2	4	0.6	13	11	0.02
			MX	15.70	25.12	3.0	37	46	93.0	190	116	0.47
753	60	39.50	AV	12.40	16.81	2.4	29	35	90.0	150	72	0.43
			SD	1.75	1.87	0.5	6	4	17.0	21	9	0.05
			MX	18.59	23.30	4.2	51	46	94.6	225	93	0.59
754	2	40.00	AV	10.41	12.47	1.8	22	23	0.0	126	66	0.38
786	32	41.00	AV	13.42	14.68	2.7	33	39	90.7	162	51	0.54
			SD	1.45	1.69	0.4	5	3	16.6	18	6	0.08
			MX	18.92	20.90	4.6	56	45	95.0	229	66	0.87
815	29	42.00	AV	13.14	14.39	2.6	32	36	94.2	159	48	0.52
			SD	0.89	0.82	0.2	2	2	0.6	11	4	0.02
			MX	14.71	16.44	2.9	35	39	95.2	178	56	0.58
858	43	43.00	AV	14.89	15.41	2.7	33	39	92.7	180	76	0.45
			SD	0.71	0.74	0.2	2	2	0.6	9	15	0.02
			MX	16.28	17.10	3.1	38	45	94.0	197	101	0.50
949	91	44.00	AV	16.35	17.86	3.0	36	51	91.5	198	107	0.41
			SD	0.97	1.05	0.2	2	7	0.4	12	8	0.02
			MX	18.84	21.07	3.7	45	61	92.5	228	123	0.46
1086	137	45.00	AV	17.21	18.03	2.9	36	52	91.0	208	120	0.35
			SD	0.76	0.76	0.2	2	3	0.4	9	10	0.03
			MX	19.33	19.66	3.3	40	59	92.1	234	140	0.42
1265	179	46.00	AV	16.71	18.46	2.8	34	55	91.1	202	113	0.33
			SD	0.94	0.95	0.2	2	5	0.4	11	7	0.01
			MX	19.00	20.99	3.3	40	66	92.7	230	135	0.37
1368	103	47.00	AV	17.21	18.17	2.8	34	61	91.2	208	114	0.34
			SD	0.83	0.96	0.3	3	2	0.4	10	6	0.02
			MX	18.76	19.91	3.3	40	66	92.4	227	126	0.39
1463	95	48.00	AV	17.51	18.71	2.9	36	62	91.1	212	114	0.35
			SD	0.56	0.70	0.2	2	3	0.3	7	5	0.02
			MX	18.67	20.08	3.3	40	67	92.1	226	127	0.39

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
1546	83	49.00	AV	15.07	15.96	2.7	33	56	92.0	182	107	0.37
			SD	1.96	1.90	0.3	4	6	0.8	24	9	0.03
			MX	17.93	19.66	3.3	40	65	95.6	217	128	0.42
1547	1	50.00	AV	10.66	13.30	2.4	29	40	0.0	129	79	0.42
1589	42	51.00	AV	15.61	20.96	2.9	35	46	92.1	189	77	0.45
			SD	0.94	0.91	0.3	3	1	0.8	11	4	0.04
			MX	20.16	24.62	4.4	54	49	93.0	244	91	0.70
1629	40	52.00	AV	16.28	21.98	2.8	35	47	91.8	197	74	0.43
			SD	0.48	0.53	0.1	2	2	0.3	6	3	0.01
			MX	17.10	22.72	3.1	38	51	92.4	207	79	0.46
1668	39	53.00	AV	17.18	22.68	3.1	38	53	91.2	208	75	0.45
			SD	0.43	0.55	0.1	2	2	0.2	5	2	0.01
			MX	18.09	23.96	3.3	40	57	91.8	219	82	0.48
1711	43	54.00	AV	17.15	21.24	3.0	37	55	91.2	208	84	0.44
			SD	0.33	1.27	0.1	2	2	0.2	4	6	0.01
			MX	17.68	23.22	3.2	39	58	91.6	214	96	0.46
1753	42	55.00	AV	16.84	19.44	3.0	37	53	91.4	204	82	0.44
			SD	0.45	0.37	0.1	2	1	0.2	5	5	0.01
			MX	17.93	20.33	3.3	40	55	91.8	217	97	0.47
1798	45	56.00	AV	16.42	19.62	2.9	35	54	91.5	199	88	0.43
			SD	0.40	0.81	0.1	2	1	0.3	5	6	0.01
			MX	17.43	21.32	3.2	39	57	92.3	211	99	0.46
1855	57	57.00	AV	16.92	20.32	3.0	37	54	91.3	205	99	0.42
			SD	0.48	0.57	0.1	2	1	0.3	6	6	0.01
			MX	17.85	21.65	3.3	40	56	91.8	216	107	0.45
1914	59	58.00	AV	17.37	21.56	3.0	37	52	90.9	210	108	0.41
			SD	0.32	0.48	0.1	1	2	0.1	4	4	0.01
			MX	18.18	22.89	3.3	40	57	91.1	220	121	0.44
2012	98	59.00	AV	17.11	21.92	2.9	35	49	91.0	207	115	0.38
			SD	0.76	0.74	0.2	3	3	0.4	9	7	0.02
			MX	18.26	22.97	3.3	40	55	92.3	221	133	0.42
2130	118	60.00	AV	15.50	19.09	3.0	36	44	90.4	188	105	0.43
			SD	1.36	1.58	0.2	3	6	8.5	16	15	0.20
			MX	16.69	20.41	3.3	40	50	98.1	202	123	2.53
2240	110	61.00	AV	15.39	18.78	2.9	35	43	91.4	186	103	0.41
			SD	0.81	0.99	0.3	3	2	0.6	10	4	0.02
			MX	16.94	20.41	3.4	41	49	92.7	205	117	0.46
2413	173	62.00	AV	15.35	18.31	2.7	33	46	91.2	186	127	0.37
			SD	0.89	0.91	0.2	3	3	3.3	11	10	0.02
			MX	17.19	20.00	3.1	38	54	93.4	208	144	0.42

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
2641	228	63.00	AV	17.45	19.13	3.2	39	54	90.7	211	152	0.38
			SD	0.77	0.73	0.2	3	2	0.3	9	10	0.01
			MX	19.00	20.57	3.7	45	58	91.6	230	167	0.40
2845	204	64.00	AV	17.72	19.20	3.3	40	55	90.5	214	152	0.39
			SD	0.42	0.52	0.1	1	2	0.1	5	4	0.01
			MX	18.59	20.57	3.5	43	59	90.9	225	162	0.40
3078	233	65.00	AV	16.02	17.59	2.7	33	57	91.1	194	142	0.35
			SD	0.74	0.96	0.2	3	2	0.4	9	3	0.02
			MX	17.68	19.50	3.3	40	61	92.3	214	152	0.39
3285	207	66.00	AV	17.03	17.75	3.0	37	60	91.0	206	148	0.36
			SD	1.21	1.10	0.3	4	2	0.6	15	7	0.03
			MX	18.92	19.91	3.5	43	65	94.6	229	161	0.40
3554	269	67.00	AV	18.09	19.04	3.3	40	59	90.7	219	176	0.37
			SD	0.92	0.99	0.3	3	2	0.5	11	10	0.02
			MX	19.25	20.41	3.5	43	66	95.6	233	195	0.40
3800	246	68.00	AV	16.76	18.98	3.0	37	44	91.0	203	153	0.40
			SD	0.57	0.72	0.2	2	1	0.3	7	5	0.01
			MX	17.93	21.15	3.4	41	46	92.4	217	164	0.42
4092	292	69.00	AV	16.95	22.54	3.1	38	45	90.8	205	172	0.39
			SD	0.34	1.26	0.1	2	1	0.2	4	11	0.01
			MX	17.76	24.62	3.4	41	48	91.1	215	186	0.41
4411	319	70.00	AV	17.86	23.64	3.4	41	44	90.6	216	188	0.39
			SD	0.59	1.47	0.2	2	2	0.2	7	4	0.01
			MX	19.17	27.43	3.9	48	48	91.1	232	201	0.41
4754	571	70.60	AV	17.35	23.00	3.3	41	43	89.7	210	192	0.43
			SD	1.17	1.61	0.2	2	6	10.5	14	28	0.24
			MX	19.00	25.86	3.8	46	48	98.3	230	217	2.45

BL#	COMMENTS
1	BEGIN PDA TEST, BPT 04-02, 12:00 18AUG04
168	PAUSE @ 29FT, DRIVE TO 29.5FT WITHOUT PDA.
753	PAUSE AT 39.5FT, DRIVE TO 40FT WITHOUT PDA, PULL TEST, ADD 10 FT.
1546	PAUSE AT 49 FT, DRIVE TO 50 FT WITHOUT PDA, PULL TEST, ADD 10 FT.
2012	STOP AT 59 FT, PULL TEST, ADD 8 FT ROD, CONTINUE.
3554	STOP AT 67 FT, PULL TEST, ADD 8 FT ROD, CONTINUE.
4445	DRILLER PULLS CASING BACK, THEN CONTINUE.

DRIVE TIME SUMMARY (2004-Aug-18 : BPT0402.MDF)			DRIVE minutes	WAIT
BN 1 -> 168, START 12:00:54 -> 12:02:44 STOP,			1.83	
				20.23
BN 169 -> 753, START 12:22:58 -> 12:30:51 STOP,			7.88	
				27.72
BN 754 -> 1546, START 12:58:34 -> 13:07:30 STOP,			8.93	
				39.30
BN 1547 -> 2012, START 13:46:48 -> 13:51:54 STOP,			5.10	
				12.02
BN 2013 -> 3554, START 14:03:55 -> 14:20:52 STOP,			16.95	
				13.47
BN 3555 -> 4445, START 14:34:20 -> 14:47:19 STOP,			12.98	
				1.07
BN 4447 -> 4754, START 14:48:23 -> 14:51:46 STOP,			3.38	
Total Elapsed time 170.87 minutes	Total Time		57.07 minutes	113.80

Pile: BPT 04-02
Info: HAV 180
AR: 12.1 in^2
LE: 32.0 ft

Proj: KCC
SP: 0.493 k/ft^3
WS: 16800 ft/s
EM: 30004 KSI

CSX: Max Measured C-Stress
CSI: Max F1 or F2 C-Stress
EMX: Max Transferred Energy
ETR: Energy Transfer Ratio
SFT: Shaft Friction Total

BPM: Blows Per Minute
RMX: Capacity - RMX
RA2: Capacity - RA2
DMX: Max Meas'd Displacement

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	RMX kips	RA2 kips	DMX inch
1	40	26.00	AV	8.76	10.08	2.3	28	24	0.0	71	71	0.44
41	40	27.00	AV	12.71	13.80	2.6	32	23	92.5	73	69	0.47
65	49	27.49	AV	14.14	16.03	2.6	32	31	92.0	96	84	0.38
BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
90	49	28.00	AV	15.45	18.35	2.8	34	40	91.3	187	107	0.37
168	78	29.00	AV	15.79	18.27	2.9	35	39	91.0	191	112	0.37
169	2	29.50	AV	10.57	13.22	1.9	23	28	0.0	128	85	0.33
212	86	30.00	AV	16.31	18.44	3.0	37	29	91.0	197	97	0.42
276	64	31.00	AV	16.64	17.86	3.0	37	32	90.8	201	103	0.42
365	89	32.00	AV	16.56	18.12	2.9	36	37	90.7	200	97	0.42
427	62	33.00	AV	16.35	17.18	2.9	36	50	90.7	198	84	0.43
471	44	34.00	AV	14.71	15.48	2.8	34	39	91.5	178	72	0.47
511	40	35.00	AV	13.26	14.21	2.6	31	31	91.8	160	64	0.46
563	52	36.00	AV	14.91	15.62	2.7	33	42	91.5	181	89	0.42
613	50	37.00	AV	16.44	17.53	3.0	37	49	91.3	199	87	0.44
667	54	38.00	AV	14.53	16.76	2.7	32	41	91.8	176	97	0.42
723	56	39.00	AV	13.63	19.15	2.7	32	34	91.9	165	97	0.44
753	60	39.50	AV	12.40	16.81	2.4	29	35	90.0	150	72	0.43
754	2	40.00	AV	10.41	12.47	1.8	22	23	0.0	126	66	0.38
786	32	41.00	AV	13.42	14.68	2.7	33	39	90.7	162	51	0.54
815	29	42.00	AV	13.14	14.39	2.6	32	36	94.2	159	48	0.52
858	43	43.00	AV	14.89	15.41	2.7	33	39	92.7	180	76	0.45
949	91	44.00	AV	16.35	17.86	3.0	36	51	91.5	198	107	0.41
1086	137	45.00	AV	17.21	18.03	2.9	36	52	91.0	208	120	0.35
1265	179	46.00	AV	16.71	18.46	2.8	34	55	91.1	202	113	0.33
1368	103	47.00	AV	17.21	18.17	2.8	34	61	91.2	208	114	0.34
1463	95	48.00	AV	17.51	18.71	2.9	36	62	91.1	212	114	0.35
1546	83	49.00	AV	15.07	15.96	2.7	33	56	92.0	182	107	0.37
1547	1	50.00	AV	10.66	13.30	2.4	29	40	0.0	129	79	0.42
1589	42	51.00	AV	15.61	20.96	2.9	35	46	92.1	189	77	0.45
1629	40	52.00	AV	16.28	21.98	2.8	35	47	91.8	197	74	0.43
1668	39	53.00	AV	17.18	22.68	3.1	38	53	91.2	208	75	0.45
1711	43	54.00	AV	17.15	21.24	3.0	37	55	91.2	208	84	0.44
1753	42	55.00	AV	16.84	19.44	3.0	37	53	91.4	204	82	0.44
1798	45	56.00	AV	16.42	19.62	2.9	35	54	91.5	199	88	0.43
1855	57	57.00	AV	16.92	20.32	3.0	37	54	91.3	205	99	0.42
1914	59	58.00	AV	17.37	21.56	3.0	37	52	90.9	210	108	0.41
2012	98	59.00	AV	17.11	21.92	2.9	35	49	91.0	207	115	0.38
2130	118	60.00	AV	15.50	19.09	3.0	36	44	90.4	188	105	0.43
2240	110	61.00	AV	15.39	18.78	2.9	35	43	91.4	186	103	0.41
2413	173	62.00	AV	15.35	18.31	2.7	33	46	91.2	186	127	0.37
2641	228	63.00	AV	17.45	19.13	3.2	39	54	90.7	211	152	0.38
2845	204	64.00	AV	17.72	19.20	3.3	40	55	90.5	214	152	0.39
3078	233	65.00	AV	16.02	17.59	2.7	33	57	91.1	194	142	0.35

BL# end	bl/ft	depth ft	TY	CSX ksi	CSI ksi	EMX K-ft	ETR %	SFT kips	BPM bl/min	FMX kips	RA2 kips	DMX inch
3285	207	66.00	AV	17.03	17.75	3.0	37	60	91.0	206	148	0.36
3554	269	67.00	AV	18.09	19.04	3.3	40	59	90.7	219	176	0.37
3800	246	68.00	AV	16.76	18.98	3.0	37	44	91.0	203	153	0.40
4092	292	69.00	AV	16.95	22.54	3.1	38	45	90.8	205	172	0.39
4411	319	70.00	AV	17.86	23.64	3.4	41	44	90.6	216	188	0.39
4754	571	70.60	AV	17.35	23.00	3.3	41	43	89.7	210	192	0.43

BL#	COMMENTS
1	BEGIN PDA TEST, BPT 04-02, 12:00 18AUG04
168	PAUSE @ 29FT, DRIVE TO 29.5FT WITHOUT PDA.
753	PAUSE AT 39.5FT, DRIVE TO 40FT WITHOUT PDA, PULL TEST, ADD 10 FT.
1546	PAUSE AT 49 FT, DRIVE TO 50 FT WITHOUT PDA, PULL TEST, ADD 10 FT.
2012	STOP AT 59 FT, PULL TEST, ADD 8 FT ROD, CONTINUE.
3554	STOP AT 67 FT, PULL TEST, ADD 8 FT ROD, CONTINUE.
4445	DRILLER PULLS CASING BACK, THEN CONTINUE.

DRIVE TIME SUMMARY (2004-Aug-18 : BPT0402.MDF)			DRIVE ----- minutes	WAIT ----
BN 1 -> 168,	START 12:00:54 -> 12:02:44	STOP,	1.83	
				20.23
BN 169 -> 753,	START 12:22:58 -> 12:30:51	STOP,	7.88	
				27.72
BN 754 -> 1546,	START 12:58:34 -> 13:07:30	STOP,	8.93	
				39.30
BN 1547 -> 2012,	START 13:46:48 -> 13:51:54	STOP,	5.10	
				12.02
BN 2013 -> 3554,	START 14:03:55 -> 14:20:52	STOP,	16.95	
				13.47
BN 3555 -> 4445,	START 14:34:20 -> 14:47:19	STOP,	12.98	
				1.07
BN 4447 -> 4754,	START 14:48:23 -> 14:51:46	STOP,	3.38	
Total Elapsed time 170.87 minutes		Total Time	57.07 minutes	113.80

Appendix C

Representative Force and Velocity Measurements

R Miner Dynamic Test

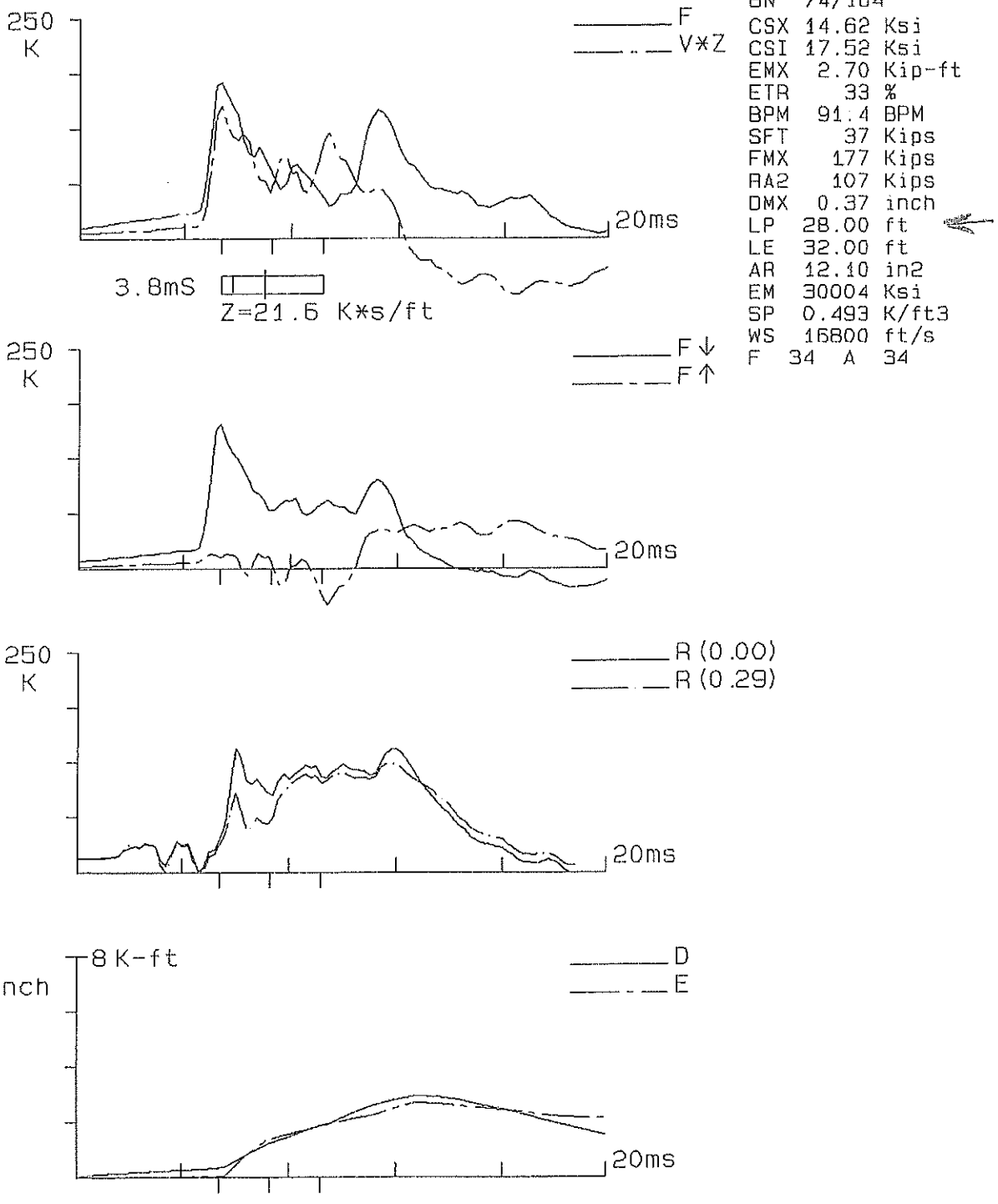
KCC, ROSE CK, BPT 04-01

PDA OP: RMDT---B. Miner

PDI PILE DRIVING ANALYZER® v5.03

02026FT 2004-08-18

HAV 180, 6-5/8" CLOSED



BN	74/164	
CSX	14.62	Ksi
CSI	17.52	Ksi
EMX	2.70	Kip-ft
ETR	33	%
BPM	91.4	BPM
SFT	37	Kips
FMX	177	Kips
RA2	107	Kips
DMX	0.37	inch
LP	28.00	ft
LE	32.00	ft
AR	12.10	in2
EM	30004	Ksi
SP	0.493	K/ft3
WS	16800	ft/s
F	34 A	34

R Miner Dynamic Test

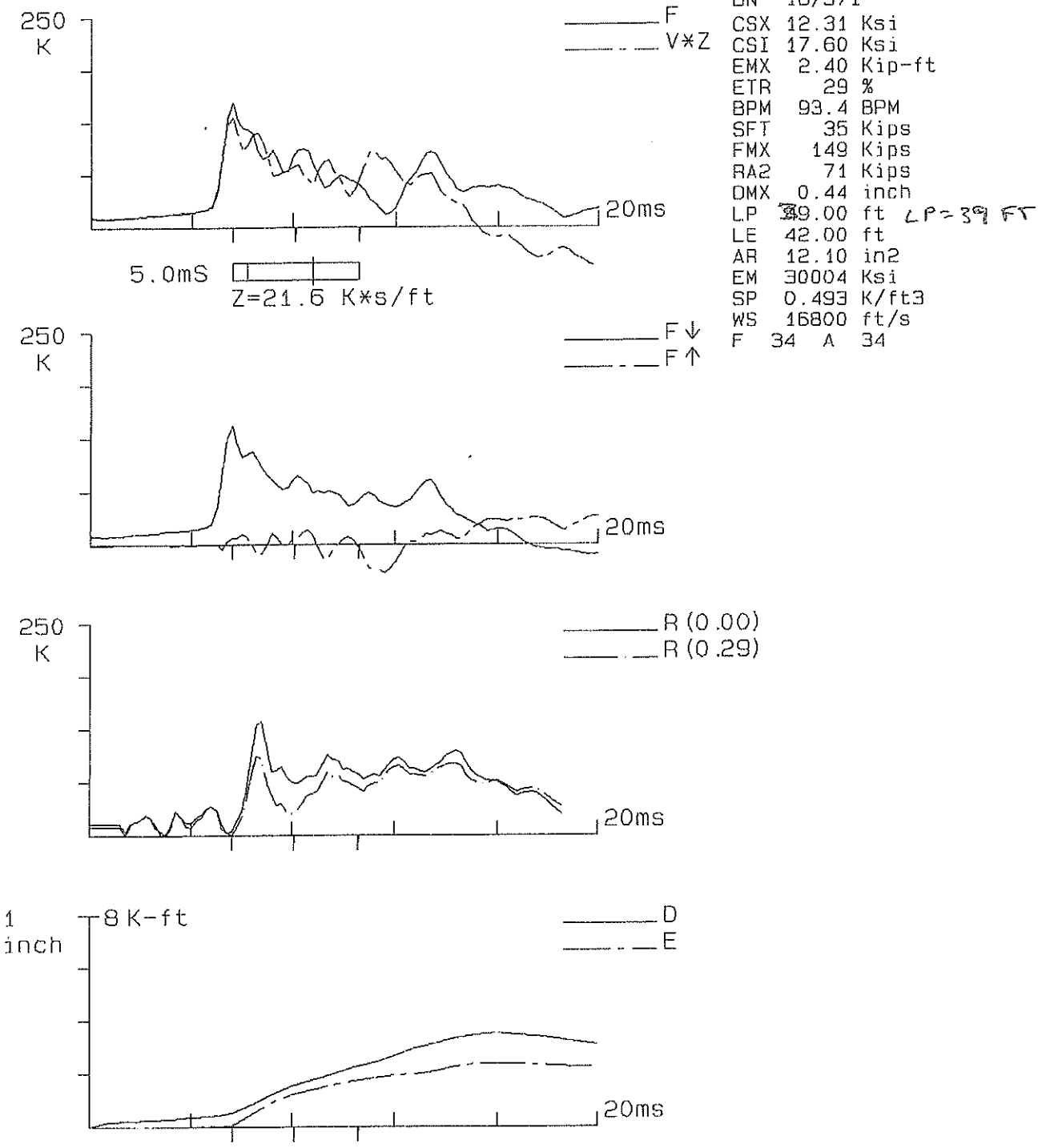
KCC, ROSE CK, BPT 04-01

PDA OP: RMDT--B. Miner

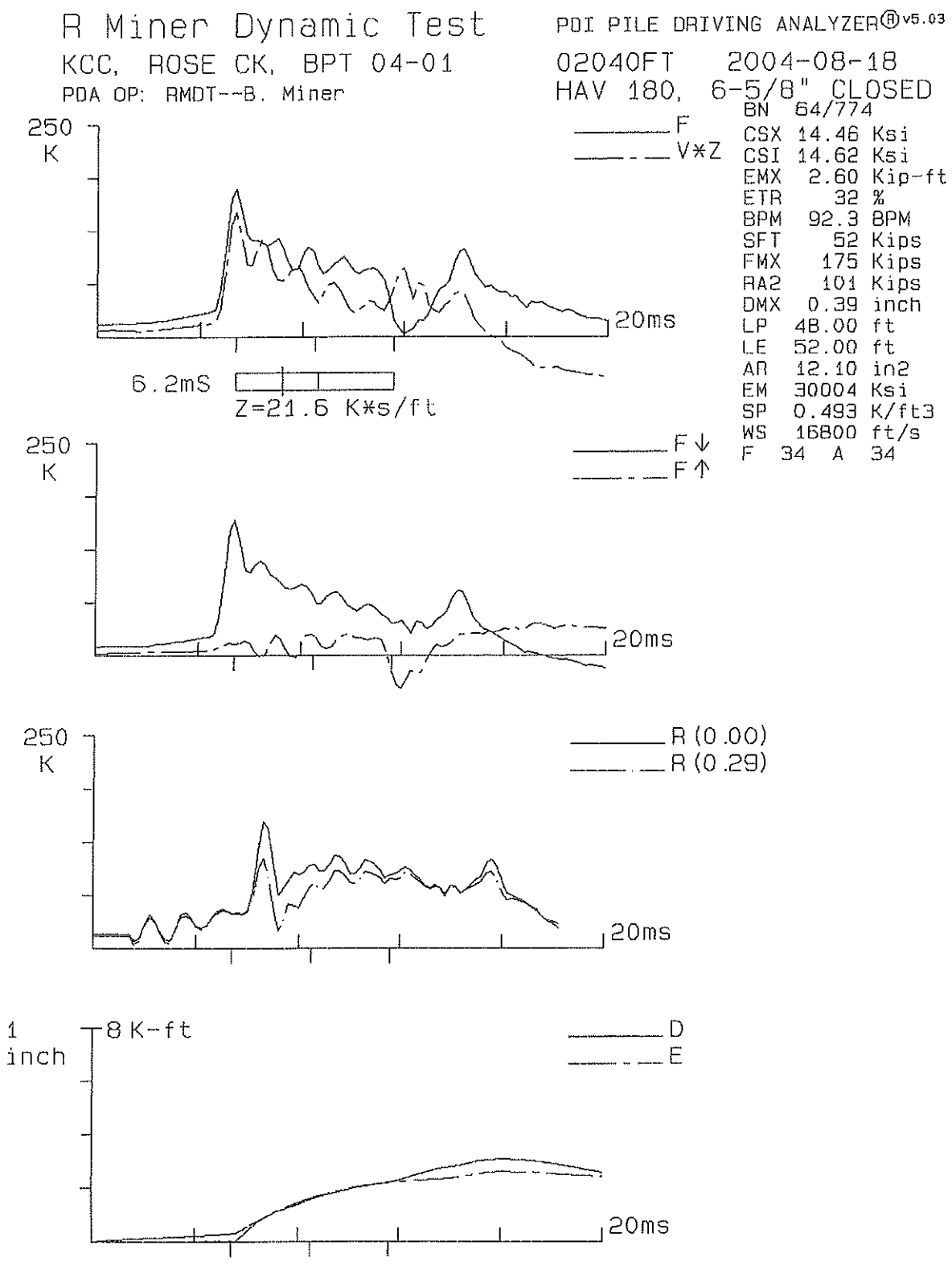
PDI PILE DRIVING ANALYZER®v5.03

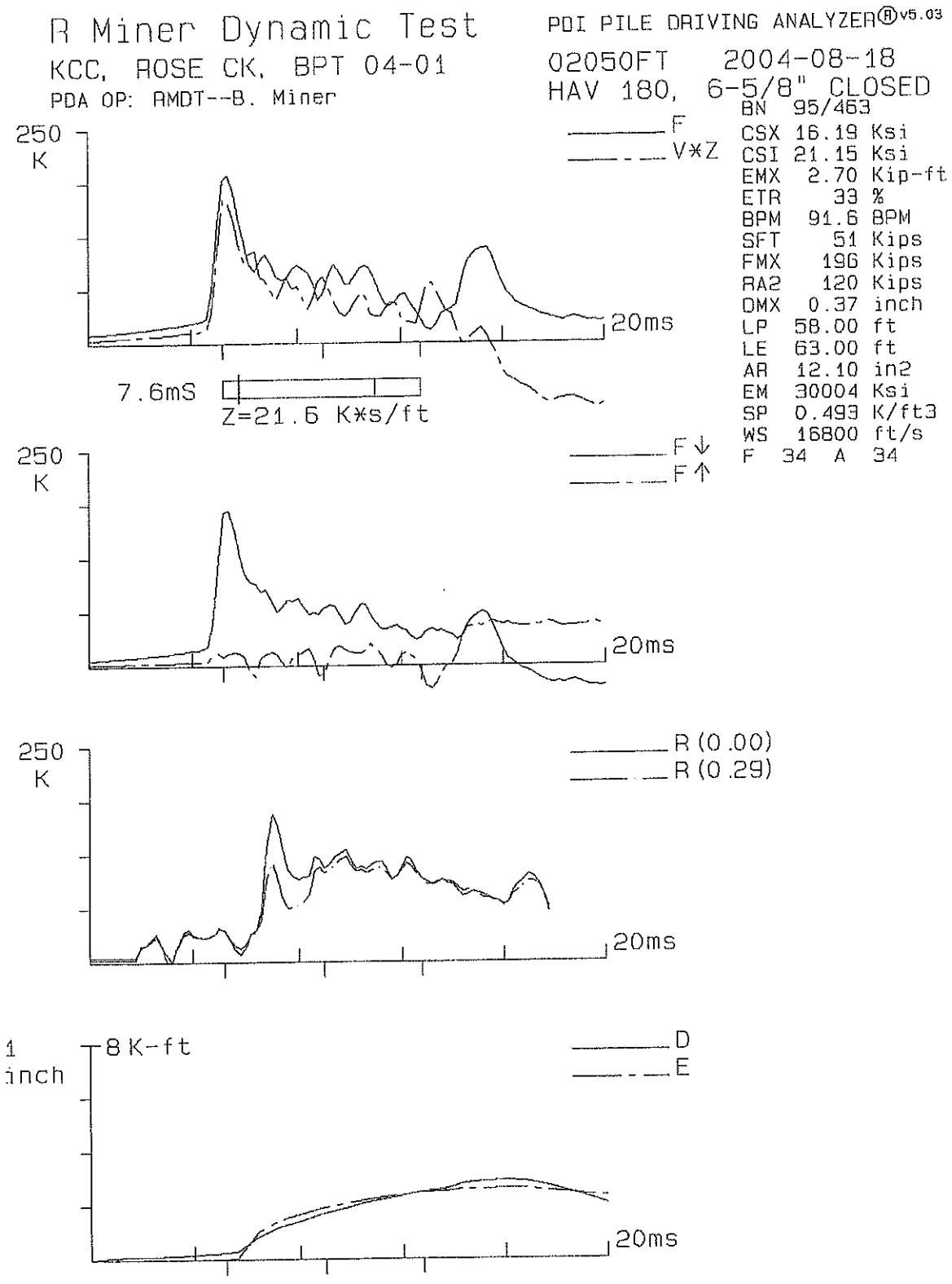
02029FT 2004-08-18

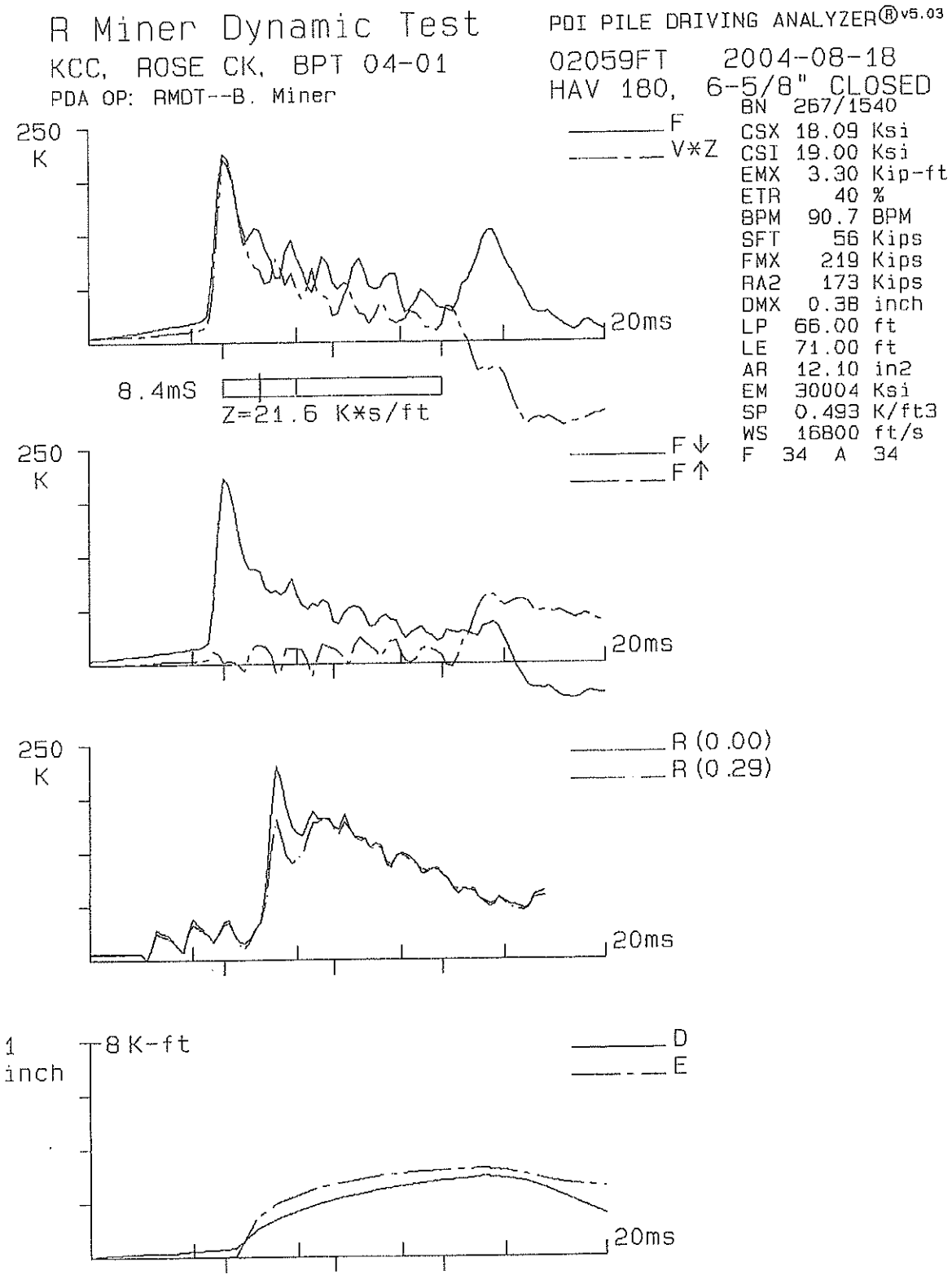
HAV 180, 6-5/8" CLOSED



BN 16/571
CSX 12.31 Ksi
CSI 17.60 Ksi
EMX 2.40 Kip-ft
ETR 29 %
BPM 93.4 BPM
SFT 35 Kips
FMX 149 Kips
RA2 71 Kips
DMX 0.44 inch
LP 39.00 ft LP=39 FT
LE 42.00 ft
AR 12.10 in2
EM 30004 Ksi
SP 0.493 K/ft3
WS 16800 ft/s
F 34 A 34


R Miner Dynamic Test
KCC, ROSE CK, BPT 04-01
PDA OP: RMDT--B. Miner
PDI PILE DRIVING ANALYZER® v5.03
02040FT 2004-08-18
HAV 180, 6-5/8" CLOSED
BN 64/774
CSX 14.46 Ksi
CSI 14.62 Ksi
EMX 2.60 Kip-ft
ETR 32 %
BPM 92.3 BPM
SFT 52 Kips
FMX 175 Kips
RA2 101 Kips
DMX 0.39 inch
LP 48.00 ft
LE 52.00 ft
AR 12.10 in2
EM 30004 Ksi
SP 0.493 K/ft3
WS 16800 ft/s
F 34 A 34
250 K
F
V*Z
20ms
6.2mS
Z=21.6 K*s/ft
250 K
F ↓
F ↑
20ms
250 K
R (0.00)
R (0.29)
20ms
1 inch
8 K-ft
D
E
20ms


R Miner Dynamic Test
KCC, ROSE CK, BPT 04-01
PDA OP: RMDT--R. Miner
PDI PILE DRIVING ANALYZER® v5.03
02050FT 2004-08-18
HAV 180, 6-5/8" CLOSED
BN 95/463
CSX 16.19 Ksi
CSI 21.15 Ksi
EMX 2.70 Kip-ft
ETR 33 %
BPM 91.6 BPM
SFT 51 Kips
FMX 196 Kips
RA2 120 Kips
DMX 0.37 inch
LP 58.00 ft
LE 63.00 ft
AR 12.10 in2
EM 30004 Ksi
SP 0.493 K/ft3
WS 16800 ft/s
F 34 A 34
F
V*Z
250
K
20ms
7.6mS
Z=21.6 K*s/ft
F↓
F↑
250
K
20ms
R (0.00)
R (0.29)
250
K
20ms
1
inch
8 K-ft
D
E
20ms


R Miner Dynamic Test
KCC, ROSE CK, BPT 04-01
PDA OP: RMDT--B. Miner
PDI PILE DRIVING ANALYZER® v5.03
02059FT 2004-08-18
HAV 180, 6-5/8" CLOSED
BN 267/1540
CSX 18.09 Ksi
CSI 19.00 Ksi
EMX 3.30 Kip-ft
ETR 40 %
BPM 90.7 BPM
SFT 56 Kips
FMX 219 Kips
RA2 173 Kips
DMX 0.38 inch
LP 66.00 ft
LE 71.00 ft
AR 12.10 in2
EM 30004 Ksi
SP 0.493 K/ft3
WS 16800 ft/s
F 34 A 34
250
K
F
V*Z
20ms
8.4mS
Z=21.6 K*s/ft
250
K
F↓
F↑
20ms
250
K
R (0.00)
R (0.29)
20ms
1
inch
8 K-ft
D
E
20ms

R Miner Dynamic Test

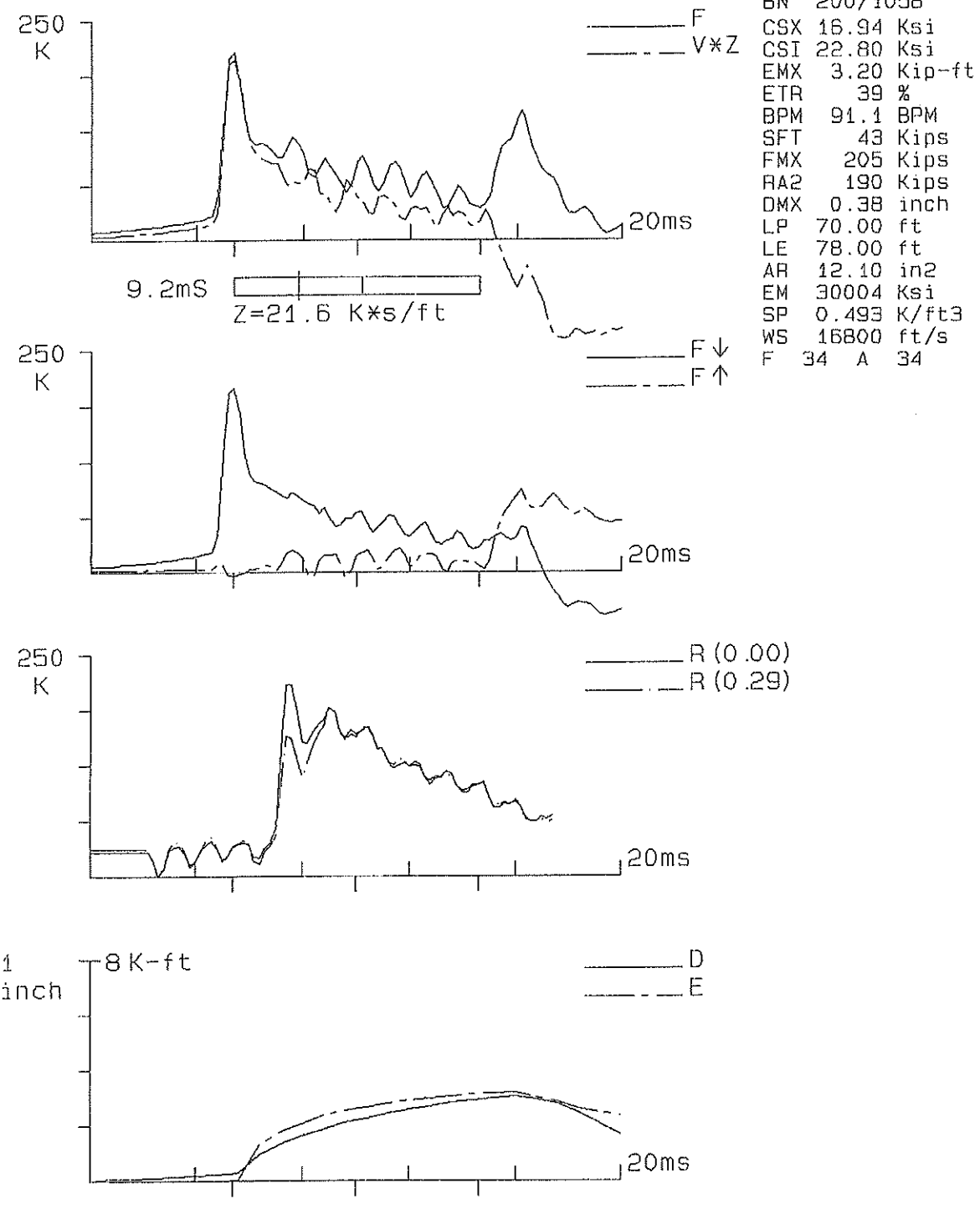
KCC, HOSE CK, BPT 04-01

PDA OP: RMDT--B. Miner

PDI PILE DRIVING ANALYZER® v5.03

02067FT 2004-08-18

HAV 180, 6-5/8" CLOSED



BN	200/1058	
CSX	16.94	Ksi
CSI	22.80	Ksi
EMX	3.20	Kip-ft
ETR	39	%
BPM	91.1	BPM
SFT	43	Kips
FMX	205	Kips
RA2	190	Kips
DMX	0.38	inch
LP	70.00	ft
LE	78.00	ft
AR	12.10	in2
EM	30004	Ksi
SP	0.493	K/ft3
WS	16800	ft/s
F	34 A	34

R Miner Dynamic Test

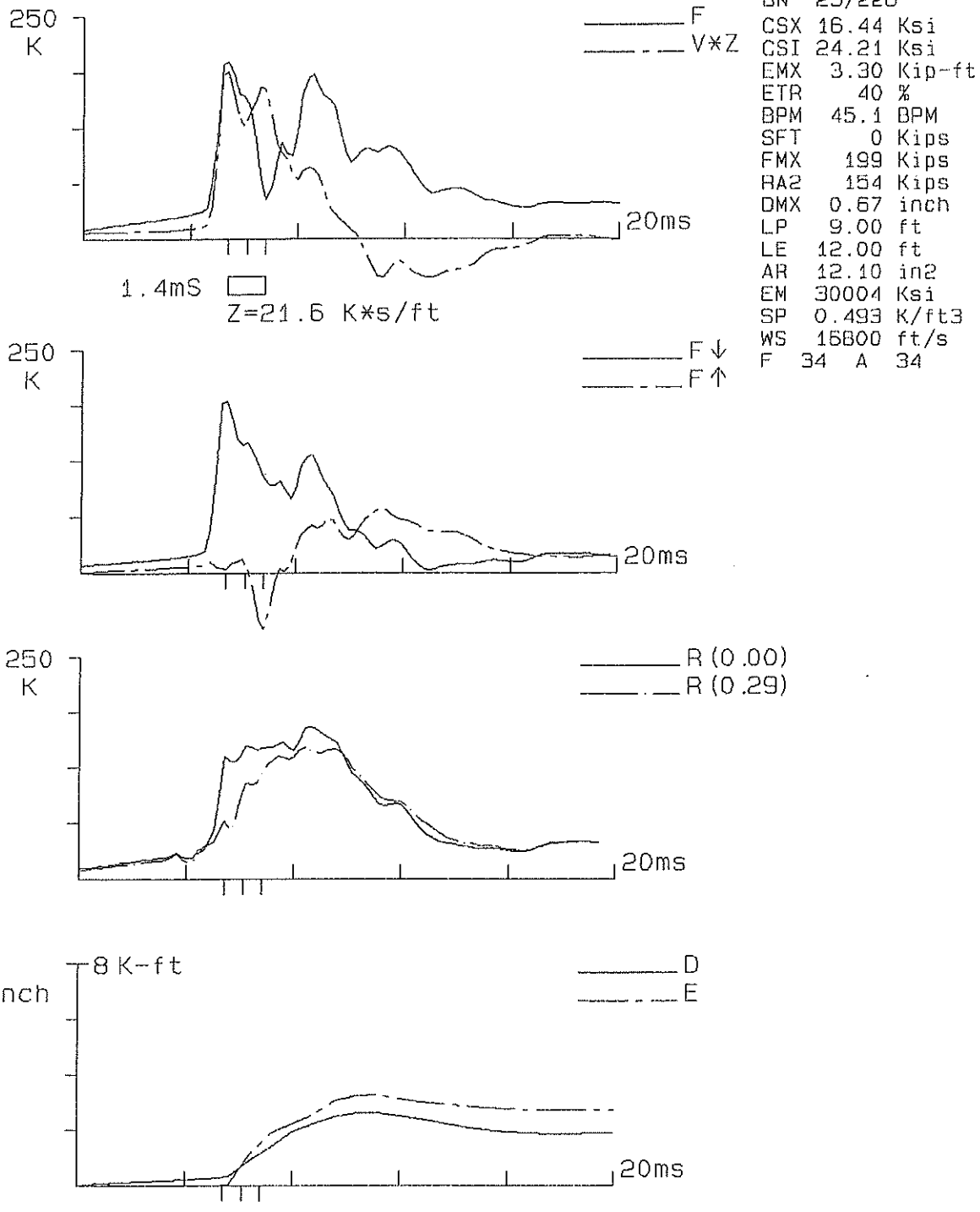
KCC, ROSE CK, BPT 04-01

PDA OP: RMDT--B. Miner

PDI PILE DRIVING ANALYZER® v5.03

01000FT 2004-08-17

HAV 180, 6-5/8" CLOSED



BN	25/226
CSX	16.44 Ksi
CSI	24.21 Ksi
EMX	3.30 Kip-ft
ETR	40 %
BPM	45.1 BPM
SFT	0 Kips
FMX	199 Kips
RA2	154 Kips
DMX	0.67 inch
LP	9.00 ft
LE	12.00 ft
AR	12.10 in2
EM	30004 Ksi
SP	0.493 K/ft3
WS	16800 ft/s
F 34	A 34

R Miner Dynamic Test

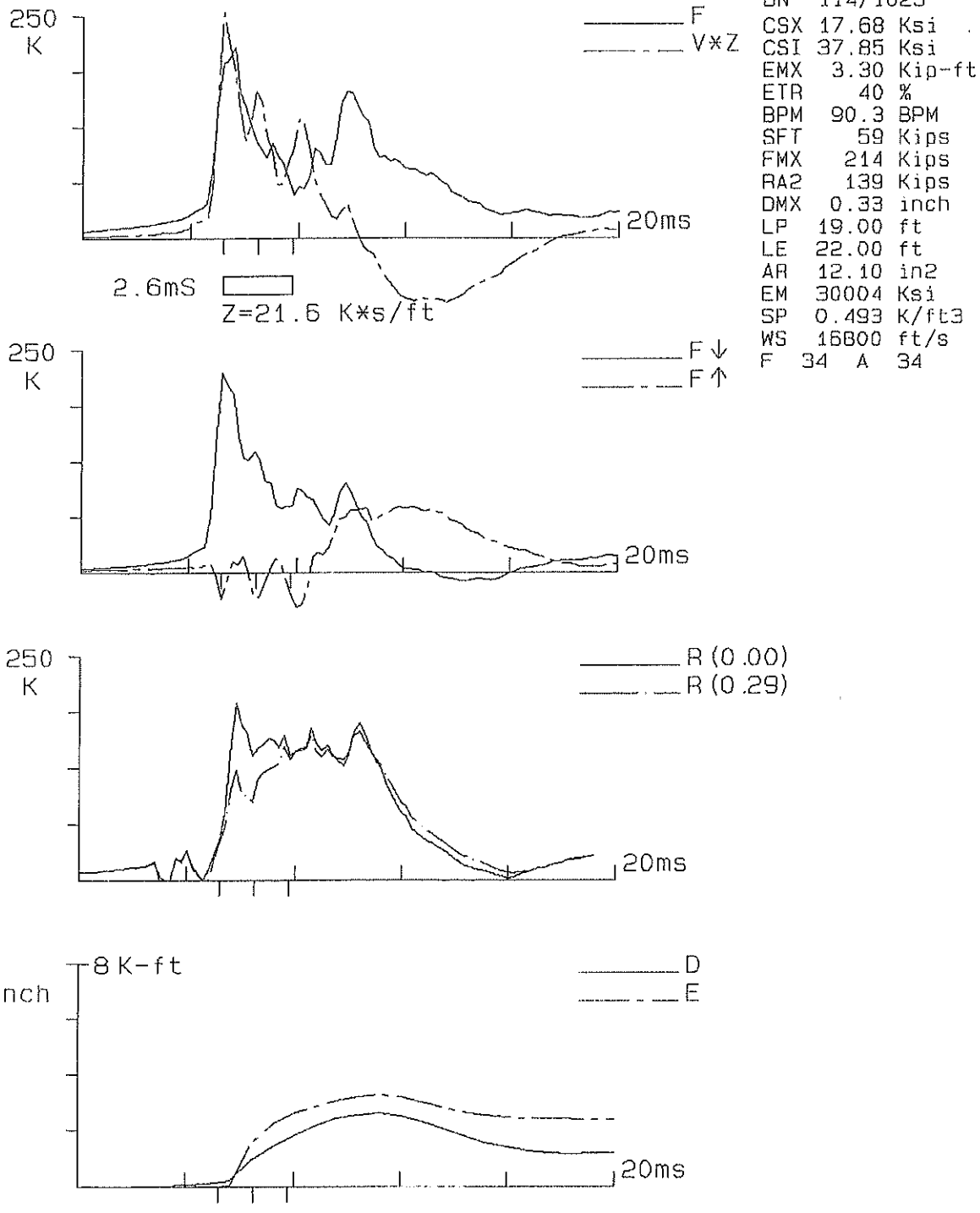
KCC, ROSE CK, BPT 04-01

PDA OP: RMDT--8 Miner

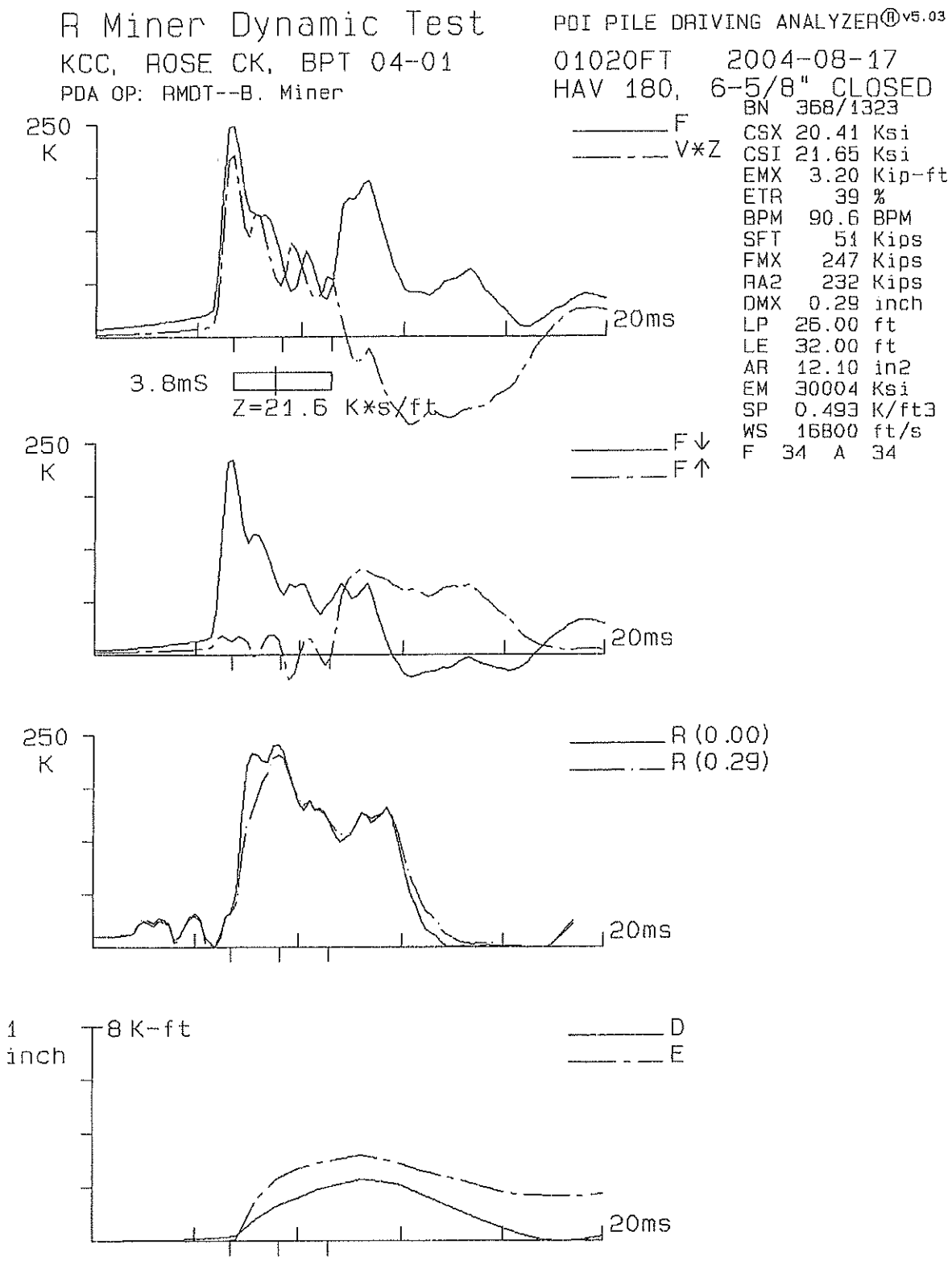
PDI PILE DRIVING ANALYZER® v5.03

01010FT 2004-08-17

HAV 180, 6-5/8" CLOSED



BN	114/1023	
CSX	17.68	Ksi
CSI	37.85	Ksi
EMX	3.30	Kip-ft
ETR	40	%
BPM	90.3	BPM
SFT	59	Kips
FMX	214	Kips
RA2	139	Kips
DMX	0.33	inch
LP	19.00	ft
LE	22.00	ft
AR	12.10	in2
EM	30004	Ksi
SP	0.493	K/ft3
WS	16800	ft/s
F	34 A	34


R Miner Dynamic Test
KCC, ROSE CK, BPT 04-01
PDA OP: RMDT--B. Miner
PDI PILE DRIVING ANALYZER® v5.03
01020FT 2004-08-17
HAV 180, 6-5/8" CLOSED
BN 368/1323
CSX 20.41 Ksi
CSI 21.65 Ksi
EMX 3.20 Kip-ft
ETR 39 %
BPM 90.6 BPM
SFT 51 Kips
FMX 247 Kips
RA2 232 Kips
DMX 0.29 inch
LP 25.00 ft
LE 32.00 ft
AR 12.10 in2
EM 30004 Ksi
SP 0.493 K/ft3
WS 16800 ft/s
F 34 A 34
250
K
F
V*Z
20ms
3.8mS
Z=21.6 K*s/ft
250
K
F↓
F↑
20ms
250
K
R (0.00)
R (0.29)
20ms
1
inch
8 K-ft
D
E
20ms

Appendix D

Results of CAPWAP Analyses

Robert Miner Dynamic Testing, Inc. 01-Oct-2004

KCC, ROSE CK, BPT 04-01, Project: 04F33
Pile: BPT 04-02 Blow: 49 41-42FT Data: HAV 180, 6-5/8" CLOSED
Collected: 04-08-18 Operator: RMDT:--R. Miner CAPWAP(R) Ver. 1997-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 50.0; along Shaft 20.0; at Toe 30.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit w. Respect to Depth kips/ft	Resist. Area kips/f2	Smith Damping Factor s/ft	Quake inch
				50.0					
1	17.3	6.3	.3	49.7	.3	.05	.03	.136	.100
2	24.3	13.3	.2	49.4	.6	.04	.02	.136	.100
3	31.2	20.2	.4	49.0	1.0	.06	.04	.136	.100
4	38.1	27.1	.4	48.5	1.4	.06	.04	.136	.100
5	45.1	34.1	8.5	40.1	9.9	1.22	.71	.136	.100
6	52.0	41.0	10.1	30.0	20.0	1.46	.84	.136	.100
Average Skin Values			3.3			.49	.28	.136	.100
	Toe		30.0				124.85	.039	.130

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		.126	.054
Unloading Quake	(% of loading quake)	50	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	65	
Soil Plug Weight	(kips)		.10

Robert Miner Dynamic Testing, Inc. 01-Oct-2004

KCC, ROSE CK, BPT 04-01, Project: 04F33
Pile: BPT 04-02 Blow: 49 41-42FT Data: HAV 180, 6-5/8" CLOSED
Collected: 04-08-18 Operator: RMDT:--R. Miner CAPWAP(R) Ver. 1997-1

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.5	142.9	-10.8	11.806	-.893	2.89	6.5	.597
2	6.9	142.9	-11.0	11.811	-.906	2.89	6.5	.598
3	10.4	142.9	-11.4	11.811	-.940	2.89	6.5	.598
5	17.3	131.6	-13.4	10.879	-1.109	2.88	7.0	.598
6	20.8	133.5	-12.1	11.030	-1.001	2.85	6.8	.597
8	27.7	140.1	-12.6	11.576	-1.043	2.83	6.3	.593
9	31.2	139.9	-13.1	11.565	-1.083	2.82	6.3	.591
11	38.1	129.0	-12.4	10.664	-1.028	2.78	6.6	.583
12	41.6	129.5	-11.6	10.704	-.960	2.73	6.5	.578
14	48.5	113.1	-6.0	9.350	-.496	2.16	7.3	.566
15	52.0	77.0	-5.8	6.365	-.482	1.42	8.5	.562
Absolute	6.9			11.811		(T=	20.6 ms)	
	17.3				-1.109	(T=	44.2 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	91.	77.	62.	47.	33.	18.	3.	0.	0.	0.
RMX	91.	81.	73.	66.	58.	56.	55.	54.	54.	53.
RSU	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

RAU 38. RA2 45.

Current CAPWAP Ru= 50.0; Corresponding J(Rs)= .28; J(Rx)=1.00

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
6.29	-.09	112.1	126.4	147.9	.589	.427	2.9	2.9	68.	67.

KCC, ROSE CK, BPT 04-01, Project: 04F33
Pile: BPT 04-02 Blow: 49 41-42FT Data: HAV 180, 6-5/8" CLOSED
Collected: 04-08-18 Operator: RMDT:--R. Miner CAPWAP(R) Ver. 1997-1

PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	12.10	30004.0	.493	1.730
52.00	12.10	30004.0	.493	1.730

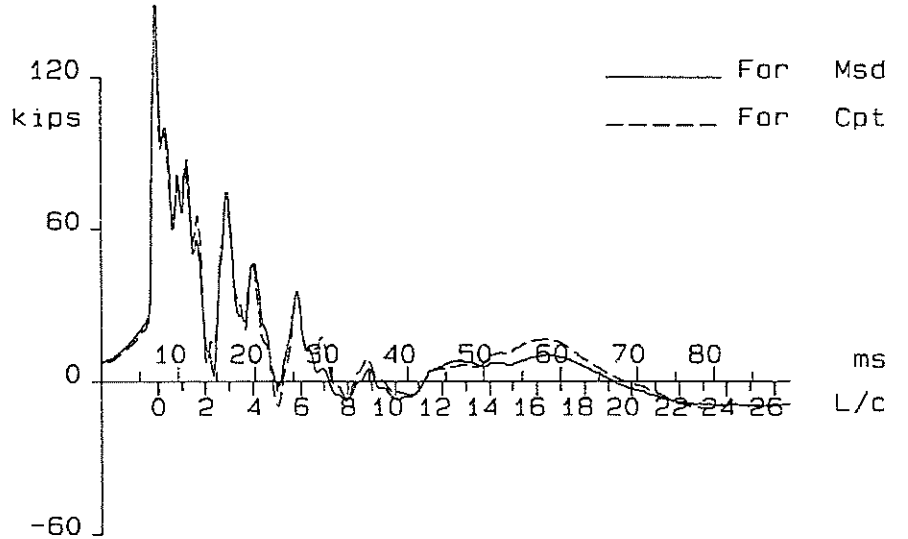
Toe Area .240 ft2

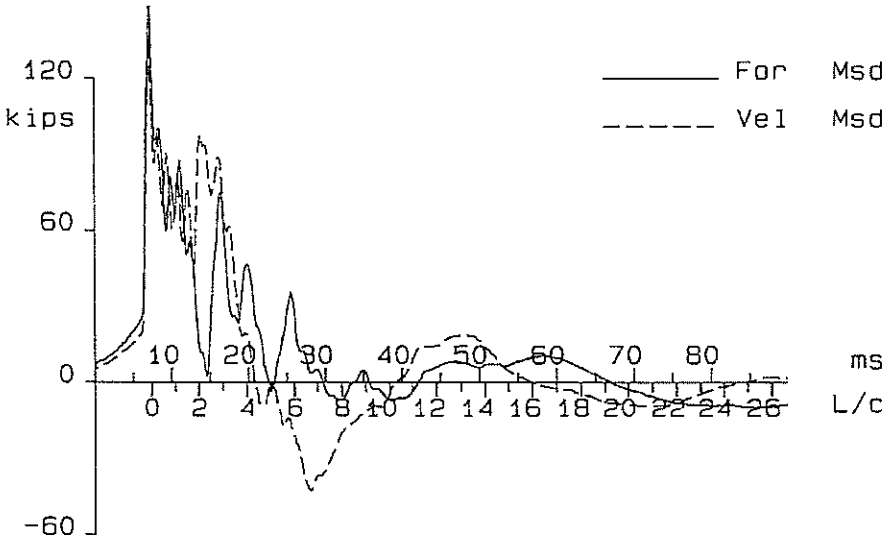
Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack inch	Tension Eff.	Compression Slack inch	Compression Eff.	Circ. ft
1	3.47	21.62	.00	.000	.000	.000	.000	1.730
7	24.27	15.00	-30.61	.000	.000	.000	.000	1.730
8	27.73	21.62	.00	.000	.000	.000	.000	1.730
13	45.07	15.00	-30.61	.000	.000	.000	.000	1.730
14	48.53	21.62	.00	.000	.000	.000	.000	1.730
15	52.00	21.62	.00	.000	.000	.000	.000	1.730

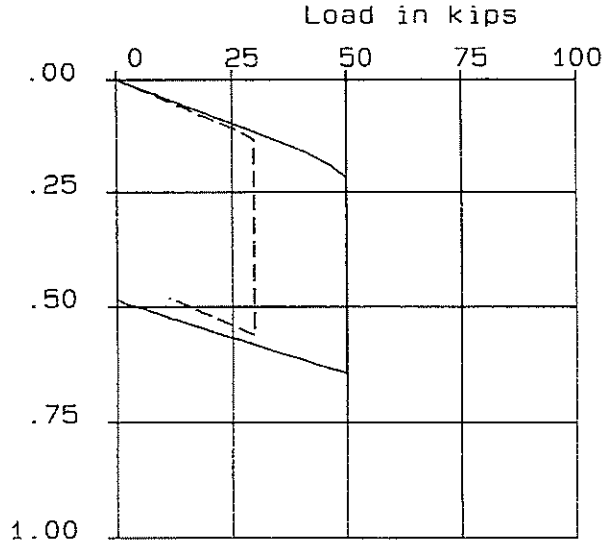
Pile Damping 4.0 %, Time Incr .206 ms, Wave Speed 16794.8 ft/s

KCC, ROSE CK, BPT 04-01, BPT 04-02, BN: 49 41-42FT 01-Oct-2004

Robert Miner Dynamic Testing, Inc. CAPWAP(R) Version 1997-1





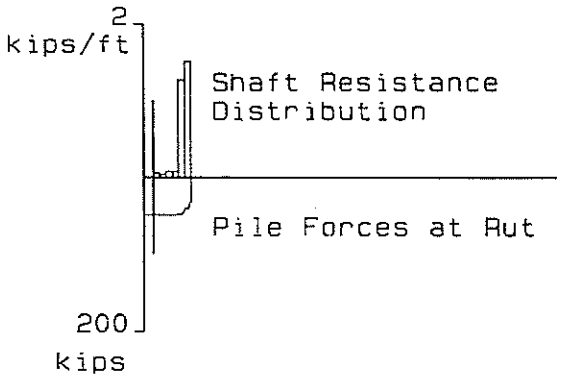


Pile Top

Bottom

Ru = 50.0 kips
Rs = 20.0 kips
Rb = 30.0 kips
Dy = .22 inch
Dmx = .65 inch





Robert Miner Dynamic Testing, Inc.

Robert Miner Dynamic Testing, Inc. 01-Oct-2004

KCC, ROSE CK, BPT 04-01, Project: 04F33
Pile: BPT 04-02 Blow: 99 51-52FT Data: HAV 180, 6-5/8" CLOSED
Collected: 04-08-18 Operator: RMDT:--R. Miner CAPWAP(R) Ver. 1997-1

CAPWAP FINAL RESULTS

Total CAPWAP Capacity: 74.0; along Shaft 32.0; at Toe 42.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile at Ru kips	Sum of Ru kips	Unit w. Respect to Depth kips/ft	Resist. Area kips/f2	Smith Damping Factor s/ft	Quake inch
				74.0					
1	16.6	5.6	.2	73.8	.2	.03	.02	.109	.100
2	23.2	12.2	.1	73.7	.3	.01	.00	.109	.100
3	29.8	18.8	.5	73.2	.8	.08	.04	.109	.100
4	36.5	25.5	.1	73.2	.8	.01	.00	.109	.100
5	43.1	32.1	4.2	69.0	5.0	.63	.36	.109	.100
6	49.7	38.7	6.2	62.8	11.2	.93	.54	.109	.100
7	56.4	45.4	10.4	52.4	21.6	1.57	.91	.109	.100
8	63.0	52.0	10.4	42.0	32.0	1.57	.91	.109	.100
Average Skin Values			4.0			.62	.35	.109	.100
	Toe		42.0				174.92	.029	.120

Soil Model Parameters/Extensions		Skin	Toe
Case Damping Factor		.162	.056
Unloading Quake	(% of loading quake)	20	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	89	
Soil Plug Weight	(kips)		.08

Robert Miner Dynamic Testing, Inc. 01-Oct-2004

KCC, ROSE CK, BPT 04-01, Project: 04F33
Pile: BPT 04-02 Blow: 99 51-52FT Data: HAV 180, 6-5/8" CLOSED
Collected: 04-08-18 Operator: RMDT:--R. Miner CAPWAP(R) Ver. 1997-1

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress kips/in2	max. Tension Stress kips/in2	max. Trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	188.2	-21.3	15.553	-1.758	3.16	8.5	.509
2	6.6	188.3	-21.8	15.560	-1.799	3.15	8.5	.504
3	9.9	188.3	-14.1	15.563	-1.166	3.13	8.5	.500
5	16.6	179.9	-11.5	14.871	-.953	3.10	8.8	.488
7	23.2	186.7	-11.7	15.431	-.970	3.04	8.4	.473
9	29.8	175.3	-13.0	14.489	-1.074	2.98	8.8	.457
11	36.5	184.6	-15.1	15.259	-1.249	2.89	8.0	.437
13	43.1	186.3	-18.8	15.397	-1.551	2.83	7.9	.421
15	49.7	182.2	-14.1	15.059	-1.164	2.62	7.6	.414
17	56.4	176.3	-10.6	14.567	-.875	2.33	7.8	.409
18	59.7	140.8	.0	11.640	.000	1.87	9.4	.405
19	63.0	89.8	.0	7.418	.000	1.36	11.0	.400
Absolute	9.9			15.563		(T=	20.9 ms)	
	6.6				-1.799	(T=	27.8 ms)	

CASE METHOD

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
RS1	110.	88.	67.	46.	24.	3.	0.	0.	0.	0.
RMX	114.	107.	99.	92.	84.	77.	70.	69.	68.	66.
RSU	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

RAU 45. RA2 75.

Current CAPWAP Ru= 74.0; Corresponding J(Rs)= .17; J(Rx)= .54

VMX	VFN	VT1*Z	FT1	FMX	DMX	DFN	EMX	EFN	RLT	REN
8.24	-.05	152.5	171.2	193.6	.507	.387	3.1	3.0	93.	94.

Robert Miner Dynamic Testing, Inc. 01-Oct-2004

KCC, ROSE CK, BPT 04-01, Project: 04F33
Pile: BPT 04-02 Blow: 99 51-52FT Data: HAV 180, 6-5/8" CLOSED
Collected: 04-08-18 Operator: RMDT:--R. Miner CAPWAP(R) Ver. 1997-1

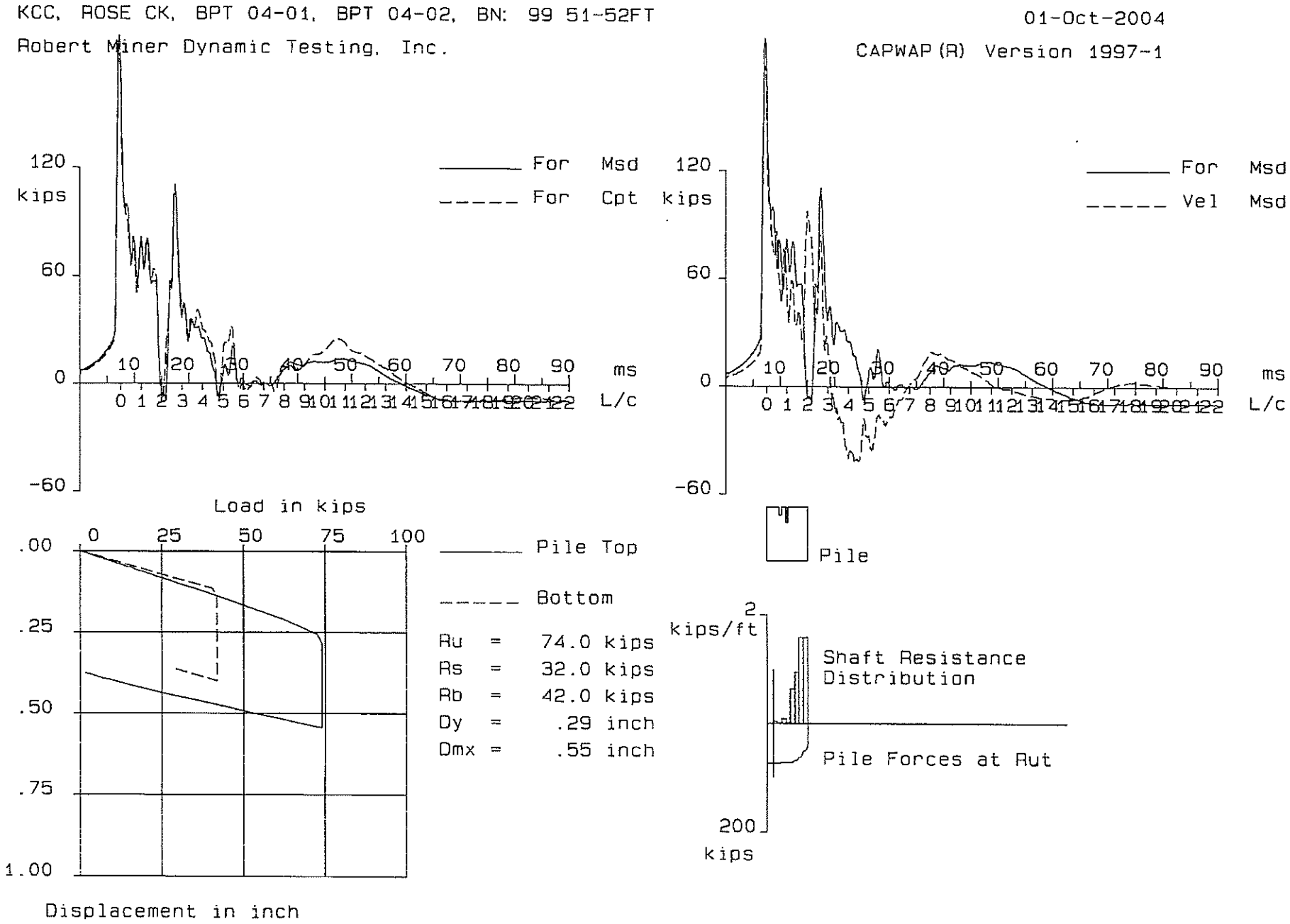
PILE PROFILE AND PILE MODEL

Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3	Circumf. ft
.00	12.10	30004.0	.493	1.730
63.00	12.10	30004.0	.493	1.730

Toe Area .240 ft2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack inch	Tension Eff.	Compression Slack inch	Compression Eff.	Circ. ft
1	3.32	21.62	.00	.000	.000	.000	.000	1.730
7	23.21	18.00	-16.73	.000	.000	.000	.000	1.730
8	26.53	21.62	.02	.000	.000	.000	.000	1.730
9	29.84	21.62	.00	.000	.000	.000	.000	1.730
10	33.16	15.00	-30.61	.000	.000	.000	.000	1.730
11	36.47	21.62	.00	.000	.000	.000	.000	1.730
13	43.11	21.62	.02	.000	.000	.000	.000	1.730
14	46.42	21.62	.00	.000	.000	.000	.000	1.730
19	63.00	21.62	.00	.000	.000	.000	.000	1.730

Pile Damping 3.0 %, Time Incr .197 ms, Wave Speed 16794.8 ft/s



Robert Miner Dynamic Testing, Inc.

APPENDIX V

BPT Data Interpretation

APPENDIX V

TABLE OF CONTENTS

PAGE

V-1. INTRODUCTION 1
V-2. HARDER APPROACH 1
V-3. SY APPROACH 2
V-4. BPT-SPT INTERPRETATION RESULTS 3

FIGURES

Figure V-1 Harder BPT Interpretation Procedures
Figure V-2 SY BPT-SPT Correlations
Figure V-3 Estimation of ENTHRU based on PDA data from BPT04-01 and BPT04-02
Figure V-4 Summary of Measured Casing Friction
Figure V-5 BPT Data Interpretation – BPT02-01
Figure V-6 BPT Data Interpretation – BPT02-02
Figure V-7 BPT Data Interpretation – BPT02-03
Figure V-8 BPT Data Interpretation – BPT02-04
Figure V-9 BPT Data Interpretation – BPT02-05
Figure V-10 BPT Data Interpretation – BPT02-08A
Figure V-11 BPT Data Interpretation – BPT02-08B
Figure V-12 BPT Data Interpretation – BPT02-09A
Figure V-13 BPT Data Interpretation – BPT02-09B
Figure V-14 BPT Data Interpretation – BPT02-10
Figure V-15 BPT Data Interpretation – BPT02-11
Figure V-16 BPT Data Interpretation – BPT02-12
Figure V-17 BPT Data Interpretation – BPT02-13
Figure V-18 BPT Data Interpretation – BPT02-14

APPENDIX V
BPT DATA INTERPRETATION

V-1. INTRODUCTION

The field observations of the performance of sites during earthquakes have formed the basis for the widely used Seed's simplified method of liquefaction potential assessment for level ground (Youd et al. 2001). The Seed procedure uses Standard penetration test (SPT) N_{60}-value as the soil density index and is applicable to sand and silty sand sites for which the performance database was compiled. For assessing soil liquefaction at gravelly sites, the current practice is to use the Becker penetration test (BPT) to determine the equivalent SPT N_{60}-values for the gravelly soils, and then evaluate the liquefaction potential using the Seed SPT-based procedure for sand and silty sand sites. Two procedures were used to convert the Becker blowcounts to equivalent SPT N_{60}-values; namely, the empirical approach proposed by Harder (1988) and the wave equation based approach proposed by Sy (1993).

V-2. HARDER APPROACH

Harder (1988) proposed an empirical BPT-SPT correlation procedure based on standardizing the BPT blowcounts to a constant combustion condition by measuring peak pressures in the bounce chamber of the double-acting diesel hammer used in the BPT. The Harder procedure involves two basic steps. First, the field measured BPT blowcount (N_b) is corrected to a reference full combustion condition using the correction chart as shown on Figure V-1a, and secondly, the bounce chamber corrected BPT blowcount (N_{bc}) is used to estimate equivalent SPT N_{60} value using the correlation curve shown on Figure V-1a. In practice, the Harder procedure has several limitations:

- The Harder reference combustion line A-A, although developed for a turbo-charged AP 1000 drill rig, is specific for the particular drill rig-

hammer system used and for the ground conditions encountered in the Harder (1988) correlation study, and is, therefore, not unique. Sy and Campanella (1993) and Sy (1997) illustrated that different reference combustion lines could be derived from the same drill rig at different sites or using different casing sizes at the same site, or from using different drilling rigs.

- The BPT-SPT correlation shown on Figure V-1b was based on test data from three sand and silty sand sites to 15 m depth. Since the Harder approach does not explicitly consider casing friction, it may not be applicable directly to depths greater than 15 m or to gravelly soil sites.

Despite the above limitations, the Harder procedure has been used, mainly in the United States, for the liquefaction assessment of gravelly soils.

V-3. SY APPROACH

An alternative and more fundamental approach to BPT-SPT correlation, based on experimental and numerical studies of the SPT and BPT, was proposed by Sy (1993). In this approach, a Pile Driving Analyzer (PDA) is used during the BPT to determine the energy transferred into the top of the casing, similar to dynamic monitoring of pile driving (ASTM D4945-89). The PDA records force and velocity time histories for each hammer blow and calculates the transferred energy for each blow in real time. The transferred energy is then used to correct the measured field blowcount to a reference energy of 30% of the manufacturer's rated energy for the ICE 180 hammer used in the Becker system:

$$N_{b30} = N_b \frac{ENTHRU}{30}$$

where N_{b30} is the BPT blowcount normalized to the 30% reference energy level, N_b is the measured Becker blowcount, and ENTHRU is the measured transferred energy expressed as percent of the rated hammer energy of 11.0 kJ for the ICE 180 hammer.

The Sy approach further considers soil friction acting on the Becker casing during driving. The soil friction and its distribution along the casing can be measured during the BPT by casing pull-up tests, or estimated from the PDA stress wave measurement using a signal-matching wave equation analysis program, CAPWAP. The energy-corrected N_{b30}, together with the measured casing friction or shaft resistance value, R_s, is then used to estimate the equivalent SPT N_{60} value from the theoretical BPT-SPT correlation shown on Figure V-2

The pull-up test consists of pulling the top of the Becker casing during each rod change or casing add-on, and measuring the pull-up force with a tension load cell connected to the top of the casing. Sy (1997) found that this pull-up tension force, although fundamentally different from the compressive soil friction acting on the casing during driving, is comparable to the CAPWAP-estimated casing friction.

V-4. BPT-SPT INTERPRETATION RESULTS

The Harder and Sy methods were applied to all BPTs. For the Harder method, the measured peak bounce chamber pressures (BCP) were used in the BPT data interpretation. For the Sy method, the measured transferred energies from the PDA, together with the load cell measured casing friction from pull-up tests, were used in the BPT data interpretation. Since PDA measurements were performed only in BPT04-01 and BPT04-02, the transferred energy required to interpret the other BPTs following the Sy procedure was estimated based on the correlations of measured energy data and the measured peak bounce chamber pressure at BPT04-01 and BPT04-02.

Figure V-3 shows the variation of BPT blow counts with the bounce chamber pressure and the transferred energy for BPT04-01 and BPT04-02. At these two BPT holes, the measured transferred energies varied between about 28% and 41%. Based on the energy

measurements, constant transferred energy contour lines for 30%, 35% and 40% were drawn on the plot of BPT blow counts versus bounce chamber pressure as shown in Figure V-3. These site-specific transferred energy-bounce chamber pressure correlations were subsequently used to estimate the transferred energy of the other BPTs. The estimated transferred energy values are listed in Table 3.2 in the main text. They varied between were 20% and 30%.

Figure V-4 shows the measured casing friction profiles at all BPT holes. These measured profiles were used in the Sy method. The measured friction values are relatively low, and lower than the average friction for the gravel sites as compiled in Sy (1997).

The BPT-SPT interpretation results are presented in Figures V-5 to V-18 for the fourteen BPTs. The figures show the recorded BPT blow counts (N_b); the measured bounce chamber pressures (BCP); the measured or estimated transferred energies (ENTHRU); the bounce chamber pressure corrected blow counts (N_{bc}); the transferred energy corrected blow counts (N_{b30}); the casing friction values from pull-up tests, and the equivalent SPT N_{60}-values from both Sy and Harder methods.

Shaft resistances were computed from CAPWAP analyses for two hammer blows at BPT04-02. These CAPWAP results are also shown in Figure V-6. The shaft resistances estimated from the CAPWAP analyses are close to those measured from the pull-up tests.

$(N_1)_{60}$ values from three SPTs conducted at the sampling hole BKS04-14 adjacent to the BPT04-14 are also shown in Figure V-18. $(N_1)_{60}$ values from SPTs at borehole BH80-46 are shown in Figure V-14.

As shown in Figure V-14 and V-18, the available $(N_1)_{60}$ values are generally close to the equivalent $(N_1)_{60}$ values estimated by Sy method than those by Harder method.

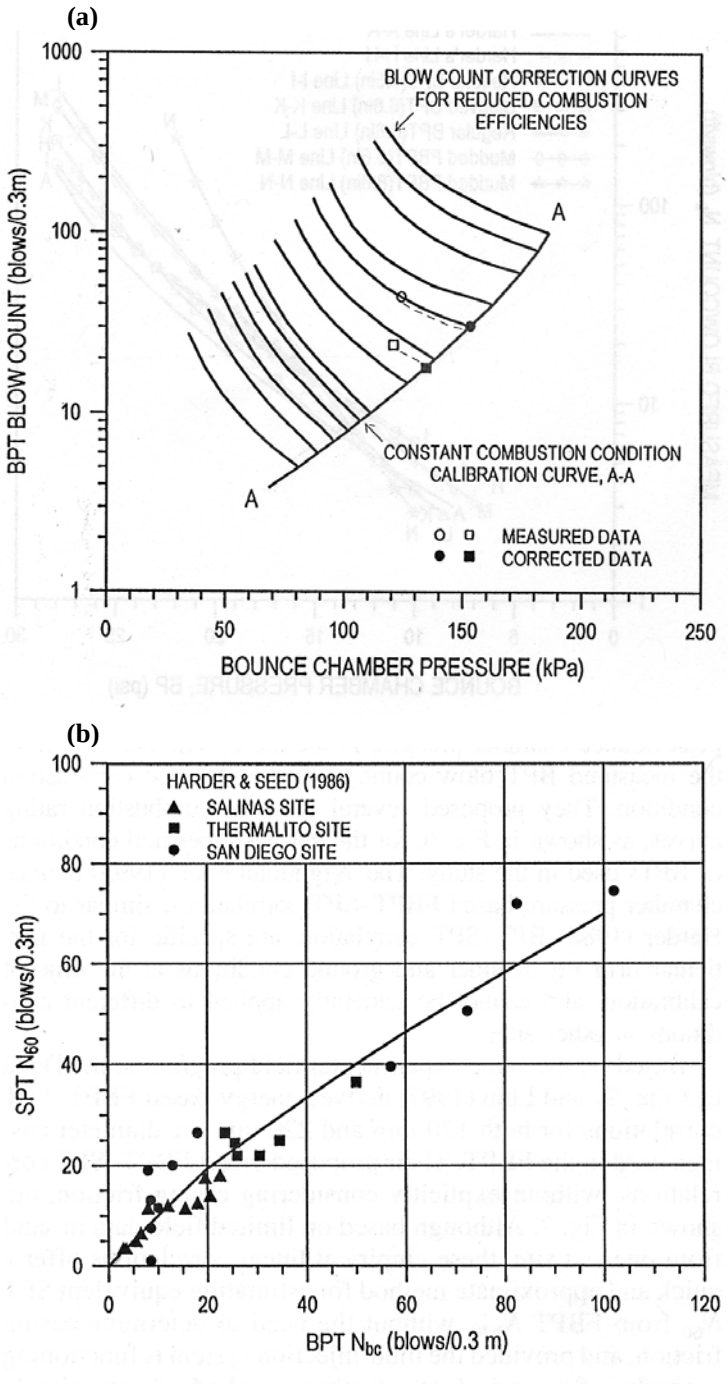


Figure V-1 Harder's BPT Interpretation Procedure

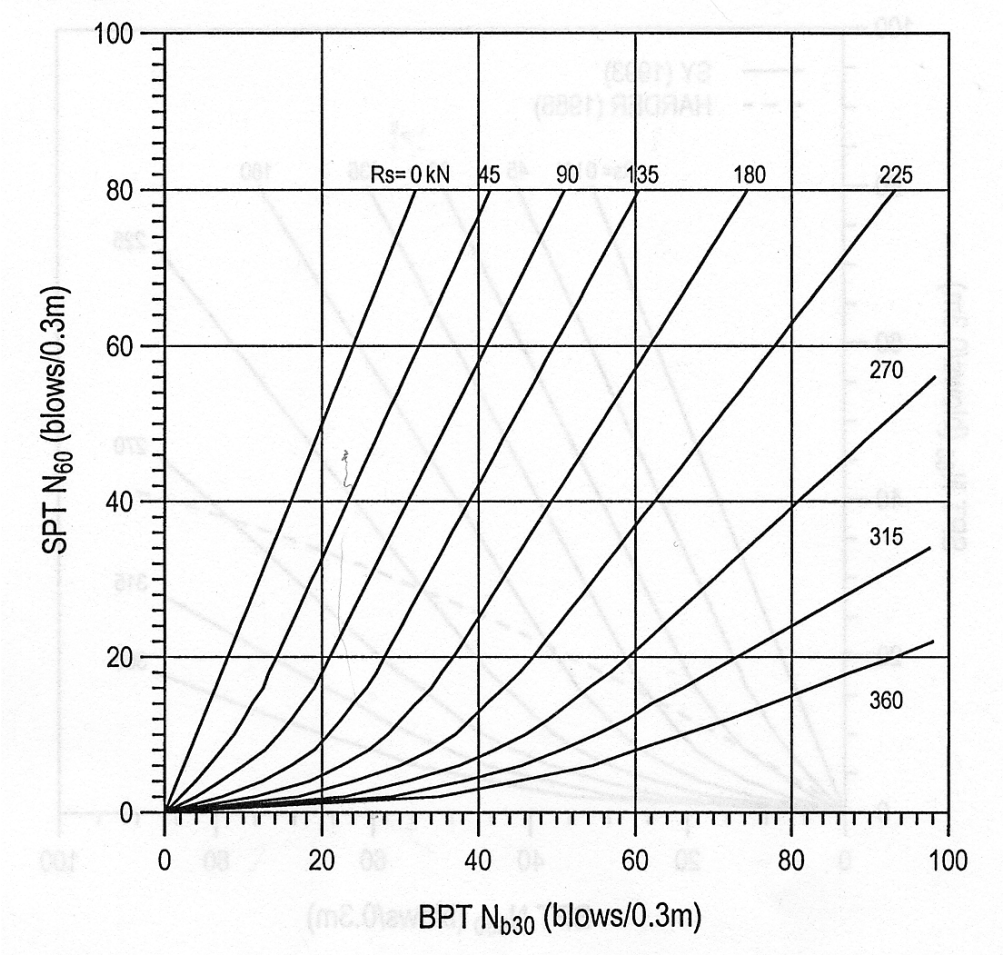


Figure V-2 Sy's BPT-SPT Correlation

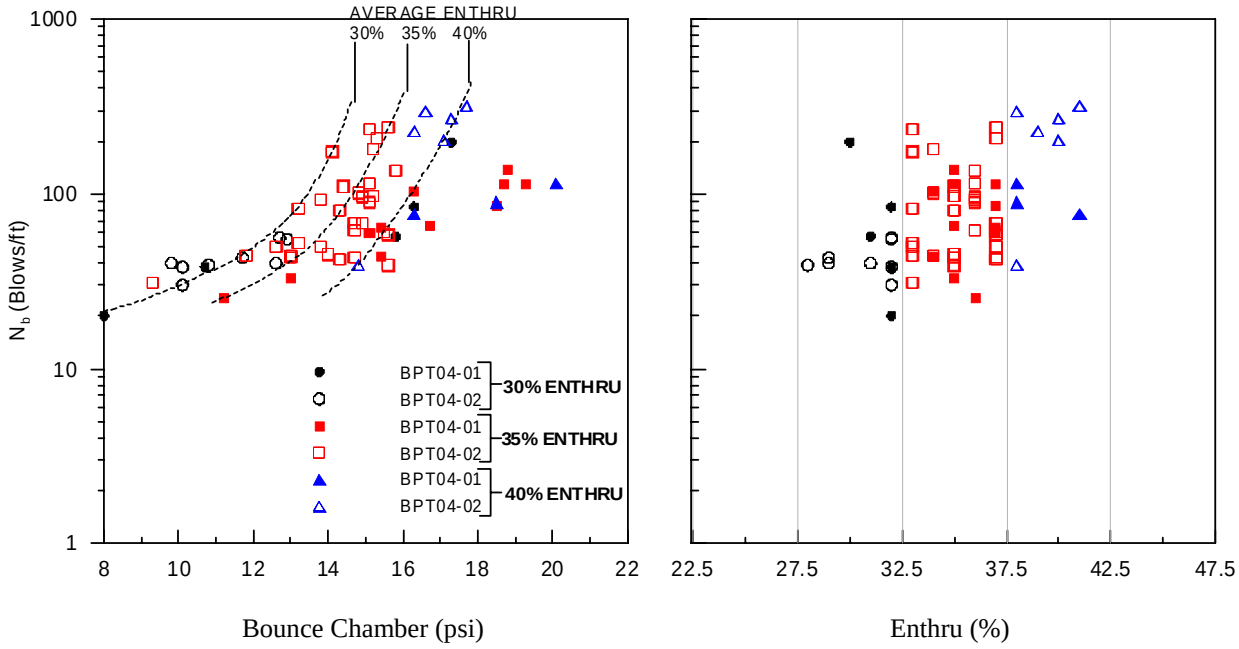


Figure V-3 Estimation of ENTHRU based on PDA data from BPT04-01 and BPT04-02

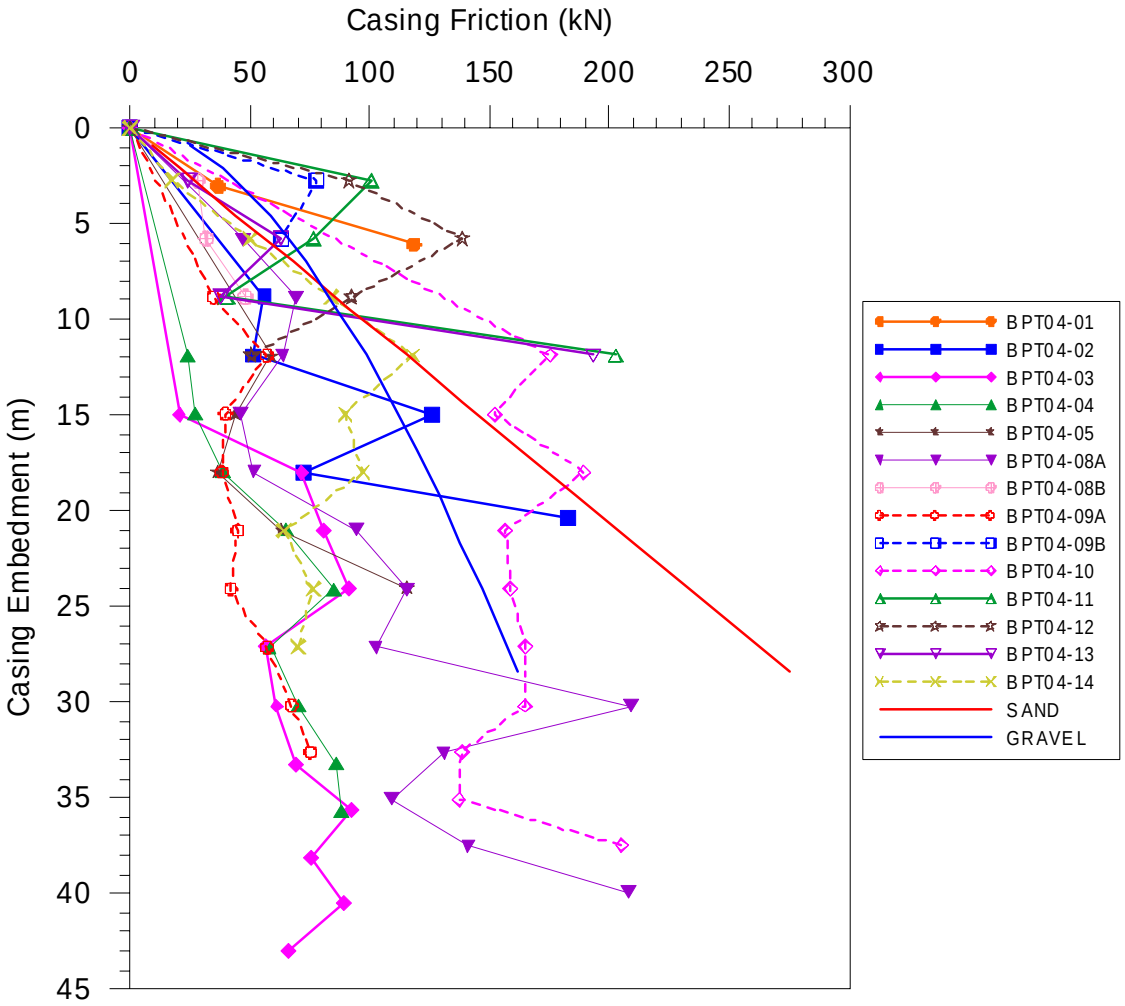
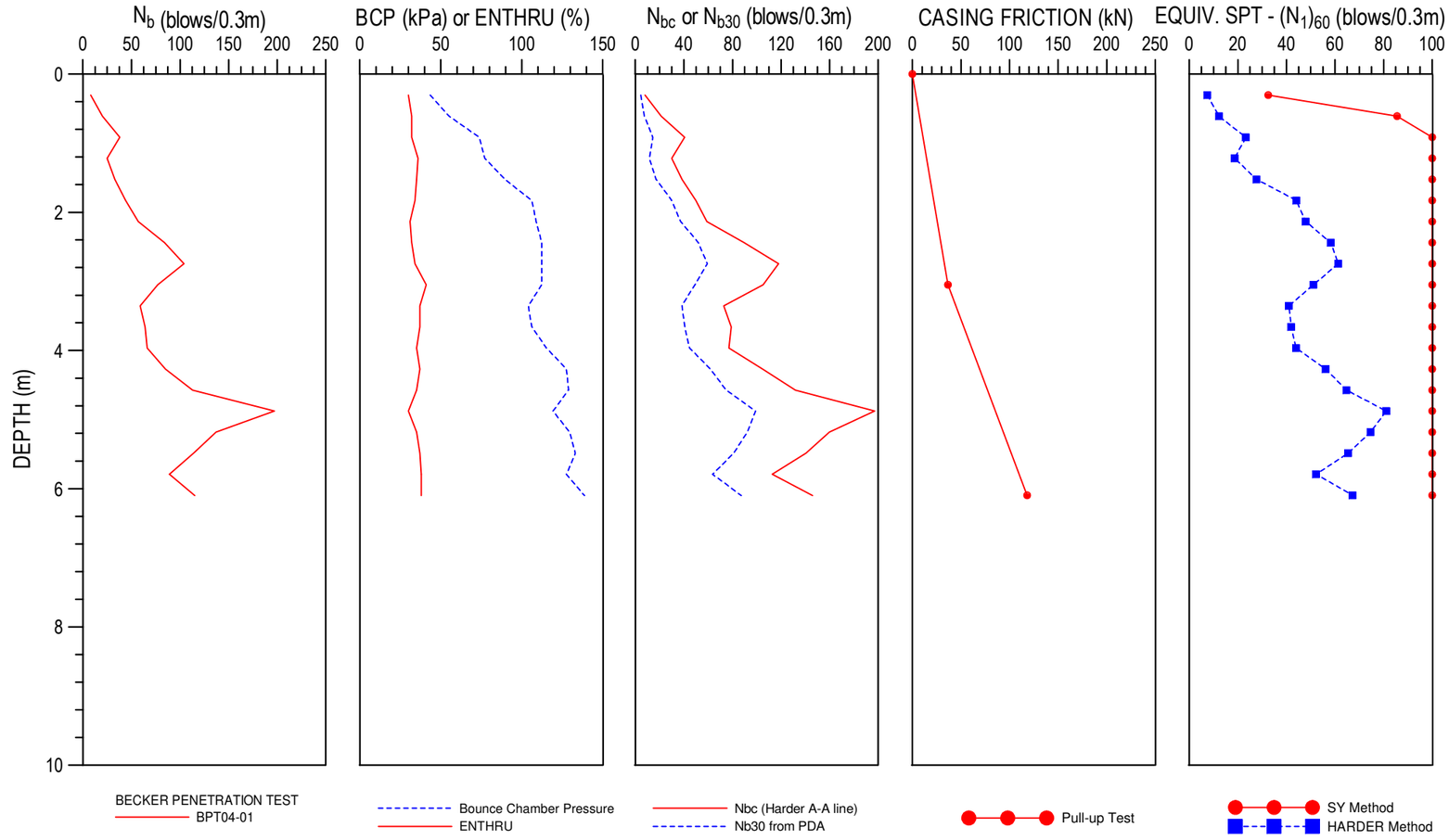


Figure V-4 Summary of Measured Casing Friction



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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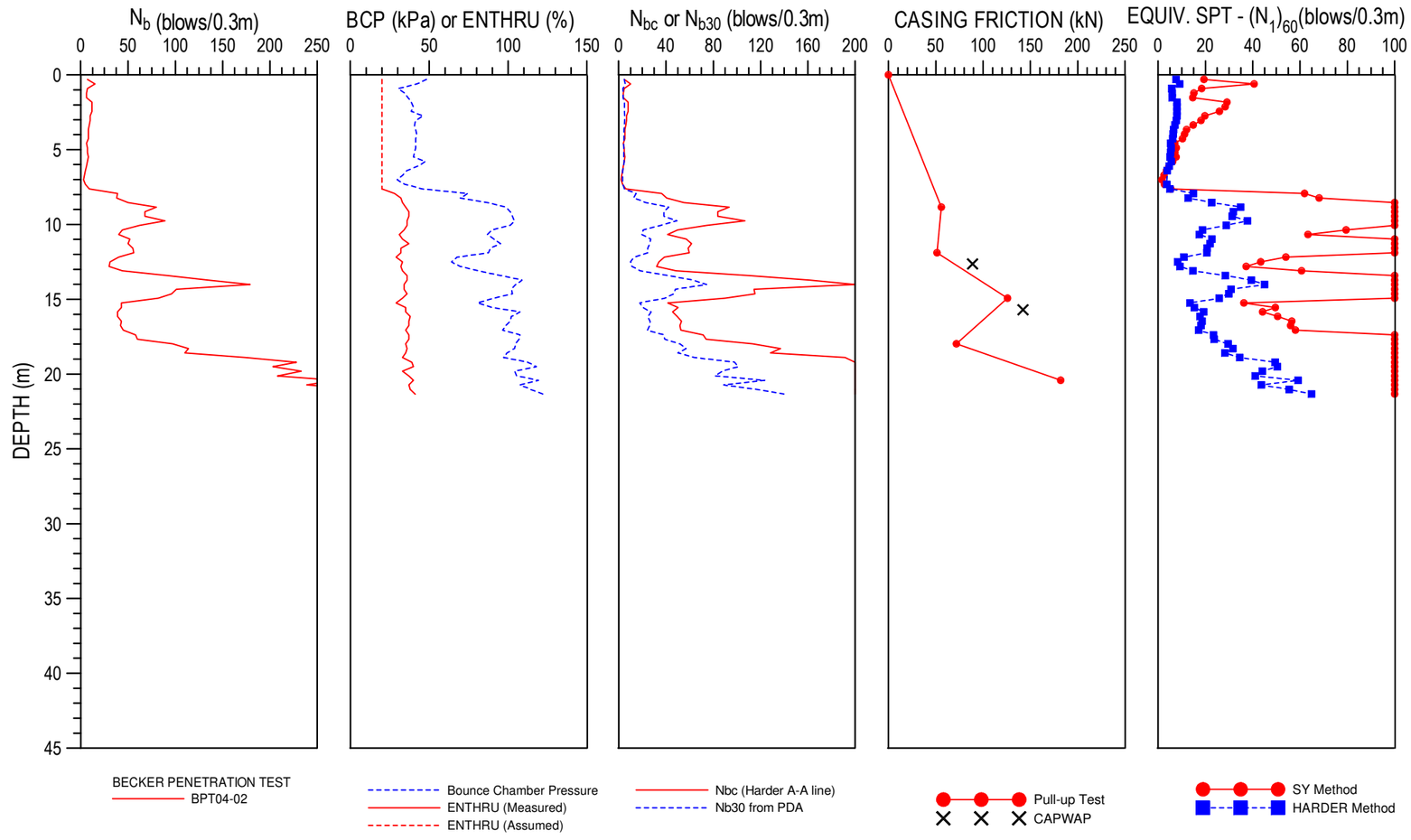
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FIGURE NO. V-5



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
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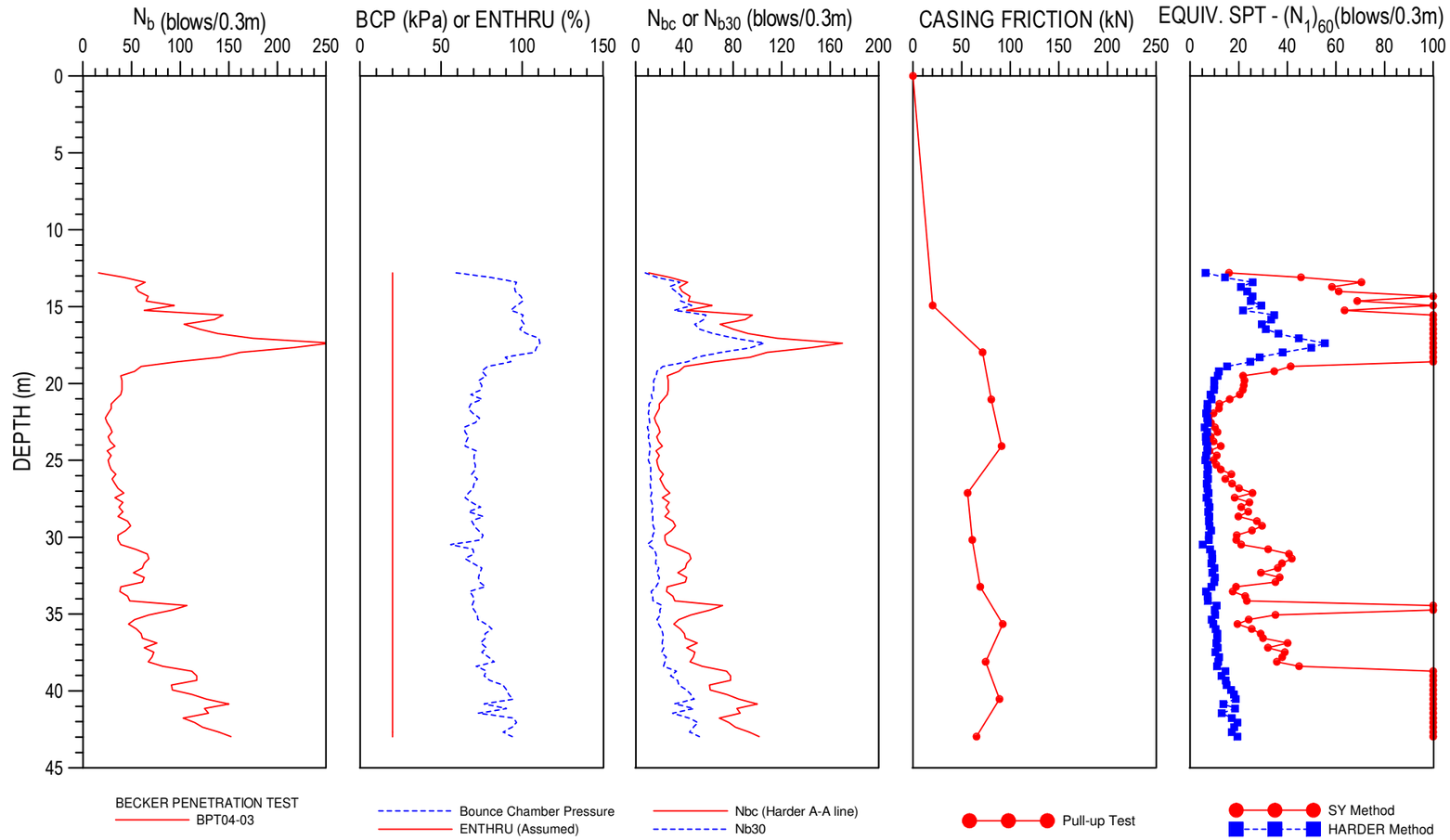
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FIGURE NO. V-6



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
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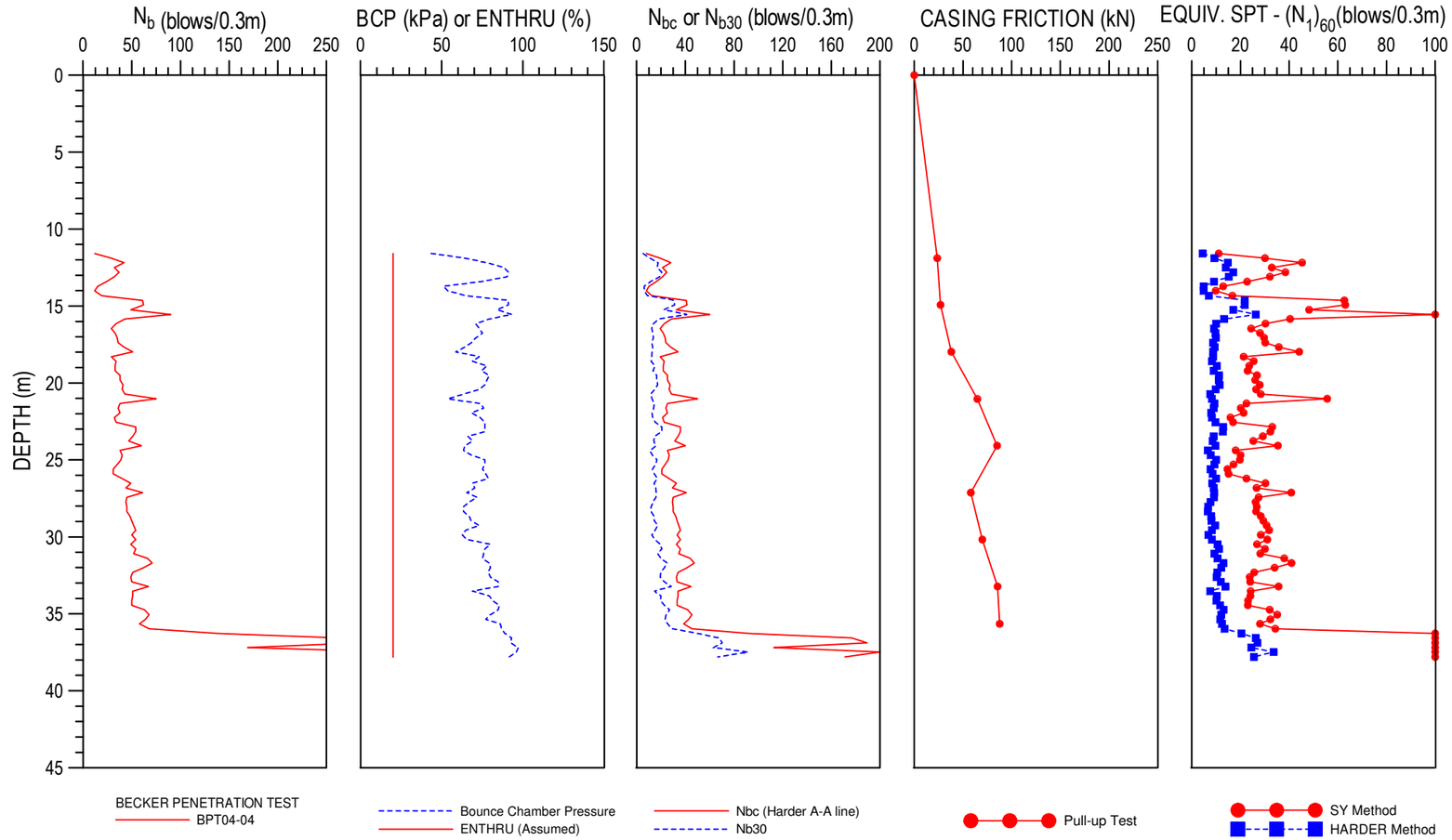
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FIGURE NO. V-7



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
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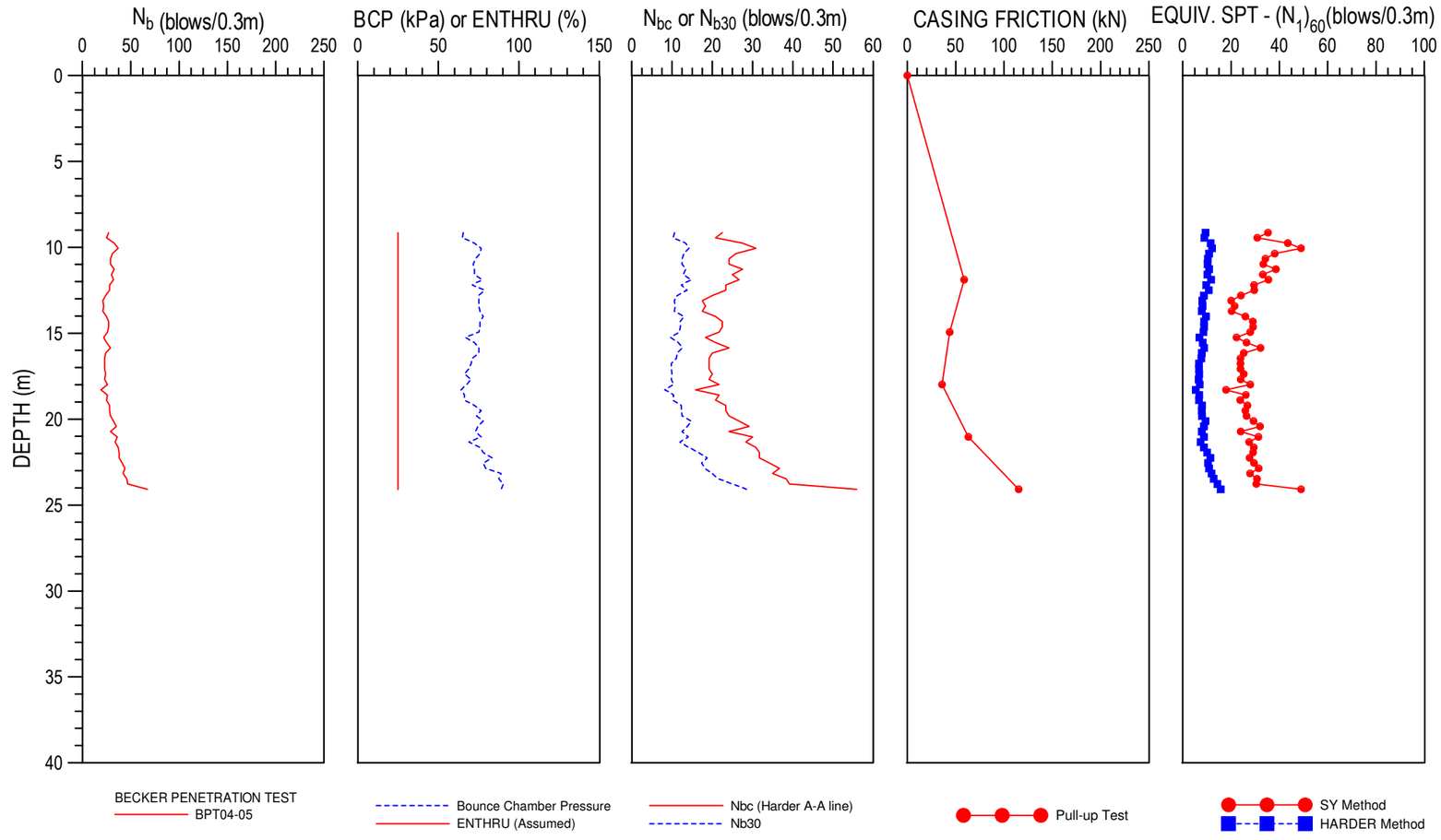
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FIGURE NO. V-8



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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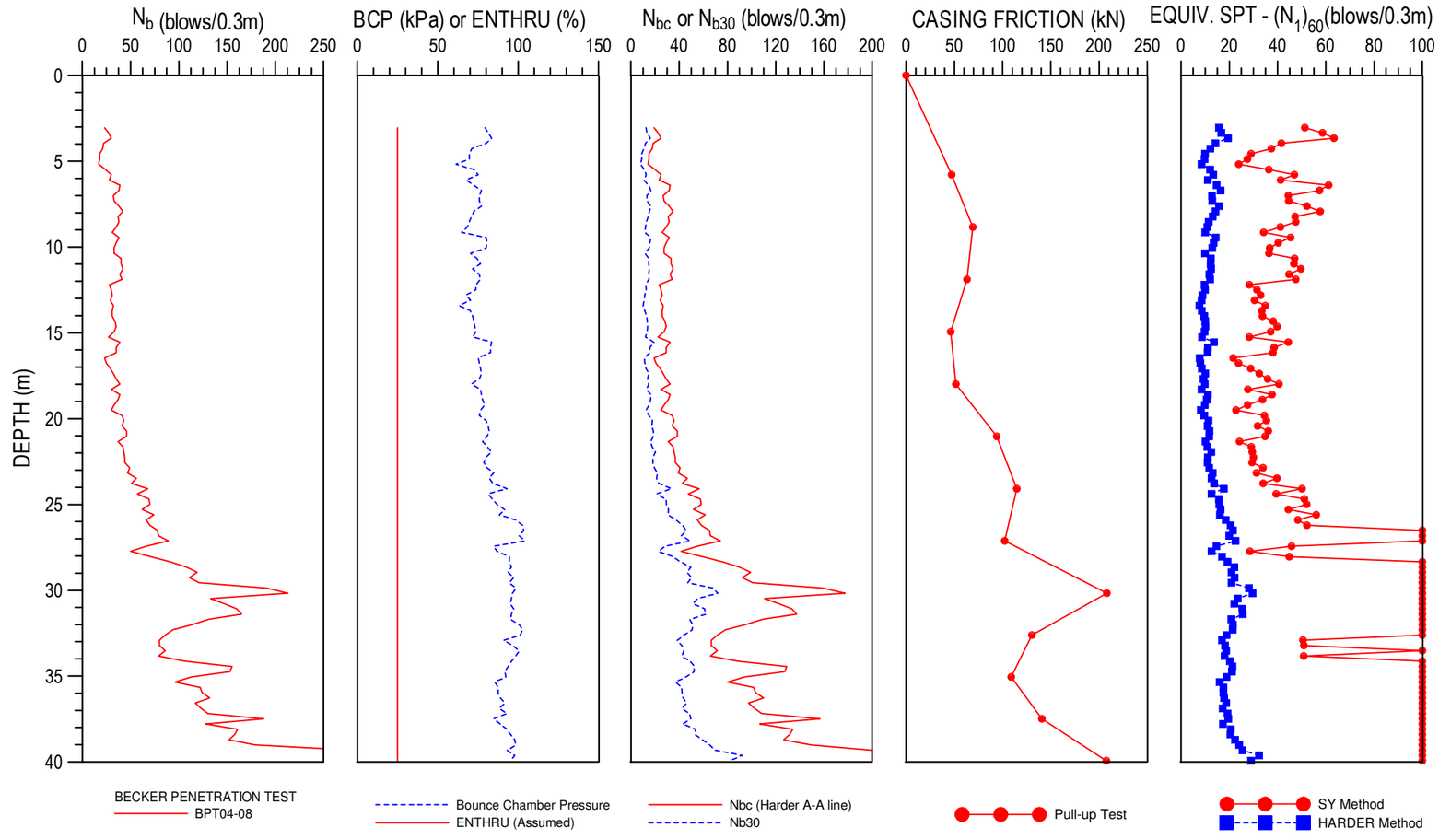
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FIGURE NO. V-9



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
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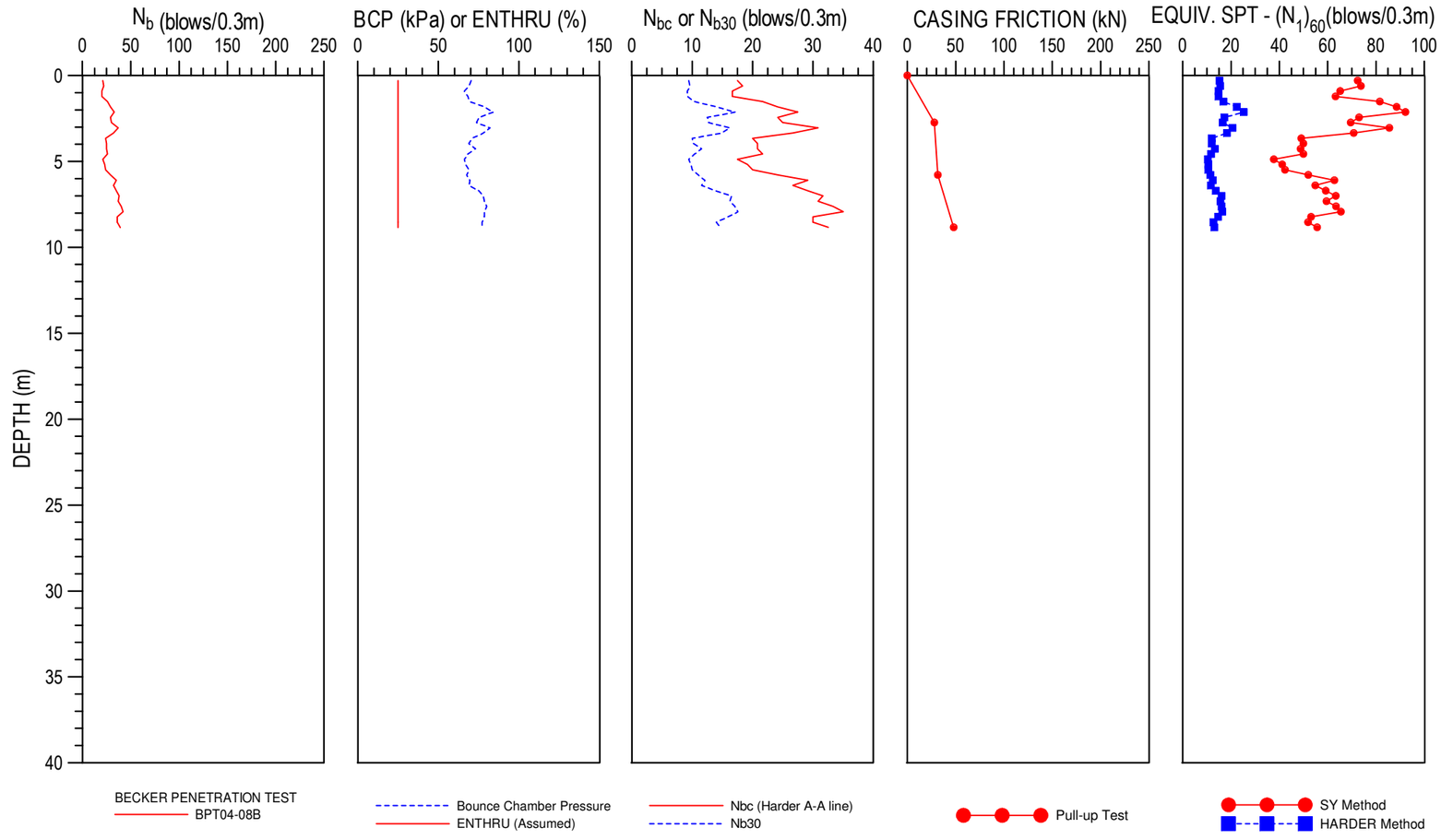
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FIGURE NO. V-10



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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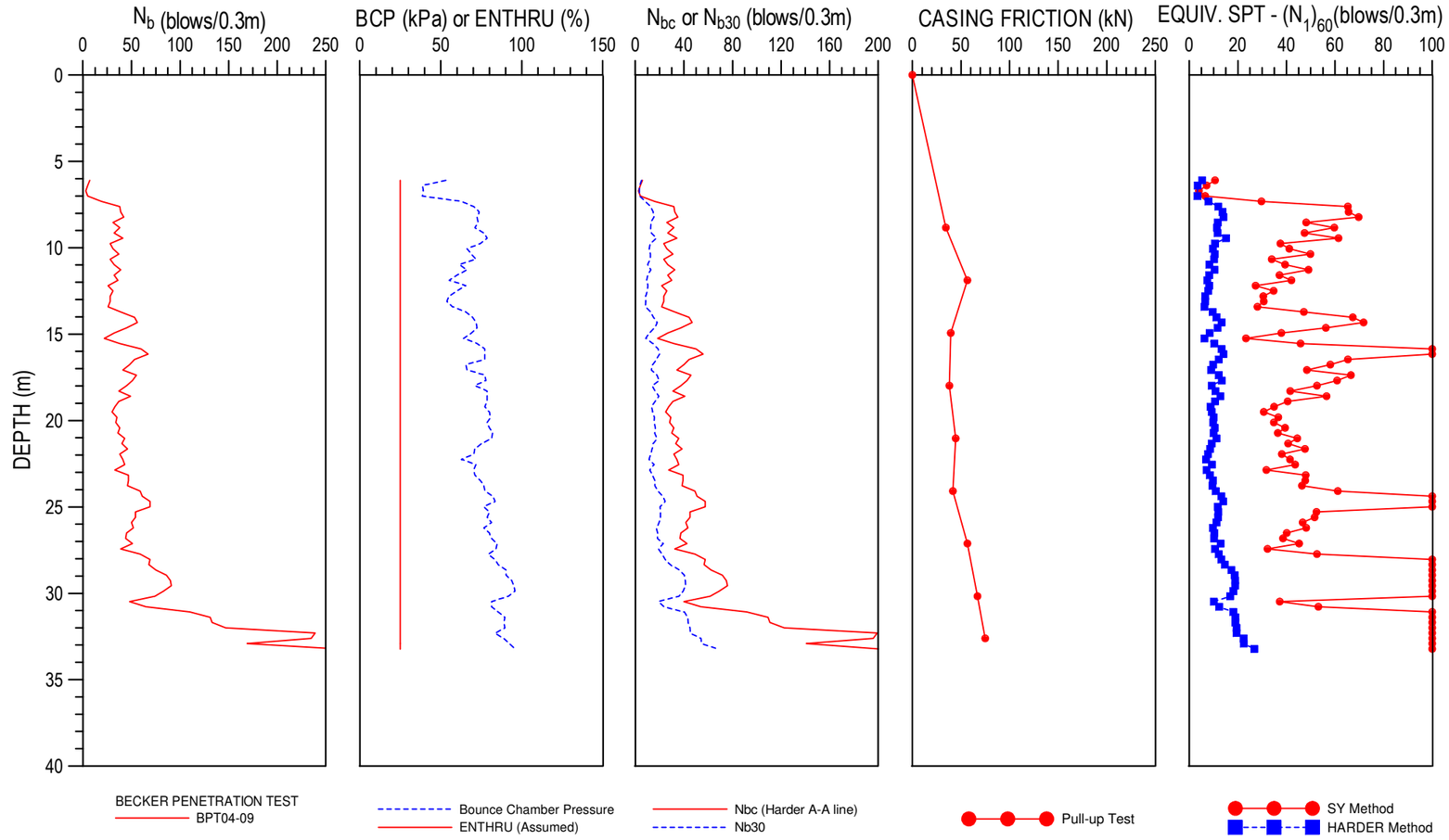
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FIGURE NO. V-11



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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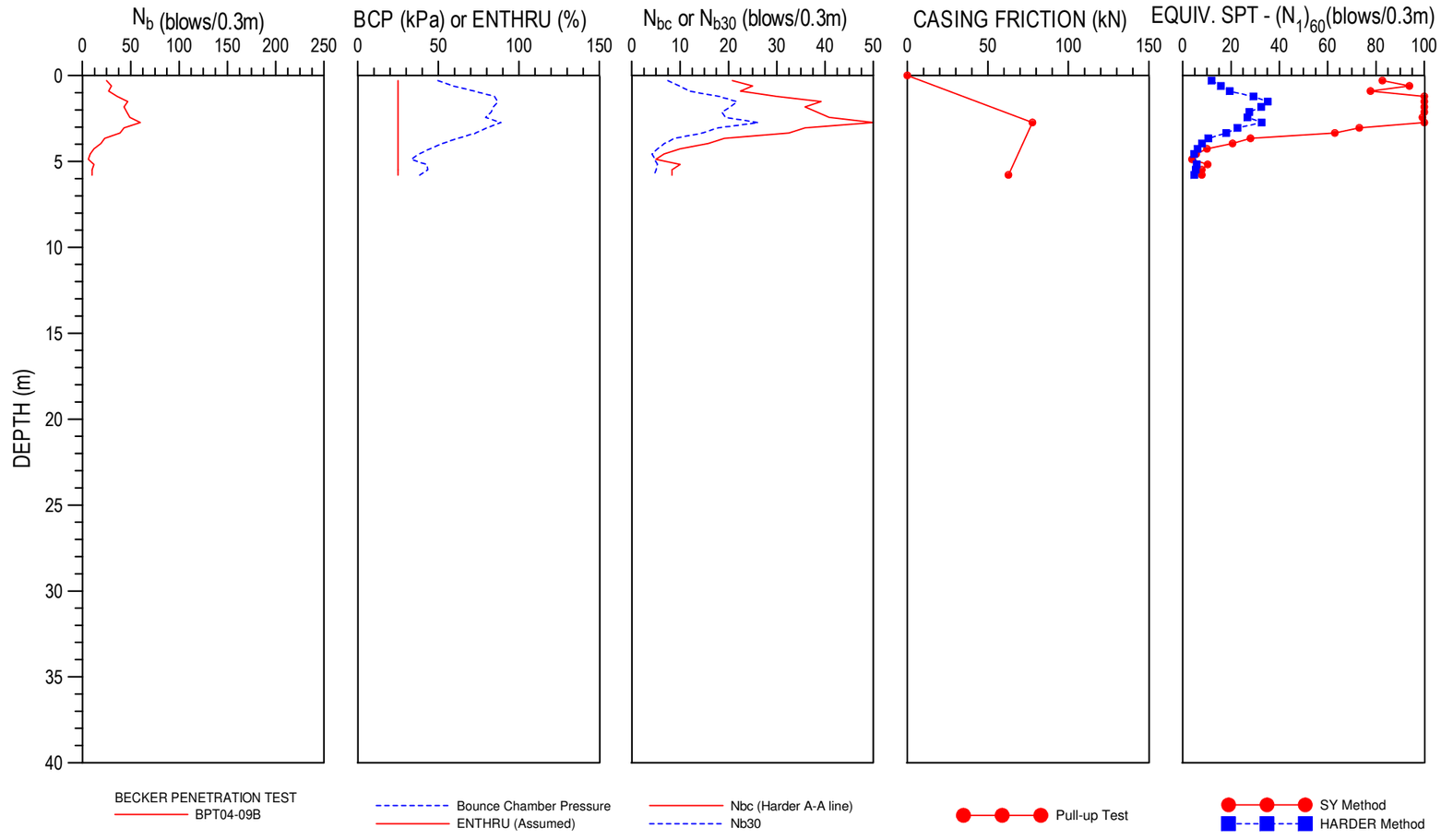
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PROJECT NO. M 09237 A01 01

FIGURE NO. V-12



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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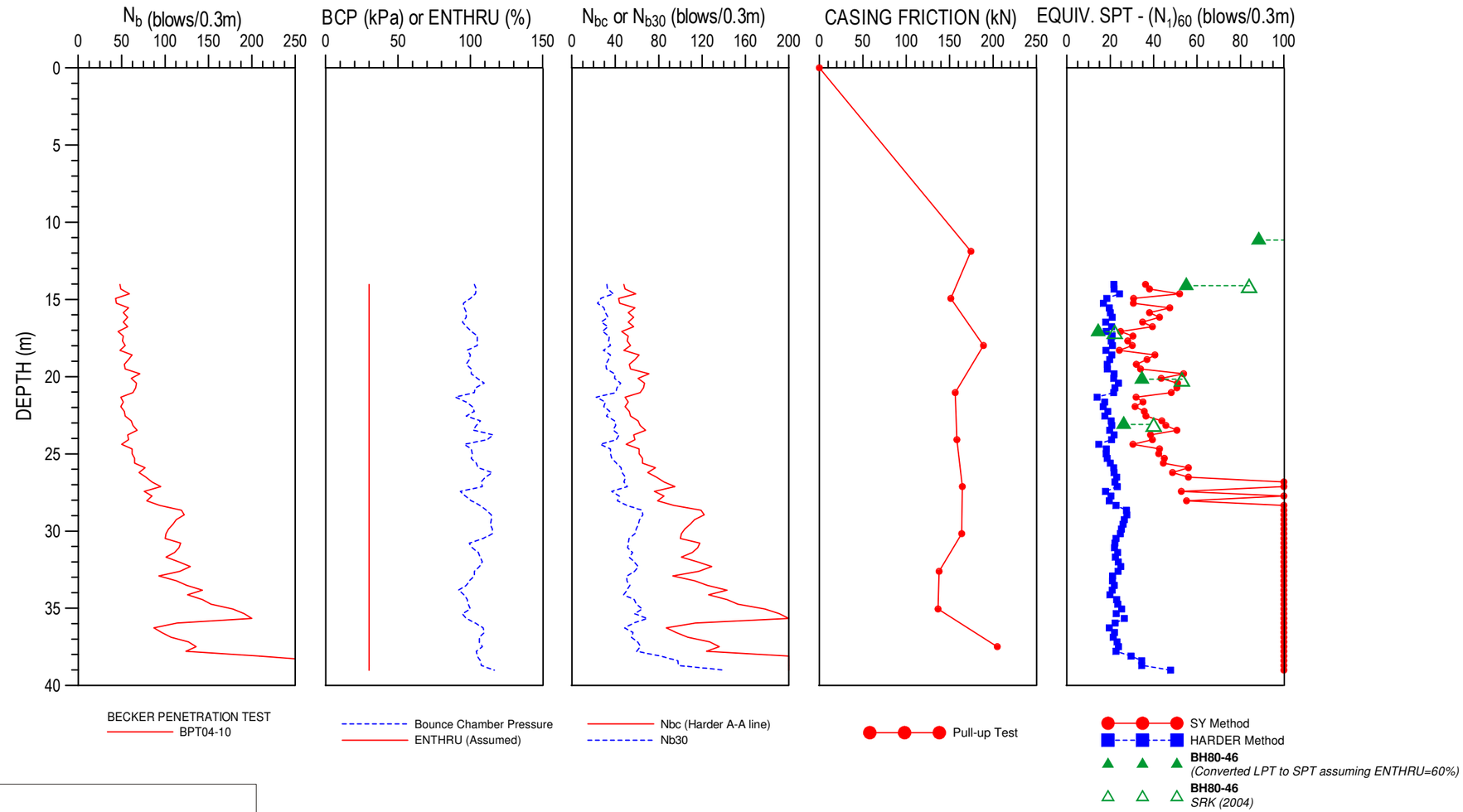
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TITLE
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PROJECT NO. M 09237 A01 01

FIGURE NO. V-13



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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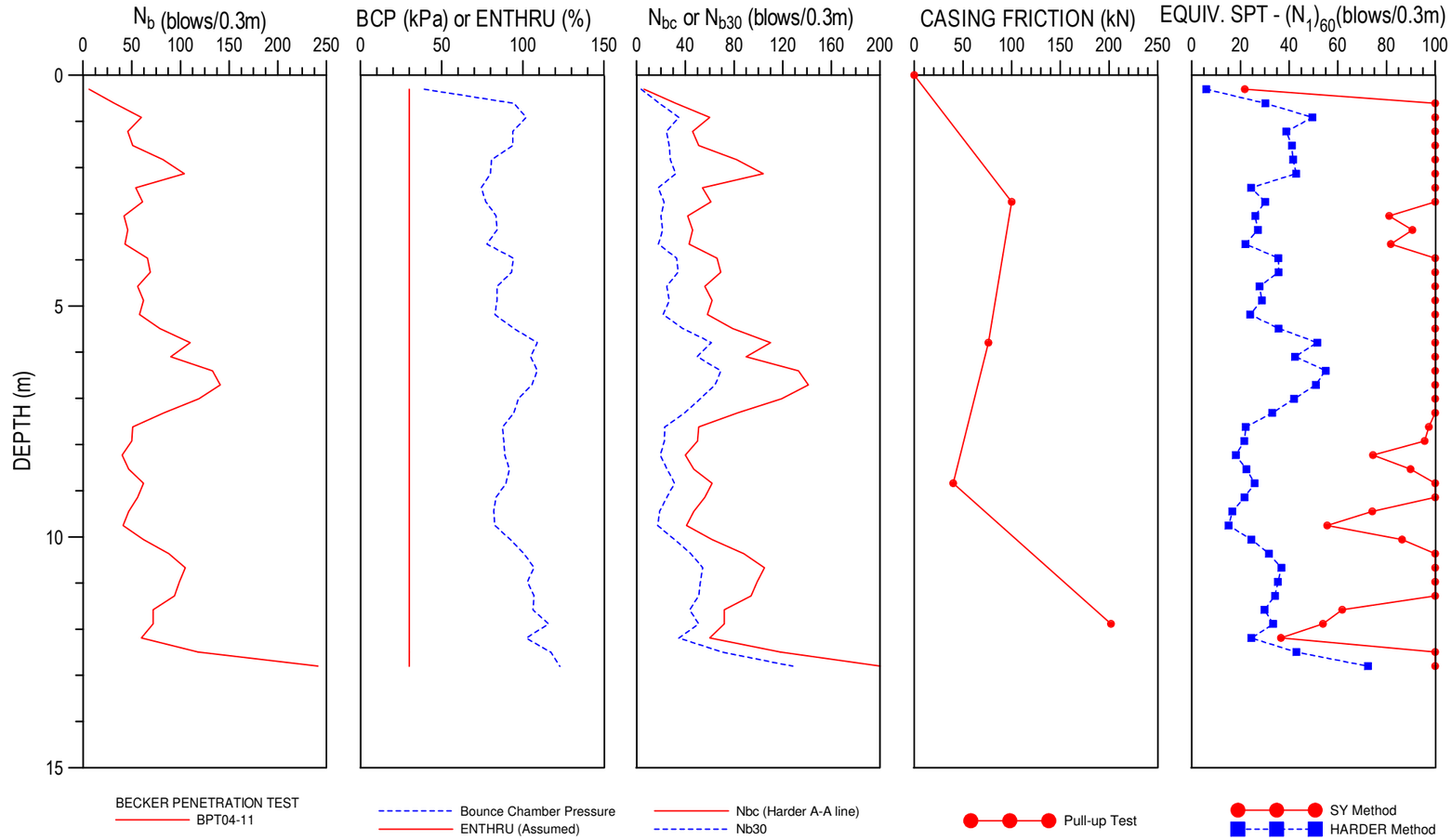
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ROSE CREEK TAILINGS IMPOUNDMENT
SEISMIC STABILITY ASSESSMENT

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BPT DATA INTERPRETATION FOR BPT04-10

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FIGURE NO. V-14



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
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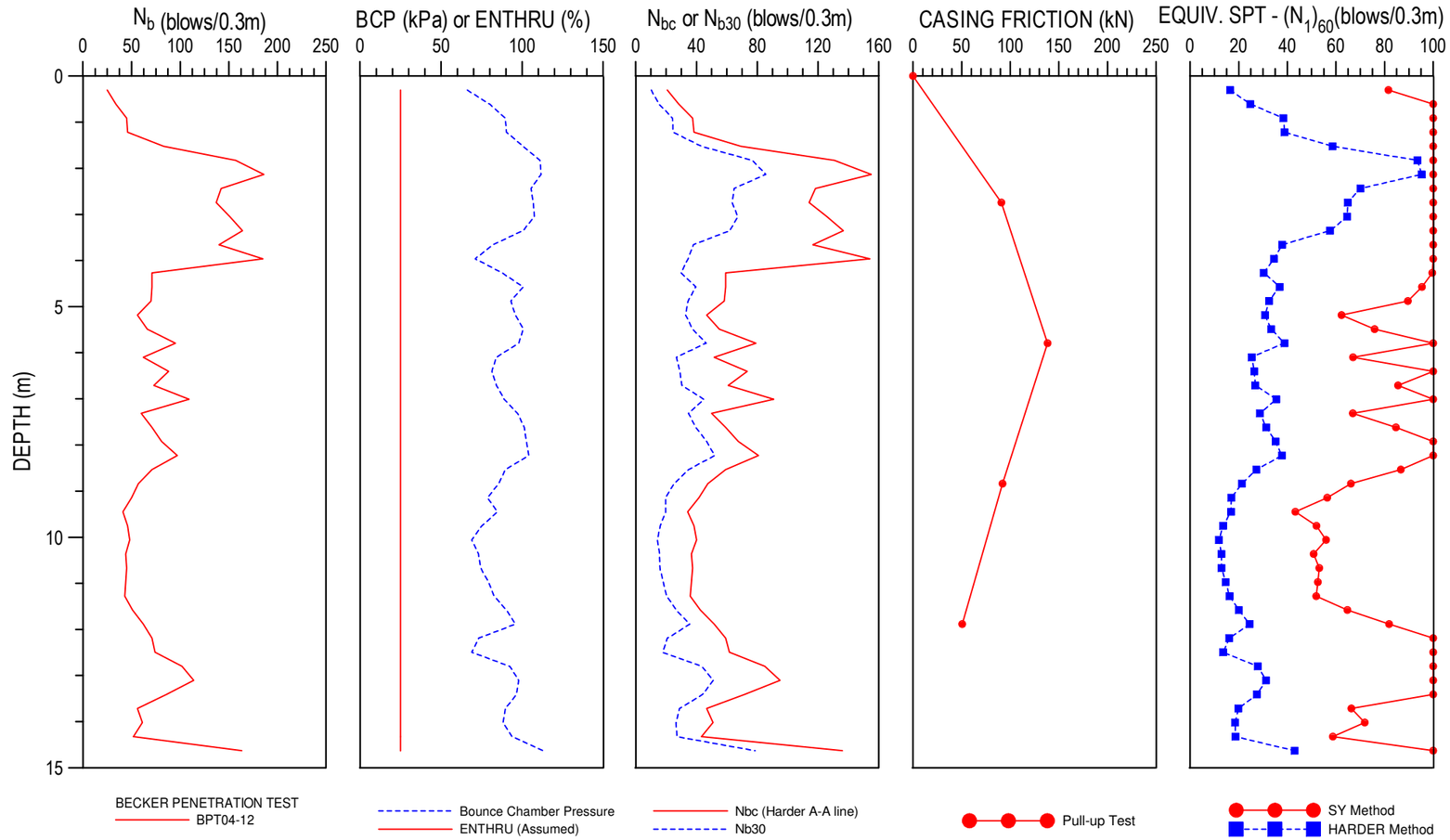
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SEISMIC STABILITY ASSESSMENT

TITLE
BPT DATA INTERPRETATION FOR BPT04-11

PROJECT NO. M 09237 A01 01

FIGURE NO. V-15



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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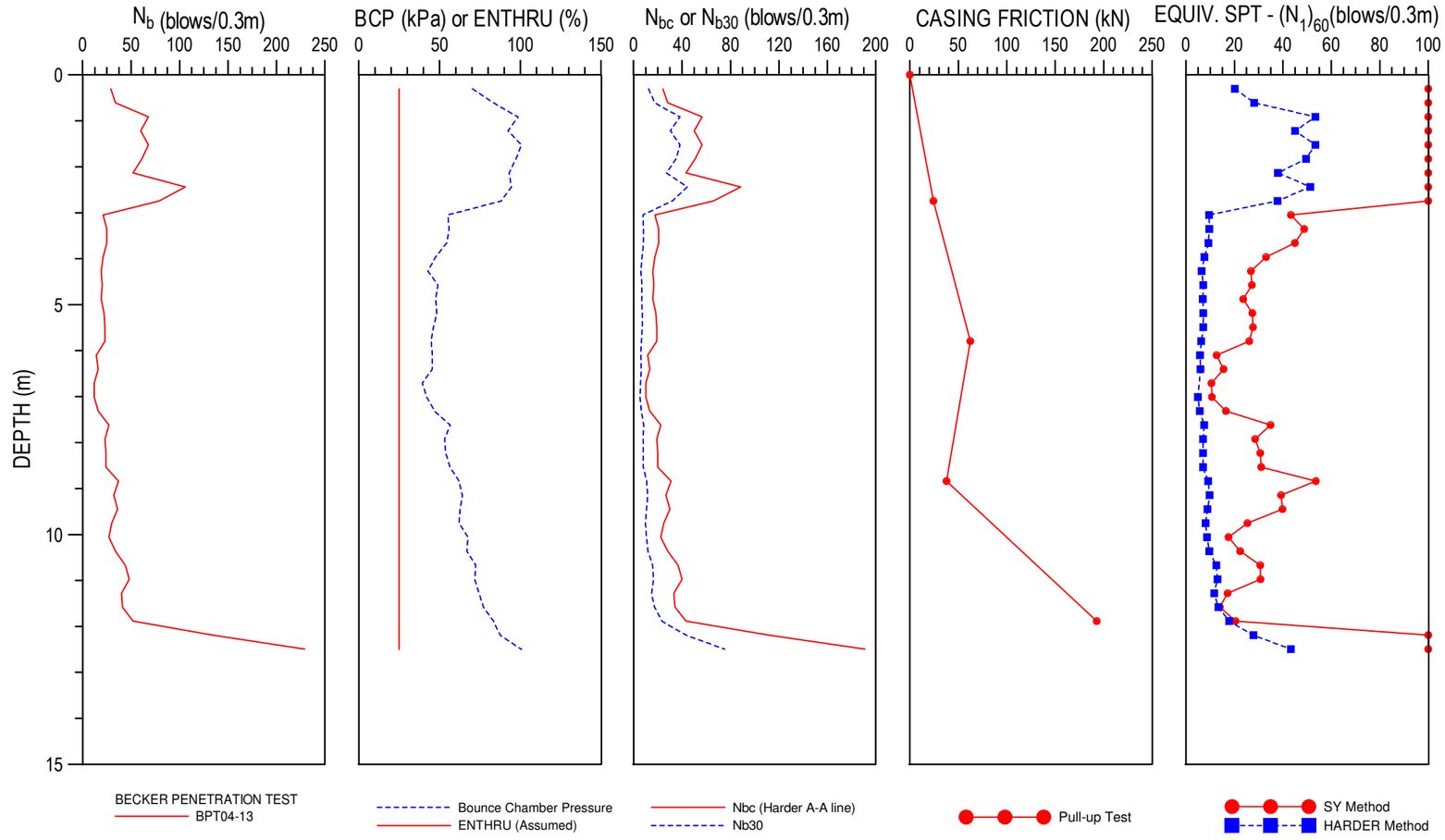
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BPT DATA INTERPRETATION FOR BPT04-12

PROJECT NO. M 09237 A01 01

FIGURE NO. V-16



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
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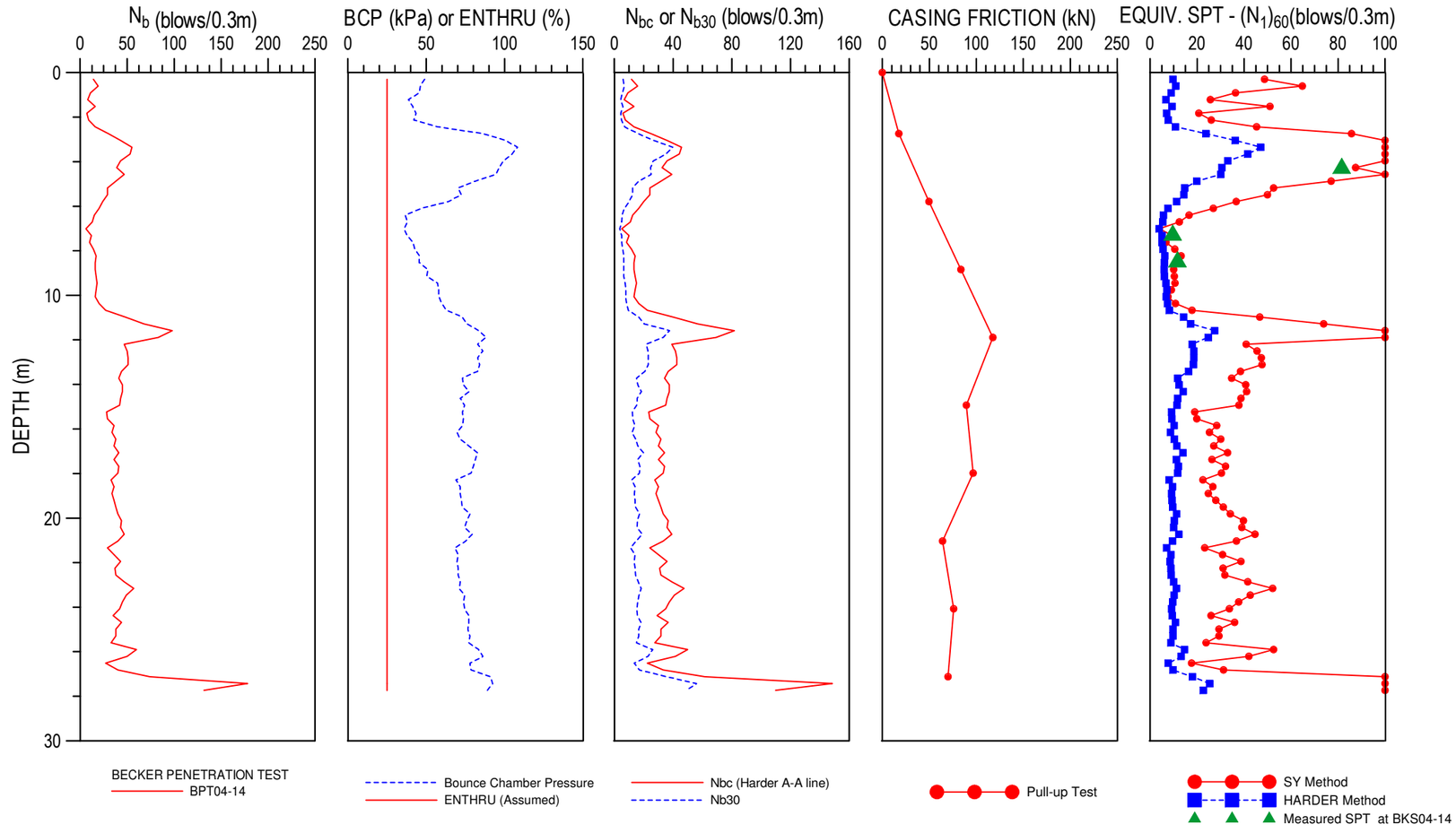
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BPT DATA INTERPRETATION FOR BPT04-13

PROJECT NO. M 09237 A01 01

FIGURE NO. V-17



LEGEND

N_b	Measured BPT blow counts per 0.3 m
ENTHRU	Measured transferred energy as % of hammer rated energy
BCP	Measured bounce chamber pressure
Nbc	Corrected BPT blow counts to a constant combustion condition using Harder rating curve (1986)
Nb30	Corrected BPT blow counts to 30% reference energy of the rated energy for the ICE 180 hammer
HARDER method	Seed and Harder (1986) method for standardizing measured BPT blow counts to a constant combustion condition using measured bounce chamber pressures and correlation of the corrected blow count, Nbc, to the equivalent SPT N_{60}
SY Method	SY (1993) method for correcting measured BPT blow counts to the 30% reference energy level and estimation of the equivalent SPT N_{60} using the corrected blow count, Nb30, and measured or computed shaft resistance to account for soil friction effect

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BPT DATA INTERPRETATION FOR BPT04-14

PROJECT NO. M 09237 A01 01

FIGURE NO. V-18

APPENDIX VI

Site Response and Liquefaction Analyses

TABLE OF CONTENTS

Page

VI-1. INTRODUCTION 1
VI-2. EARTHQUAKE INPUT MOTIONS 2
VI-3. GROUND RESPONSE ANALYSES 3
VI-3.1 Soil Models 3
VI-3.2 Low Strain Shear Modulus, G_{max} 3
VI-3.3 Shear Wave Velocity Profiles 4
VI-3.4 Modulus Reduction and Damping Curves 5
VI-3.5 SHAKE Analyses 6
VI-3.5.1 Intermediate Dam 6
VI-3.5.2 Secondary Dam West Limb 6
VI-3.5.3 Secondary Dam East Limb 6
VI-4. LIQUEFACTION EVALUATION OF GRANULAR SOILS 6
VI-4.1 Methodology for Liquefaction Evaluation Using BPT 6
VI-5. RESULTS OF LIQUEFACTION ANALYSES 8
VI-5.1 Intermediate Dam 8
VI-5.2 Secondary Dam-West Limb 9
VI-5.3 Secondary Dam East-Limb 10
VI-6. LIQUEFACTION EVALUATION OF FINE GRAINED SOILS 10

TABLES

Table VI-1 Details of SHAKE Analyses
Table VI-2 Earthquake Records Used in the Site Response Analyses

FIGURES

Figure VI-1 Six Input Acceleration Time Histories Used in the Site Response Analyses
Figure VI-2 Comparison of Acceleration Response Spectra of Six Input Time Histories with the Target Spectra Provided by Atkinson (2003)

Figure VI-3 Comparison of Shear Wave Velocity Profiles Computed Using Various Correlations at BPT04-10
Figure VI-4 Computed and Idealized Shear Wave Velocity Profiles at the Intermediate Dam Used in the Site Response Analyses
Figure VI-5 Computed and Idealized Shear Wave Velocity Profiles at the Secondary Dam (West Limb) Used in the Site Response Analyses
Figure VI-6 Computed and Idealized Shear Wave Velocity Profiles at the Secondary Dam (East Limb) Used in the Site Response Analyses
Figure VI-7 Modulus Reduction and Damping Curves Used in the SHAKE Analyses
Figure VI-8 Liquefaction Assessment at BPT04-10 for Condition Through the Crest of the Intermediate Dam
Figure VI-9 Liquefaction Assessment at BPT04-08 for Condition Through the Crest of the Intermediate Dam
Figure VI-10 Liquefaction Assessment at BPT04-09 for Condition Through the Crest of the Intermediate Dam
Figure VI-11 Liquefaction Assessment at BPT04-10 for Condition through the Toe of the Intermediate Dam
Figure VI-12 Liquefaction Assessment at BPT04-08 for Condition through the Toe of the Intermediate Dam
Figure VI-13 Liquefaction Assessment at BPT04-09 for Condition through the Toe of the Intermediate Dam
Figure VI-14 Liquefaction Assessment at BPT04-03 for Condition with the Tailings at the Secondary Dam
Figure VI-15 Liquefaction Assessment at BPT04-04 for Condition with the Tailings at the Secondary Dam
Figure VI-16 Liquefaction Assessment at BPT04-02 for Condition with the Tailings at the Secondary Dam
Figure VI-17 Liquefaction Assessment at BPT04-03 for Condition without the Tailings at the Secondary Dam
Figure VI-18 Liquefaction Assessment at BPT04-04 for Condition without the Tailings at the Secondary Dam
Figure VI-19 Liquefaction Assessment at BPT04-02 for Condition without the Tailings at the Secondary Dam
Figure VI-20 Liquefaction Assessment at BPT04-11 for As-Drilled Condition at the Secondary Dam East Limb
Figure VI-21 Liquefaction Assessment at BPT04-12 for As-Drilled Condition at the Secondary Dam East Limb
Figure VI-22 Liquefaction Assessment at BPT04-13 for As-Drilled Condition at the Secondary Dam East Limb
Figure VI-23 Liquefaction Assessment at BPT04-13 for Condition Through the Dam Crest at the Secondary Dam East Limb
Figure VI-24 Liquefaction Assessment of Silt Samples from BPT04-09 Using the Fine Grained Soils Criteria

APPENDIX VI

SITE RESPONSE AND LIQUEFACTION ANALYSES

VI-1. INTRODUCTION

One dimensional total-stress site response analyses were carried out at selected locations on the Intermediate Dam and east and west limbs of the Secondary Dam. The objective of the site response analyses was to determine the variations of peak horizontal accelerations and cyclic shear stresses with depth for liquefaction assessment of the two dams. Table VI-1 lists the locations and conditions for which the analyses were carried out:

Table VI-1 Details of SHAKE Analyses

Dam	Representative Location	Condition
Intermediate Dam	BPT04-01&10	With the dam (through the crest) and without the dam (through the toe ?)
	BPT04-05&08	
	BPT04-09	
Secondary Dam West Limb	BPT04-03	With and without tailings at the downstream
	BPT04-04	
	BPT04-05	
Secondary Dam East Limb	BPT04-11	As – Drilled condition
	BPT04-12	
	BPT04-13	As – Drilled condition and through the dam crest

The computer program SHAKE (SHAKE91 version by Idriss and Sun, 1992) was used to analyze the response of the soil columns under imposed acceleration time histories at the base. SHAKE computes the response of a horizontally layered soil deposit overlying a uniform half-space subjected to vertically propagating shear waves. The analysis is done in the frequency domain. An equivalent linear method is used to account for the non-linearity of the soil using an iterative procedure to obtain values for modulus and damping that are compatible with the shear strain induced in each soil layer.

VI-2. EARTHQUAKE INPUT MOTIONS

The earthquake records used in the analyses are those recommended by Atkinson (2003). They included four records from the 1989 Loma Prieta earthquake and two records from the 1994 Northridge earthquake. Figure VI-1 shows the acceleration time histories of these six records. Atkinson (2003) recommended these records, without any scaling, as they generally match the design spectra for Faro with 10,000 year return period in the median (50^{th} percentile) to mean range. Figure VI-2 shows comparison of acceleration spectra of the six records with the 10, 000 year return period spectra corresponding to mean, median, 16^{th} and 84^{th} percentile recommended by Atkinson (2003). The peak acceleration of the selected six records varied between 0.37 g and 0.54 g. Note that the range of peak accelerations of the selected records is within the median (0.3 g) and mean (0.5 g) peak ground accelerations (PGAs) for the 10, 000 year return period earthquake as illustrated in Figure IV-2.

Table VI-1 shows the derails of the parameters of the earthquake records used in the analyses.

Table VI-2 Earthquake Records Used in the Site Response Analyses

Record Number	Earthquake and Station	Magnitude	Component	PGA (g)
1	1989 Loma Prieta Earthquake, Gilroy No.3	M_L7	0 deg	0.55
2	1989 Loma Prieta Earthquake, Gilroy No.3	M_L7	90 deg	0.37
3	1989 Loma Prieta Earthquake, UCSC/Lick Lab.	M_L7	0 deg	0.44
4	1989 Loma Prieta Earthquake, UCSC/Lick Lab.	M_L7	90 deg	0.41
5	1994 Northridge EQ, Pacoima Dam Downstream	M_L6.6	175 deg	0.42
6	1994 Northridge EQ, Pacoima Dam Downstream	M_L6.6	265 deg	0.43

VI-3. GROUND RESPONSE ANALYSES

VI-3.1 Soil Models

In the site response analyses, the soil column was divided into layers primarily on the open-ended Becker test hole data. The original ground elevations estimated from the open-ended Becker test hole data were checked, where possible, against the elevations estimated from the pre-mining contours shown in Figure 2.1. The bedrock or the dense till layer at the bottom of each hole was treated as a half space and the input acceleration time histories were applied at the top of the bedrock or till layer. For the analyses through the dam crest at the Intermediate Dam, the dam fill was treated as one unit. The total unit weight of the sand and gravel and the dam fill was taken as 21 kN/m^3 and that of the silt was taken as 20 kN/m^3. The moist and saturated unit weights of the tailings were estimated as 19.5 kN/m^3 and 24 kN/m^3, respectively, based on the tailings data in Golder Associates (2004).

VI-3.2 Low Strain Shear Modulus, G_{max}

Foundation Materials

The low strain shear modulus, G_{max}, is normally estimated from shear wave velocity using the relationship $G_{max}=\rho V_s^2$ where ρ is the density and V_s is the shear wave velocity. However, for foundation materials at the Intermediate Dam and at the Secondary Dam, measured shear wave velocities were not available. Thus, the low strain shear modulus, G_{max} was estimated from empirical correlations between G_{max} and SPT $(N_1)_{60}$. Various correlations for gravelly soils are available from Seed et al. (1986), Rollins et al. (1986), Lum and Yan (1994), Ohta and Goto (1976) and Yoshida et al. (1988). Seed et al. (1986) suggested the following correlation for gravels.

$$Gmax=434\ (N_1)_{60}^{1/3}\ Pa\ (\sigma m'/P_a)^{0.5}\ F$$

where P_a is the atmospheric pressure, σ_m' is the mean effective normal stress and F is a factor that can range between 1.35-2.5 for gravels. Note that for sand, F=1.

Figure VI-3 shows the comparison of shear wave velocities at BPT04-10 estimated using correlations proposed by various researchers. As can be seen from the figure, the correlation proposed by Rollins et al. (1998) for Pleistocene gravel is close to that by the Seed et al. (1986) with F=2.5 (dense gravels). The correlations by Lum and Yan (1994), Yoshida (1988) and Ohta and Goto (1978) agree generally well with the Seed et al. (1986) relationship for loose gravels with F=1.35. Thus, in our analyses, the Seed et al. (1986) correlation with F=1.5 was used to estimate the shear wave velocity of gravelly soils. For sand, the same correlation with F=1 was used. For limited thickness of silt encountered at BKS04-09, the same correlation with F=1 was used as neither shear wave velocity nor undrained shear strength data were available.

Dam Fill

As measured shear wave velocities or small strain moduli were not available for the dam materials, the following relationship was used to estimate the low strain modulus, G_{max}=3255 Pa $(\sigma_m'/P_a)^{0.5}$. Note that the limited readings taken in the dam fill at BPT04-01 confirmed that the dam fill is dense.

Tailings

The following relationship for clean sand to sandy silt suggested by Golder Associates (2004) based on the shear wave velocity measurements taken during the Seismic Cone Penetration Test (SCPT) in the tailings was used.

G_{max}=109 $(\sigma_m'/100)^{0.531}$ MPa

VI-3.3 Shear Wave Velocity Profiles

Shear wave velocities were calculated from the estimated low strain modulus, G_{max}. Figure VI-4 shows the shear wave velocity profiles at the Intermediate Dam for conditions with and without the dam fill. The shear wave velocities were estimated based on the $(N_1)_{60}$ at BPT04-10, BPT04-08 and BPT04-09. As these estimated profiles reflect

the local variation in the soil layering, they were idealized and used in the analyses. The idealized profiles are shown as dotted lines in Figure VI-4.

Figure VI-5 shows the estimated shear wave velocity profiles with and without the downstream tailings at locations BPT04-02, BPT04-03 and BPT04-04 at the Secondary Dam. Idealized profiles used in the analyses are also shown as dotted lines in Figure VI-5.

Figure VI-6 shows the estimated and idealized profiles for the as drilled condition at the east limb of the Secondary Dam. It is noted that the SPT $(N_1)_{60}$ blow counts at BPT04-11 and at BPT04-12 were generally greater than 30 and liquefaction will not be triggered at these locations. However, at BPT04-13 where liquefaction triggering is likely to occur due to the relatively low $(N_1)_{60}$ blow counts, analyses were carried out for as drilled condition and for condition through the crest of the dam. Figure VI-6 shows the estimated and idealized profiles for these two conditions for BPT04-13.

For the bedrock or dense till layer where the input motions were applied in the SHAKE analyses, the shear wave velocity was taken as 1500 m/s.

VI-3.4 Modulus Reduction and Damping Curves

The modulus reduction and damping curves as a function of shear strain are required as part of the input to SHAKE for the equivalent linear analysis. Modulus reduction and damping curves, generally considered characteristics of soil type, were obtained from the curves suggested by Rollins et al. (1998) and Seed et al. (1986). The modulus reduction and damping curves for sand and gravel, sand, silt and tailings used in the SHAKE analyses are shown in Figure VI-7.

VI-3.5 SHAKE Analyses

VI-3.5.1 Intermediate Dam

The results of the SHAKE analyses for the six earthquake records input ground motions are shown in Figures VI-8 to VI-10 for condition with the dam (through the crest) at BPT04-10, BPT04-08 and BPT04-09, respectively. Each figure shows the computed peak accelerations and the cyclic shear stress ratios (CSR) versus elevation. The CSR is the earthquake induced shear stress divided by the initial effective overburden stress. The average profiles for the six input motions are also shown. Figures VI-11 to VI-13 show the same set of figures for the condition without the dam (through the dam toe).

VI-3.5.2 Secondary Dam West Limb

The results of the SHAKE analyses are shown in Figures VI-14 to VI-16 for condition with the tailings at BPT04-02, BPT04-03 and BPT04-04, respectively. Figures VI-17 to VI-19 show the same set of figures for conditions without the tailings.

VI-3.5.3 Secondary Dam East Limb

The results of the SHAKE analyses are shown in Figures VI-20 to VI-22 for as-drilled condition at BPT04-11, BPT04-12 and BPT04-13, respectively. Figures VI-23 shows the same figure for condition through the dam crest at BPT04-13.

VI-4. LIQUEFACTION EVALUATION OF GRANULAR SOILS

VI-4.1 Methodology for Liquefaction Evaluation Using BPT

The liquefaction assessment of granular soils was carried out in general accordance with the Seed simplified approach as described in Youd et al. (2001), which summarizes the final recommendations from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. In this approach, the cyclic shear stress ratios (CSR) induced by the design earthquake are compared with the cyclic resistance

ratios (CRR) derived from field observations of soils with known standard penetration test (SPT) resistances, at sites that have or have not liquefied during earthquakes. For this project, the induced CSRs were computed from the SHAKE ground response analyses. The CRRs were evaluated based on the equivalent $(N_1)_{60}$ values derived from the Becker penetration test (BPT) blow counts using the Sy (1997) method (Appendix V).

The procedure used for assessment of the liquefaction potential is described as follows:

- The cyclic shear stress ratios (CSR) induced by the design earthquake are computed directly using results from the SHAKE analyses, where CSR is 0.65 times the maximum earthquake-induced cyclic shear stress divided by the initial effective overburden pressure at the depth of interest.

- With the above computed CSR and a factor of safety (FOS) of 1.0, the required cyclic resistance ratio (CRR) is computed as:

  $$CRR = FOS \times CSR$$

- Since the Seed's liquefaction chart is for a M7.5 earthquake at an overburden stress of 100 kPa (1 tsf), the above CRR is corrected as follows:

  $$CRR_{M7.5} = \frac{CRR}{K_m K_\sigma}$$

  where K_m is the correction factor for design earthquake magnitude other than M7.5, and K_σ is the correction factor for initial static confining stress greater than 100 kPa (1 tsf).

- The required SPT $(N_1)_{60}$ value corresponding to the above corrected $CRR_{M7.5}$ is determined from the liquefaction resistance chart originally developed by Seed et al. (1985), and recently modified by Youd et al. (2001). The liquefaction curve for fines content of 5% was used

- The required $(N_1)_{60}$ values for a factor of safety of 1.0 are then compared with the equivalent $(N_1)_{60}$ values estimated from BPTs to identify the liquefiable zones. If the required $(N_1)_{60}$ values (demand) are greater than the equivalent $(N_1)_{60}$ values (resistance), the factor of safety against

liquefaction is less than one, and liquefaction is expected during the design earthquake at the depths where this condition occurs.

M7.1 was used as design earthquake as suggested in Atkinson (2003) to determine the correction factor K_m. The correction factor, K_σ was conservatively estimated using the correlation recommended in Youd et al. (2001) for soils with relative density of 80%. Fines content (FC) of fifteen samples from Becker open holes BKS04-03, BKS04-04 and BKS04-13 that are primarily gravel and/or sand but suspected to contain fines and have relatively low SPT $(N_1)_{60}$ blow counts were tested in the laboratory (see Appendix III for lab test results). The test results showed that the FC was less than 5% except for three samples that had a FC about 8-9%. Therefore, the liquefaction curve corresponding to FC<5% curve used in the analysis is considered appropriate.

VI-5. RESULTS OF LIQUEFACTION ANALYSES

The required $(N_1)_{60}$ values for a factor of safety against liquefaction of 1.0 are plotted in Figures VI-8 to VI-23 to assess the liquefaction potential at the Intermediate Dam and the Secondary Dam. Also included in the figures are the equivalent $(N_1)_{60}$ profiles determined from the BPTs following the Sy (1997) method. Liquefaction is deemed to occur under a design earthquake, if the equivalent $(N_1)_{60}$ values estimated from BPT are less than the required $(N_1)_{60}$.

VI-5.1 Intermediate Dam

Figures VI-8 to VI-10 shows the liquefaction evaluation for conditions at the dam crest. The key results are as follows:

- The upper bound of required $(N_1)_{60}$ (from all six earthquake records) to prevent liquefaction in the granular soils in the foundation at all three locations was at the limiting value of 30.

- Liquefaction was not predicted at BPT04-10 and at BPT04-09. Marginal liquefaction triggering was predicted at BPT04-08 between about EL.

1010 m to 1019 m as the equivalent $(N_1)_{60}$ from the measured BPT in this range was about 25.

- Relatively low equivalent $(N_1)_{60}$ at BPT04-09 between El. 1024 m and 1029 m is apparently due to the presence of silt layer. The liquefaction potential of this fine-grained layer is evaluated in Section VI-6.

Figures VI-11 to VI-13 show the liquefaction evaluation for conditions at the downstream toe. The triggering potential is similar to that at the dam crest. However, the required $(N_1)_{60}$ values to prevent liquefaction for all the records are at the limiting value of 30, apparently due to the higher amplification at the toe than at the crest.

VI-5.2 Secondary Dam-West Limb

Figures VI-14 to VI-16 shows the liquefaction evaluation for existing conditions with the tailings. The key results are as follows:

- The upper bound of required $(N_1)_{60}$ (from all six earthquake records) to prevent liquefaction in the granular soils in the foundation at all three locations was at the limiting value of 30.

- The liquefaction triggering was predicted between EL. 1023 m and 1030 m at BPT04-03 where the lower bound equivalent $(N_1)_{60}$ from the measured BPT values was about 10 only. Below elevation EL. 1023 m, marginal liquefaction was predicted at BPT04-03. However, the measured $(N_1)_{60}$ was generally greater than 22.

- Liquefaction triggering was predicted at BPT04-04 between EL.1016 and 1038 m in intermittent layers of about 2-3 m thick. However, the average equivalent $(N_1)_{60}$ of these layers was about 22.

- Liquefaction triggering is predicted in the tailings at BPT04-02 as can be seen from Figure VI-16 which shows the required $(N_1)_{60}$ values for soils with FC > 35%.

Figures VI-17 to VI-19 show the same figures when the downstream tailings is removed. The triggering potential is similar to the same for the existing conditions. However, the

required $(N_1)_{60}$ values to prevent liquefaction for all six records are at the limiting value of 30.

VI-5.3 Secondary Dam East-Limb

Figures VI-20 to VI-22 shows the liquefaction evaluation for as-drilled condition at the east limb of the dam. Figure VI-23 shows the same for condition through the dam crest at BPT04-13. The key results are as follows:

- Due to very dense soils at BPT04-11 and BPT04-12, liquefaction triggering is not expected. Equivalent $(N_1)_{60}$ values were higher than the limiting value for triggering, $(N_1)_{60}$ of 30.
- At BPT04-13, the required $(N_1)_{60}$ to prevent liquefaction triggering was about 30 for both as-drilled condition and condition through the dam crest. Liquefaction triggering was predicted between EL. 1049.5 and 1053.0 m as the lower bound $(N_1)_{60}$ of this layer is about 10. A meter thick layer at EL. 1042 m was also predicted. However, the average $(N_1)_{60}$ of this layer was about 25.

VI-6. LIQUEFACTION EVALUATION OF FINE GRAINED SOILS

A silt layer about 3 m thick was encountered at BPT04-09 at shallow depth. The liquefaction susceptibility of the fine-grained soils is typically assessed using the Chinese liquefaction criteria (Seed and Idriss, 1982) as a screening tool. Recent criteria by Andrews and Martin criteria (2000) and by Bray et al. (2000) can also be used to determine the liquefaction susceptibility of fine-grained soils. The Chinese criteria is outlined below:

- Clay content (percent finer than 0.005 mm) <15%;
- Liquid limit (LL) <35%; and
- Water content (W) >0.9 x LL.

All three criteria must be satisfied for the soil to be considered potentially liquefiable and recommended for further laboratory testing. The criteria proposed by Andrews and Martin (2000) for fine grained soils are as follows:

- Clay content (percent finer than 0.002 mm) < 10%
- Liquid limit < 32 %.

Both criteria need to be satisfied for the soil to be considered potentially liquefiable. If only one of the two criteria is satisfied, further testing is recommended by Andrews and Martin (2000).

Recently, Bray et al. (2004) proposed a new criterion to evaluate the liquefaction susceptibility of fine-grained soils. They noted that, it is not the amount of clay-size particles in the soil; rather it is the amount of clay minerals that best indicate the liquefaction susceptibility to liquefaction. Hence, they used the soils plasticity index and the ratio of water content to the liquid limit in their proposed criteria.

Disturbed samples from the silt layer at BKS04-09 collected during the open-ended Becker drilling were tested in the laboratory for grain size and Atterberg limits (See Appendix III for laboratory test results). The results of these tests were used to assess the liquefaction potential of the silt using the Chinese criteria, Andrews and Martin criteria and Bray et al. criteria. Figure VI-24 summarizes the results from liquefaction assessment of two samples taken from BKS04-09 at about 4 m and 5.2 m depth. Note only one sample (~4 m depth) was tested for grain size. The percentage passing 0.005 mm was about 20% and the percentage passing <0.002 mm was about 12%. Therefore, the grain size defining clay content in Chinese criteria (< 0.005 mm) was violated and the same in the Andrews and Martin criteria was also violated and recommended further testing. However, according to the Bray et al. (2003) criteria, both samples suggested the silt layer is susceptible to liquefaction.

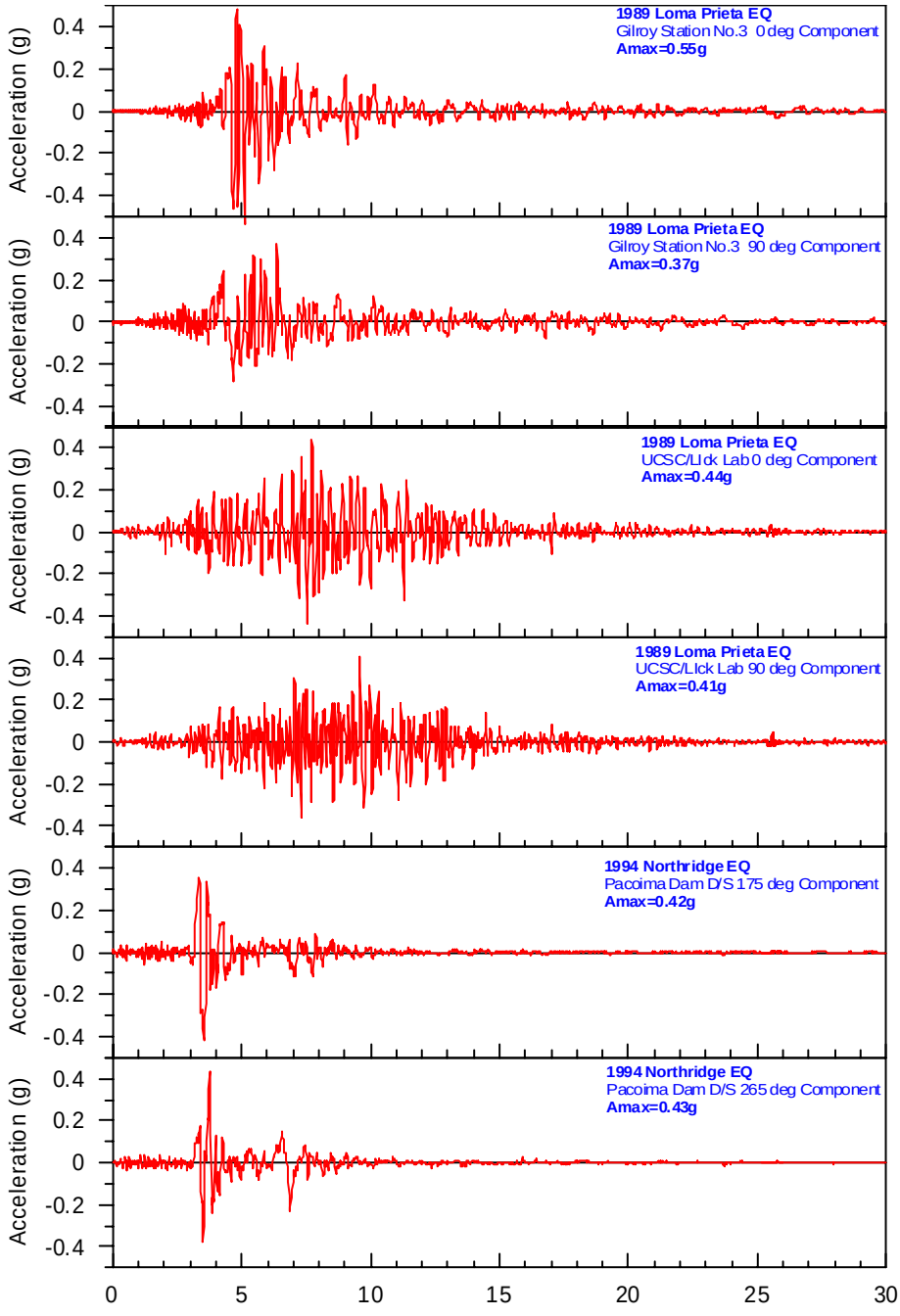


Figure VI-1 Six Input Acceleration Time Histories Used in the Site Response Analyses

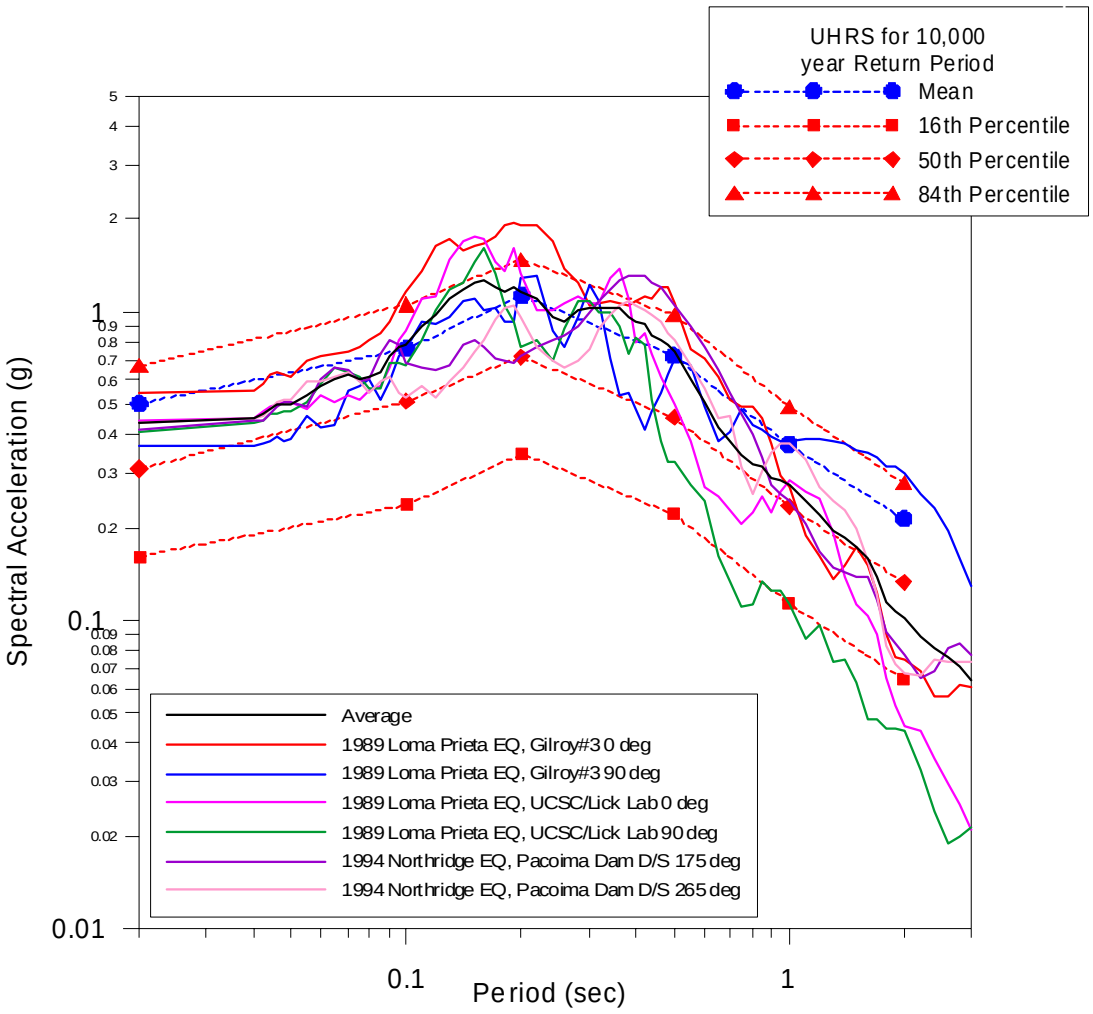


Figure VI-2 Comparison of Acceleration Response Spectra of Six Input Time Histories with the Target Spectra Provided by Atkinson (2003)

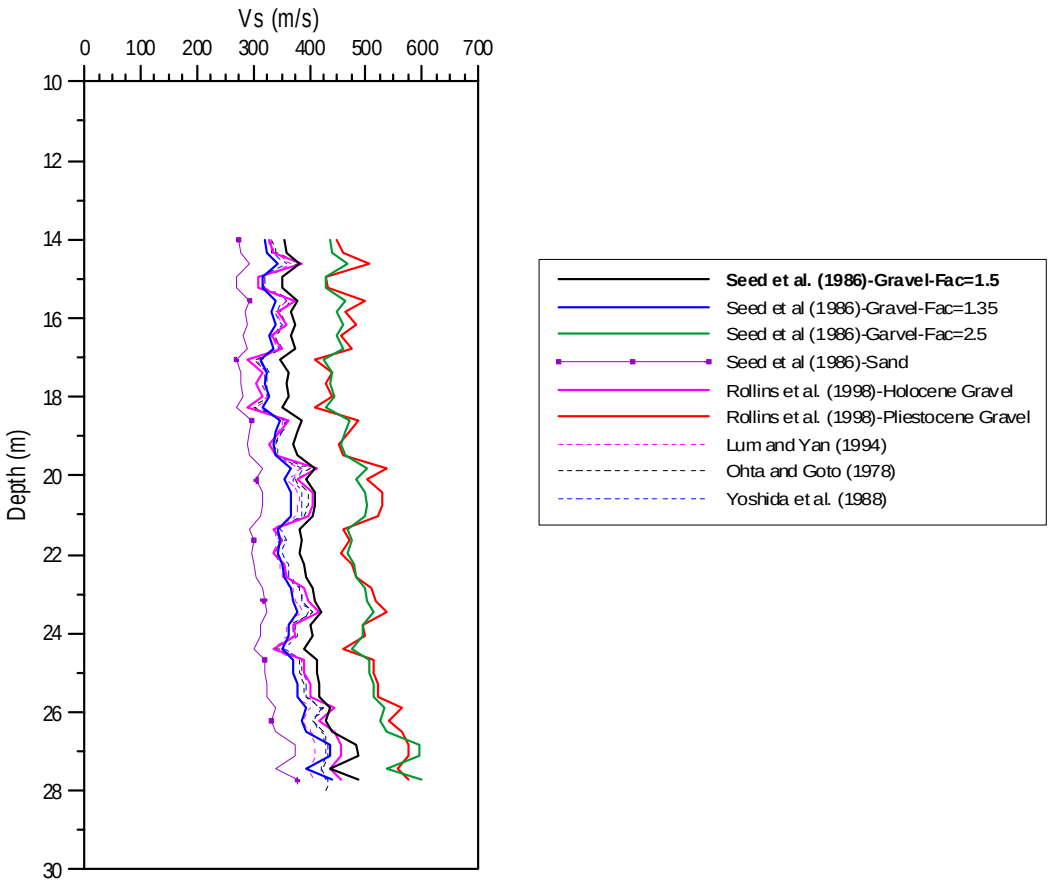


Figure VI-3 Comparison of Shear Wave Velocity Profiles Computed Using Various Correlations at BPT04-10

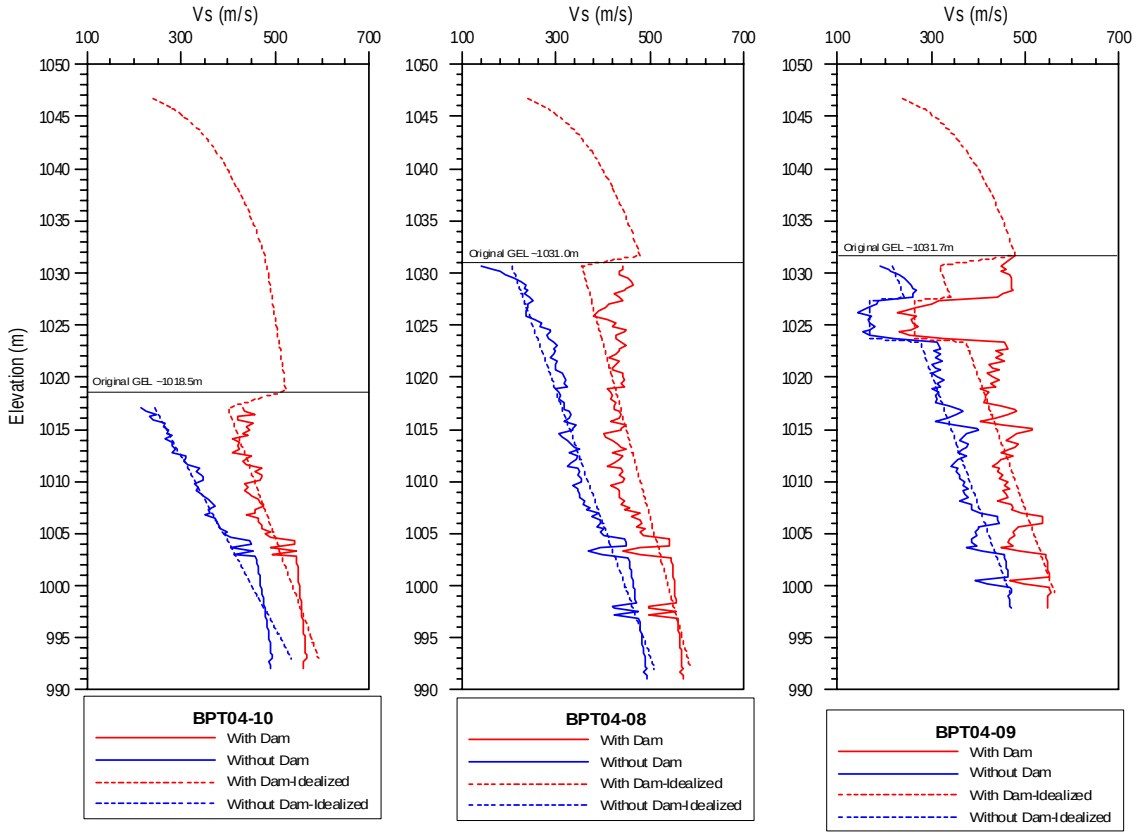


Figure VI-4 Computed and Idealized Shear Wave Velocity Profiles at the Intermediate Dam Used in the Site Response Analyses

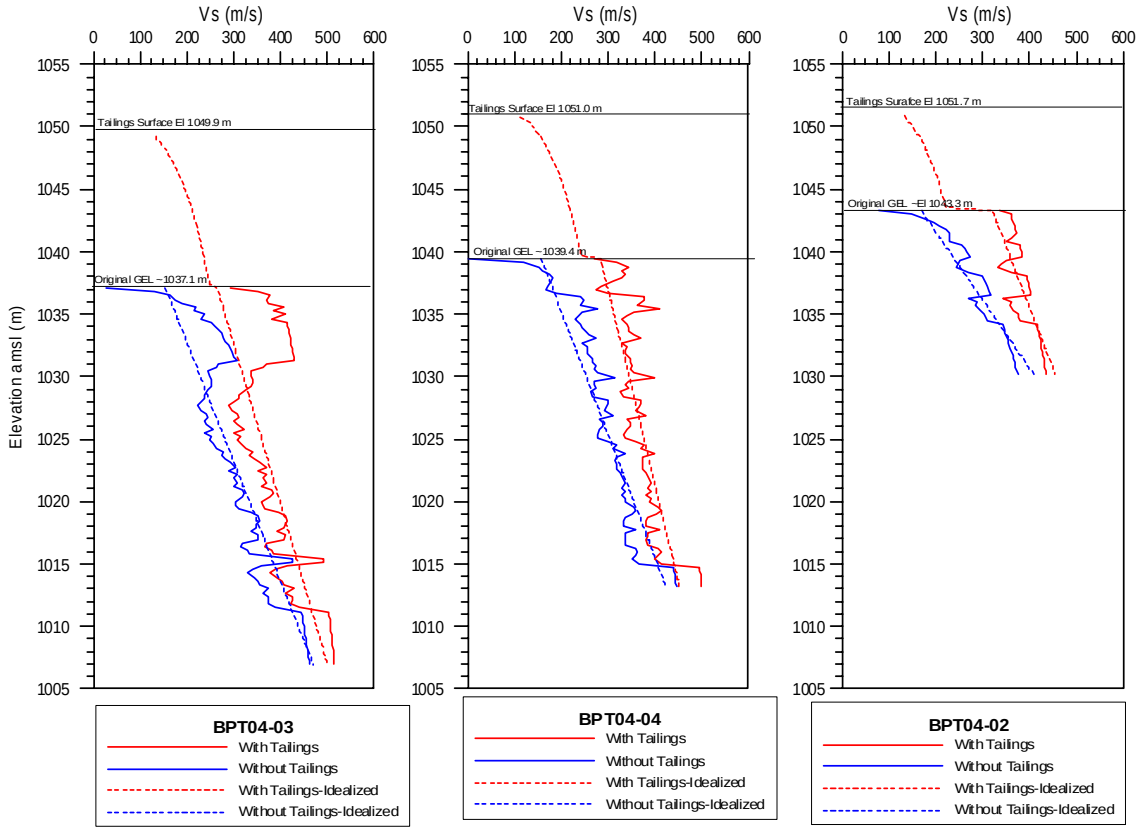


Figure VI-5 Computed and Idealized Shear Wave Velocity Profiles at the Secondary Dam (West Limb) Used in the Site Response Analyses

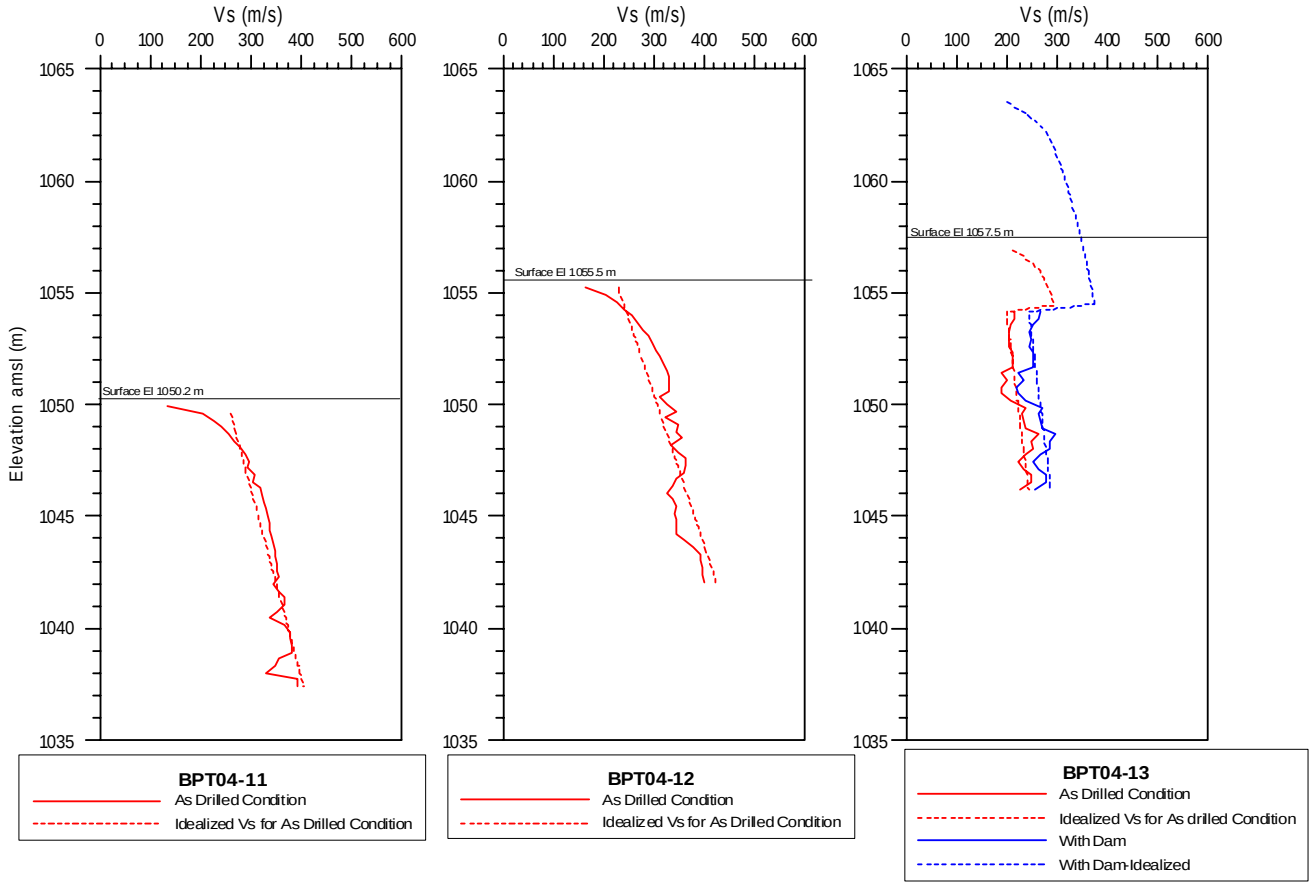


Figure VI-6 Computed and Idealized Shear Wave Velocity Profiles at the Secondary Dam (East Limb) Used in the Site Response Analyses

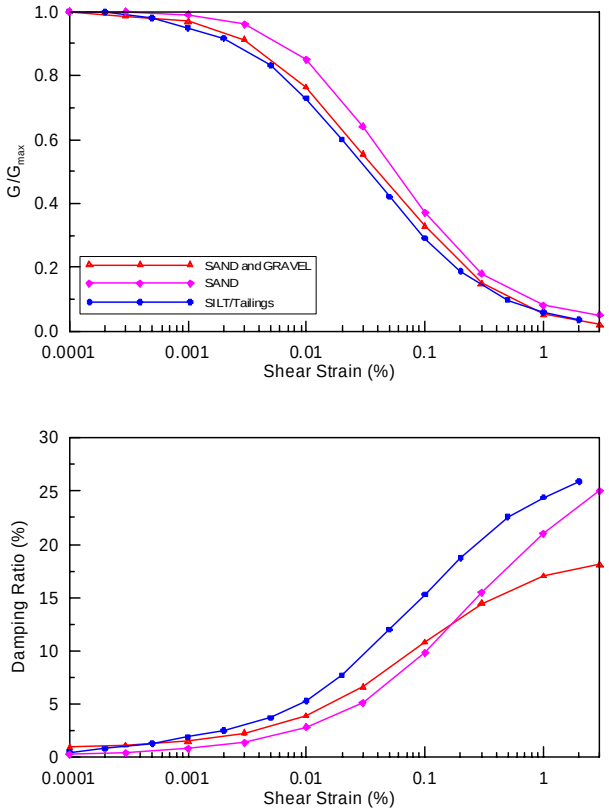


Figure VI-7 Modulus Reduction and Damping Curves Used in the SHAKE Analyses

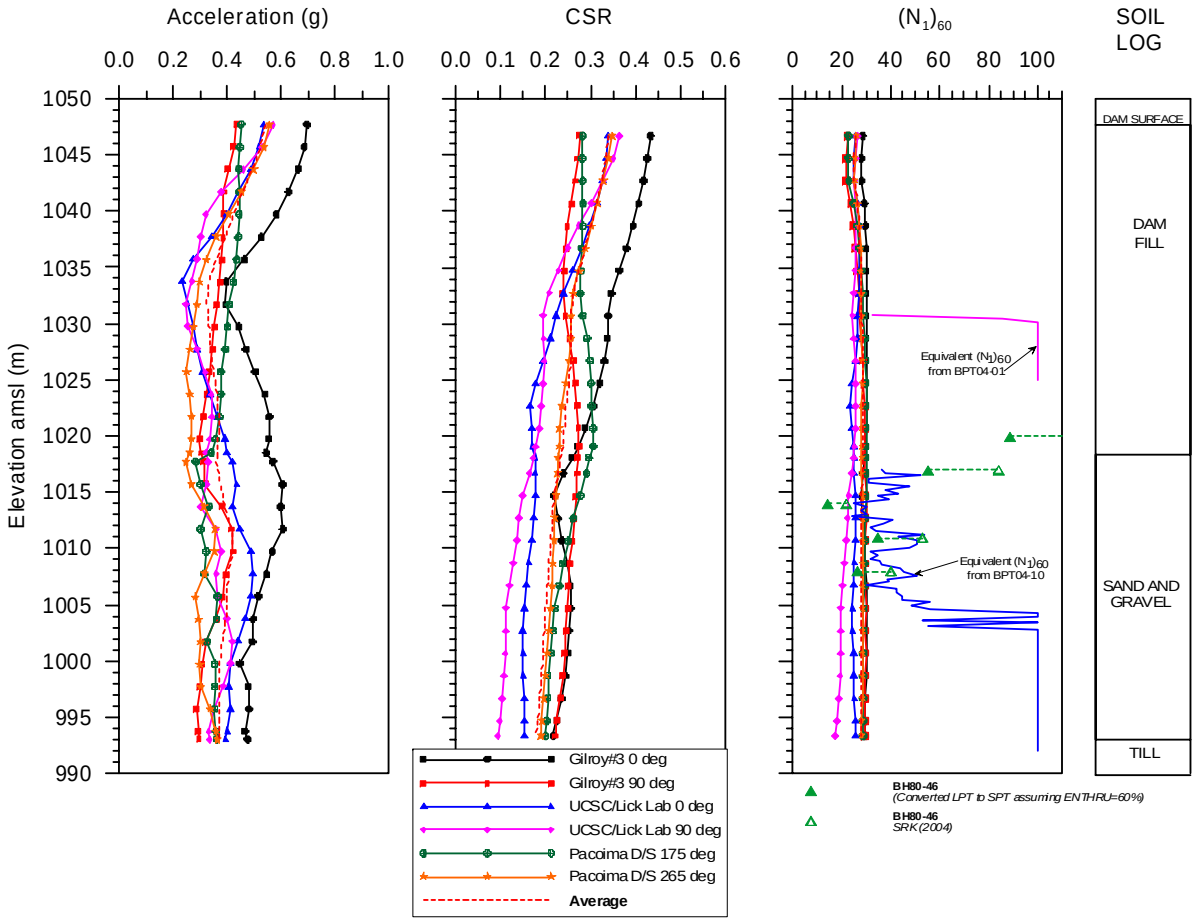


Figure VI-8 Liquefaction Assessment at BPT04-10 for Condition Through the Crest of the Intermediate Dam

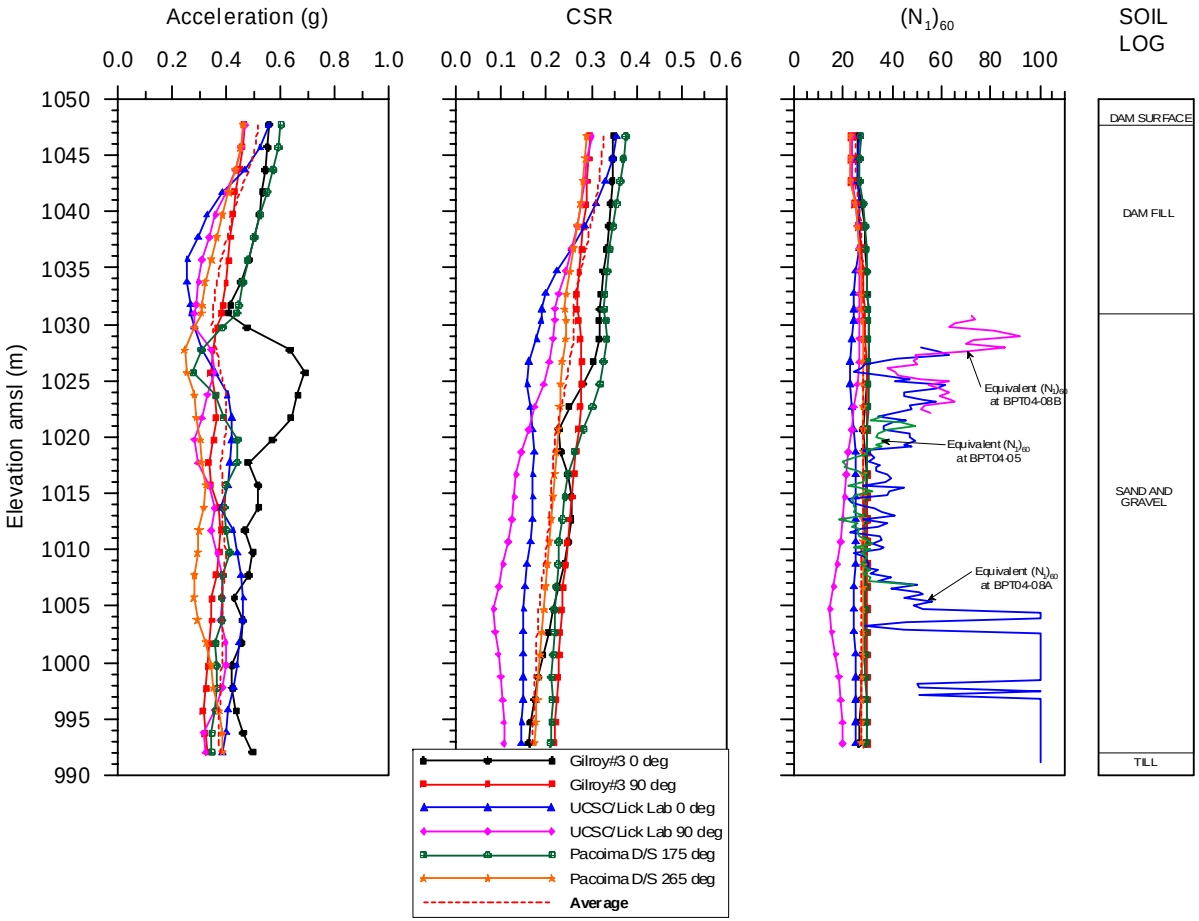


Figure VI-9 Liquefaction Assessment at BPT04-08 for Condition Through the Crest of the Intermediate Dam

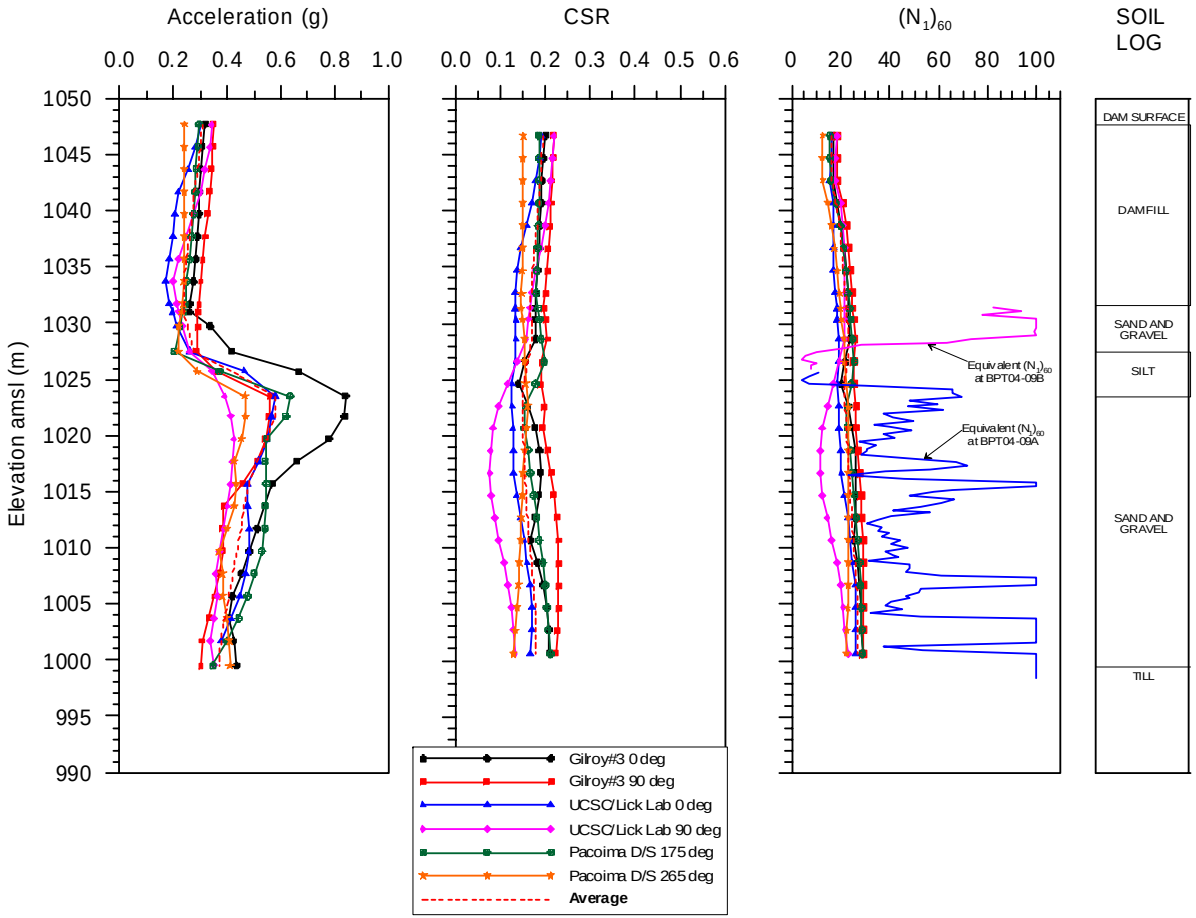


Figure VI-10 Liquefaction Assessment at BPT04-09 for Condition Through the Crest of the Intermediate Dam

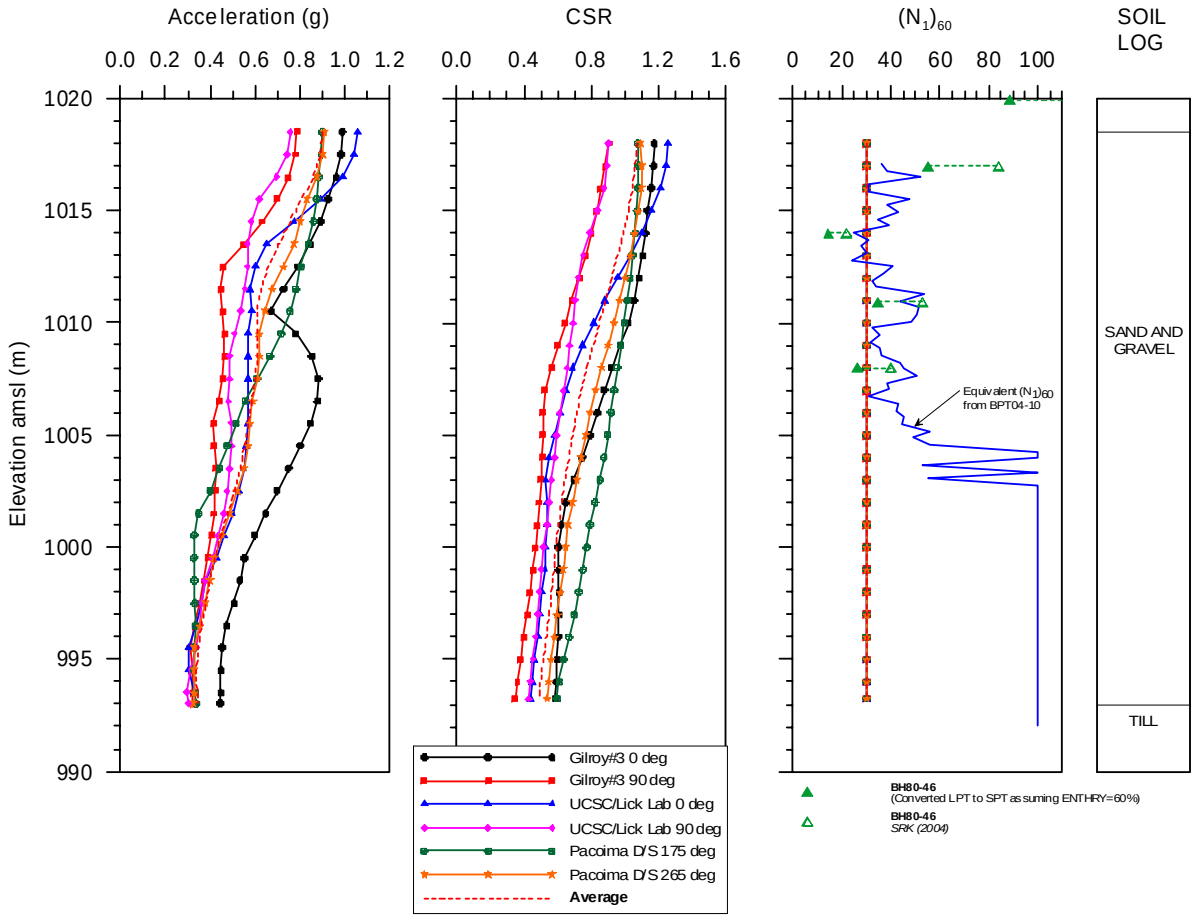


Figure VI-11 Liquefaction Assessment at BPT04-10 for Condition through the Toe of the Intermediate Dam

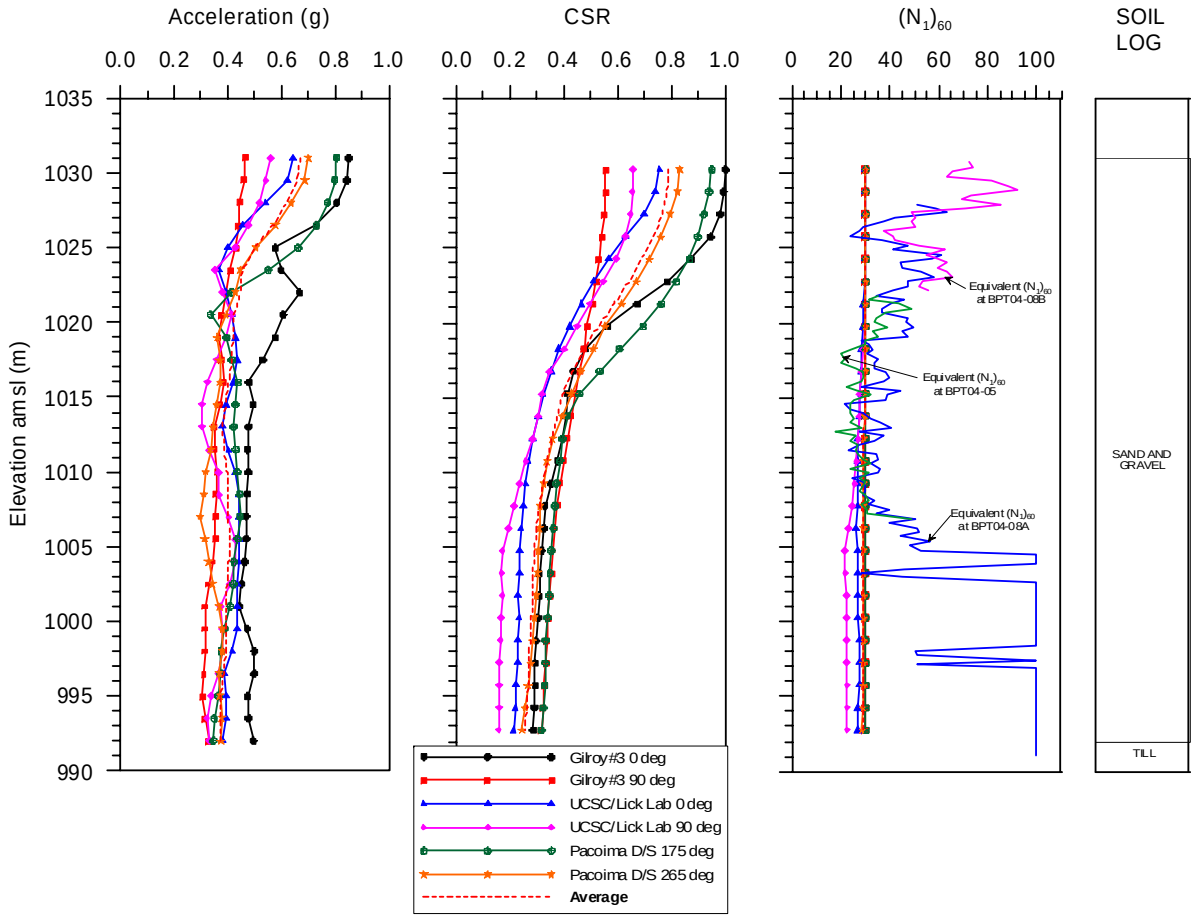


Figure VI-12 Liquefaction Assessment at BPT04-08 for Condition through the Toe of the Intermediate Dam

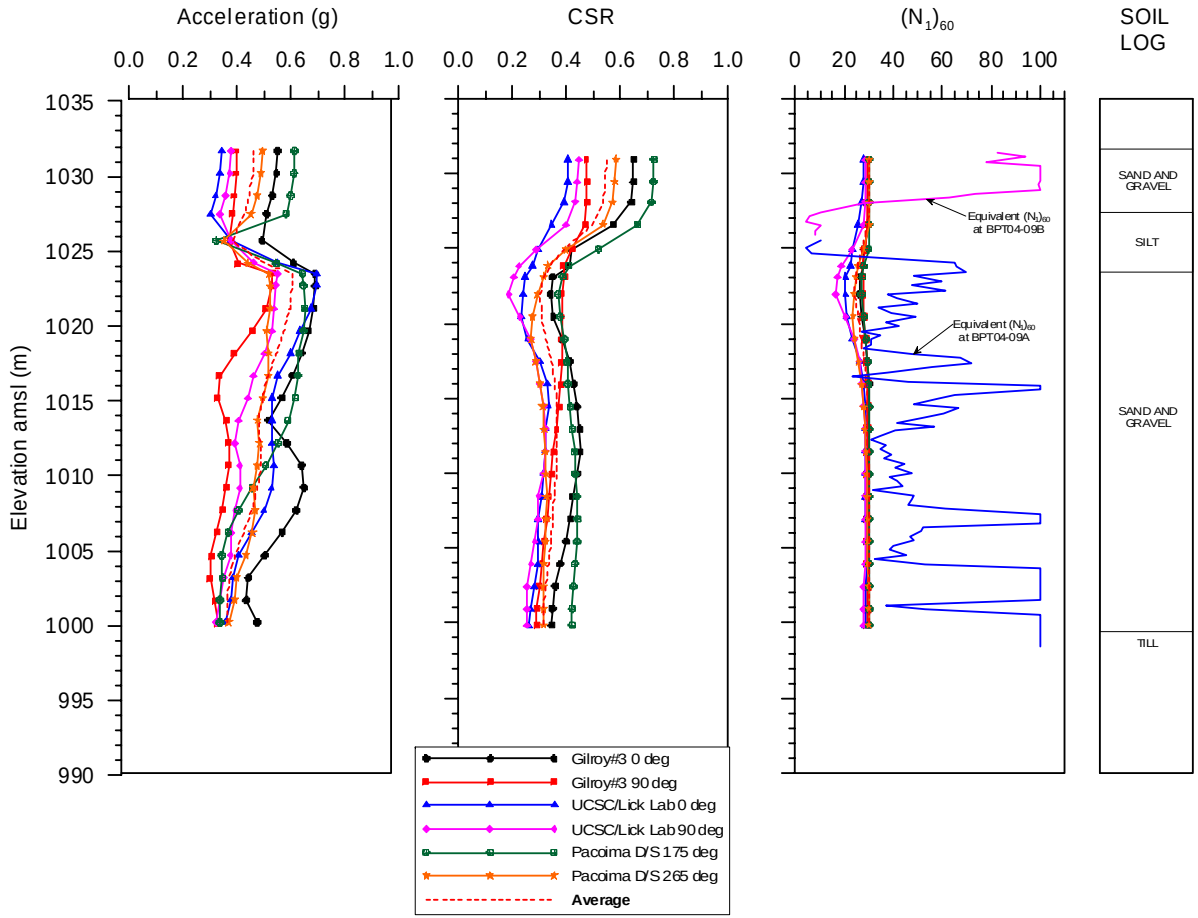


Figure VI-13 Liquefaction Assessment at BPT04-09 for Condition through the Toe of the Intermediate Dam

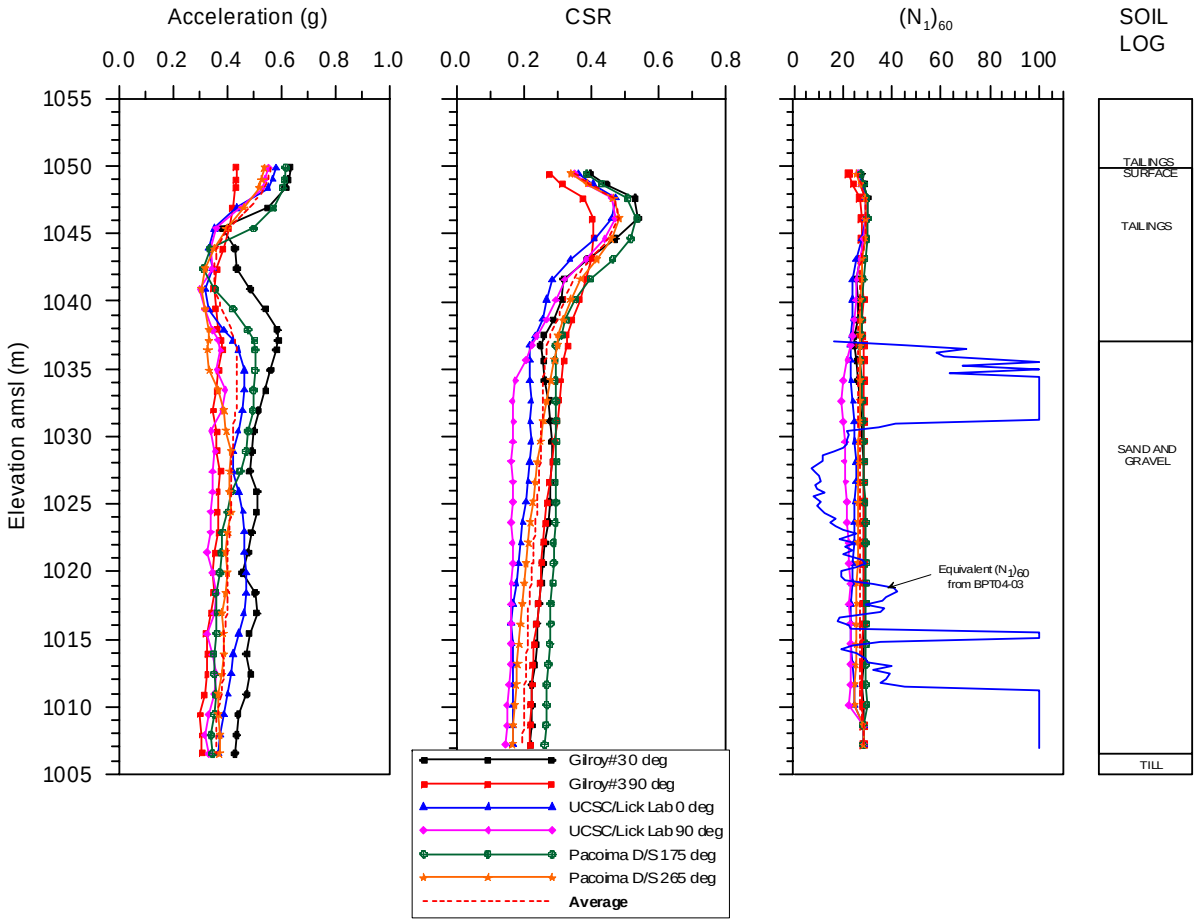


Figure VI-14 Liquefaction Assessment at BPT04-03 for Condition with the Tailings at the Secondary Dam

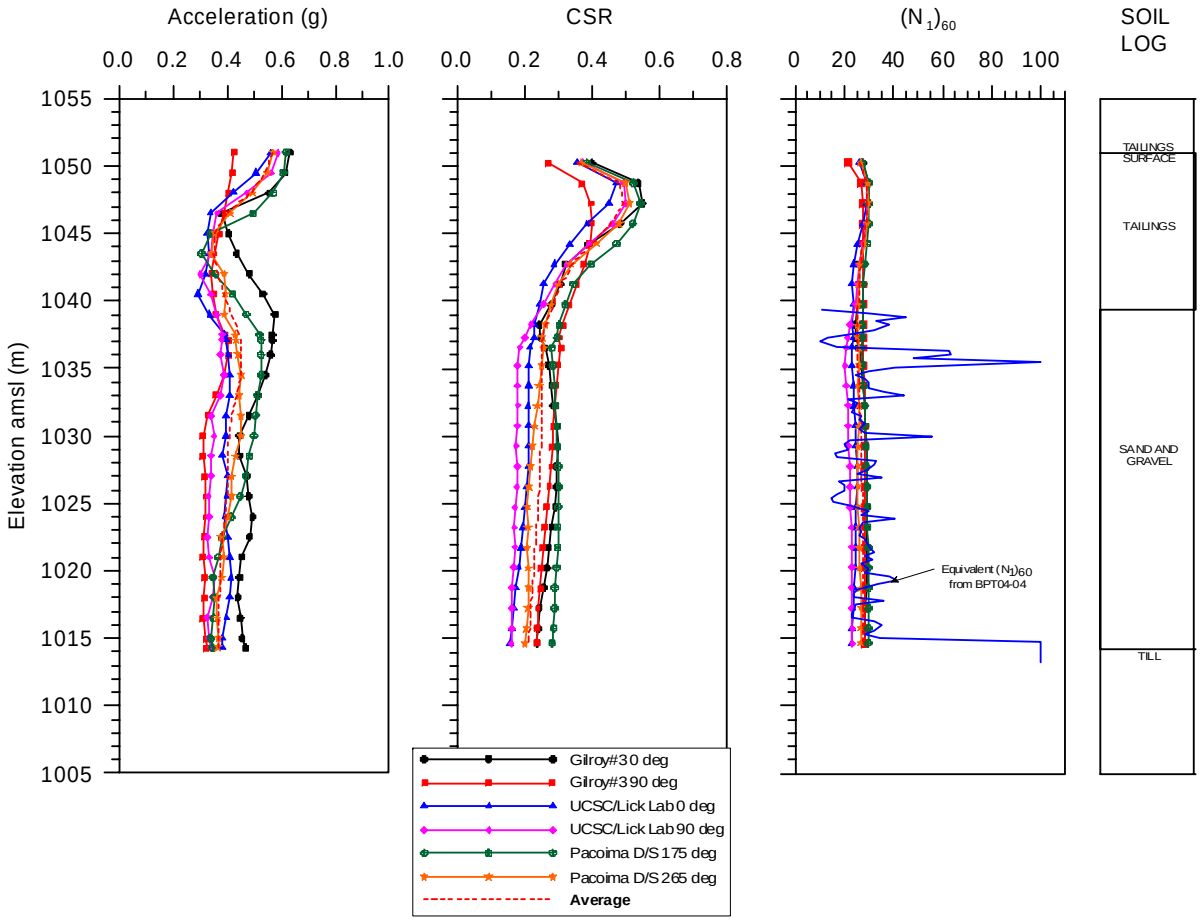


Figure VI-15 Liquefaction Assessment at BPT04-04 for Condition with the Tailings at the Secondary Dam

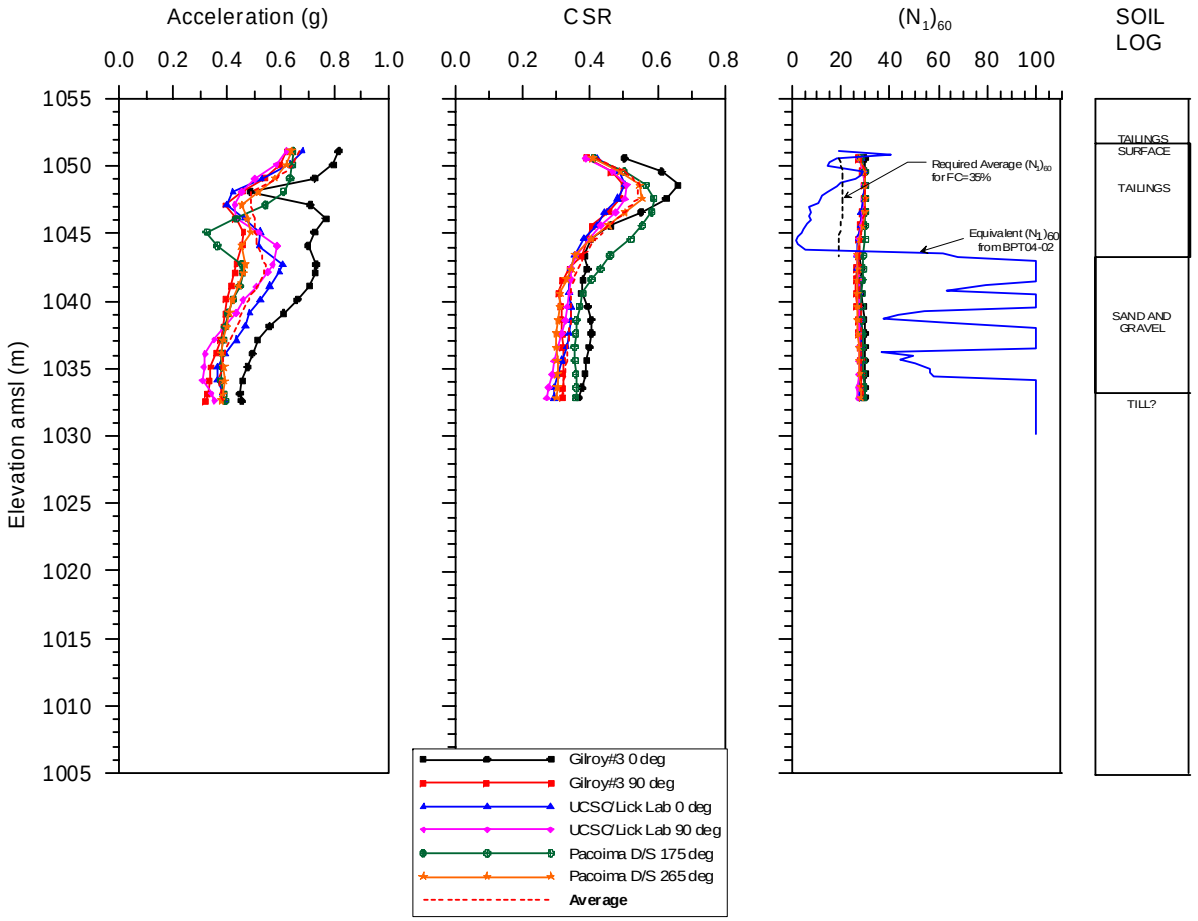


Figure VI-16 Liquefaction Assessment at BPT04-02 for Condition with the Tailings at the Secondary Dam

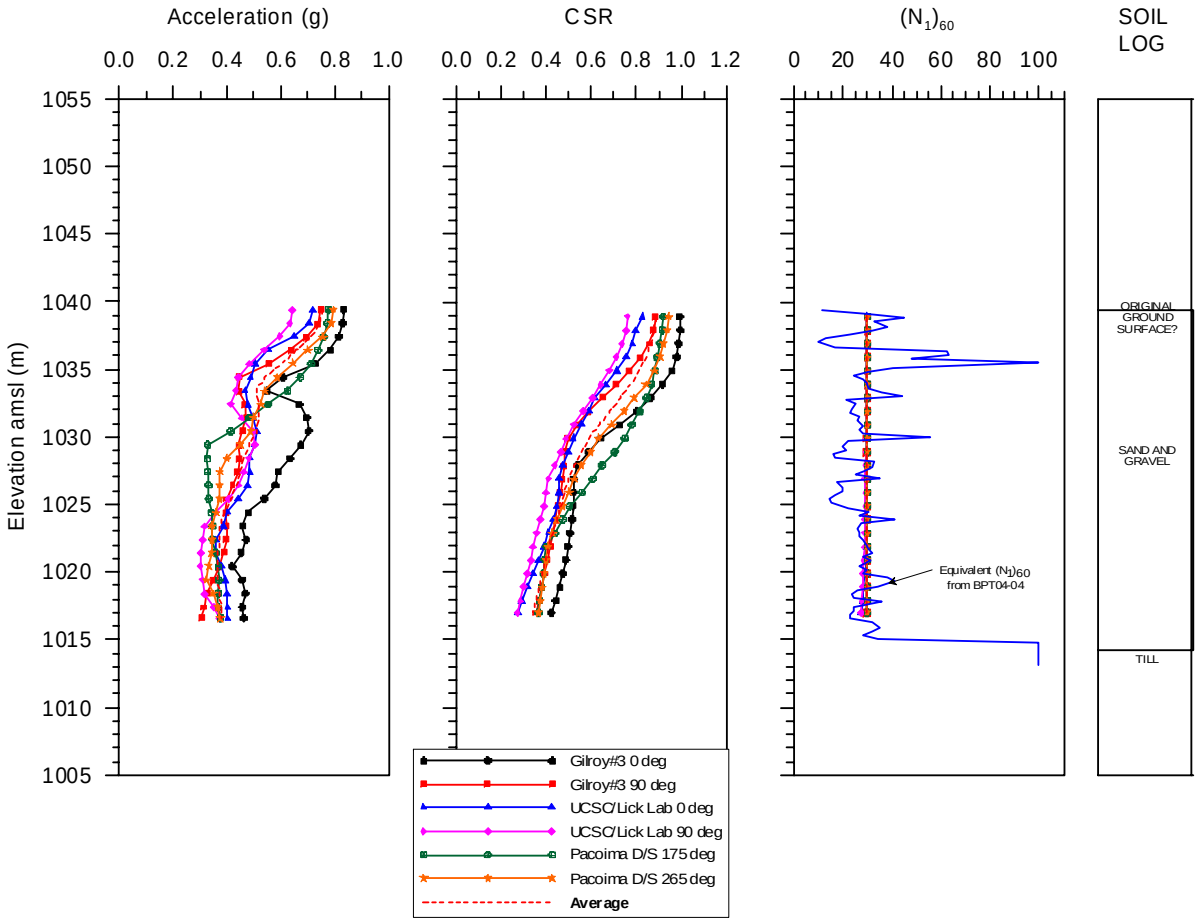


Figure VI-17 Liquefaction Assessment at BPT04-03 for Condition without the Tailings at the Secondary Dam

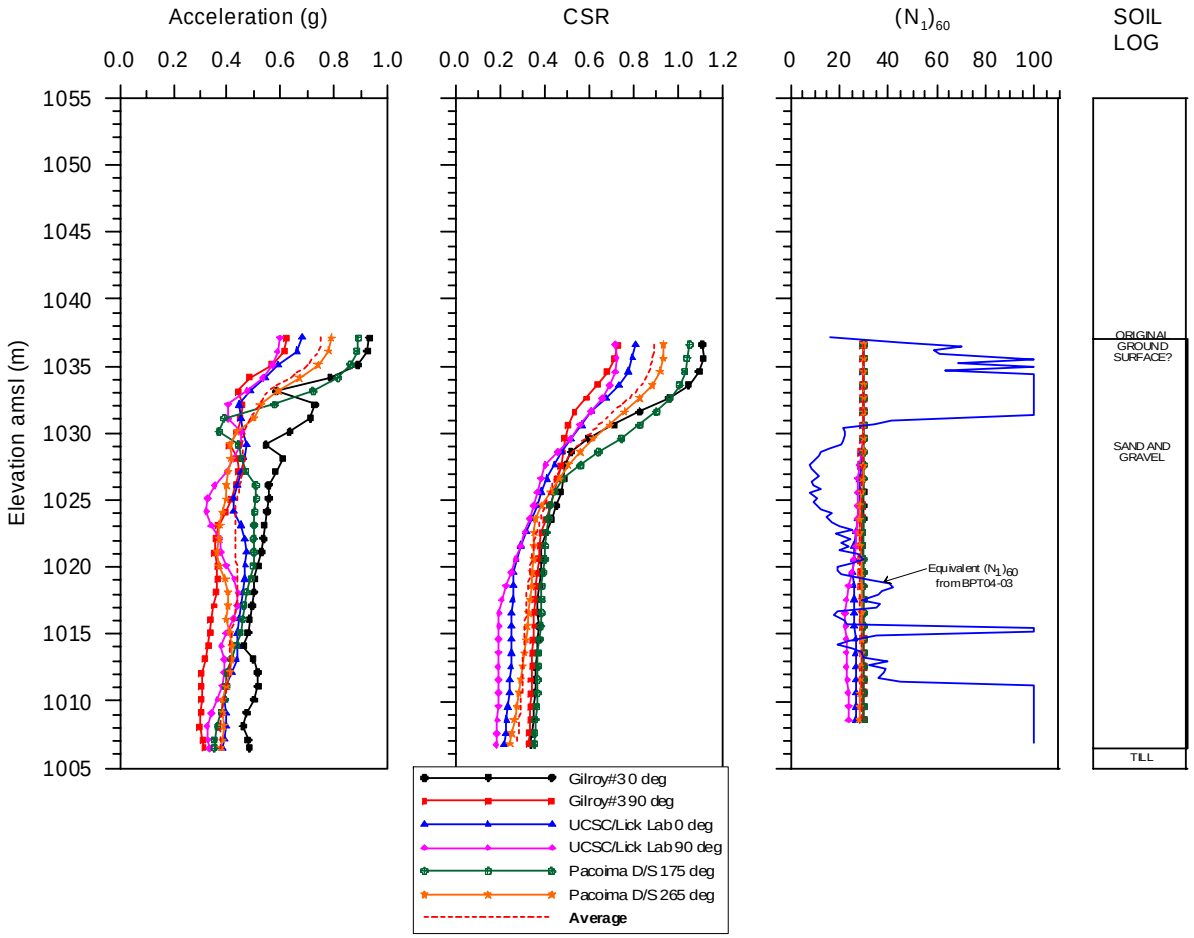


Figure VI-18 Liquefaction Assessment at BPT04-04 for Condition without the Tailings at the Secondary Dam

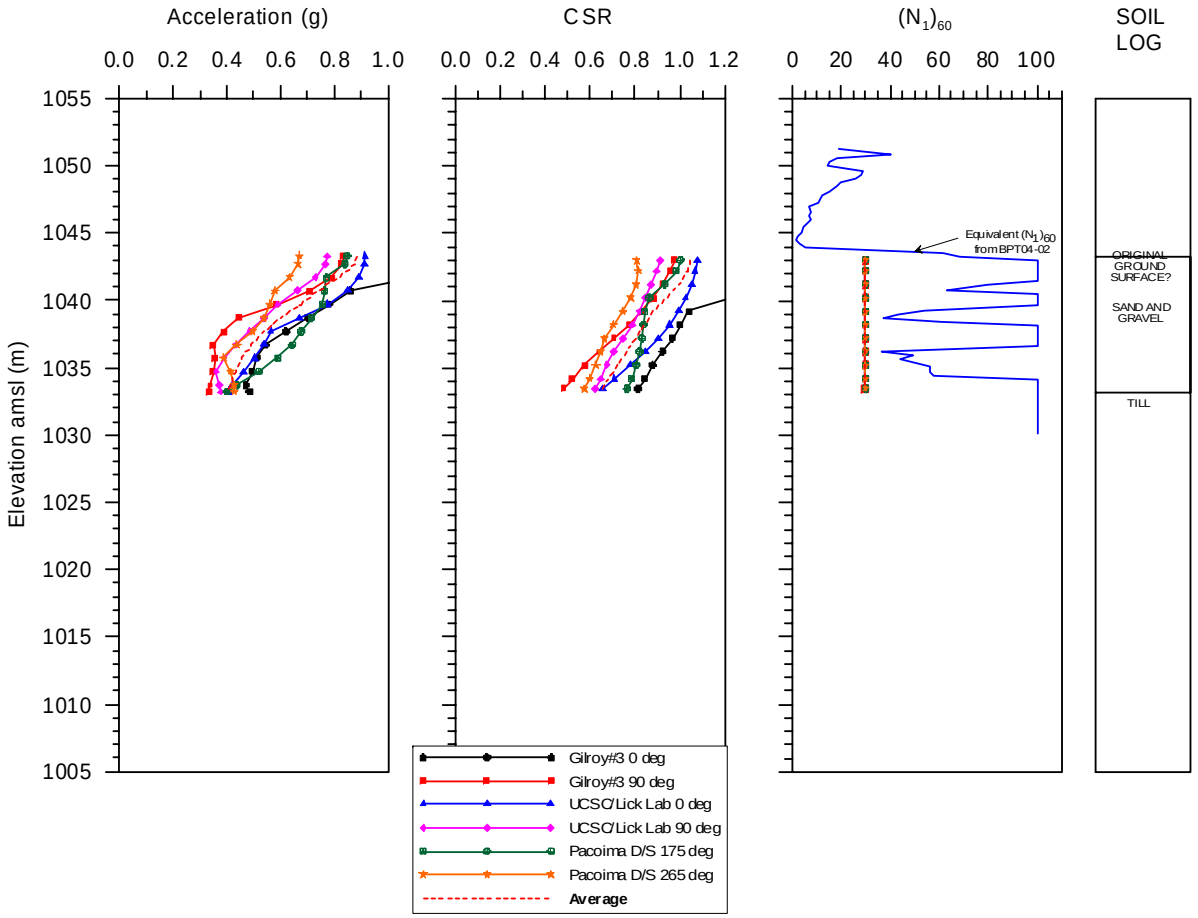


Figure VI-19 Liquefaction Assessment at BPT04-02 for Condition without the Tailings at the Secondary Dam

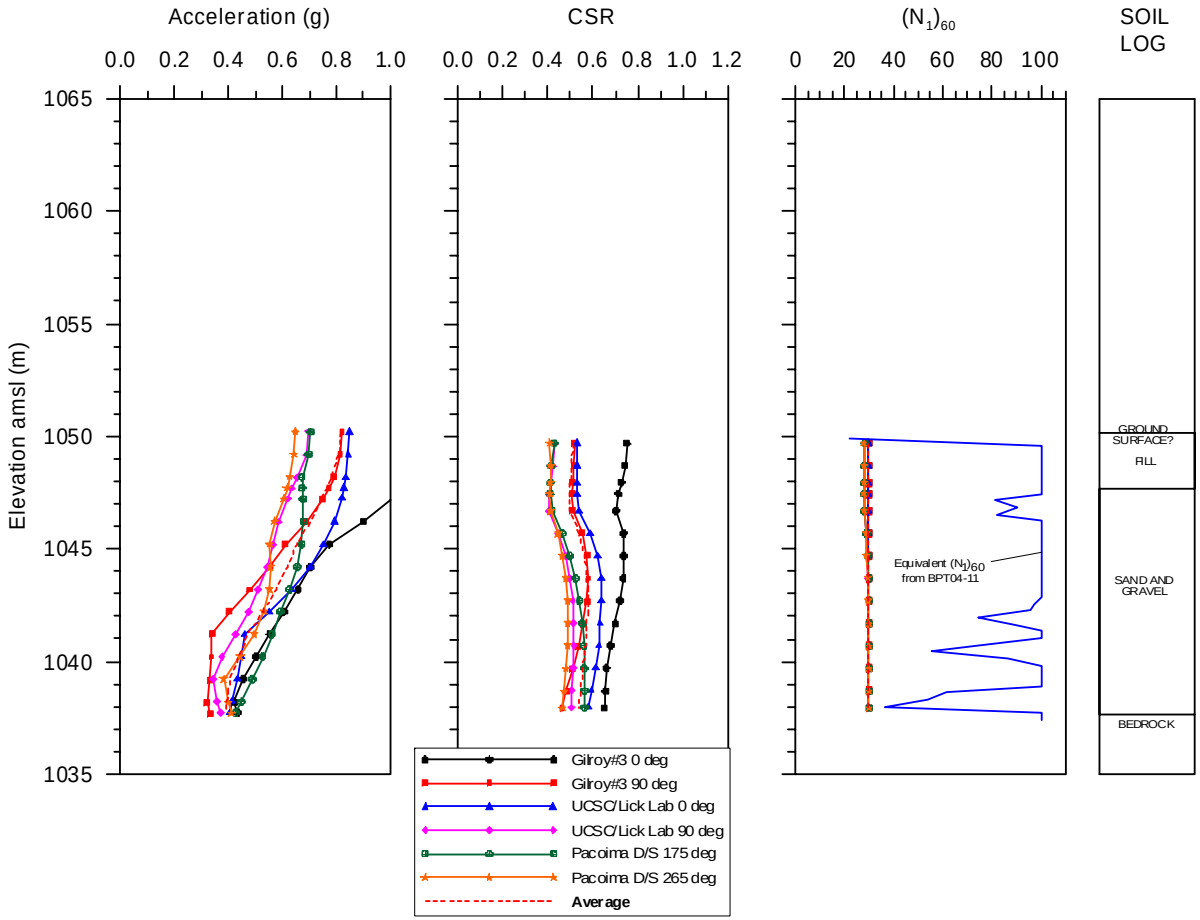


Figure VI-20 Liquefaction Assessment at BPT04-11 for As-Drilled Condition at the Secondary Dam East Limb

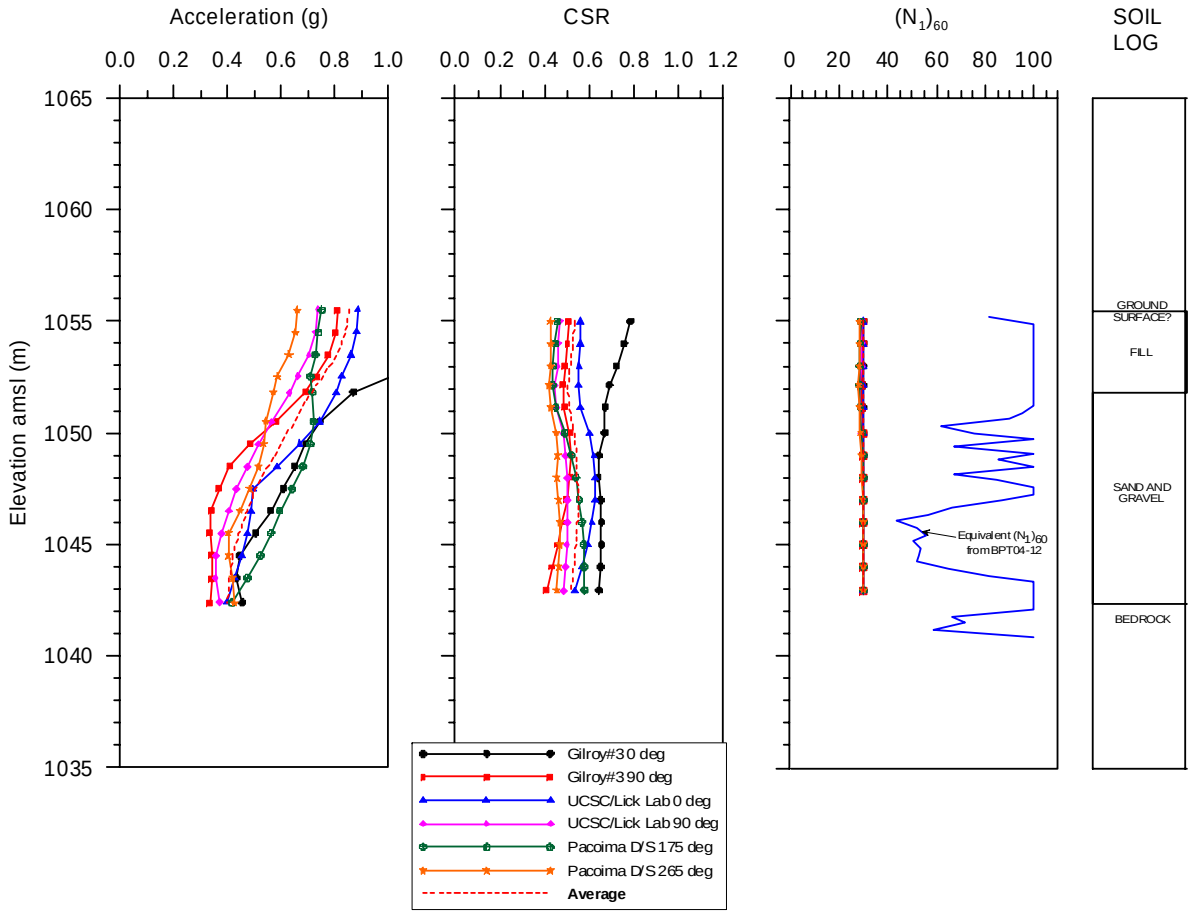


Figure VI-21 Liquefaction Assessment at BPT04-12 for As-Drilled Condition at the Secondary Dam East Limb

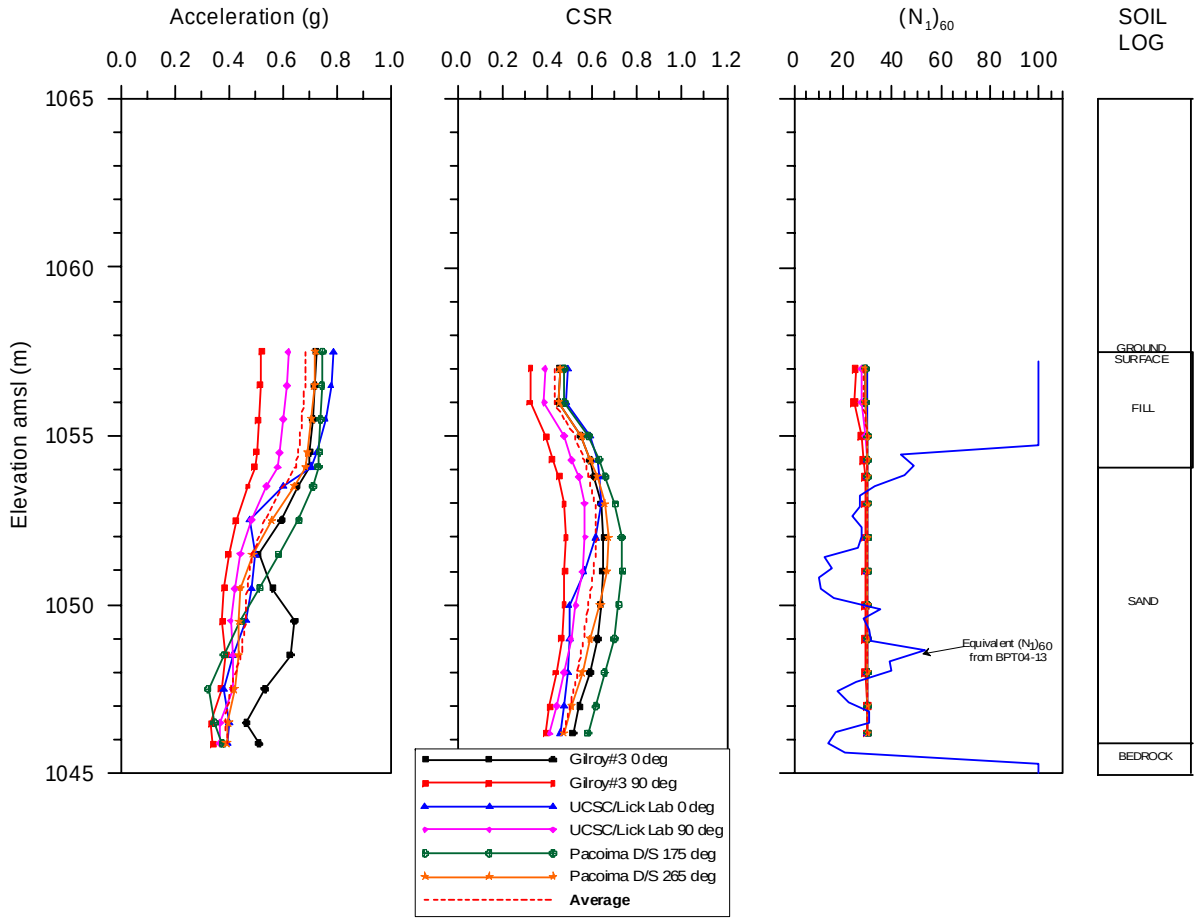


Figure VI-22 Liquefaction Assessment at BPT04-13 for As-Drilled Condition at the Secondary Dam East Limb

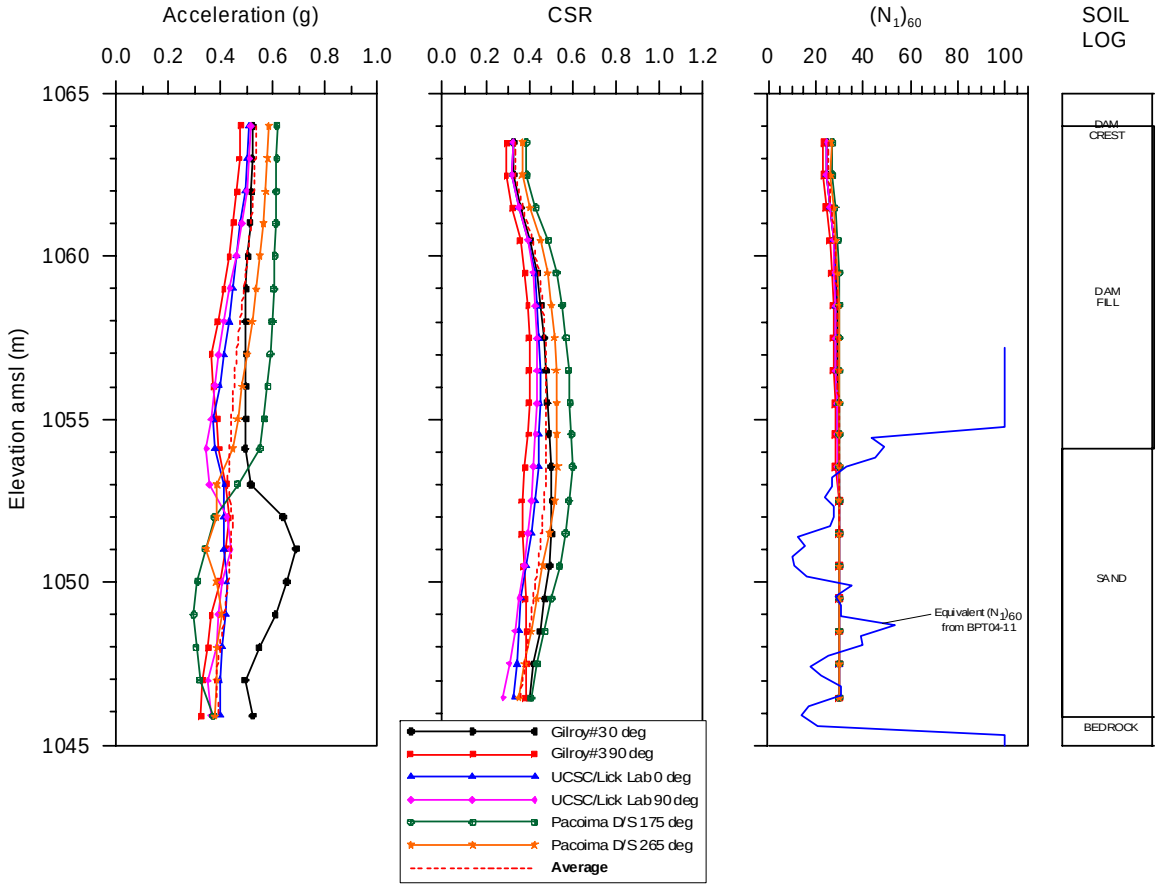


Figure VI-23 Liquefaction Assessment at BPT04-13 for Condition Through the Dam Crest at the Secondary Dam East Limb

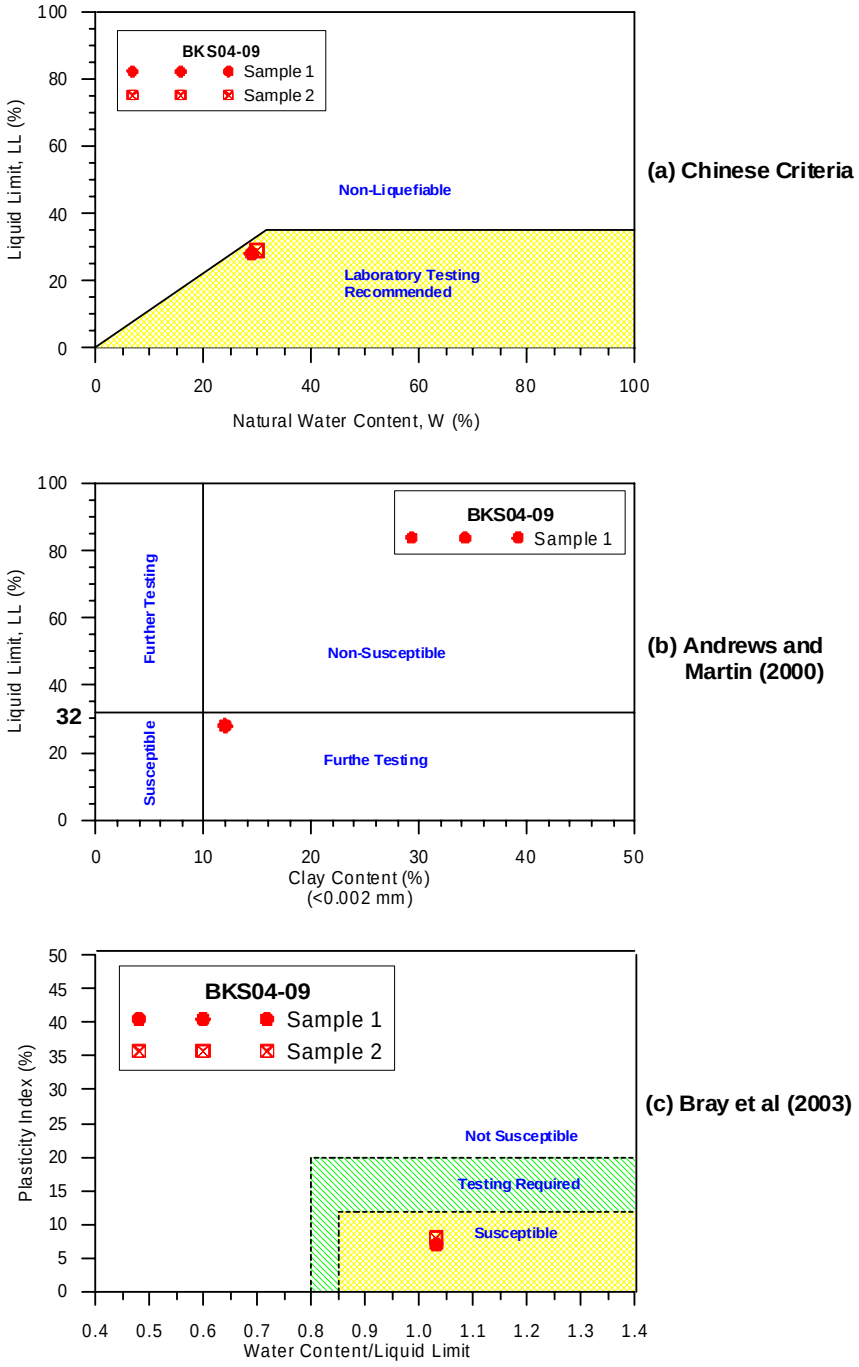


Figure VI-24 Liquefaction Assessment of Silt Samples from BKS04-09 Using the Fine Grained Soils Criteria

APPENDIX VII

Stability Analyses and Seismic Displacements

TABLE OF CONTENTS

Page

VII-1. INTRODUCTION 1

VII-2. STABILITY ANALYSES OF UNREMEDIATED SECTIONS 1

VII-2.1 Soil Properties 2

VII-2.2 Results of Post-Liquefaction Stability Analyses 4

VII-2.3 Seismic Permanent Displacements 4

VII-3. STABILITY ANALYSES OF REMEDIATED SECTIONS 7

VII-3.1 Soil properties 7

VII-3.2 Results of Post-Liquefaction Stability Analyses of Remediated Sections ...8

VII-3.3 Sensitivity Analyses on the Stability of the Intermediate Dam 9

VII-3.4 Sensitivity Analyses on the Stability of the Secondary Dam 10

TABLES

Table VII-1 Summary of Stability Analyses of Unremediated Section

Table VII-2 Soil Properties Used in the Post-Liquefaction Stability Analyses of the Intermediate Dam

Table VII-3 Soil Properties Used in the Post-Liquefaction Stability Analyses of the Secondary Lam West Limb Section

Table VII-4 Soil Properties Used in the Post-Liquefaction Stability Analyses of the Secondary Dam East Limb Section

Table VII-5 Post-Liquefaction Factors of Safety of Unremediated Sections of the Intermediate and the Secondary Dam

Table VII-6 Permanent Seismic Displacements by Various Sliding Block Methods for PGA of 0.5g and M7.1

Table VII-7 Soil Properties of Improved Soils and Compacted Berm

Table VII-8 Post-Liquefaction Factors of Safety of Remediated Sections of the Secondary Dam

FIGURES

Figure VII-1 Simplified Model of the Intermediate Dam with the Downstream Pond Used in the Post-Liquefaction Stability Analyses in Case IU1

Figure VII-2 Simplified Model of the Intermediate Dam without the Downstream Pond Used in the Post-Liquefaction Stability Analyses in Case IU2

Figure VII-3 Simplified Model of the Secondary Dam (West Limb) with the Downstream Tailings Used in the Post-Liquefaction Stability Analyses in Case SU1
Figure VII-4 Simplified Model of the Secondary Dam (West Limb) without the Downstream Tailings Used in the Post-Liquefaction Stability Analyses in Case SU2
Figure VII-5 Simplified Model of the Secondary Dam (East Limb) Used in the Post-Liquefaction Stability Analyses in Case SU3
Figure VII-6 Relationship between the Residual Strength Ratio S_{ur}/σ_{vo}' and SPT $(N_1)_{60cs}$ Proposed by Idriss (2004)
Figure VII-7 Results from the Stability Analysis of the Intermediate Dam with the Downstream Pond (Case IU1)
Figure VII-8 Results from the Stability Analysis of the Intermediate Dam without the Downstream Pond (Case IU2)
Figure VII-9 Results from the Stability Analysis of the Secondary Dam (West Limb Section) with the Downstream Tailings (Case SU1
Figure VII-10 Results from the Stability Analysis of the Secondary Dam (West Limb Section) without the Downstream Tailings (Case SU2)
Figure VII-11 Results from the Stability Analysis of the Secondary Dam (East Limb Section) (Case SU3)
Figure VII-12 Model of the Secondary Dam (West Limb Section) with Remedial Option 1 (Case SR1)
Figure VII-13 Model of the Secondary Dam (West Limb Section) with Remedial Option 2 (Case SR2)
Figure VII-14 Model of the Secondary Dam (East Limb Section) with Remedial Option 1 (Case SR3)
Figure VII-15 Model of the Secondary Dam (East Limb Section) with Remedial Option 2 (Case SR4)
Figure VII-16 Results from the Stability Analysis of the Secondary Dam (West Limb Section) with Remedial Option 1 (Case SR1)
Figure VII-17 Results from the Stability Analysis of the Secondary Dam (West Limb Section) with Remedial Option 2 (Case SR2
Figure VII-18 Results from the Stability Analysis of the Secondary Dam (East Limb Section) with Remedial Option 1 (Case SR3)
Figure VII-19 Results from the Stability Analysis of the Secondary Dam (East Limb Section) with Remedial Option 2 (Case SR3)
Figure VII-20 Results from the Sensitivity Analysis of the Intermediate Dam with Downstream Pond
Figure VII-21 Results from the Sensitivity Analysis of the Intermediate Dam with Liquefied Silt Layer in the Foundation (M-P FOS=0.54)
Figure VII-22 Results from the Sensitivity Analysis of the Secondary Dam with Downstream Tailings (M-P FOS=1.48)

APPENDIX VII
STABILITY ANALYSES AND SEISMIC DISPLACEMENTS

VII-1. INTRODUCTION

In the site response analyses and liquefaction evaluation described in Appendix VI, certain zones within the foundation under the Intermediate Dam and Secondary Dams were identified as susceptible to liquefaction. The tailings retained by these two dams are also assumed to liquefy during the design earthquake. Therefore, limit equilibrium stability analyses were carried out to determine the potential for flow slide and the factor of safety (FOS) of critical sections through the Intermediate Dam and the Secondary Dam under post-liquefaction conditions. The computer program SLOPE/W (Geo-Slope, 2003) and the Morgenstern Price method were used in the analyses.

VII-2. STABILITY ANALYSES OF UNREMEDIATED SECTIONS

Analyses were performed for the following conditions:

- Intermediate Dam with and without downstream pond.
- Secondary Dam with and without downstream tailings.

Table VII-1 summarizes the analyses carried out on the unremediated section of the Intermediate and the Secondary dams. Figures VII-1 to VII-5 shows the models of the dam used in the limit equilibrium analyses of Cases IU1, IU2, IU3, SU1, SU2 and SU3, respectively.

Table VII-1 Summary of Stability Analyses of Unremediated Section

Dam	Section[a]	Liquefaction Potential	Condition	Case
Intermediate Dam	Through BPT04-01&10 (through Rose Creek?)	Marginal liquefaction in the foundation sand and gravel with $22<(N_1)_{60}<30$	With D/S pond	IU1
			Without D/S pond	IU2
Secondary Dam- West Limb	Through BPT04-03 (through Rose Creek?)	Liquefaction in foundation sand and gravel with $(N_1)_{60}$ ~10 and marginal liquefaction in foundation sand and gravel with $22<(N_1)_{60}<30$	With D/S tailings	SU1
			Without D/S tailings	SU2
Secondary Dam- East Limb	Through BPT04-13 (at the foot of a creek?)	Liquefaction in foundation sand with $(N_1)_{60}$ ~10 and marginal liquefaction in foundation sand with $22<(N_1)_{60}<30$	No D/S Tailings	SU3

[a]See Figure 4.1 for the approximate location of sections

VII-2.1 Soil Properties

The properties of various soil units used in the post-liquefaction limit equilibrium analyses are listed in Table VII-2. Post-liquefaction residual strengths were assigned to loose foundation sand and gravel or sand layers that had $(N_1)_{60}$ of about 10. The ratio of residual strength over the initial effective overburden stress ratio, S_{ur}/σ_{vo}' was taken as 0.1 based on the relationship between the S_{ur}/σ_{vo}' and the clean sand SPT $(N_1)_{60cs}$ proposed by Idriss (2004). Idriss (2004) relationship is shown in Figure IV-6. For the dense foundation sand and gravel layers with SPT $(N_1)_{60cs}$ greater than 22 that showed marginal liquefaction, the strengths was not reduced and friction angle of 35 degrees was assumed in the limit equilibrium analyses.

The tailings was assumed to liquefy and a post –liquefaction residual strength ratio, S_{ur}/σ_{vo}' of 0.05 was used in the post-liquefaction stability analyses. Tables VII-2, VII-3 and VII-4 lists the soil properties used in the post-liquefaction limit equilibrium analyses.

Table VII-2 Soil Properties Used in the Post-Liquefaction Stability Analyses of the Intermediate Dam

Soil Unit	Unit Weight, γ (kN/m^3)	Friction, ϕ (deg)	Cohesion, C (kPa)	Post-Liquefaction Residual Strength Ratio, S_{ur}/σ_{vo}'
Dam Core and Fill	21	35	0	-
Unsaturated Tailings	19.5	30	0	-
Saturated Tailings	24	-	-	0.05
Foundation Sand and Gravel Layer 1 with $(N_1)_{60cs} > 22$	21	35	0	-
Foundation Sand and Gravel Layer 2 with $(N_1)_{60cs} > 30$	21	35	0	-

[a]See Figure 4.1 for location of BPT holes

Table VII-3 Soil Properties Used in the Post-Liquefaction Stability Analyses of the Secondary Lam West Limb Section

Soil Unit	Unit Weight, γ (kN/m^3)	Friction, ϕ (deg)	Cohesion, C (kPa)	Post-Liquefaction Residual Strength Ratio, S_{ur}/σ_{vo}'
Dam Core and Fill	21	35	0	-
Unsaturated Tailings	19.5	30	0	-
Liquefied Saturated Tailings	24	-	-	0.05
Foundation Sand and Gravel Layer 1 with $(N_1)_{60cs} > 22$	21	35	0	-
Liquefied Foundation Sand and Gravel Layer 2 with $(N_1)_{60cs}$ ~10	21	-	-	0.10
Foundation Sand and Gravel Layer 3 with $(N_1)_{60cs} > 22$	21	35	0	-

Table VII-4 Soil Properties Used in the Post-Liquefaction Stability Analyses of the Secondary Dam East Limb Section

Soil Unit	Unit Weight, γ (kN/m^3)	Friction, ϕ (deg)	Cohesion, C (kPa)	Post-Liquefaction Residual Strength Ratio, S_{ur}/σ_{vo}'
Dam Fill	21	35	0	-
Unsaturated Tailings	19.5	30	0	-
Liquefied Saturated Tailings	24	-	-	0.05
Foundation Sand and Gravel Layer 1 with $(N_1)_{60cs} > 30$	21	35	0	-
Liquefied Foundation Sand Layer 1 with $(N_1)_{60cs}$ ~10	20	-	-	0.10
Foundation Sand Layer 1 with $(N_1)_{60cs} > 22$	20	35	0	-

VII-2.2 Results of Post-Liquefaction Stability Analyses

Figures VII-7 to VII-11 show the slip surfaces for the unremediated dam sections and Table VII-4 lists the factors of safety.

Table VII-5 Post-Liquefaction Factors of Safety of Unremediated Sections of the Intermediate and the Secondary Dam

Dam	Section	Condition	Case	Factor of Safety (Morgenstern-Price)	Reference Figure
Intermediate Dam	Through BPT04-01&10 (through old Rose Creek?)	With D/S pond	IU1	1.47[a]	VII-7
		Without D/S pond	IU2	1.76[b]	VII-8
Secondary Dam- West Limb	Through BPT04-03 (through old Rose Creek?)	With D/S tailings	SU1	1.56[c]	VII-9
		Without D/S tailings	SU2	0.62	VII-10
Secondary Dam- East Limb	Through BPT04-13 (at the foot of an old creek?)	No D/S Tailings	SU3	0.78	VII-11

[a]FOS>1.5 for slip surfaces through the crest and foundation of the dam and FOS of shallower slip surfaces are smaller

[b]FOS>1.8 for or slip surface through the crest and foundation of the dam.

[C] FOS for slip surfaces within the D/S slope not extending to the foundation are slightly smaller.

Flow slide and significant displacements are not expected to occur in Cases IU1, IU2 and SU1. However, flow slide will likely occur in Cases SU2 and SU3 as the post-liquefaction FOSs in these cases were less than 0.8. Therefore, remedial measures are required to stabilize the dam under the design earthquake loading.

VII-2.3 Seismic Permanent Displacements

Newmark sliding block type of analyses were carried out to estimate the permanent seismic displacements of the Intermediate and the Secondary Dams sections that were not predicted to undergo flow slide. For these sections, the yield acceleration, which is required to bring the post-liquefaction factor of safety to unity, was estimated using the computer program SLOPE/W (Geo-Slope, 2003) and the Morgenstern-Price method.

Newmark's (1965), Makdisi and Seed (1978) and Hynes and Franklin (1984) methods were then used to estimate the displacements.

The displacements were estimated for the mean PGA of 0.5 g suggested by Atkinson (2003) and for a design earthquake magnitude of M7.1 suggested by Atkinson (2003). In the Makdisi and Seed (1978) method, the crest acceleration of the dam was estimated using the upper bound of the amplification factors proposed by Harder (1991) and the ratio of average acceleration along the slip surface to the crest acceleration was estimated from the relationship proposed by Makdisi and Seed (1978) as a function of the depth of the failure surface. The amplification factor from Harder (1991) is about 1.4 for peak base acceleration of 0.5 g. In the Hynes and Franklin method (1984), an amplification factor of 2.2 and 2.6 were used in the estimation of mean and mean plus one standard deviation (σ) displacements, respectively. SHAKE analyses of 1-D columns described in Appendix VI showed that the maximum amplification factor is about 2.4 and the average is about 1.5. The calculated displacements are listed in Table VII-8.

Table VII-6 Permanent Seismic Displacements by Various Sliding Block Methods for PGA of 0.5g and M7

Dam	Section[a]	Condition	Post-Liquefaction M-P FOS	Yield Acceleration (g)	Permanent Seismic Displacements (cm)				
					Newmark (1965) Upper Bound	Makdisi and Seed (1978)-Mean		Hynes and Franklin (1984) Mean	
						Lower Bound	Upper Bound	Mean	Mean+σ
Intermediate Dam	Through BPT04-01&10	With D/S pond	1.47	0.13	38	2	13	22	55
		Without D/S pond	1.76	0.25	5	<1	<1	<10	25
Secondary Dam- West Limb	Through BPT04-03	With D/S tailings	1.56	0.08	96	8	46	36	91

[a]See Figure 4.1 for location of BPT holes

VII-3. STABILITY ANALYSES OF REMEDIATED SECTIONS

The following two remedial options were considered to stabilize the dam in Cases SU2 and SU3:

Option 1: Insitu-densification to improve the foundation soils.

Option 2: Compacted downstream berm with earthquake drains in the potentially liquefiable foundation soils.

Figures VII-12 and VII-13 show the remediated sections of the Secondary Dam (West Limb) without the downstream tailings and with remedial Option 1 and 2, respectively. The Option 1 remediation comprises a 45 m wide densified zone in the loose foundation sand and gravel with $(N_1)_{60cs}$ of about 10. The Option 2 consists of a 50 m wide and 15 m high downstream berm and 70 m wide zone in the foundation with earthquake drains. The procedure to be used for the installation of earthquake drains is assumed not to increase density significantly.

Figures VII-14 and VII-15 show the similar remedial concepts for the Secondary Dam at the east limb. In this case, the Option 1 remediation comprises a 10 wide densified zone in the loose foundation sand with $(N_1)_{60cs}$ of about 10. The Option 2 consists of about 12 m wide and 3 m high downstream berm on the existing road fill and about 10 m wide zone in the foundation with earthquake drains but without any densification.

VII-3.1 Soil properties

In the stability analyses of the remediated sections of the Secondary Dam, the same properties listed in Table VII-2 were used for the unimproved soil units. Table VII-6 lists the properties that were used for the improved foundation soils and compacted berm.

Table VII-7 Soil Properties of Improved Soils and Compacted Berm

Secondary Dam	Remedial Option	Soil Unit	Unit Weight, γ (kN/m^3)	Friction, ϕ (deg)	Cohesion, C (kPa)	Excess Porewater Pressure Ratio, $\Delta u/\sigma_{vo}'$
West Limb without D/S Tailings	Option 1	Densified Foundation Sand and Gravel Layer 2	21	38	0	0
	Option 2	Compacted Berm	20	38	0	0
		Foundation Sand and Gravel Layer 2 with EQ Drains	21	30	0	0.5
East Limb	Option 1	Densified Foundation Sand Layer 1	21	38	0	0
	Option 2	Compacted Berm	20	38	0	0
		Foundation Sand and Gravel Layer 1 with EQ Drains	20	30	0	0.5

VII-3.2 Results of Post-Liquefaction Stability Analyses of Remediated Sections

A target FOS of 1.1 was selected for the remediated sections of the dam, which will ensure that flow slide will not occur under the design earthquake loading. Figures VII-16 to VII-19 show the slip surfaces and the Table VII-5 lists the minimum factors of safety.

Table VII-8 Post-Liquefaction Factors of Safety of Remediated Sections of the Secondary Dam

Secondary Dam	Section	Remedial Option	Case	Factor of Safety (Morgenstern-Price)	Reference Figure
West Limb without D/S Tailings	Through BPT04-03 (through old Rose Creek?)	Option 1	SR1	1.21	VII-16
		Option 2	SR2	1.13	VII-17
East Limb	Through BPT04-13 (at the foot of an old creek?)	Option 1	SR3	1.42	VII-18
		Option 2	SR4	1.12	VII-19

VII-3.3 Sensitivity Analyses on the Stability of the Intermediate Dam

Case IS1

A sensitivity analysis was carried out to check the stability of the intermediate dam with following assumptions:

1. Excess pore water pressure ratio of 50% was assumed in the foundation dense sand and gravel layers with $(N_1)_{60cs} > 22$ that showed marginal liquefaction.
2. Liquefied tailings was assumed to act as a heavy liquid with no shear strength.

Figure IV-20 shows the slip surface from the analysis of Intermediate Dam with downstream pond (Case IU1 above) repeated with the above assumptions. The minimum Morgenstern-Price FOS reduced to 1.14 from about 1.47.

Case IS2

Silt layer was encountered at BKS04-09. Liquefaction potential of this fined grained layer was evaluated using screening tools such as the Chinese criteria, Andrews and Martin criteria (2000) and Bray et al. (2003) criteria. The Chinese and Andrews and martin criteria for liquefaction was not satisfied as the clay content of a sample tested in the laboratory was slight high. However, the Bray et al. (2003) criteria suggested that the silt layer is susceptible to liquefaction.

An analysis was carried by assuming that a continuous liquefiable silt layer exists underneath the dam as shown in Figure IV-21. As no post-liquefaction strength data was available for this silt, the residual strength of this silt layer was assumed as 10% of the initial effective overburden stress (i.e. S_{ur}/σ_{vo}' of 0.1). In this analysis the foundation sand and gravel layers were treated as non-liquefiable and assigned a friction angle of 35 degrees. Figure IV-21 shows the slip surface with a minimum Morgenstern-Price FOS of 0.54.

Note that a review of geology using the available data including the adjacent borehole data from Golder Associates (1980) showed no strong evidence for the existence of a continuous silt layer underneath the dam.

VII-3.4 Sensitivity Analyses on the Stability of the Secondary Dam

A sensitivity analysis was carried out by treating the liquefied tailings upstream and downstream of the dam as a heavy liquid with no shear strength. Figure IV-22 shows the results of this stability analysis. The Morgenstern-Price FOS reduced to 1.49 from 1.52 in the corresponding Case SU1, in which a residual strength ratio, S_{ur}/σ_{vo}' of 0.05 was assigned for the liquefied tailings.

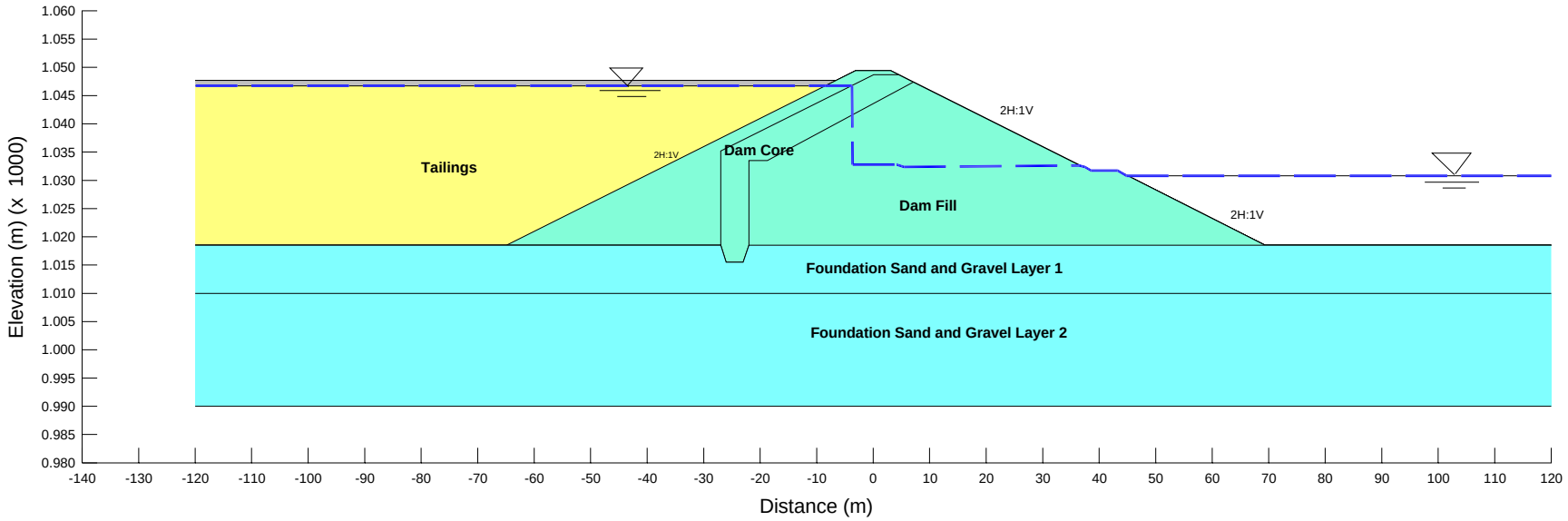


Figure VII-1 Simplified Model of the Intermediate Dam with the Downstream Pond Used in the Post-Liquefaction Stability Analyses in Case IU1

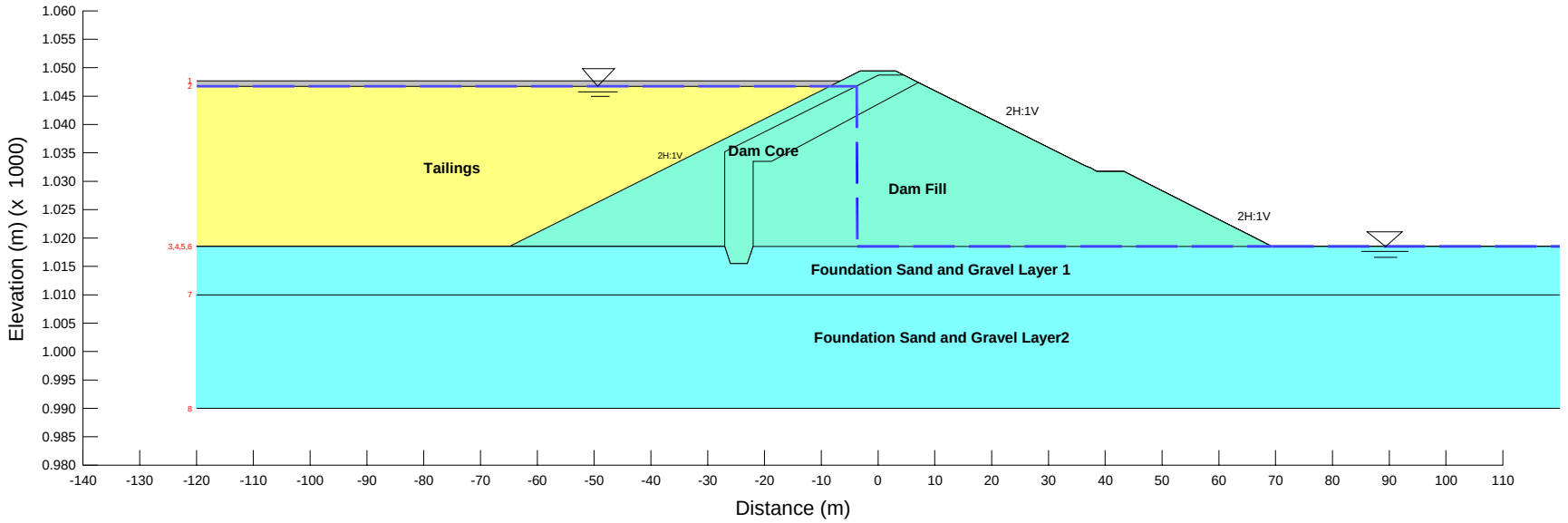


Figure VII-2 Simplified Model of the Intermediate Dam without the Downstream Pond Used in the Post-Liquefaction Stability Analyses in Case IU2

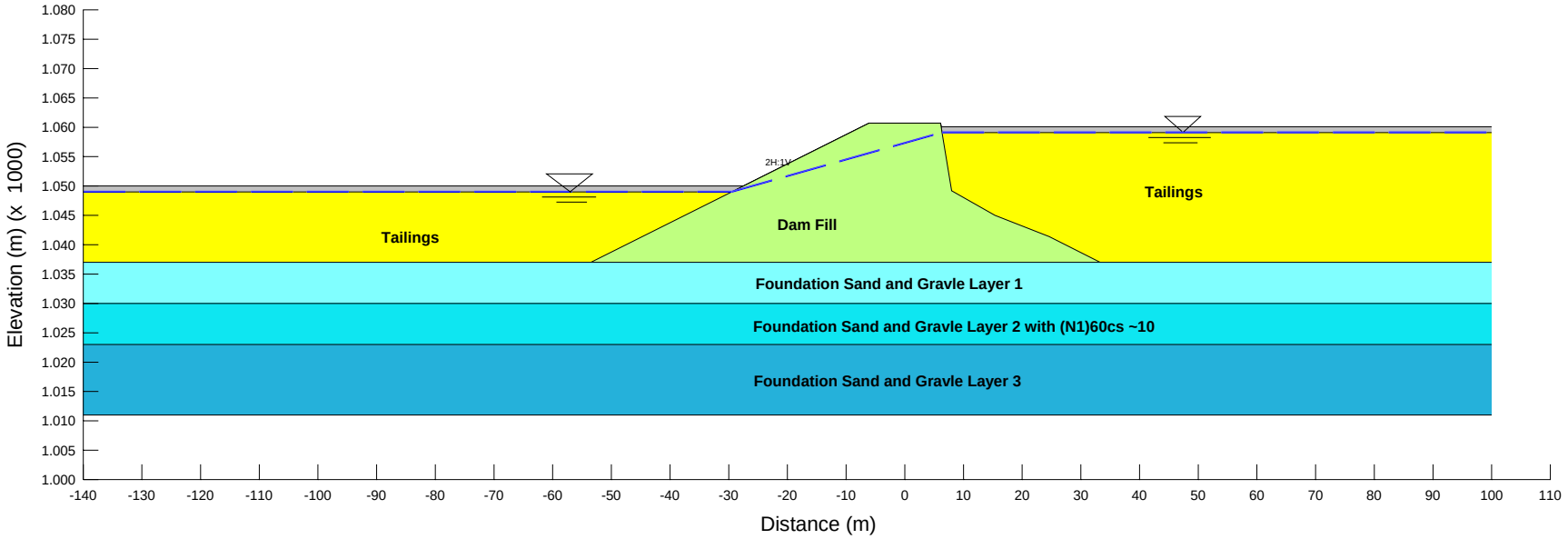


Figure VII-3 Simplified Model of the Secondary Dam (West Limb) with the Downstream Tailings Used in the Post-Liquefaction Stability Analyses in Case SU1

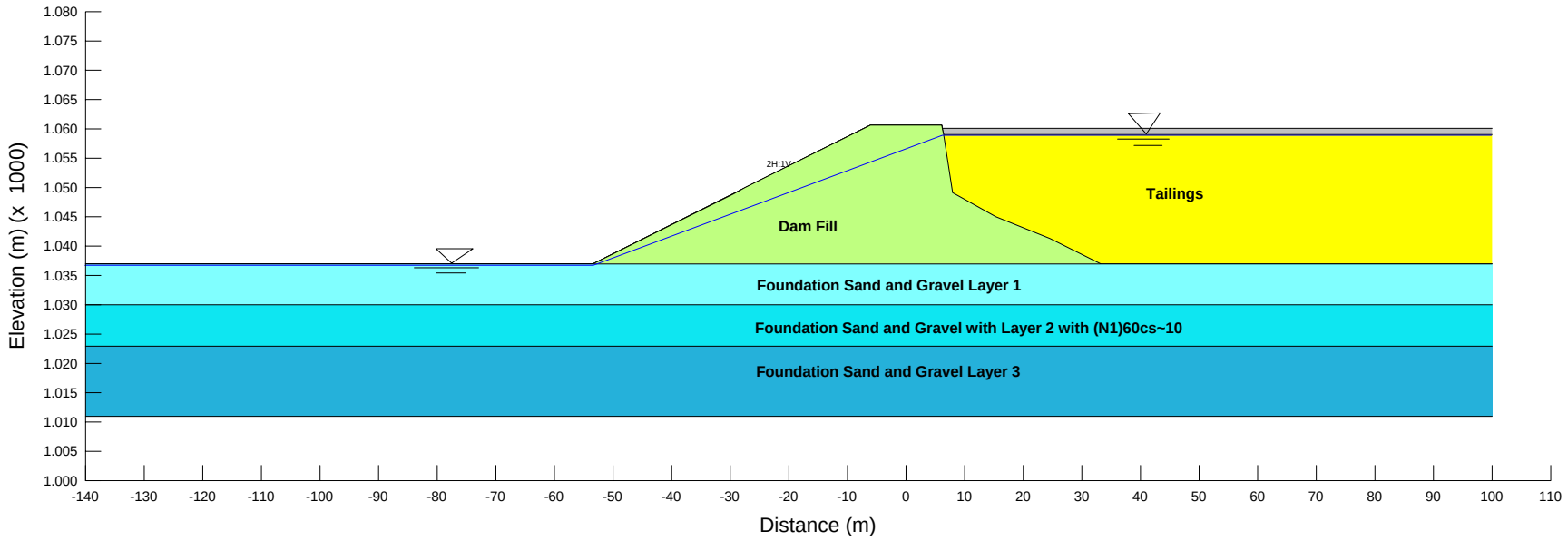


Figure VII-4 Simplified Model of the Secondary Dam (West Limb) without the Downstream Tailings Used in the Post-Liquefaction Stability Analyses in Case SU2

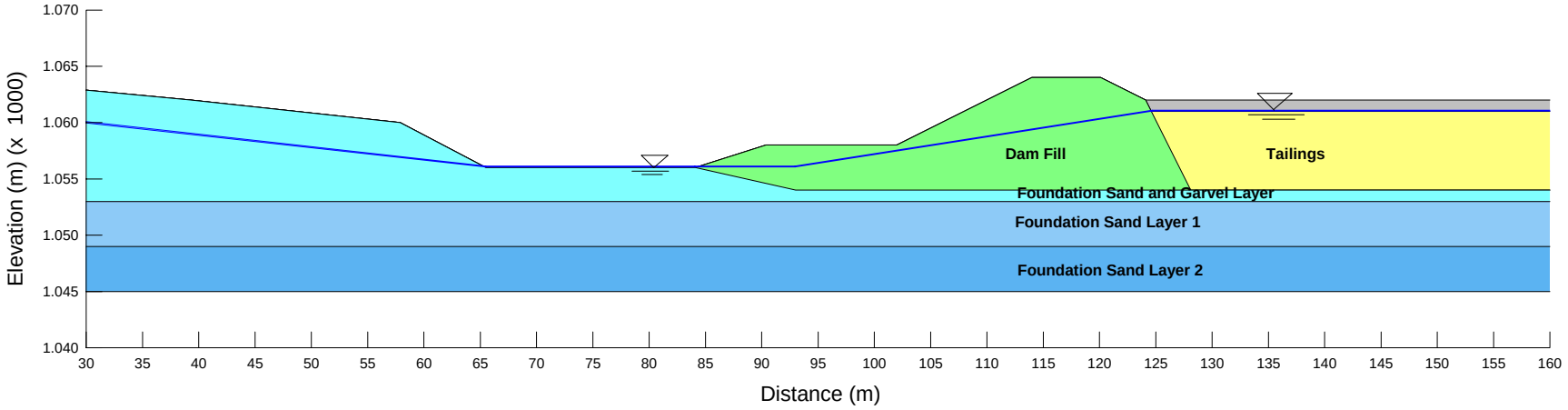


Figure VII-5 Simplified Model of the Secondary Dam (East Limb) Used in the Post-Liquefaction Stability Analyses in Case SU3

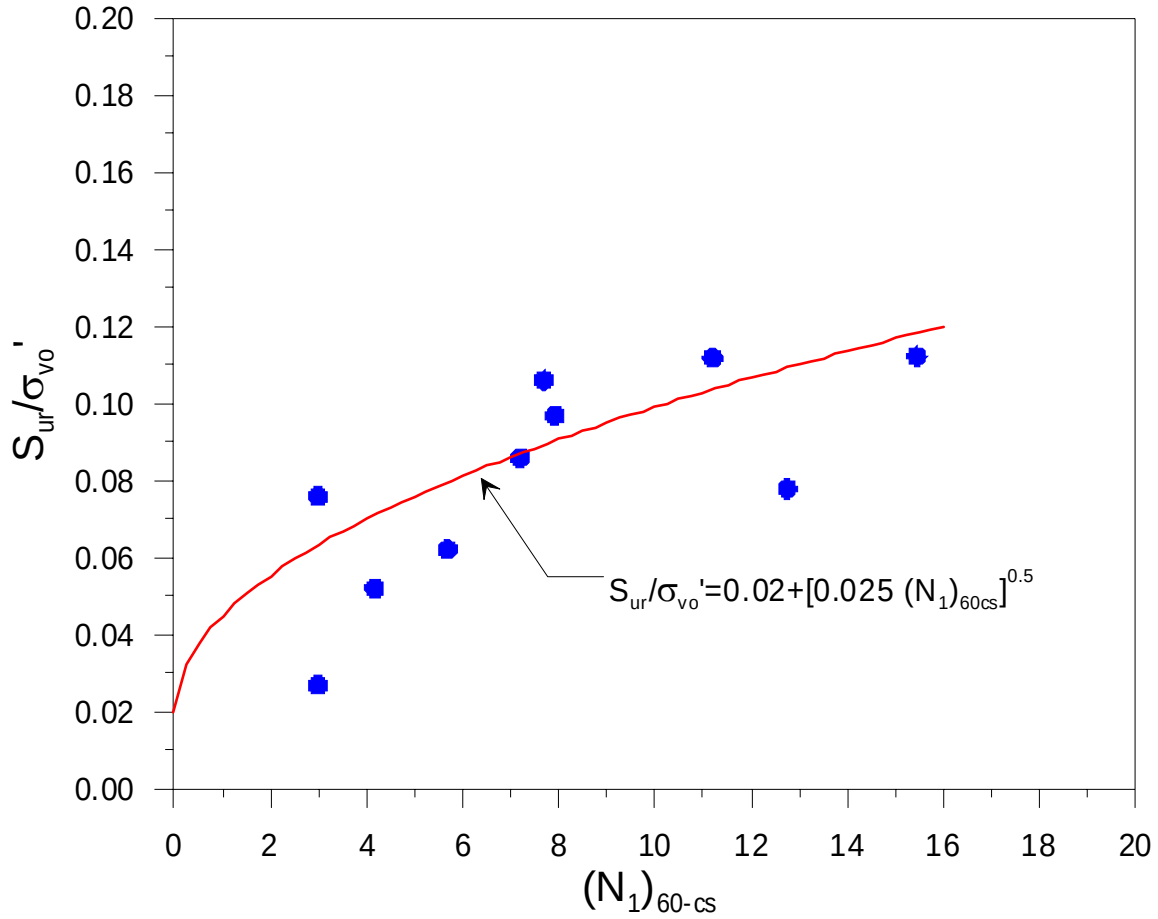


Figure VII-6 Relationship between the Residual Strength Ratio S_{ur}/σ_{vo}' and SPT $(N_1)_{60c}$s Proposed by Idriss (2004)

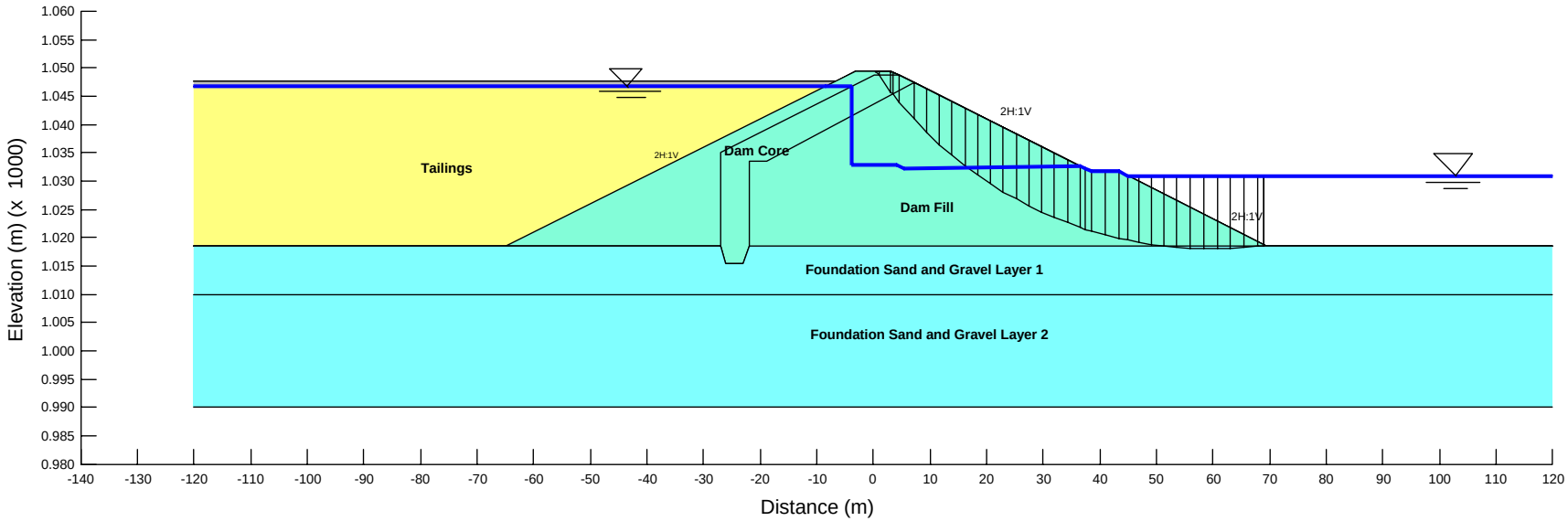


Figure VII-7 Results from the Stability Analysis of the Intermediate Dam with the Downstream Pond (Case IU1) (M-P FOS=1.47, For slip surfaces through foundation M-P FOS> 1.5)

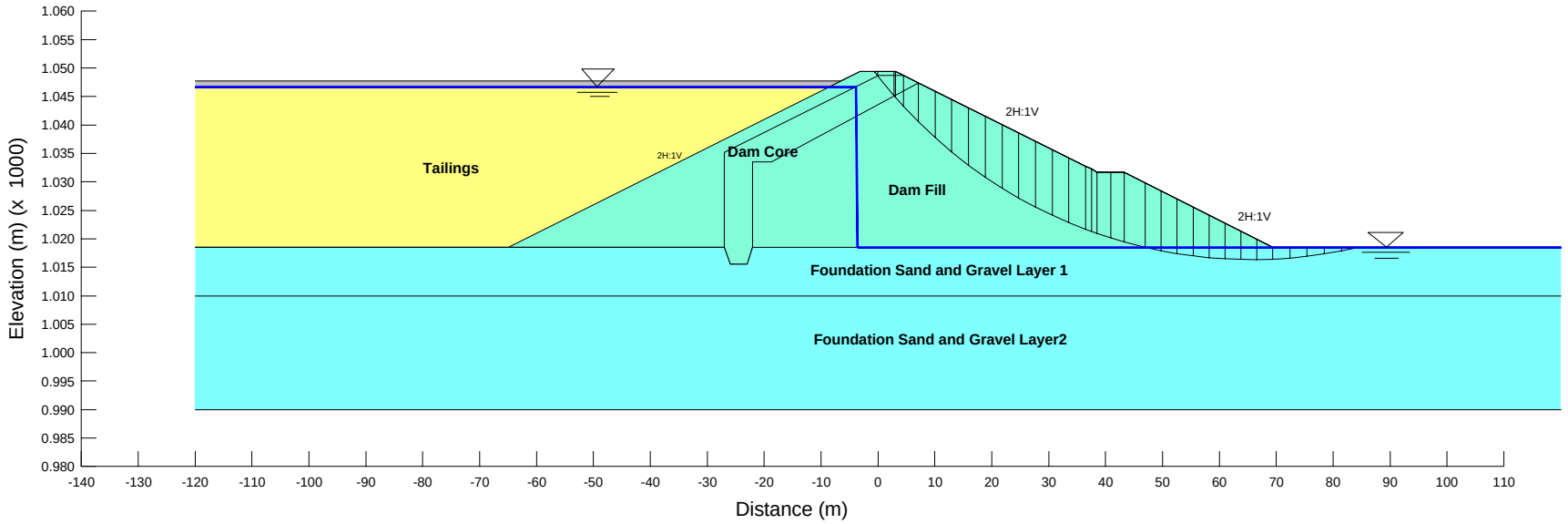


Figure VII-8 Results from the Stability Analysis of the Intermediate Dam without the Downstream Pond (Case IU2) (M-P FOS=1.76, For slip surfaces deep through foundation M-P FOS> 1.8)

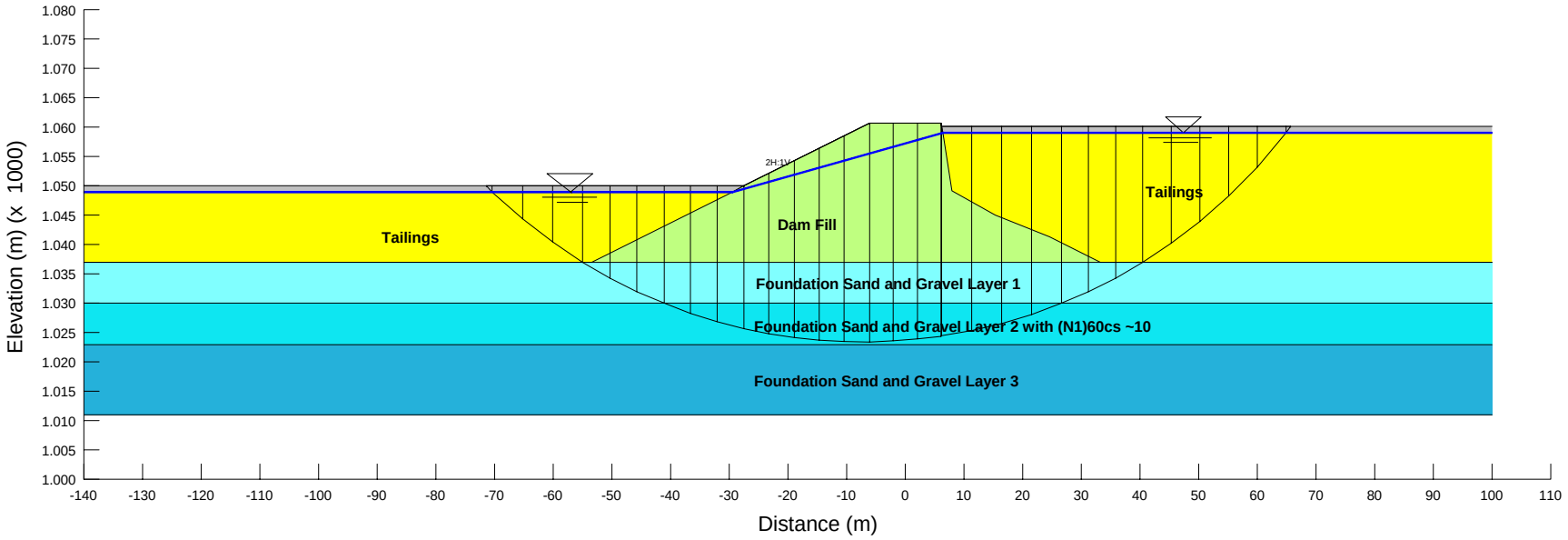


Figure VII-9 Results from the Stability Analysis of the Secondary Dam (West Limb Section) with the Downstream Tailings (Case SU1) -(M-P FOS=1.56)

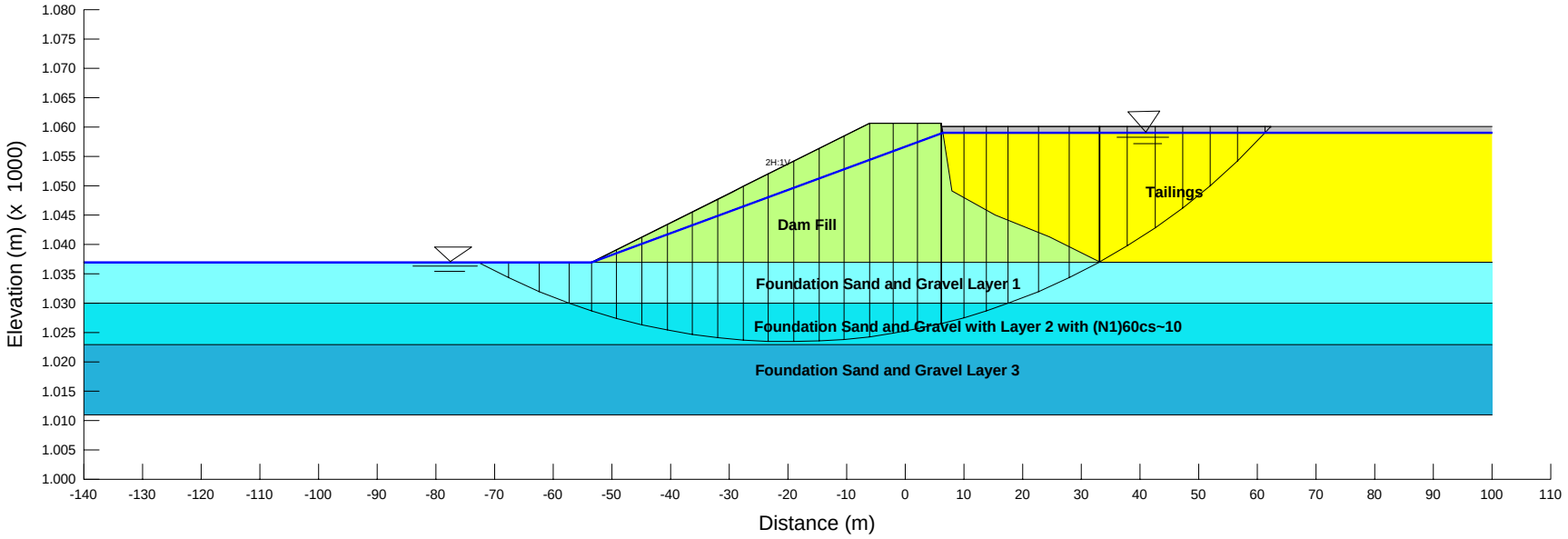


Figure VII-10 Results from the Stability Analysis of the Secondary Dam (West Limb Section) without the Downstream Tailings (Case SU2)- (M-P FOS=0.62)

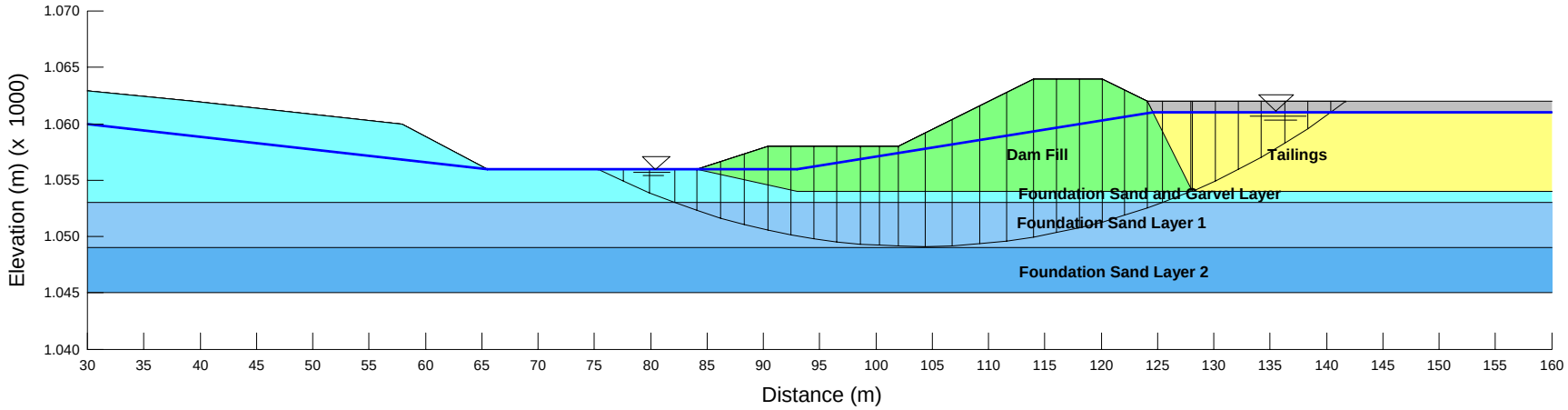


Figure VII-11 Results from the Stability Analysis of the Secondary Dam (East Limb Section) (Case SU3) (M-P FOS=0.78)

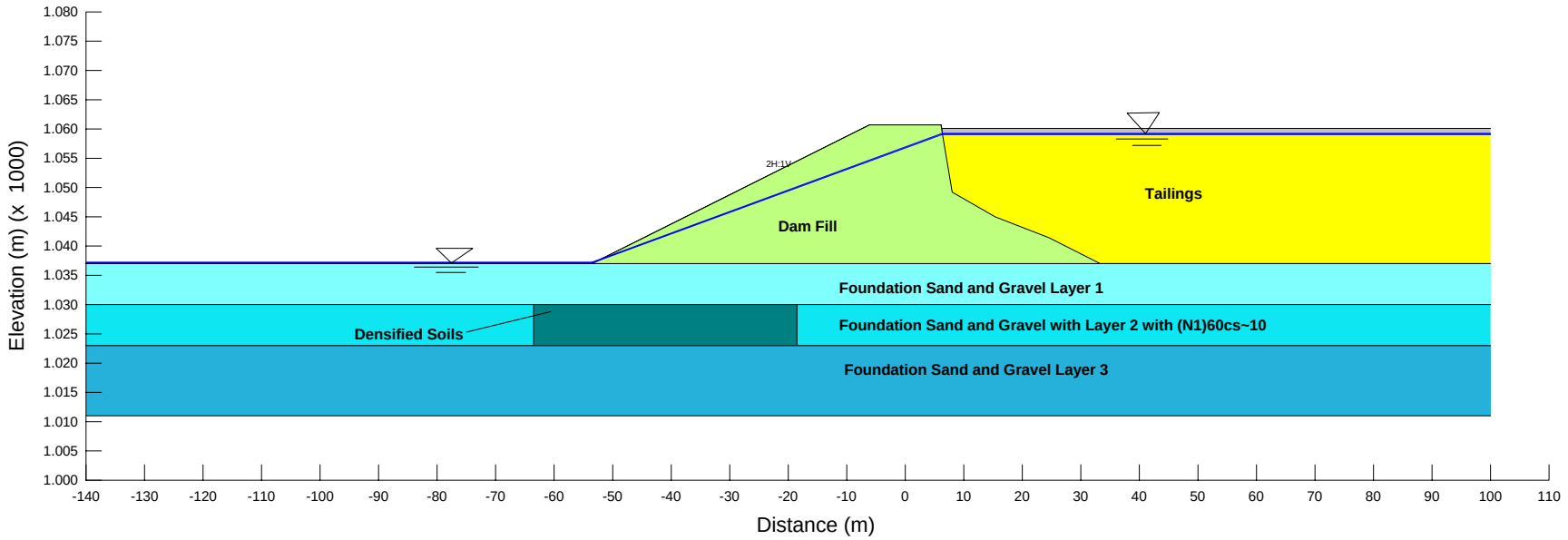


Figure VII-12 Model of the Secondary Dam (West Limb Section) with Remedial Option 1 (Case SR1)

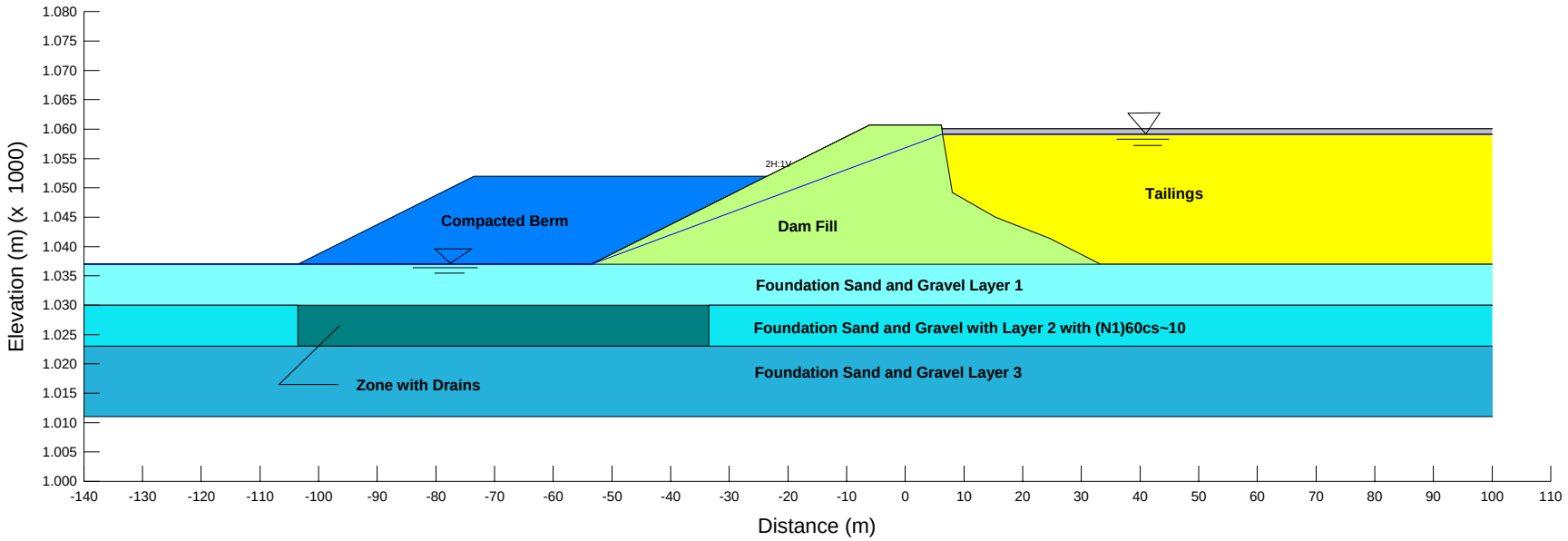


Figure VII-13 Model of the Secondary Dam (West Limb Section) with Remedial Option 2 (Case SR2)

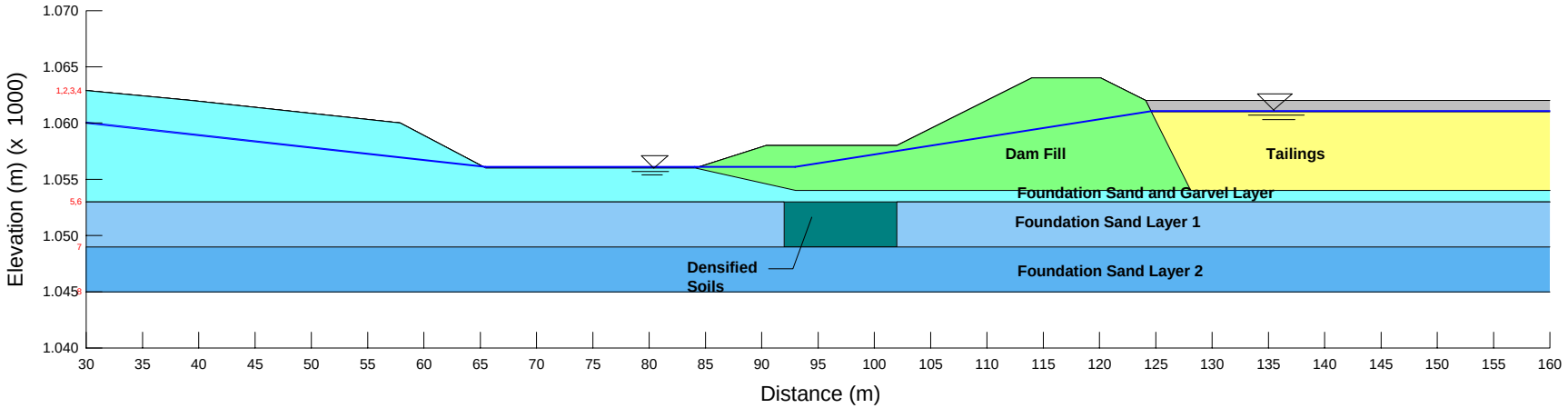


Figure VII-14 Model of the Secondary Dam (East Limb Section) with Remedial Option 1 (Case SR3)

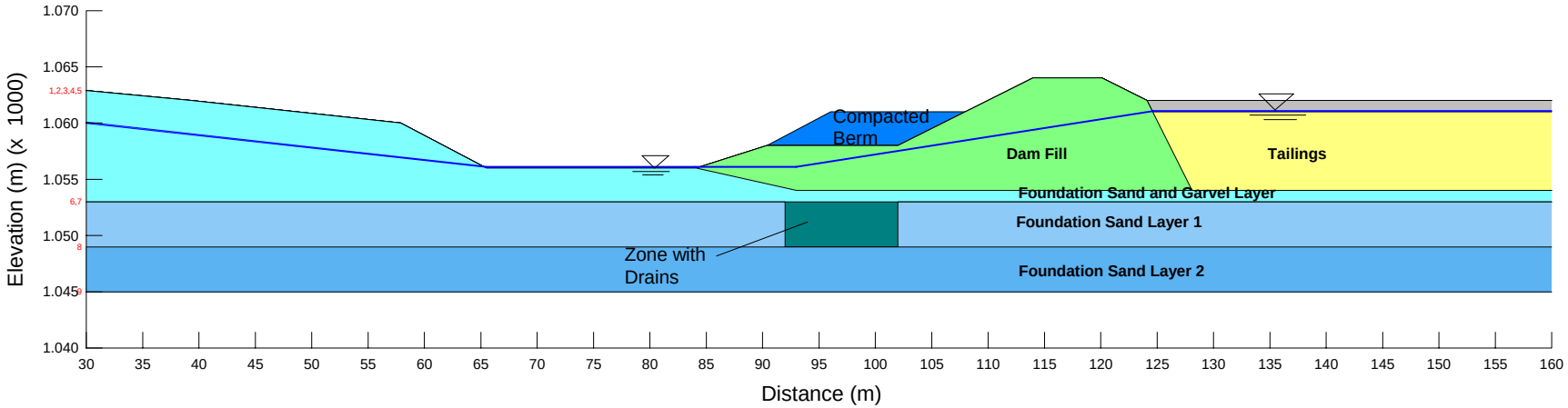


Figure VII-15 Model of the Secondary Dam (East Limb Section) with Remedial Option 2 (Case SR4)

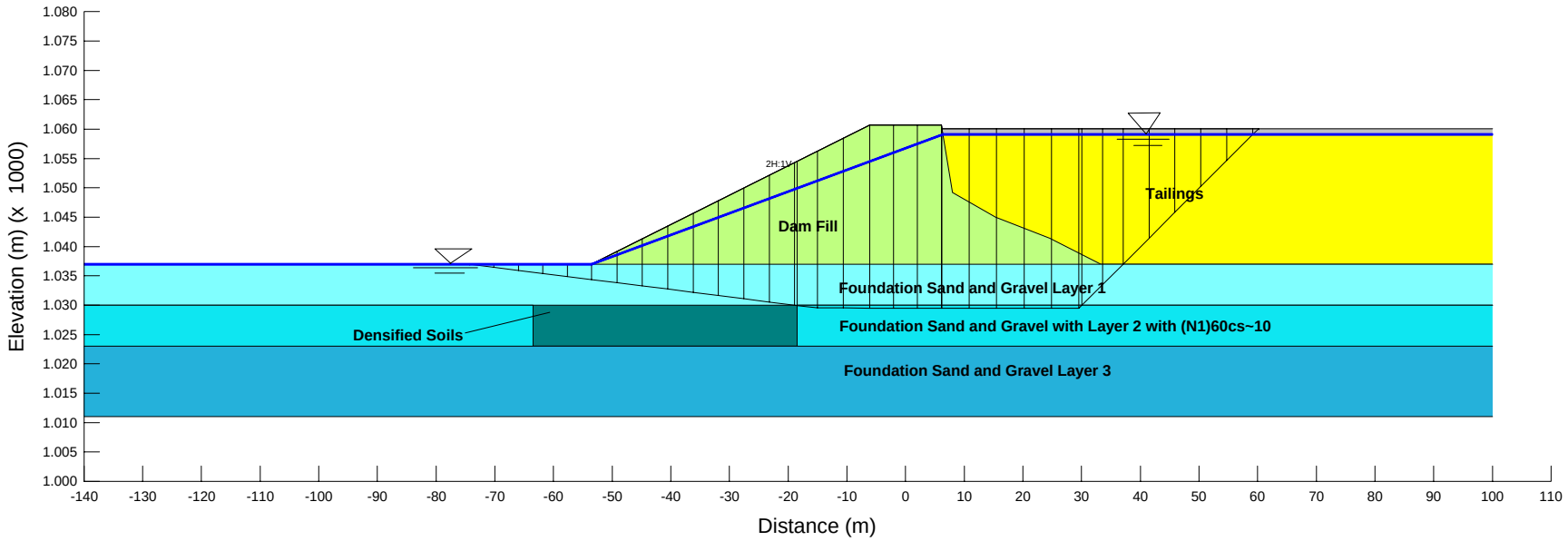


Figure VII-16 Results from the Stability Analysis of the Secondary Dam (West Limb Section) with Remedial Option 1 (Case SR1) (M-P FOS=1.21)

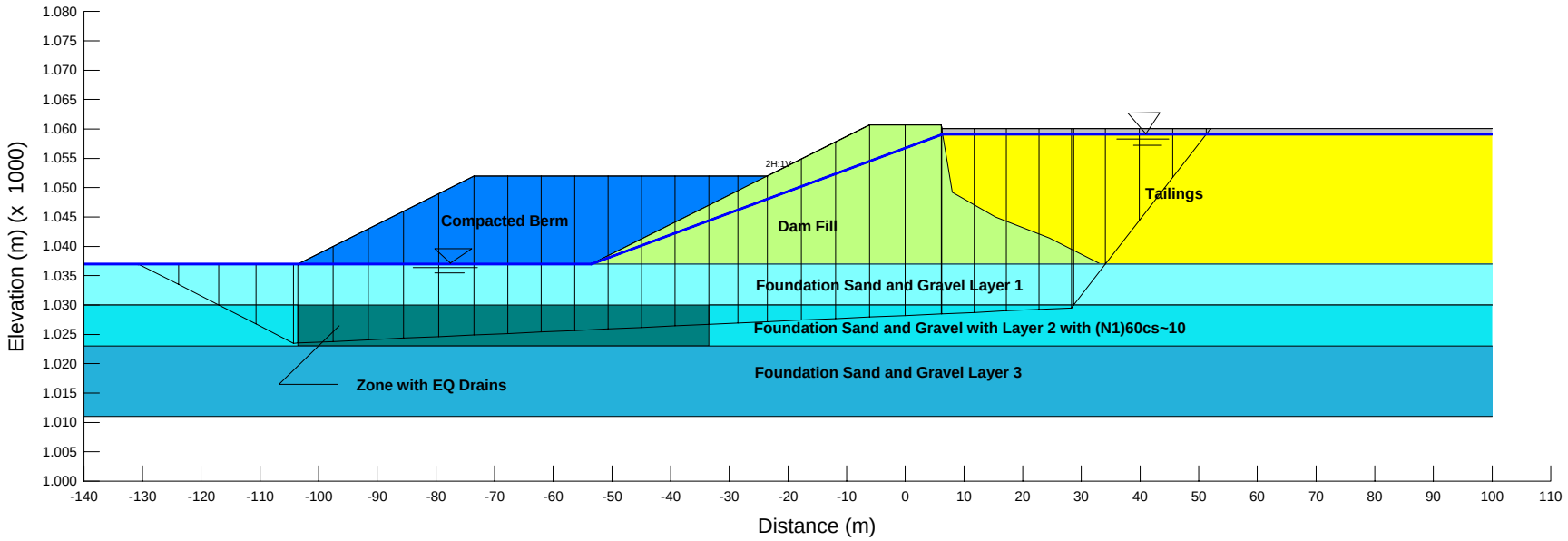


Figure VII-17 Results from the Stability Analysis of the Secondary Dam (West Limb Section) with Remedial Option 2 (Case SR2) (M-P FOS=1.13)

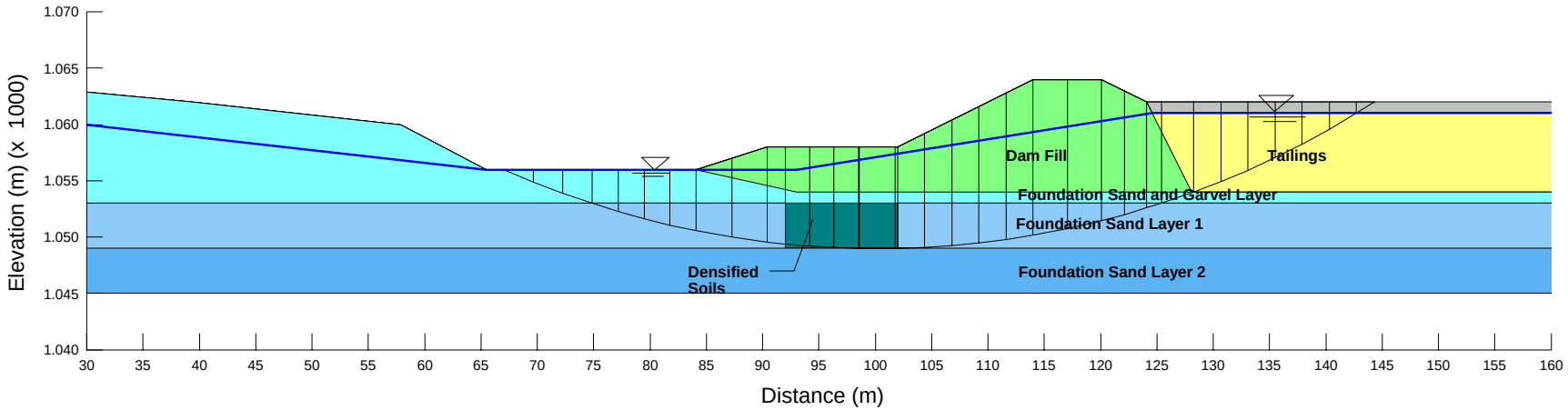


Figure VII-18 Results from the Stability Analysis of the Secondary Dam (East Limb Section) with Remedial Option 1 (Case SR3) (M-P FOS=1.42)

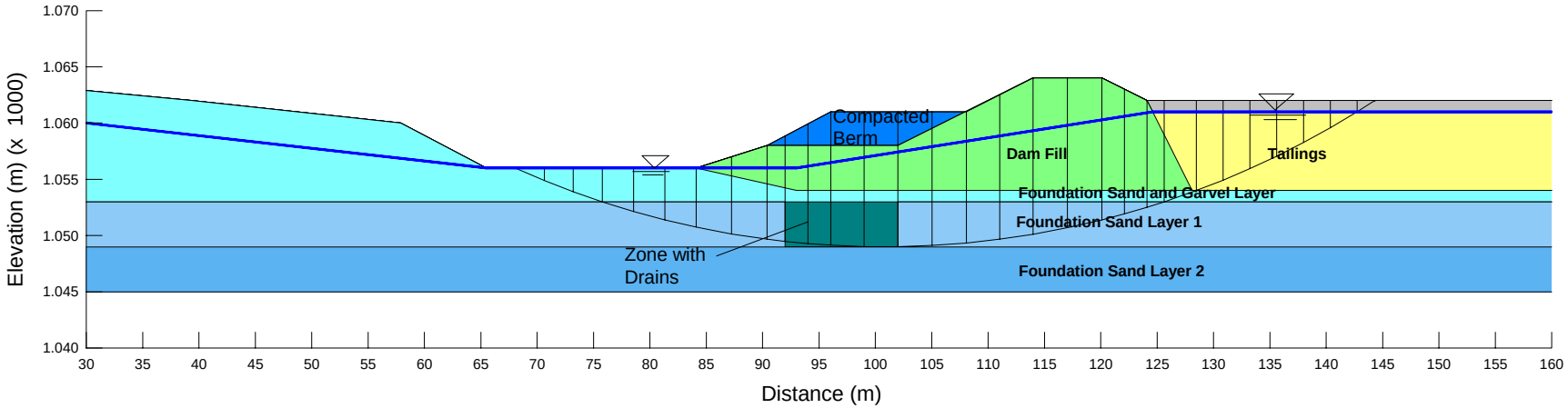


Figure VII-19 Results from the Stability Analysis of the Secondary Dam (East Limb Section) with Remedial Option 2 (Case SR3) (M-P FOS=1.12)

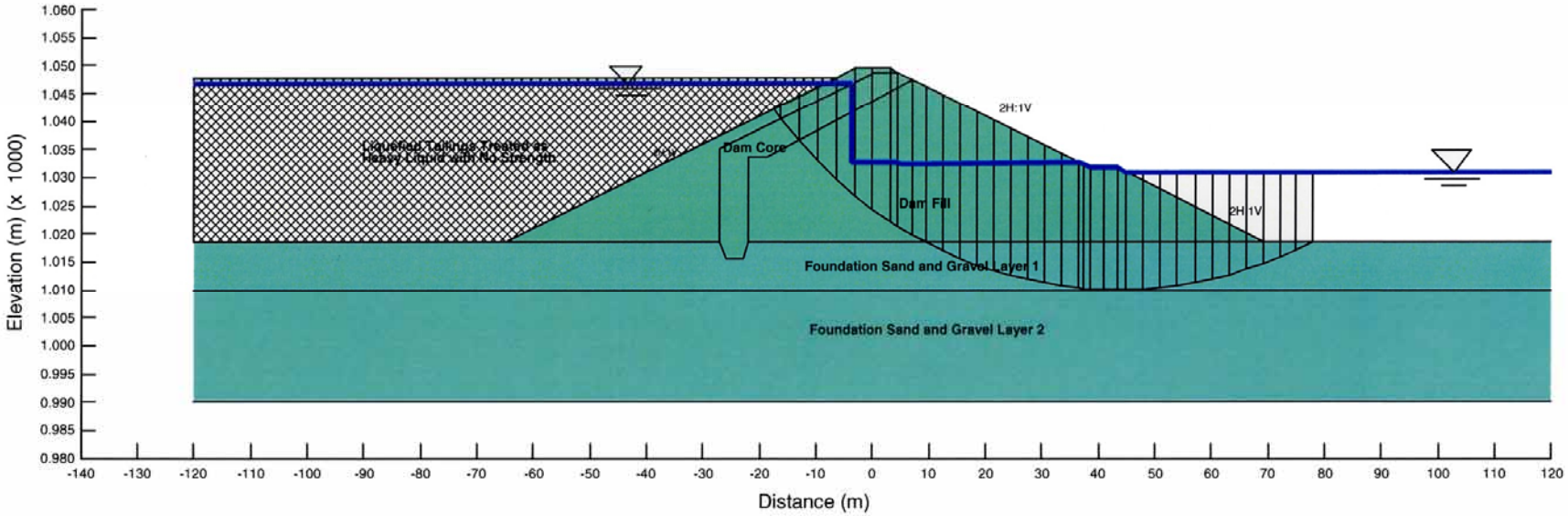


Figure VII-20 Results from the Sensitivity Analysis of the Intermediate Dam with Downstream Pond (M-P FOS=1.14)

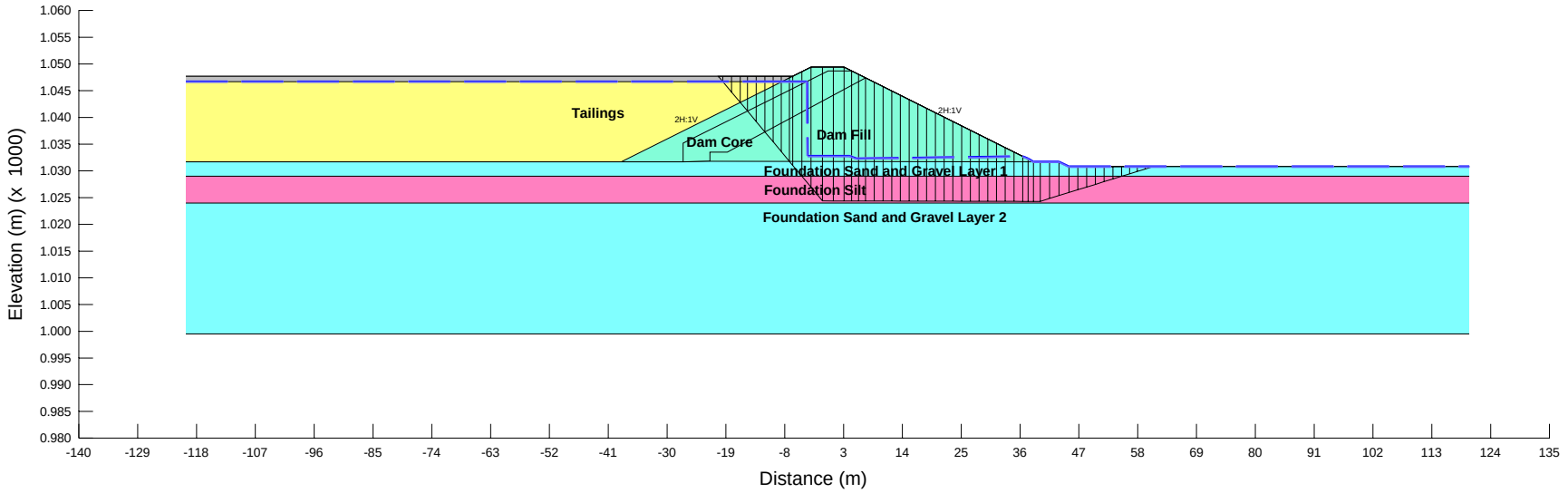


Figure VII-21 Results from the Sensitivity Analysis of the Intermediate Dam with Liquefied Silt Layer in the Foundation (M-P FOS=0.54)

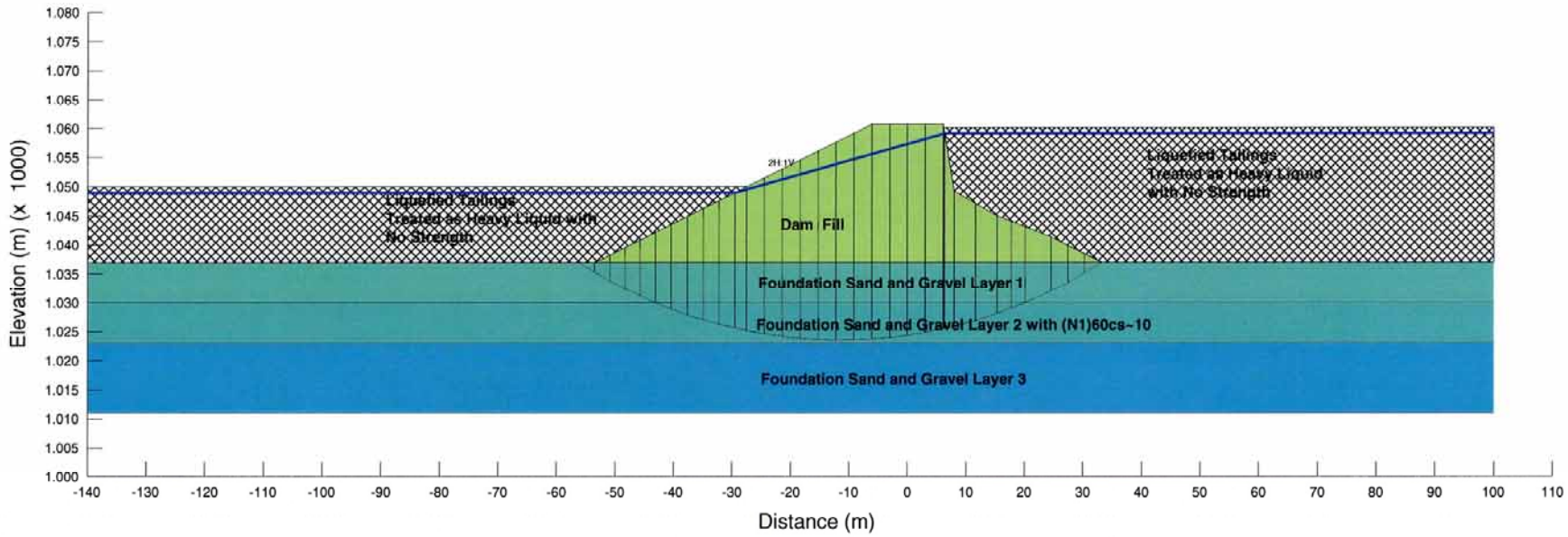


Figure VII-22 Results from the Sensitivity Analysis of the Secondary Dam with Downstream Tailings (M-P FOS=1.48)