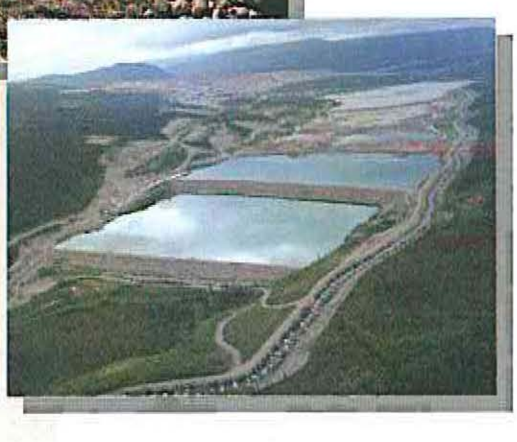
Deloitte & Touche

Dam Safety Studies for the Intermediate Dam
Anvil Range Mining Complex



Prepared for:

DELOITTE & TOUCHE INC.
Interim Receiver of Anvil Range Mining Corporation
Suite 1900, 79 Wellington Street West
Toronto, ON M5K 1B9

Prepared by:

SRK Consulting
Engineers and Scientists

Project Reference Number:
SRK 1CD003.29

April 2004

Dam Safety Studies for the Intermediate Dam
Anvil Range Mining Complex

Deloitte & Touche

Suite 1900, 79 Wellington Street West
Toronto, ON M5K 1B9
Canada

SRK Project Number 1CD003.27

Steffen Robertson and Kirsten (Canada) Inc.
Suite 800, 1066 West Hastings Street
Vancouver, B.C. V6E 3X2

Tel: 604.681.4196 Fax: 604.687.5532
E-mail: vancouver@srk.com Web site: www.srk.com

SRK Contact:
Cam Scott: cscott@srk.com

April 2004

Compiled by:

Cam Scott

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1 Introduction

The Anvil Range Mining Complex, located in Faro, Yukon (Figure 1), ceased operations in January 1998 when Anvil Range Mining Corporation filed for creditor protection under the Companies' Creditor Arrangement Act. Deloitte & Touche Inc. was appointed Interim Receiver of Anvil Range Mining Corporation ("Interim Receiver") on April 21, 1998. The Interim Receiver has overseen the management of the property under the terms of two water licences since that time.

The vast majority of the tailings associated with the processing of ore at the Anvil Range Mining Complex are stored in a tailings impoundment situated in Rose Creek (Figure 2). From east to west, the Rose Creek tailings impoundment comprises the original 1969 tailings area (approximately 42 ha), the 1974 tailings area (approximately 55 ha), the Intermediate Dam tailings area (approximately 88 ha) and the Cross Valley Dam which provides a polishing pond area of approximately 22 ha. These facilities are also referred to as the Down Valley structures.

In 2002, Deloitte & Touche Inc. commissioned Klohn Crippen Consultants Ltd. (Klohn Crippen) to undertake a safety review of four dams at the Anvil Range Mining Complex. The Klohn Crippen dam safety report recommended that the following studies be completed in relation to the Intermediate Dam:

1. A dam breach analysis to confirm the consequence classification and project design criteria for flood and seismic loading;
2. An assessment of the capacity of the emergency spillway; and
3. An assessment of the performance of the foundation soils during the design earthquake.
4. Execution of specific maintenance items;
5. Preparation of an operation, maintenance and surveillance manual;
6. Preparation of an emergency preparedness plan; and
7. Assessment of the upstream diversion channels (North Fork rock drain and the Rose Creek diversion, including the fuse plug).

Items 1 through 3 are the subject of the current report by SRK Consulting Inc. (SRK). Items 4 through 7 are excluded from this report as they are being handled independently by Deloitte & Touche Inc.

2 Scope of Work

During the discussions at the outset of this project, Deloitte & Touche Inc. requested that the scope of work be at the "low end" of the technically acceptable range. This request has been used as a guide to establish the depth of effort performed during the course of this project.

3 Dam Breach Analysis and Consequence Classification

3.1 Background

The consequence classification of a dam is of fundamental significance because it provides the basis for the selection of the flood and seismic events used in design. Although Klohn Crippen did not complete a classification assessment for the Intermediate Dam, their report indicates that the structure would likely be classified as a "very high consequence" structure, due to the high costs associated with the consequences of a dam breach. The Klohn Crippen report recommended that a breach of the Intermediate Dam be studied and that a screening level estimate of the risks and impacts of such a potential failure be completed.

3.2 Previous Dam Breach Assessment

Concurrent with the Klohn Crippen dam safety review, and in support of the Environmental Assessment of the slot cut at the Fresh Water Supply Dam (FWSD), SRK undertook breach evaluations related to the FWSD and the Down Valley structures that, at least in part, address downstream impacts. A summary of the modelling approach that was used for the FWSD breach evaluations is provided below:

- The ability of the FWSD to withstand a variety of floods was determined and, in those cases where the flood lead to an overtopping of the FWSD, it was assumed that this lead to a complete breach of the FWSD.
- The downstream effects of the FWSD breaches and floods were simulated using a software package named DAMBRK and relatively detailed topographic data (1:2,000 scale, with a contour interval of 1 m). DAMBRK is an unsteady-state flood routing model that predicts water outflow due to spillway and/or dam failures. The model then routes the predicted outflow through downstream valleys and predicts the extent of flooding. The results of the DAMBRK runs were used to evaluate the impacts as far downstream as the Intermediate Dam and the Cross Valley Dam. The results indicated that all breaches of the Intermediate Dam resulted in breaches of the Cross Valley Dam.
- The downstream effects caused by the breaching of the Intermediate and Cross Valley Dams were simulated using the computer program, HEC RAS, and the available mapping downstream of the Cross Valley Dam (1:50,000 scale, with a contour interval of about 30 m).

The HEC RAS modelling indicated that a floodwave would flow downstream to the Pelly River, approximately 45 km distant. The floodwave would be about 1½ metres deep immediately downstream of the Cross Valley Dam and would gradually get thinner as the wave progressed towards the Pelly River. Given the remoteness of this site, it is unlikely that there would be loss of human life as a result of this event. However, tailings would travel downstream with the floodwave and be deposited across the valley floor as a layer that would likely vary in thickness from a metre or so close to the impoundment to perhaps only a few millimetres close to the Pelly River. In relation to impact on vegetation, this thin layer of tailings is likely to kill the existing vegetation and to "sterilize" the soil, thereby preventing the growth of new vegetation. This conclusion is based on the fact that the tailings behind the Intermediate Dam are strongly acid generating and the observation that an area downstream of the Intermediate Dam, where tailings spilled approximately 30 years ago during operations, remains barren of any vegetation to this day. Photos taken in 2003 of these tailings are provided in Appendix A.

The total volume of tailings that would likely escape from the Rose Creek tailings impoundment following a breach of the Intermediate and Cross Valley Dams cannot be determined with a high degree of certainty. The amount that might be lost depends on factors such as the shape of the impoundment and the volume of water typically maintained on the impoundment surface. Case studies of tailings dam failures suggest that usually less than half of the tailings are lost during the initial breach. Based on the long shape of the Rose Creek tailings impoundment and the relatively modest volume of water that will be kept on the facility, the previous analyses assumed that 33% of the tailings in the Intermediate Dam tailings area would escape downstream as a result of a dam breach.

3.3 Estimated Cost of Cleanup

The following factors will negatively impact the ease and cost of cleanup following a breach of the containment structures at the Rose Creek tailings impoundment:

- As noted previously, the thickness of the spilled tailings is likely to be a maximum close to the impoundment, gradually thinning to probably only a few millimetres close to the Pelly River.
- The thickness of tailings close to the dam may be suited to cleanup using conventional earthmoving equipment but, over much of the impacted area, the thickness of the tailings is likely to be only a few centimetres to a few millimetres, thereby possibly requiring the movement of all of the organic cover and, likely, some portion of the natural mineral soil. At some point, the cleanup causes more damage than the event.
- The ground conditions in the impacted area are often soft due to the presence of a high water table and soft organic soils at the ground surface, and are therefore usually not well suited to conventional earthmoving equipment.

Based on these considerations, it may be impractical to try to recover the tailings from most of the impacted area. As a consequence, there will likely be large areas over which the impacts of the tailings spill will be very long term. A cost estimate has been developed for the recovery of tailings, where practical, on the basis of the following assumptions:

- The volume of tailings in the Intermediate Dam tailings area is about 11.9 million m^3 (ICAP, 1996)
- The volume of tailings that escapes following the breach is 3.9 million m^3 (based on a loss of 33%, which is a typical percentage based on the range of cases in the literature and engineering judgment).
- About 2.0 million m^3 of tailings can be recovered using conventional earthmoving equipment (judgment).
- The unit cost of recovering these tailings is $8 per cubic metre (judgment based on typical unit costs of excavation using small equipment in a very inefficient setting).

These assumptions lead to a cost of about $16 million dollars. It is reasonable to assume that the total costs of a failure would be higher than this due to other factors such as related cleanup activities, possible fines, engineering and environmental studies and long-term monitoring requirements.

3.4 Selection of Design Flood and Design Seismic Events

3.4.1 Dam Classification

The dam classification system recommended in the Canadian Dam Safety Guidelines (Canadian Dam Association, 1999) is shown in Table 3.1.

Table 3.1: CDA Dam Classification in Terms of Consequences of Failure

Consequence Category	Potential Incremental Consequences of Failure[a]	
	Life Safety[b]	Socioeconomic, Financial & Environmental[c]
Very High	Large number of fatalities	Extreme damages
High	Some fatalities	Large damages
Low	No fatalities anticipated	Moderate damages
Very Low	No fatalities	Minor damages beyond owner's property

Notes to Table 3.1

a) *Incremental to the impacts which would occur under the same natural conditions (flood, earthquake or other event) but without the failure of the dam. The consequence (i.e. loss of life or economic loses) with the higher rating determines which category is assigned to the structure. In the case of tailings dams, consequence categories should be assigned for each stage in the life cycle of the dam.*

b) *The criteria which define the Consequence Categories should be established between the Owner and the regulatory authorities, consistent with societal expectations. Where regulatory authorities do not exist, or do not provide guidance, the criteria should be set by the owner to be consistent with societal expectations. The criteria may be based on levels of risk which are acceptable or tolerable to society.*

c) *The owner may wish to establish separate corporate financial criteria which reflect their ability to absorb or otherwise manage the direct financial loss to their business and their ability to pay for damages to others.*

The potential incremental consequences of failure with regard to life safety factors is classified as *"no fatalities anticipated"*, corresponding to the *"low"* consequence category. This selection is based on the remote nature of the site.

The potential incremental consequence of failure in regard to socioeconomic, financial and environmental factors is probably classified as *"extreme damages"*, due to the long-term damage over a large area for a very long time. This corresponds to the *"very high"* consequence category. The CDA guidelines recommend that the selection of the classification be based on the more severe consequences, which leads to the *"very high"* consequence category.

3.4.2 Design Flood

Table 3.2, taken from the Canadian Dam Safety Guidelines, provides a basis for selecting the inflow design flood (IDF).

Table 3.2: Usual Minimum Criteria for Inflow Design Floods

Consequence Category	Inflow Design Flood (IDF)
Very high	Probable Maximum Flood (PMF)
High	Annual Exceedance Probability (AEP) between 1/1,000 and the PMF
Low	AEP between 1/100 and 1/1,000

Based on Table 3.2 and the "very high" consequence classification discussed previously, the IDF for the Intermediate Dam would be the PMF. However, modification of the emergency spillway to pass the PMF would be technically challenging and very costly. In addition, the process of developing a Final Closure and Reclamation Plan is currently underway. It would be imprudent to consider large expenditures associated with spillway modifications that may well not conform to the Final Closure and Reclamation Plan. In view of these considerations, the IDF should remain as it was for the original design, namely the 1:500 year flood flow event (Golder, 1991), until the Final Closure and Reclamation Plan has been finalized.

3.4.3 Design Earthquake

Table 3.3, taken from the Canadian Dam Safety Guidelines, provides a basis for selecting the maximum design earthquake (MDE).

Table 3.3: Usual Minimum Criteria for Design Earthquakes

Consequence Category	Maximum Design Earthquake (MDE)	
	Deterministically Derived	Probabilistically Derived (Annual Exceedance Probability)
Very High	MCE[1]	1/10,000
High	50% to 100% MCE	1/1,000 to 1/10,000
Low	-	1/100 to 1/1,000

Note 1: MCE is the maximum credible earthquake

Based on Table 3.3 and the "very high" consequence classification discussed previously, the MDE for the Intermediate Dam would be either the MCE or an earthquake with a 1:10,000 annual probability of exceedance, depending on whether the database provides a deterministically or probabilistically derived earthquake. Like the IDF, any decision to design for the earthquake indicated by Table 3.3 should consider the financial implications for the same reasons noted in Section 3.4.3. However, unlike the IDF, it is unclear what the cost implications will be in relation to the selection of one of these earthquakes for further studies. Such studies are currently being performed by others, but as an input to these studies, this report will consider the foundation conditions beneath the Intermediate Dam. Further comments are provided in Section 5.

4 Emergency Spillway Assessment

4.1 Background

Klohn Crippen raised concerns about the ability of the spillway to pass a flow of 100 m^3/sec, which corresponds to a 1:500 year flood flow event (Golder, 1991). In addition, Klohn Crippen concluded that failure of the spillway could lead to failure of the Intermediate Dam right abutment area and consequent release of tailings to the Rose Creek channel. Based on these concerns, the Klohn Crippen report recommended that the capacity of the emergency spillway be reviewed.

4.2 New Ground Survey

The crest of the dam and the spillway were surveyed by Yukon Engineering Services, Inc. (YES) of Whitehorse in early August 2003 in order to obtain precise information regarding the elevation of the dam crest, the elevation of the spillway inlet, and the width and gradient of the spillway. The survey results provided by YES are summarized on Figure 3.

4.3 Engineering Assessment of the Spillway

Mr. Barry Chilibeck, P.Eng., of Northwest Hydraulic Consultants (nhc) completed an assessment of the Intermediate Dam spillway based on (1) a brief onsite assessment undertaken on July 27-28, 2003; (2) hydraulic modelling and estimation of flow depths and velocities based on the YES and as-built survey information; and (3) the results of an erosion and stability assessment of the in situ spillway as of fall 2003.

The onsite review of the spillway identified several potential issues that required further assessment, based on either capacity or stability issues. The initial concerns were:

- The presence of additional material along the bottom section of the spillway near the dam crest that could result in loss of freeboard across the dam section during the 1:500 flood flow;
- Potential left bank overtopping of the spillway along most of its length downstream from

the dam crest centreline;

- Relatively small size of riprap materials along the banks of the spillway;
- Lack of riprap along portions of the bank, notably on the right bank downstream of the dam crest centreline;
- Potential scour and erosion resulting from the placement of large boulders and weir section in the lower section of the spillway were velocities and depth of flow likely increase; and
- The lack of riprap thickness in areas of potential scour.

Following an erosion and stability assessment, nhc provided a report that outlined their recommendations to address these deficiencies. Figure 4 summarizes the recommendations on a plan view of the spillway. The complete nhc report is included in Appendix B.

5 Seismic Stability Assessment

5.1 Background

Klohn Crippen noted that they could find very little information about the results of stability analyses for the Intermediate Dam, although they noted reference to a stability assessment for the Cross Valley Dam which was based on a 1 in 200 year seismic loading criterion. Klohn Crippen recommended that the performance of the Intermediate Dam under the MDE be checked and that this assessment consider the resistance capability of the natural foundation materials in relation to this performance.

In defining the scope of work for this study, the concurrent execution of the following studies by others related to the development of the Final Closure and Reclamation Plan was considered:

- Provision of a definitive opinion on the magnitude of the maximum credible earthquake (MCE) based on available information;
- Completion of a program of cone penetration tests (CPTs) within the tailings impoundment in order to provide soil classifications and property estimates for use in stability and deformation analyses;
- Completion of initial screening-level seismic stability analyses and, if appropriate, screening-level deformation analyses; and
- If necessary, completion of conceptual design upgrades to meet long-term closure requirements.

Based on these other studies, the scope of work for this study was limited to the assessment of the performance of the foundation soils during the design earthquake.

5.2 Available Geotechnical Data

The available drilling records for the foundation conditions at the Intermediate and Cross Valley Dams have been reviewed. Most of the available geotechnical information within the foundation soils coincides with the location of the Cross Valley Dam. Standard penetration test (SPT) results, which provide an indication of the relative density of the foundation soils, have been compiled and converted to $(N1)_{60}$ values, which is the form used in seismic stability assessments. The results of these analyses are provided in a technical memorandum in Appendix C.

5.3 Seismic Risk Assessment

A site-specific seismic hazard analysis in relation to the ongoing development of a Final Closure and Reclamation Plan for the Anvil Range Mining Complex was completed by Dr. Gail Atkinson (2004). The seismic hazard analysis assumed that the northwest trending fault structures associated with the Tintina Trench near the Town of Faro are active, which was the conclusion of a recent review of the regional bedrock geology and faults (Smith, 2003).

The analysis results included six ground motion curves, reflecting uncertainty in the results according to uncertainty in the input parameters. The response spectra were modified in consideration of amplification due to foundation conditions in the Rose Creek Valley. The design event corresponds to a design earthquake magnitude of M7.2 approximately 10 to 20 km from the Intermediate Dam. Based on a probability of exceedance criterion of 1 in 10,000 (Table 3.3), the peak ground acceleration is approximately 0.37g to 0.55g, depending on whether the median or mean value is selected for design purposes.

5.4 PROSHAKE Analysis of Foundation Soils

A screening level study has been carried out by Dr. Peter Byrne in order to assess the potential for liquefaction of the foundation soil beneath the Intermediate Dam during the design earthquake, which is assumed to be the 1 in 10,000 year event. Due to the limited data beneath the Intermediate Dam, soil information from beneath the Cross Valley Dam was also used to estimate materials properties for liquefaction assessment. Six different soil profiles representing different dam foundation conditions were analyzed for the design earthquake. Response analyses were carried out for the six soil profiles and for the six input motions provided by Atkinson using the PROSHAKE (EduPro Civil Systems, Inc.) computer code. This program uses a frequency domain equivalent linear method to provide the ground response during the design earthquake. These outputs have been used to evaluate the liquefaction potential of the foundation soils. The results indicate that in some cases, widespread liquefaction is predicted. Further assessment of this seismic stability of the Intermediate Dam will be the subject of studies related to the development of the Final Closure and Reclamation Plan.

The complete report by Dr. Byrne is included in Appendix D.

6 Conclusions

In response to a safety review of four dams at the Anvil Range Mining Complex, the Interim Receiver commissioned SRK to respond to the following three recommendations for the Intermediate Dam:

- Confirm the consequence classification and project design criteria for flood and seismic loading based on a dam breach analysis;
- Assess the capacity of the emergency spillway; and
- Assess the performance of the foundation soils during the design earthquake.

SRK, in conjunction with mine site personnel, YES, nhc and Dr. Peter Byrne of UBC, completed a scope of work that addresses the three recommendations noted above. The conclusions arising from this work scope are summarized below.

Dam breach analyses were performed previously in relation to works proposed for a dam upstream of the Intermediate Dam. Based on these analyses and reasonable assumptions on the probable impacts (largely environmental), and the costs of remediation (likely greater than $16 million), the Intermediate Dam lies within the "*very high*" consequence category. The CDA guidelines indicate that the PMF and MCE should be used in relation to the design flood and stability analyses, respectively. However, the spillway should be assessed based on its original design flood, namely the 1:500 year flood, until the Final Closure and Reclamation Plan, which is currently being developed, has been finalized.

An assessment of the performance capabilities of the emergency spillway was completed. Deficiencies relative to the 1:500 year flood were noted and recommendations are included in this report as to how these deficiencies should be corrected.

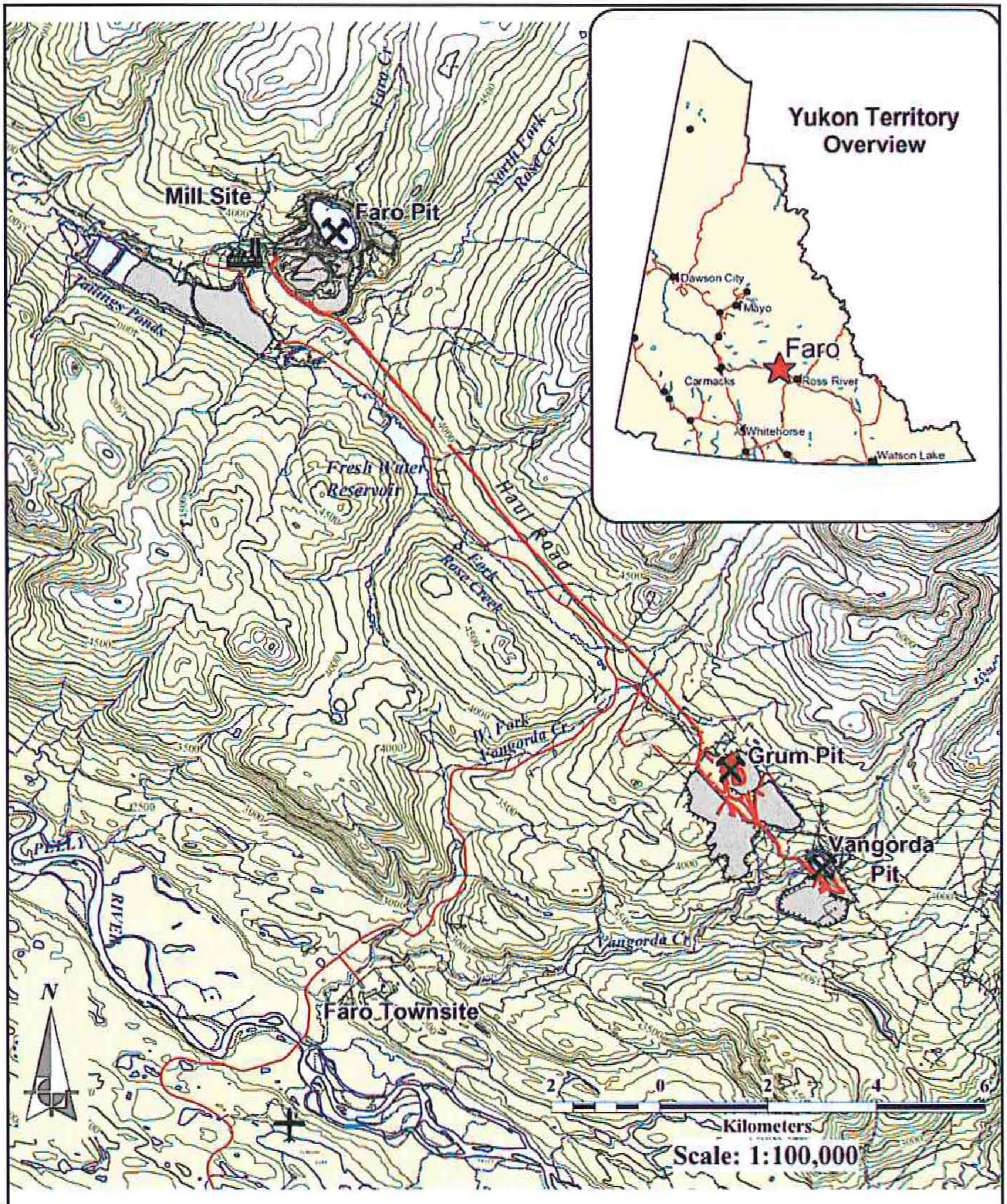
An assessment of the seismic stability of the foundation soils at the Intermediate Dam was completed based on the existing borehole database and an updated seismic risk assessment. The results indicate that in some cases, widespread liquefaction is predicted. Further assessment of this seismic stability of the Intermediate Dam will be the subject of studies related to the development of the Final Closure and Reclamation Plan.

7 References

Atkinson, G., 2004. Seismic Hazard Assessment for Faro, YK. Draft report dated January 9, 2004.

Smith, R., (2003). Tectonic Setting, Faro, Yukon. Draft letter dated December 23, 2003.

FIGURES



Dwg Ref: Regional Loc Map.dwg

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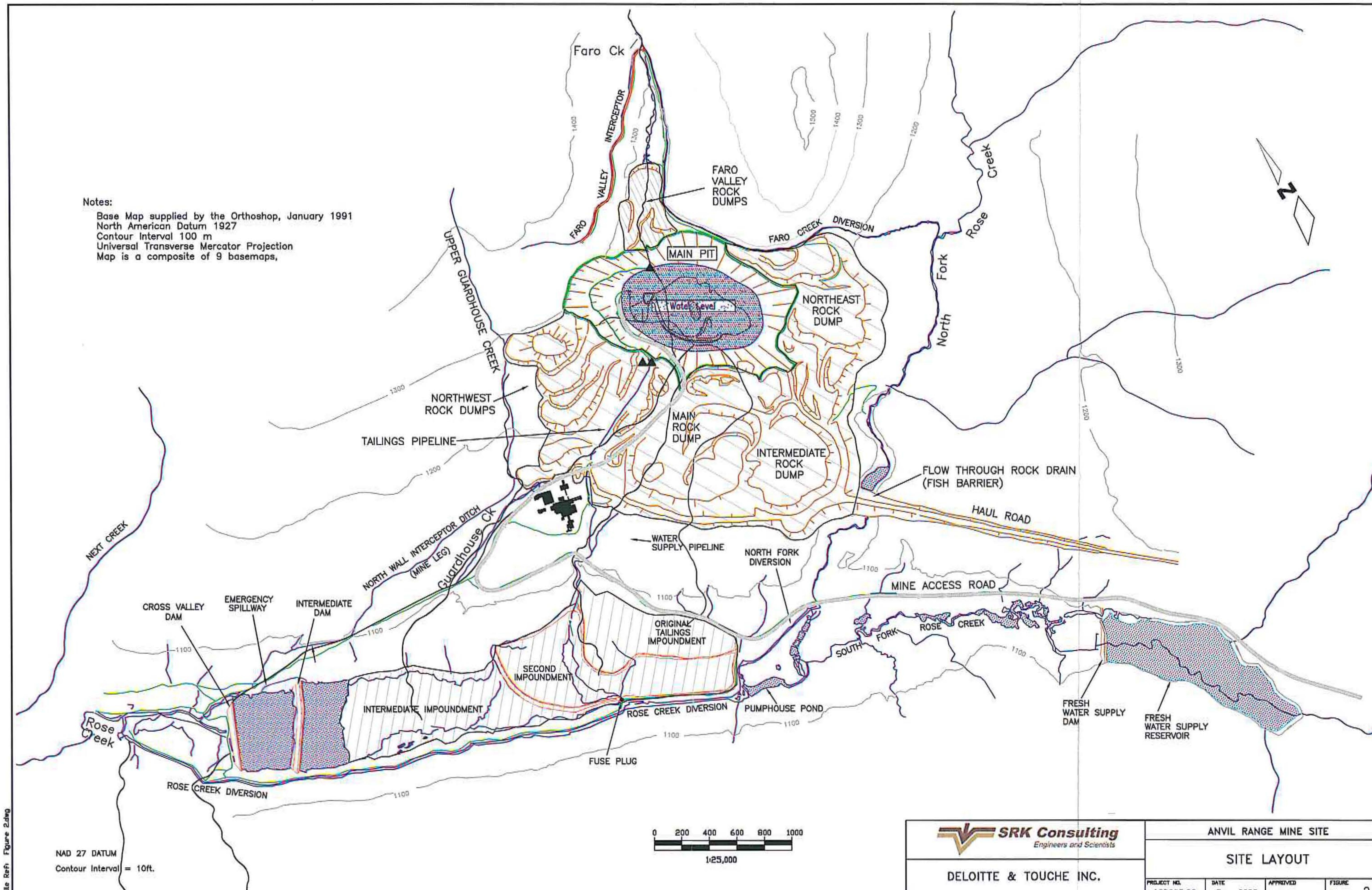
Engineers and Scientists

Deloitte & Touche Inc.

FARO–VANGORDA PLATEAU MINE SITES

LOCATION MAP

PROJECT NO.	DATE	APPROVED	FIGURE
1CD003.27	Dec. 2003		1

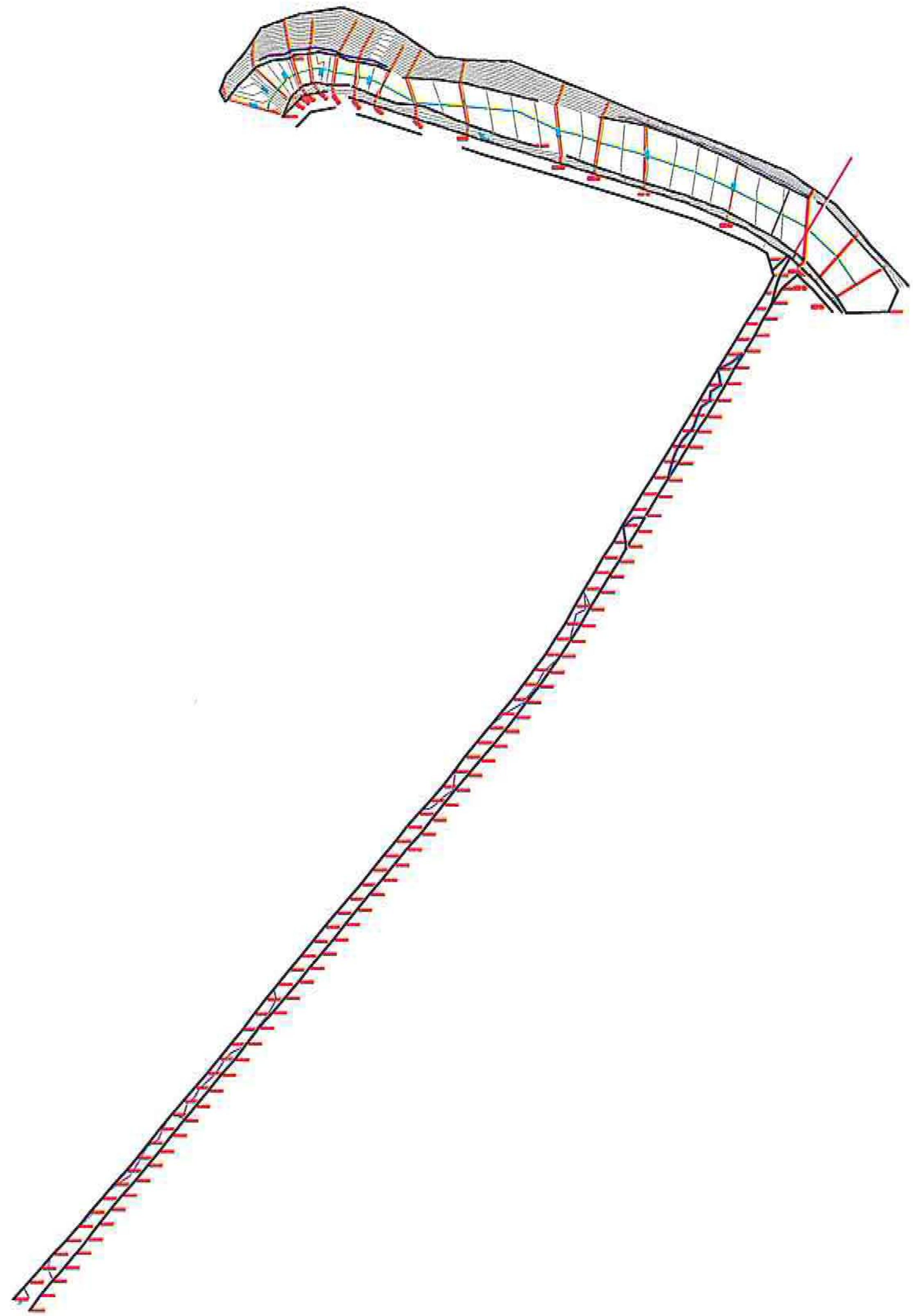


Notes:
Base Map supplied by the Orthoshop, January 1991
North American Datum 1927
Contour Interval 100 m
Universal Transverse Mercator Projection
Map is a composite of 9 basemaps,

NAD 27 DATUM
Contour Interval = 10ft.

File Ref: Figure 2.dwg

SRK Consulting Engineers and Scientists	ANVIL RANGE MINE SITE
	SITE LAYOUT
DELOITTE & TOUCHE INC.	PROJECT NO. 1CD003.26 — DATE Dec. 2002 — APPROVED — FIGURE 2



Deloitte & Touche Inc.

FARO–VANGORDA PLATEAU MINE SITES

INTERMEDIATE DAM
SPILLWAY MAINTENANCE

PROJECT NO.	DATE	APPROVED	FIGURE
1CD003.37	Dec. 2003		3

Dwg Ref: Regional Loc Map.dwg

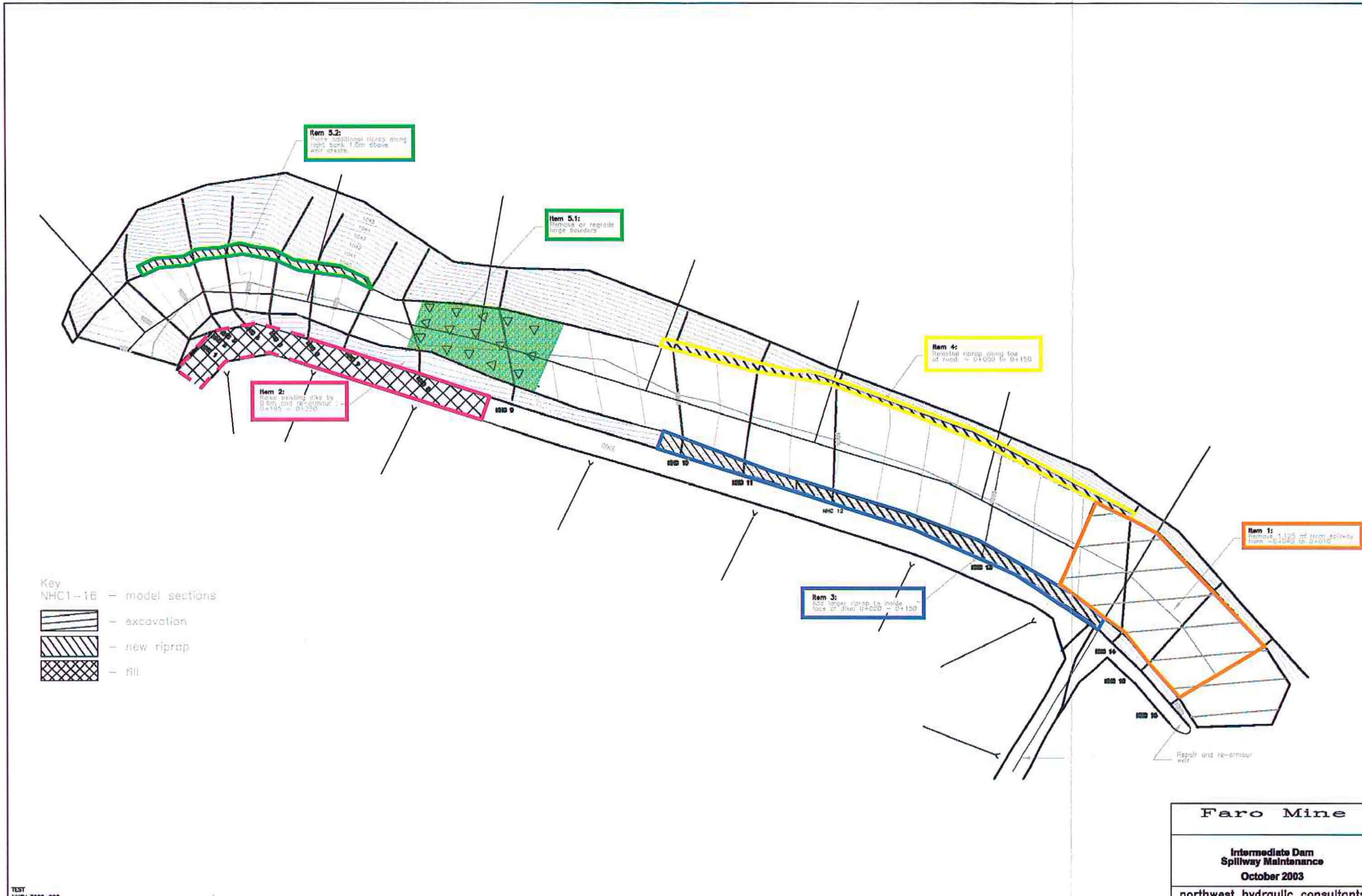

Item 5.2:
Place additional riprap along right bank 1.0m above weir crests
Item 5.1:
Remove or regrade large boulders
Item 4:
Reinstall riprap along toe of road ~ 0+000 to 0+150
Item 2:
Raise existing dike by 0.6m and re-armour ~ 0+195 – 0+250
Item 3:
Add larger riprap to inside face of dike 0+000 – 0+150
Item 1:
NHC 9
NHC 10
NHC 11
NHC 12
NHC 13
NHC 14
NHC 15
NHC 16
DIKE
Repair and re-armour weir
Key
NHC1–16 – model sections
– excavation
– new riprap
– fill
Faro Mine
Intermediate Dam
Spillway Maintenance
October 2003
northwest hydraulic consultants
TEST
NHCV 3928-002

FIGURE 4

APPENDIX A

Photo 1 - Overview of area downstream of the Cross Valley Dam that was covered by a tailings spill in about 1974 or 1975.

Photo 2 - Tailings spill area, September 12, 2003.


Photo 3 - Small pit excavated in the tailings spill area on September 12, 2003.


Photo 4 - Profile through small pit, exposing tailings and natural soil.

APPENDIX B

water
• • • •
environment
• • • •
solutions

nhc

3-3928

October 24, 2003

SRK Consulting Inc.
800 – 1066 W. Hastings St.
Vancouver, BC
V6E 3X2

Attn: Mr. Cam Scott, P.Eng.

Re: **Faro Mine**
Intermediate Dam Spillway Assessment

Please find enclosed a brief update on our analysis of the capacity and stability of the intermediate dam spillway for the estimated extreme event, 1:500 year flood flows of approximately 100 $m^{-3} \cdot s^{-1}$ (Golder, 1991).

Our analysis was based on: (1) brief onsite assessment undertaken on July 27-28, 2003; (2) hydraulic modelling and estimation of flow depths and velocities based on as-built survey information; and (3) erosion and stability assessment of the insitu spillway as of fall 2003. Additional technical information included the original design reports, hydrological and hydraulic studies, and design drawings – specifically Golder Assoc. Ltd. drawing No. 912-2402-4, Spillway Details.

The onsite review of the spillway identified several potential issues that required further assessment, based on either capacity or stability issues. The initial concerns were:

1. The addition of materials along the bottom section in the spillway section through -0+060 to 0+000 that could result in loss of freeboard across the dam section during the 1:500 flood flow,
2. Potential left bank overtopping of the spillway from section 0+000 through 0+250,
3. Relatively small size of riprap materials along the banks of the spillway,
4. Lack of riprap along potions of the bank – notably the right bank from sections 0+050 through 0+150,
5. Potential scour and erosion resulting from the placement of large boulders and weir section in the lower section of the spillway were velocities and depth of flow likely increase, and
6. The potential lack of riprap thickness in areas of potential scour.

northwest
hydraulic
consultants

30 gostick place
north vancouver
british columbia
canada V7M 3G2
tel (604) 980-6011
fax (604) 980-9264
www.nhcweb.com

To address these potential deficiencies, hydraulic analysis and engineering was undertaken, and the following actions are recommended.

Item 1:
Our initial hydrotechnical assessment, based on the as-built survey information (Figure 1), is that water surface elevations at the crest of the spillway will reach el. 1486.47 m. during the 1:500 flood flow, which is approximately 0.5 m. below the dam crest. However, to include the potential effects of the spillway upstream of the design crest, we extended our hydraulic analysis. As a result, the water surface elevation behind the dam peaks at 1049.00 m. approximately 50 m. upstream (section 16 - Figure 2). These elevations approximate the dam crest elevation at several locations away from the spillway, along the center and left side of the dam face, based on existing as-built survey information provided by Yukon Engineering Services (YES).

Design drawings also indicate that the core of the dam was constructed approximately 0.2 – 0.3 m. below the crest. With recent modifications, the elevation of the core is approximately 0.4 m. below the current crest of the dam at el. 1048.5 m. (section 14 – Figure 1). Canadian Dam Association (CDA) Guidelines also suggest that the maximum reservoir elevation be at or below the top of the impervious core for earth-fill embankment dams.

In order to maintain dam safety both for overtopping and protection of the dam core, we recommend the removal of materials in the base of the spillway to provide a maximum pool elevation of 1048.5 m. on the intermediate dam for 1:500 flood flow conditions – the approximate elevation of the dam core. Excavation of sections 14 (dam crest) through 16 (upstream limit) to a minimum el. 1047.0 m. is required. Based on existing elevations and a spillway width of 30 m., approximately 1,125 m^3 of excavation is required with re-armouring of the bed and banks with new riprap if existing materials were not salvaged. Modelling of the new invert elevations (Figure 3) indicates a maximum water surface elevation behind the dam of el. 1047.48 m.

Item 2:
Second, the water surface elevations along the left bank at flood conditions provide approximately 0.5 m. of freeboard. However, near Sta. 0+195 through 0+250, water surface elevations are approximately at existing top-of-bank along the left bank of the spillway with little or no freeboard at 1:500 flood flows. Loss of stability through overtopping and downcutting would result in loss of the spillway section and potential downcutting through the axis of the spillway. Additional riprap and raising of the left bank berm by 0.5 m. is required for approximately 55 m. Based on a 3 m. top width and 0.5 m. freeboard, these works require approximately 100 m^3 of bulk fill materials and 50 m^3 of riprap.

Item 3:
Preliminary assessment of insitu materials sizes along the upper section of the spillway – notably the left bank – suggest that these will not be stable during the flood flows. Additional larger riprap – preferably angular material with a D_{50} of 400 mm – needs to be placed from left bank face from Sta. 0+000 to 0+150 to provide primary stability of the berm during the 500-year event. We suggest placement of protection approximately 1.0 m. up the bank, and a total of 120 m^3 of riprap is required.

Item 4:
There are long sections of the right bank supporting an existing access road that either have no riprap protection or the riprap has been buried with installation of the road. If the riprap is not installed flood flows less than the flood flows will cause erosion of this bank and potential loss of the road. We suggest further field investigation to determine if the original riprap is still installed from approximately Sta. 0+00 to 0+150. If the original needs to be replaced, approximately 120 m^3 of rock (D_{50} = 400 mm) will be required.

Item 5:
In the lower section of the spillway (design section 3), larger boulders and large riprap weirs were designed and installed – likely on the premise that they would break up the supercritical flow and dissipate energy. The large boulders are unanchored and simply rest of the bed of the spillway. The initial modelled velocities were estimated at 4 $m \cdot s^{-1}$ and depths of flow are approximately and 1 m in this section. The local velocities around these elements would likely result in loss of the bed armour and scour through to the underlying materials. The weirs in design section 4 have not been explicitly modelled but there may be potential for scour between weir crests and overtop of the bank riprap at the weir section – especially along the right bank where the water surface profile could be superelevated or bulked due to aeration.

We suggest either removal of the large boulders and re-contouring the weirs out (Item 5.1) – using the materials to line the existing channel bed – providing a relatively rough surface with good dissipation characteristics, or installation of additional protection along the top of the existing riprap on the right bank to an elevation approximately 1.0 m. above the weir crests (Item 5.2) and analysis of the potential scour between weirs (50 m^3 of riprap).

nhc

* * * * * *

We hope this provides the required information for your design and budgeting purposes. Please call me if you have any questions, or wish to discuss our findings.

Sincerely,

northwest hydraulic consultants

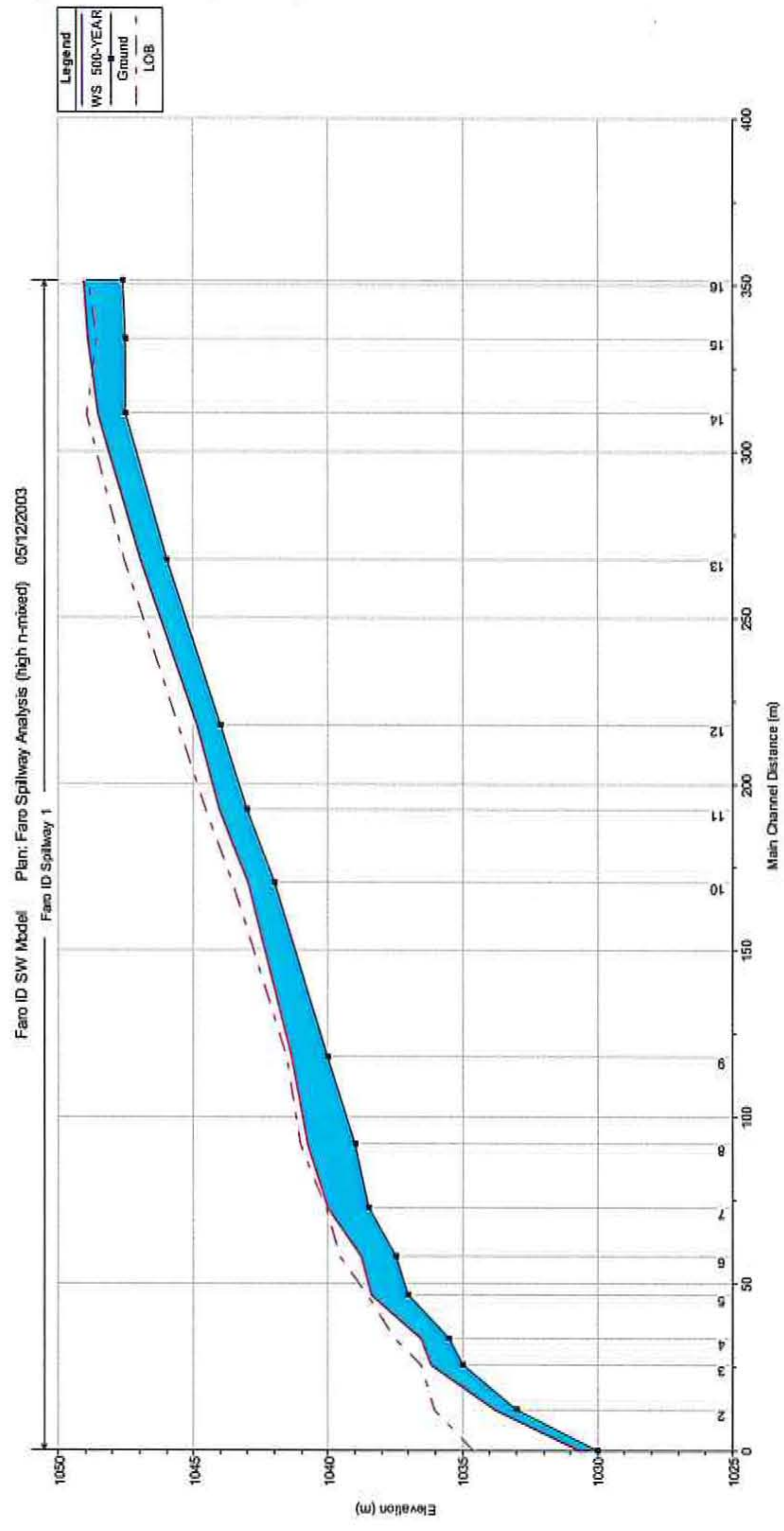
BM Chilibeck

Barry Chilibeck, P.Eng.
Principal

Attach.

nhc

Figure 1 Spillway WSL and Invert Profile



Project: ***Faro Intermediate Dam (ID) Hydraulics***
Proj. No. ***3-3928***
Date: ***Oct. 7, 2003***

HEC-RAS Spillway Model Results

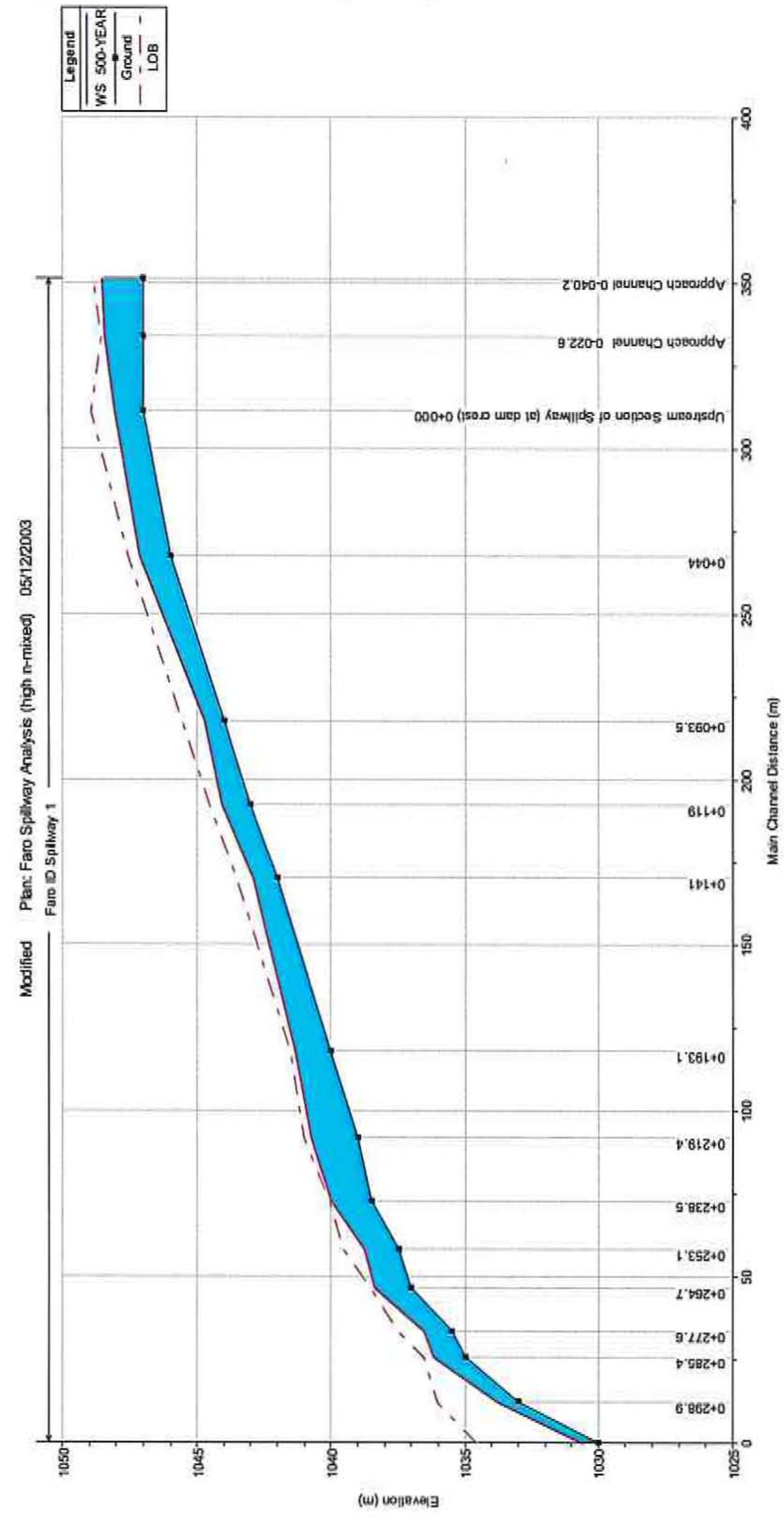
River (m)	Sta (m)	Profile (m)	Q500 m3/s	Bed Elev (m)	WS m	Crit WS m	EG Elev. m	EG Slope m/m	Vel Chnl m/s	Flow Area m2	Top W m	Froude No.
1	16	500-YEAR	100	1047.6	1049.0	1048.7	1049.3	0.006	2.44	41.06	33.91	0.71
1	15	500-YEAR	100	1047.5	1048.9		1049.2	0.007	2.56	39.02	34.25	0.77
1	14	500-YEAR	100	1047.5	1048.5	1048.5	1048.9	0.021	3.00	33.37	36.83	1.01
1	13	500-YEAR	100	1046.0	1046.9	1047.1	1047.7	0.039	3.87	25.85	30.90	1.35
1	12	500-YEAR	100	1044.0	1044.8	1045.0	1045.6	0.045	3.92	25.52	33.29	1.43
1	11	500-YEAR	100	1043.0	1044.0	1044.1	1044.6	0.030	3.56	28.10	31.26	1.20
1	10	500-YEAR	100	1042.0	1042.9	1043.1	1043.7	0.065	3.79	26.36	30.86	1.31
1	9	500-YEAR	100	1040.0	1041.3	1041.2	1041.8	0.026	2.94	34.06	29.28	0.87
1	8	500-YEAR	100	1039.0	1040.7		1041.1	0.021	2.94	34.05	24.87	0.80
1	7	500-YEAR	100	1038.5	1040.0	1040.0	1040.6	0.034	3.49	28.63	23.24	1.00
1	6	500-YEAR	100	1037.5	1038.8	1039.1	1039.9	0.076	4.65	21.50	20.70	1.46
1	5	500-YEAR	100	1037.0	1038.4	1038.5	1039.1	0.043	3.74	26.75	23.44	1.12
1	4	500-YEAR	100	1035.5	1036.5	1037.0	1038.1	0.134	5.55	18.01	20.35	1.88
1	3	500-YEAR	100	1035.0	1036.1	1036.4	1037.1	0.077	4.46	22.42	23.27	1.45
1	2	500-YEAR	100	1033.0	1033.7	1034.2	1035.5	0.209	5.83	17.14	25.33	2.26
1	1	500-YEAR	100	1030.0	1030.6	1031.1	1032.5	0.280	6.08	16.44	28.39	2.55

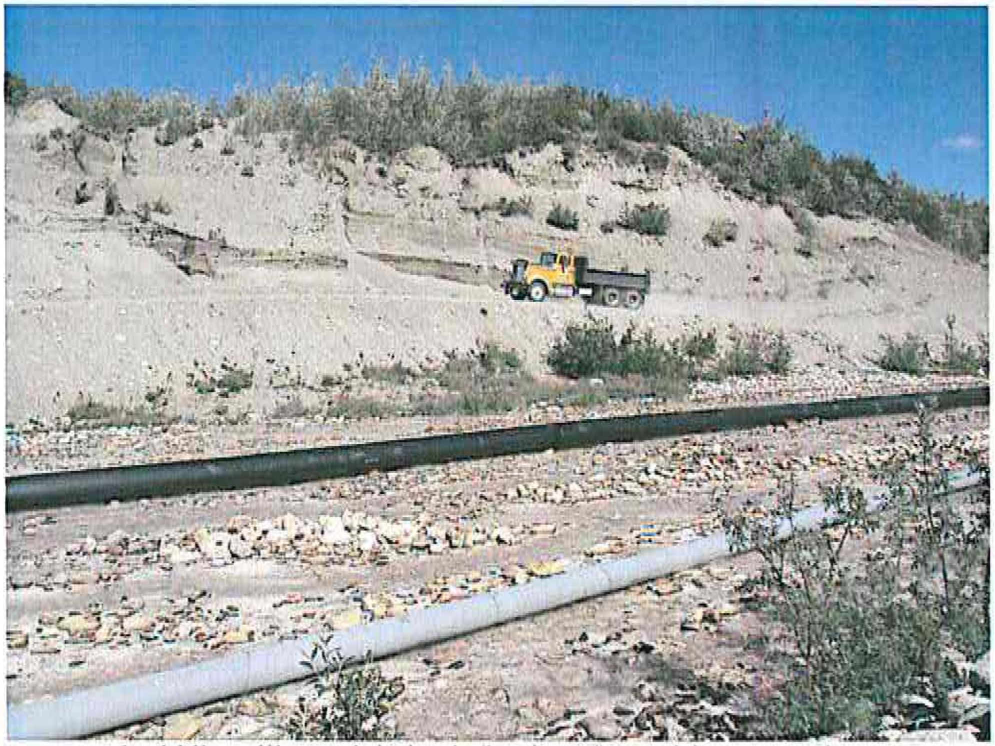
Notes:
1. Overbank distances may require adjustment
2. Manning's 'n' set to 0.045
3. Q500: 100 cms (taken from Cam Scott e-mail)
4. No PMF has been modelled
5. Depths vary from 0.5m at XS 1 to about 1.4m at XS 7 (transition to narrower channel downstream)

nhc

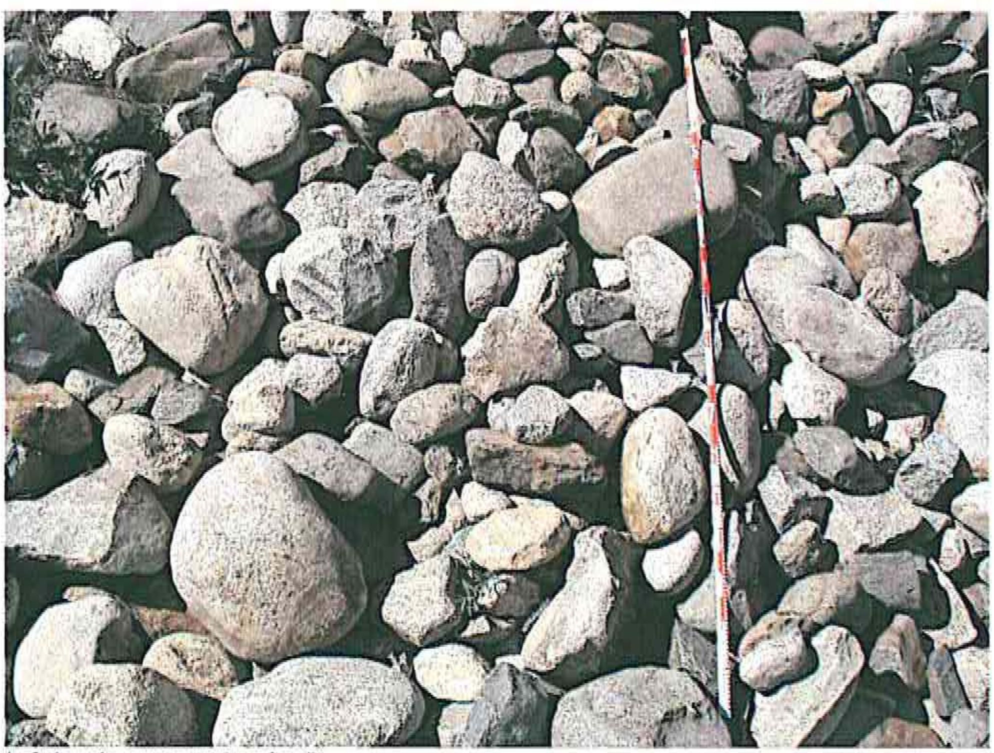
nhc

Figure 3 Modified Spillway WSL and Invert Profile



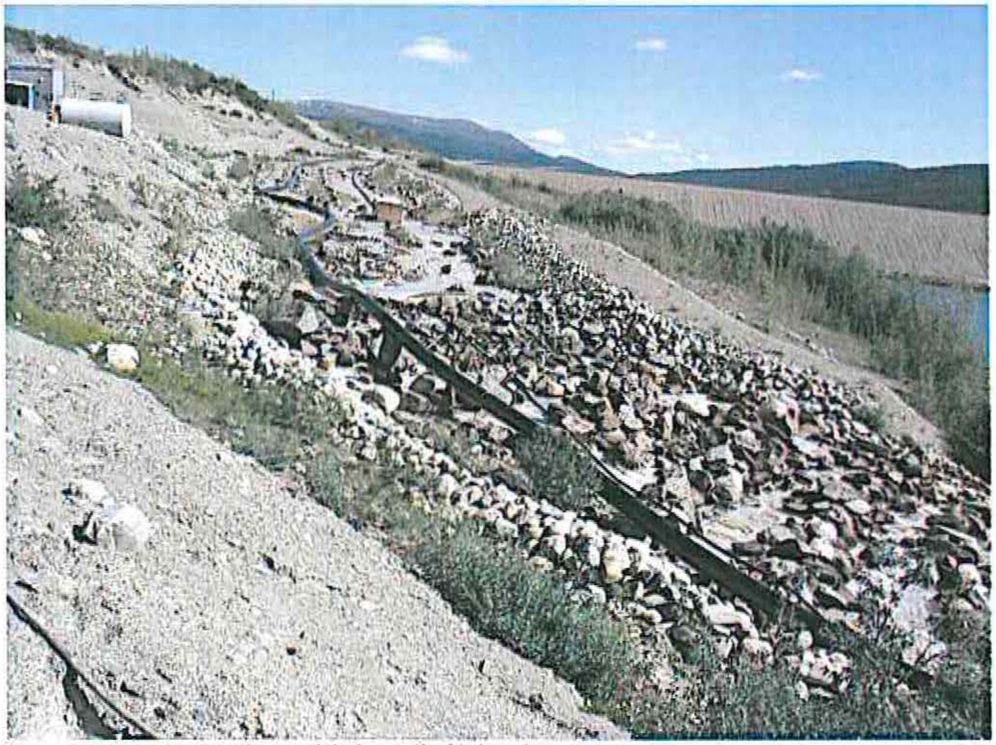


upper and middle spillway right bank (haul road) requiring protection



left bank armour (typical)

nhc



lower spillway section with low (left) bank

upper area (partial) requiring suggested invert lowering

APPENDIX C

SRK Consulting
Engineers and Scientists

Steffen, Robertson and Kirsten (Canada) Inc.
Suite 800 – 1066 West Hastings Street
Vancouver, B.C. V6E 3X2
Canada

vancouver@srk.com
www.srk.com

Tel: 604.681.4196
Fax: 604.687.5532

Technical Memorandum

To:	Cam Scott	**Date:**	December 2, 2003
cc:		**From:**	Joe Pun
Subject:	SPT Data from the Main Dams at the Rose Creek Tailings Facility	**Project #:**	1CD003.27

This memorandum summarizes the results of an assessment of the foundation soils in the vicinity of the Cross Valley and Intermediate Dams at the Rose Creek tailings impoundment, Anvil Range Mining Complex, Yukon.

1.0 Introduction

The Cross Valley and Intermediate Dams at the Rose Creek tailings impoundment retain water and tailings, respectively. Retention of the tailings solids at this facility depends on the satisfactory performance of these two dams.

The investigation and design of these dams was undertaken by Golder Associates in 1979 and 1980. The Golder assessment of the dynamic stability of these dams was geared mainly to operational criteria (a relatively modest seismic event) and was based on a simplified analysis that looked primarily at the Standard Penetration Test (SPT) blow counts that would be needed to withstand the design event (Golder, 1980). It appears that, since the SPT blow counts, or N-values, obtained from the field investigations generally exceeded this recommended value, no further assessment was undertaken. It also appears that the field N-values were not converted to normalized values, referred to as $(N_1)_{60}$ values (Golder, 1980 and Gilchrist, 2003).

This memo summarizes the source of available SPT data and the conversion method, i.e. N-values to $(N_1)_{60}$ values. It is intended that this information be used in subsequent one-dimensional computer analyses to assess the performance of these soils during the design earthquake, the magnitude of which will depend on current dam design practise and a seismic study being undertaken by others.

2.0 Available SPT Data

A series of boreholes were completed near the Cross Valley and Intermediate Dam (Figure TM-1) during the 1979 and 1980 field investigations. SPTs were undertaken in ten boreholes at the Cross Valley Dam and three boreholes at the Intermediate Dam. The locations of the holes in which SPTs were undertaken are shown on Figures TM-2 and TM-3, which were adapted from figures in the Golder (1980) report (refer to the information at each borehole label).

Figure TM-2 indicates that the SPT data covers most of the soil profile at the Cross Valley Dam, although SPT tests were not conducted at several boreholes in the mid-region of the valley to provide further details of that region.

Figure TM-3 indicates that most of the SPT data at the Intermediate Dam is confined to the soils in the channel on the south (left) side of the valley. There is no SPT data in the shallow soils that comprise the "bench" that was present immediately north of the channel. However, based on the seismic data, the seismic velocities within the soils in the upper 10 m or so of the bench are about half of those within the underlying soils. In general, the seismic velocity increases with increased relative density.

3.0 Conversion of Blow Counts from Field Values to Normalized Values

The SPT is a test in which a standard sampling tube is driven into the ground using standardized methods and equipment. The number of blows required over a select 300 mm (12 inch) interval corresponds to the field "N-value". The method used to convert the N-values to $(N_1)_{60}$ values is described below.

First, the N-values were corrected to account for differences in the energy ratios associated with different SPT test procedures. The drilling company responsible for the boreholes, Midnight Sun Drilling Co. Ltd., was contacted to verify that a safety hammer was used to perform the SPT tests. The energy ratio delivered by the safety hammer used by Midnight Sun Drilling Co. Ltd. was 60% (see Table 3.1), which is the ratio associated with equipment commonly used in North America. Conversion from the N-value to N_{60} is detailed in Seed, et al (1985) and Coduto (2001), with reference to Liao and Whitman (1986), using the following equation and the correction factors from Tables 3.1 and 3.2:

$N_{60} = E_m{*}C_B{*}C_S{*}C_R{*}N / 60$ .... (1)

where:

N_{60} = SPT N-value corrected for different test procedures

E_m = hammer efficiency (from Table 2.1)

C_B = borehole diameter correction (from Table 2.2)

C_S = sampler correction (from Table 2.2)

C_R = rod length correction (from Table 2.2)

N = SPT N-value

Table 3.1 SPT Hammer Efficiency Correction Factors (Adapted from Clayton, 1990)

Country	Hammer Type	Hammer Release Mechanism	Hammer Efficiency E_m
Argentina	Donut	Cathead	0.45
Brazil	Pin weight	Hand dropped	0.72
China	Automatic	Trip	0.60
	Donut	Hand dropped	0.55
	Donut	Cathead	0.50
Colombia	Donut	Cathead	0.50
Japan	Donut	Tombi trigger	0.78-0.85
	Donut	Cathead 2 turns + special release	0.65-0.67
UK	Automatic	Trip	0.73
US	Safety	2 turns on cathead	0.55-**0.60**
	Donut	2 turns on cathead	0.45
Venezuela	Donut	Cathead	0.43

Note: - Bold value has been adopted for use in equation (1)

Table 3.2 Borehole, Sampler and Rod Length Correction Factors (adapted from Skempton, 1986).

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	65-115 mm (2.5-4.5 in)	**1.00**
	150 mm (6 in)	1.05
	200 mm (8 in)	1.15
Sampling method factor, C_S	Standard sampler	**1.00**
	Sampler without liner (not recommended)	1.20
Rod length factor, C_R	3-4 m (10-13 ft)	**0.75***
	4-6 m (13-20 ft)	**0.85***
	6-10 m (20-30 ft)	**0.95***
	>10 m (>30 ft)	**1.00***

Note: - Bold values have been adopted for use in equation (1)
- *Dependent on depth

Information from Midnight Sun Drilling Co. Ltd. and Golder Associates indicates that a safety hammer with a standard split spoon sampler was used and that the borehole diameters were less than 115 mm. Therefore, the only correction factor from Table 3.2 that is necessary to apply for the N_{60} conversion was the rod length factor, C_R. The rod length factors were applied with respect to the various depths of SPT tests conducted, plus an additional stickup length of the drill rod from the ground surface to the top end, where it was connected to the drill rig. An exact measurement of the stickup length was unobtainable. However, a 3 m stickup was assumed for this assessment.

Another correction was made to account for the effect of overburden pressure. Uniform soil at greater depths has a higher N-value due to the effect of overburden pressure. The conversion from N_{60} to $(N_1)_{60}$ is detailed in Coduto (2001) and is based on the following equation:

$$(N_1)_{60} = N_{60}*(100\text{kPa} / \sigma_z')^{1/2}$$

where:

$(N_1)_{60}$ = SPT N-value corrected for different test procedures and overburden pressure

σ_z' = vertical effective stress at the test depth.

In the absence of dry unit weights for the overburden soils, the vertical effective stress could not be accurately calculated. The overburden pressure was therefore estimated on the basis of the borehole logs and typical dry unit weights for the soil types encountered (Holts et al, 1981).

4.0 Results

Summaries of the $(N_1)_{60}$ results for all the SPT test performed at various depths in the vicinity of the Cross Valley Dam and the Intermediate Dam are presented in Tables 4.1 and 4.2, respectively; the detailed calculations on which these summaries are based are included in Appendix A. The data are arranged by borehole in a south to north sequence across the valley, i.e. the same orientation used in the sections provided on Figures TM-2 and TM-3. The results highlighted are the $(N_1)_{60}$ values below 30, which represents soil that has a higher likelihood of experiencing strength loss during cyclic loading associated with a large earthquake.

Table 4.1 Summary of $(N_1)_{60}$ Values at the Cross Valley Dam

Elevation (m)	$(N_1)_{60}$ Values by Borehole									
	BH 79-6	BH 79-21	BH 79-7	BH 80-41	BH 79-18	BH 80-38A	BH 79-19	BH 79-16	BH 79-1	BH 79-15
1052	-	-	-	-	-	-	-	-	-	14
1051	-	-	-	-	-	-	-	-	-	16
1050	-	-	-	-	-	-	-	-	-	-
1049	-	-	-	-	-	-	-	-	-	22
1048	33	-	-	-	-	-	-	-	-	30
1047	-	-	-	54	-	-	-	-	49	-
1046	14	11	113	50	-	-	-	22	73	19
1045	-	36	-	-	154*	-	-	-	-	18
1044	-	-	-	-	-	32	-	20	59	-
1043	33	116	74	-	-	-	21	18	31	42
1042	27	40	-	-	131*	-	-	-	-	-
1041	-	-	33	-	117*	-	-	20	24	38
1040	33	51	14	-	-	-	-	20	32	-
1039	-	-	-	-	105*	-	-	-	-	-
1038	-	-	21	-	11	-	-	28	-	-
1037	18	37	11	-	-	-	-	51	48	56
1036	-	-	-	-	11	-	-	-	-	-
1035	-	-	19	-	11	-	-	31	22	-
1034	-	74	35	-	-	-	-	23	53	24
1033	-	-	-	-	16	-	-	-	-	-
1032	-	-	13	-	10	-	-	29	-	-
1031	-	67	21	-	-	-	-	7	-	5
1030	-	-	-	-	11	-	-	-	-	-
1029	-	-	-	-	14	-	-	18	-	-
1028	-	62	-	-	-	-	-	-	-	10
1027	-	-	41	-	-	-	-	23	-	-
1026	-	-	-	-	14	-	-	-	-	-
1025	-	58	-	-	-	-	-	-	-	46
1024	-	-	-	-	-	-	-	23	-	-
1023	-	-	58	-	-	-	-	58	-	-
1022	56	-	-	-	31	-	-	-	-	-
1021	-	-	-	-	-	-	-	-	-	43
1020	-	-	-	-	-	-	-	-	-	-
1019	-	-	-	-	22	-	-	-	-	-
1018	-	-	-	-	-	-	-	-	-	-
1017	-	50	-	-	-	-	-	-	-	-
1016	-	-	-	-	-	-	-	-	-	-
1015	-	-	-	-	41	-	-	-	-	-

*Permafrost layers with frozen soil observed. SPT N-values not representative

Table 4.2 Summary of $(N_1)_{60}$ Values at the Intermediate Dam

Elevation (m)	$(N_1)_{60}$ Values by Borehole		
	BH 80-37	BH 79-33	BH 80-46
1053	-	-	-
1052	-	-	154
1051	-	-	-
1050	-	-	-
1049	-	-	84
1048	11	-	-
1047	-	-	-
1046	-	-	22
1045	18	-	-
1044	-	-	-
1043	-	-	53
1042	-	12	-
1041	-	-	-
1040	-	-	40

5.0 References

Coduto, Donald P., 2001. *Foundation Design: Principles and Practices,* Second Edition, pp. 119 – 120.

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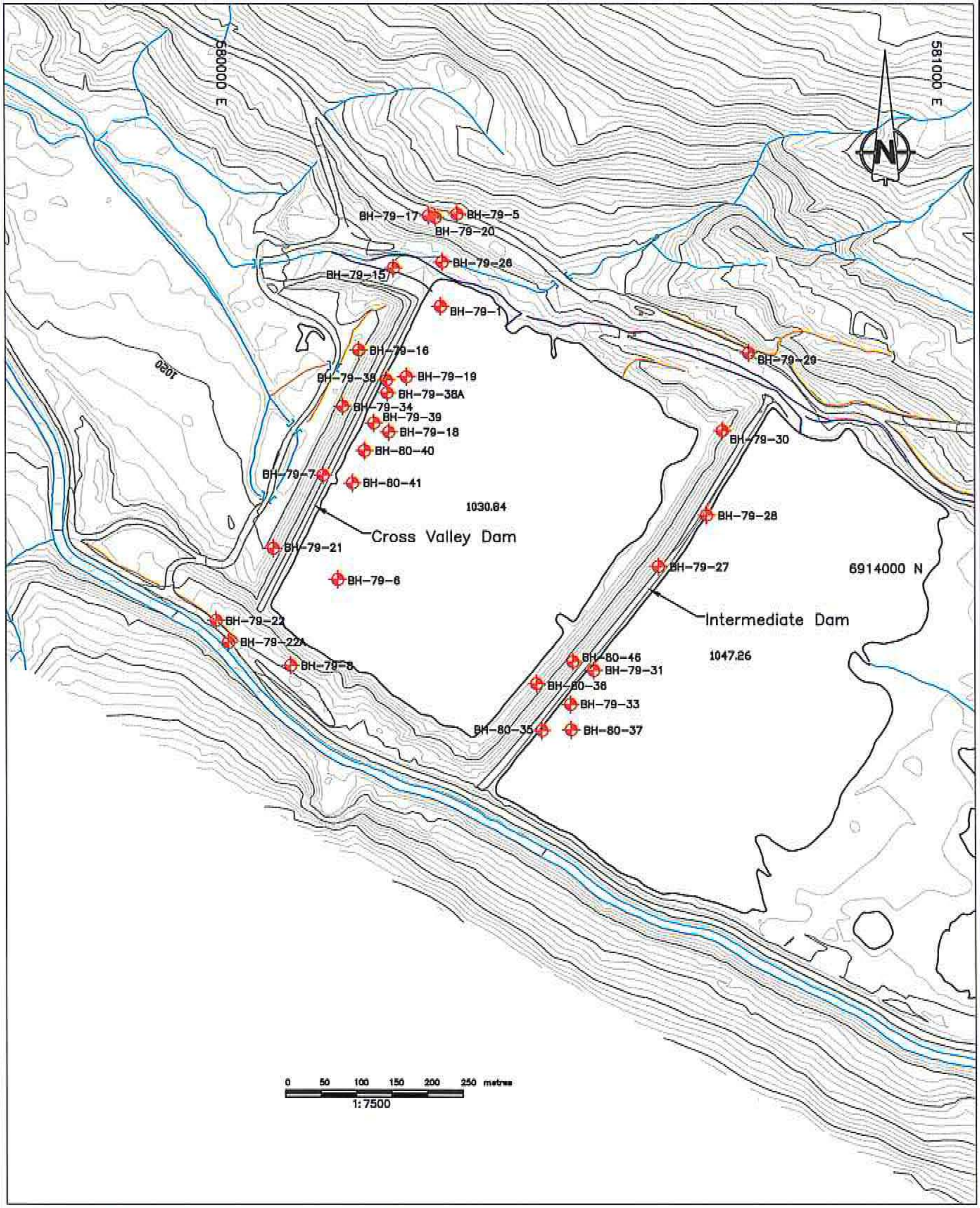
Golder Associates, 1980. Final Design Recommendations for the Down Valley Tailings Disposal Project.

Holtz, Robert D., and Kovacs, William D., 1981. *An Introduction to Geotechnical Engineering,* pp. 15.

Liao, S.S.C., and Whitman, R.V., 1986. "Overburden Correction Factors for SPT in Sand," *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No.3, p.373-377.

Seed, H. Bolton, Tokimatsu, K., Harder, L. F., and Chung, Riley M., 1985. "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," *Journal of Geotechnical Engineering,* Vol. 111, No. 12, December, pp. 1425 – 1445.

FIGURES



Dwg Ref: 2003 FARO SITE PLAN-Seep.dwg

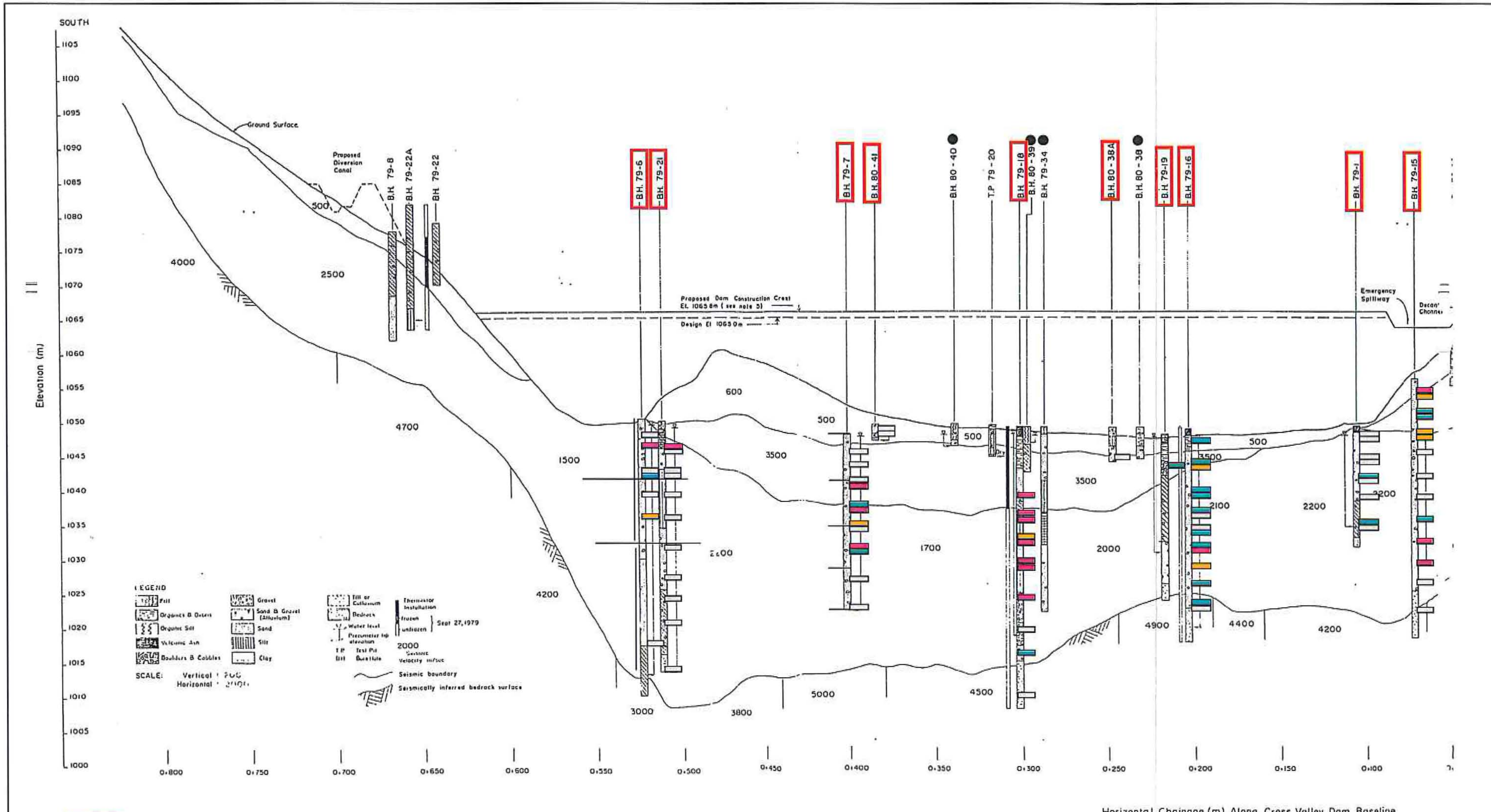
SRK Consulting
Engineers and Scientists

DELOITTE & TOUCHE INC.

FARO DAM GEOTECH INVESTIGATION

Borehole Locations at Cross Valley and Intermediate Dams

PROJECT NO.	DATE	APPROVED	FIGURE
1CD003.27	Oct. 2003	C.S.S.	TM-1



Borehole with SPT Data

- $(N_1)_{60}$ value > 30
- $(N_1)_{60}$ value 20 – 30
- $(N_1)_{60}$ value 15 - 20
- $(N_1)_{60}$ value < 15

● Borehole without SPT Data

X Borehole log not located

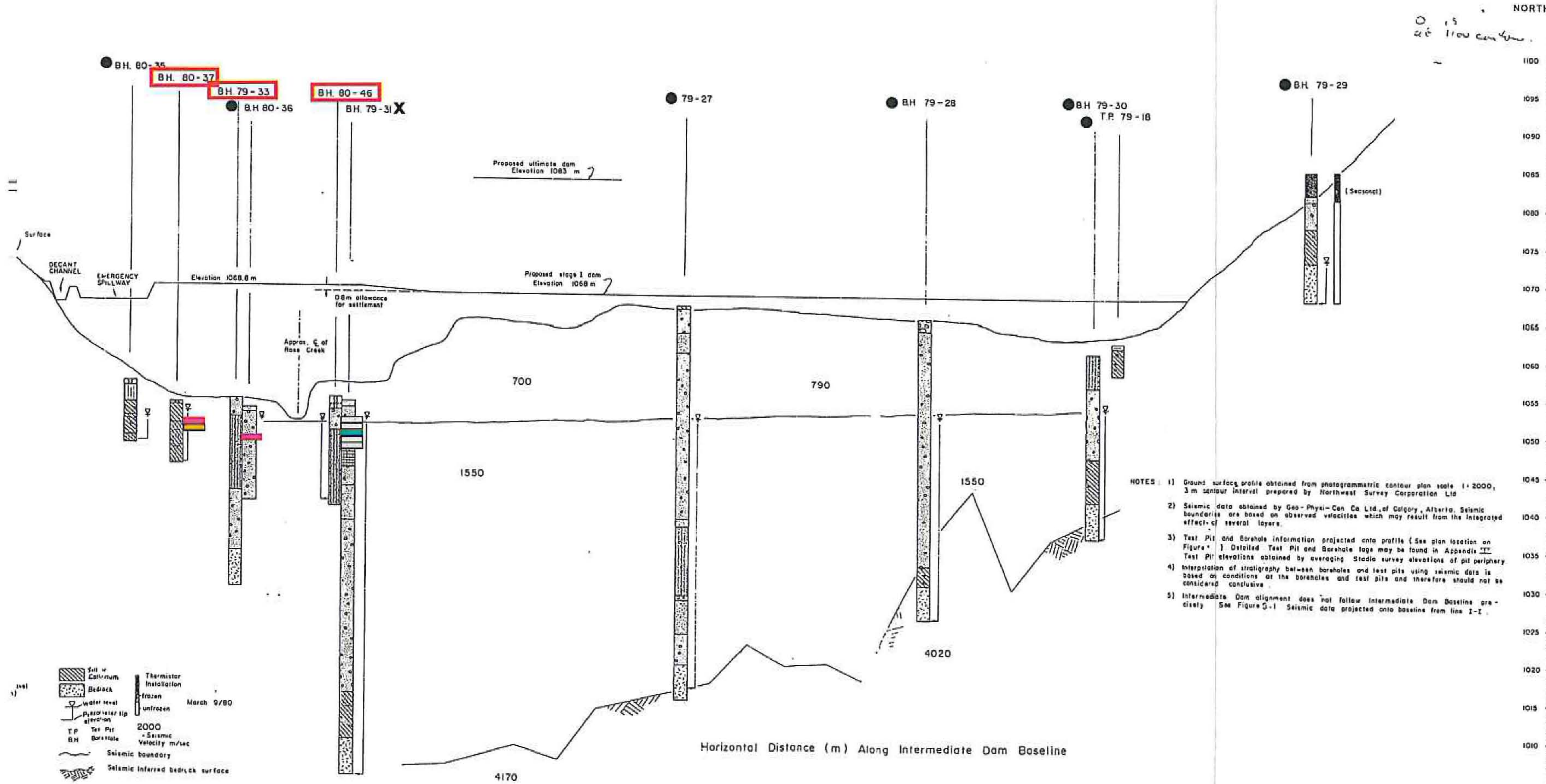
SRK Consulting
Engineers and Scientists

DELOITTE TOUCHE INC.

Faro Dam Geotech Investigation

Cross Valley Dam SPT Data

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.27	Oct. 2003		TM-2



Borehole with SPT Data

- $(N_1)_{60}$ value > 30
- $(N_1)_{60}$ value 20 – 30
- $(N_1)_{60}$ value 15 - 20
- $(N_1)_{60}$ value < 15

● Borehole without SPT Data

X Borehole log not located

SRK Consulting
Engineers and Scientists

DELOITTE TOUCHE INC.

Faro Dam Geotech Investigation

Intermediate Dam SPT Data

PROJECT:	DATE:	APPROVED:	FIGURE:
1CD003.27	Oct. 2003		TM-3

APPENDIX A

Calculations

Summary of SPT Data at the Cross Valley Dam
(in order from South to North along the valley)

	$(N_1)_{60}$									
Elevation	Borehole No.									
(m)	BH 79-6	BH 79-21	BH 79-7	BH 80-41	BH 79-18	BH 80-38A	BH 79-19	BH 79-16	BH 79-1	BH 79-15
1052	-	-	-	-	-	-	-	-	-	14
1051	-	-	-	-	-	-	-	-	-	16
1050	-	-	-	-	-	-	-	-	-	-
1049	-	-	-	-	-	-	-	-	-	22
1048	33	-	-	-	-	-	-	-	-	30
1047	-	-	-	54	-	-	-	-	49	-
1046	14	11	113	50	-	-	-	22	73	19
1045	-	36	-	-	154*	-	-	-	-	18
1044	-	-	-	-	-	32	-	20	59	-
1043	33	116	74	-	-	-	21	18	31	42
1042	27	40	-	-	131*	-	-	-	-	-
1041	-	-	33	-	117*	-	-	20	24	38
1040	33	51	14	-	-	-	-	20	32	-
1039	-	-	-	-	105*	-	-	-	-	-
1038	-	-	21	-	11	-	-	28	-	-
1037	18	37	11	-	-	-	-	51	48	56
1036	-	-	-	-	11	-	-	-	-	-
1035	-	-	19	-	11	-	-	31	22	-
1034	-	74	35	-	-	-	-	23	53	24
1033	-	-	-	-	16	-	-	-	-	-
1032	-	-	13	-	10	-	-	29	-	-
1031	-	67	21	-	-	-	-	7	-	5
1030	-	-	-	-	11	-	-	-	-	-
1029	-	-	-	-	14	-	-	18	-	-
1028	-	62	-	-	-	-	-	-	-	10
1027	-	-	41	-	-	-	-	23	-	-
1026	-	-	-	-	14	-	-	-	-	-
1025	-	58	-	-	-	-	-	-	-	46
1024	-	-	-	-	-	-	-	23	-	-
1023	-	-	58	-	-	-	-	58	-	-
1022	56	-	-	-	31	-	-	-	-	-
1021	-	-	-	-	-	-	-	-	-	43
1020	-	-	-	-	-	-	-	-	-	-
1019	-	-	-	-	22	-	-	-	-	-
1018	-	-	-	-	-	-	-	-	-	-
1017	-	50	-	-	-	-	-	-	-	-
1016	-	-	-	-	-	-	-	-	-	-
1015	-	-	-	-	41	-	-	-	-	-

*Permafrost layers with frozen soil observed. SPT N-values not representative

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-1
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 1.25 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1049.38	0.00	1.00		gravel (fill)	21.6							
1047.63	1.00		1.75	medium to coarse sand & fine to medium gravel	18.6	21.6	31	0.90	28	32.13	49	N/A
1046.13			3.25				54	0.96	52	49.79	73	N/A
1044.63			4.75				50	0.97	49	67.44	59	N/A
1043.13			6.25				29	0.99	29	85.10	31	N/A
1041.63			7.75				24	1.00	24	102.76	24	N/A
1040.13		11.28	9.25				35	1.00	35	120.42	32	N/A
1037.13	11.28		12.25	fine sandy silt		19.6	60	1.00	60	153.83	48	N/A
1035.63			13.75				28	1.00	28	168.55	22	N/A
1034.13		16.46	15.25				72	1.00	72	183.26	53	N/A

*Bedrock at depth = 16.46 m

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-6
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 0.65 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1050.03	0.00	1.68		gravel, cobbles, boulders	20.6	22.6						
1048.28	1.68		1.75	oxidized sandy and silty gravel, compact		21.6	23	0.90	21	27.35	40	N/A
1046.73			3.30				10	0.96	10	45.60	14	N/A
1043.73			6.30				30	0.99	30	80.91	33	N/A
1042.13		8.50	7.90				27	1.00	27	99.75	27	N/A
1040.63	8.50		9.40	boulder		19.6	36	1.00	36	115.64	33	N/A
1037.88		20.12	12.15				22	1.00	22	142.62	18	N/A
1022.28	20.12	32.80	27.75	very dense silty sand		22.1	100	1.00	100	314.37	56	N/A

*Bedrock at depth = Not Encountered

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-7
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 0.25 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m³)	Saturated Unit Weight (kN/m³)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m²)	$(N_1)_{60}$	Gravel Content (%)
1048.34	0.00		1.75	very dense sand & gravel	21.6	22.6	62	0.90	56	24.53	113	N/A
1043.59		5.79	4.75				60	0.97	58	62.78	74	N/A
1041.97	5.79		6.38	coarse sand & fine to medium gravel, compact		21.6	30	0.99	30	82.93	33	N/A
1040.34			8.00				14	1.00	14	102.06	14	31
1038.84			9.50				23	1.00	23	119.72	21	N/A
1037.34		12.20	11.00				13	1.00	13	137.38	11	N/A
1035.72	12.20		12.63	fine to coarse sand, compact		21.6	24	1.00	24	156.51	19	N/A
1034.22			14.13				46	1.00	46	174.17	35	N/A
1032.59			15.75				18	1.00	18	193.30	13	N/A
1031.09		20.73	17.25				30	1.00	30	210.95	21	[1] 40
1027.47	20.73		20.88	very dense sandy silt		20.6	66	1.00	66	253.49	41	N/A
1023.47		25.91	24.88				100	1.00	100	296.65	58	[2] 54

[1]additional 1% cobble was present as well
[2]additional 1% cobble was present as well

*Bedrock at depth = Not Encountered

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-15
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 8.30 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1056.1	0.00	2.44		sandy gravel, cobbles	20.6							
1052.75	2.44		3.35	coarse sand & fine to medium gravel, compact	21.6		12	0.96	11	69.91	14	N/A
1051.30			4.80				16	0.98	16	101.20	16	N/A
1049.75		7.16	6.35				26	0.99	26	134.65	22	N/A
1048.20	7.16		7.90	sandy to silty till, high proportion of bedrock schist	22.6	23.5	39	1.00	39	168.83	30	N/A
1046.70			9.40				27	1.00	27	192.96	19	N/A
1045.05			11.05				26	1.00	26	215.62	18	N/A
1043.60			12.50				64	1.00	64	235.54	42	N/A
1041.65			14.45				61	1.00	61	262.32	38	N/A
1037.85			18.25				100	1.00	100	314.51	56	N/A
1034.40		23.78	21.70				46	1.00	46	361.89	24	N/A
1031.45	23.78		24.65	silty fine sand with pebbles, dense		21.6	11	1.00	11	400.70	5	N/A
1028.30			27.80				20	1.00	20	437.78	10	N/A
1025.35		31.10	30.75				100	1.00	100	472.51	46	N/A
1021.00	31.10	38.11	35.10	BEDROCK		26.0	100	1.00	100	541.37	43	N/A

*Bedrock at depth = 31.10 m

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-16
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 1.50 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1048.25	0.00	0.91		gravel (fill)	21.6							
1046.50	0.91	2.74	1.75	organics	10.8	15.7	13	0.90	12	27.48	22	N/A
1044.88	2.74		3.38				13	0.96	12	40.78	20	61
1043.38			4.88				14	0.98	14	58.44	18	N/A
1041.75			6.50				18	0.99	18	77.57	20	N/A
1040.25			8.00				20	1.00	20	95.23	20	[1] 31
1038.75			9.50	medium to coarse sand & gravel, compact			30	1.00	30	112.88	28	N/A
1037.19			11.06			21.6	59	1.00	59	131.28	51	[2] 64
1035.69			12.56				38	1.00	38	148.94	31	N/A
1034.19			14.06				30	1.00	30	166.59	23	65
1032.63			15.63				40	1.00	40	184.99	29	N/A
1031.06			17.19				10	1.00	10	203.38	7	N/A
1029.50		19.21	18.75				27	1.00	27	221.77	18	N/A
1027.88	19.21		20.38	medium to coarse sandy silt, dense		20.6	36	1.00	36	239.76	23	70
1024.75		23.78	23.50				38	1.00	38	273.48	23	N/A
1023.00	23.78	25.61	25.25	silty sand, dense		20.6	100	1.00	100	292.37	58	54

[1]additional 1% cobble was present as well
[2]additional 8% cobble was present as well

*Bedrock at depth = 25.61 m

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-18
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 1.00 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1049.23	0.00	1.52		gravel (fill)	21.6	22.6						
1045.93	1.52	6.10	3.30	frozen organic silt**		15.7	100	0.96	96	38.69	154	N/A
1042.98	6.10		6.25	frozen, fine to medium sand, trace gravel**		20.6	100	0.99	99	56.79	131	N/A
1041.43			7.80				100	1.00	100	73.52	117	N/A
1039.88		10.36	9.35				100	1.00	100	90.24	105	N/A
1038.23	10.36		11.00	fine to coarse sand & gravel		21.6	11	1.00	11	108.68	11	N/A
1036.73		13.11	12.50				12	1.00	12	126.33	11	N/A
1035.23	13.11		14.00	fine to medium sand & silt, compact		19.6	13	1.00	13	142.25	11	N/A
1033.73			15.50				20	1.00	20	156.96	16	N/A
1032.13			17.10				13	1.00	13	172.66	10	N/A
1030.58			18.65				15	1.00	15	187.86	11	N/A
1029.08			20.15				20	1.00	20	202.58	14	N/A
1026.03		24.38	23.20				22	1.00	22	232.50	14	N/A
1022.98	24.38		26.25	sandy gravel, dense		22.6	51	1.00	51	267.92	31	N/A
1019.98		32.00	29.25				38	1.00	38	306.18	22	N/A
1015.38	32.00	34.14	33.85	sand & gravel, v. dense		22.6	79	1.00	79	364.84	41	N/A

*Bedrock at depth = 34.14 m

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-19
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 0.50 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1048.26	0.00	1.52		gravel (fill)	21.6	22.6						
1043.39	1.52	6.19	4.88	organic silt		15.7	14	0.98	14	43.55	21	55

*Bedrock at depth = 21.95 m

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 79-21
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 0.90 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1050.07	0.00	2.13		cobbles & boulders	16.7	19.6						
1046.82	2.13	3.96	3.25	fine sandy silt, loose		18.6	7	0.96	7	36.96	11	25
1045.13	3.96		4.94	coarse sand & fine gravel, compact		21.6	27	0.98	26	54.74	36	55
1043.57		7.01	6.50				100	0.99	99	73.13	116	N/A
1042.07	7.01		8.00	silty fine sand & sandy silt, dense		21.6	38	1.00	38	90.79	40	N/A
1040.45			9.63				53	1.00	53	109.92	51	[1] 37
1037.38		15.54	12.69				45	1.00	45	145.97	37	[2] N/A
1034.26	15.54		15.81	medium sand, sandy silt & till, very dense		22.6	100	1.00	100	183.03	74	N/A
1031.13			18.94				100	1.00	100	222.88	67	N/A
1028.07			22.00				100	1.00	100	261.94	62	N/A
1025.07			25.00				100	1.00	100	300.20	58	N/A
1017.20		24.38	32.88				100	1.00	100	400.63	50	N/A

[1]additional 18% cobble was present as well
[2]no gravel exist but with 5% cobble present

*Bedrock at depth = 34.14 m

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 80-38A
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 1.50 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1048.90	0.50	1.20		frozen organic silt	10.8							
1045.96	1.20	3.44		silt and fine sand	16.7	19.6						
1044.21	3.44	5.00	4.69	sand & gravel		20.6	23	0.97	22	50.45	32	72

*Bedrock at depth = Not Encountered

Cross Valley Dam
Borehole Data - SPT

Borehole # : BH 80-41
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 2.00 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1049.15	0.00	1.40		frozen sand & gravel		20.6						
1047.53	1.40		1.63	organic silt & fine sand	14.7	17.7	34	0.89	30	32.15	54	N/A
1046.96		2.30	2.19				34	0.92	31	39.14	50	N/A

*Bedrock at depth = Not Encountered

Summary of SPT Data at the Intermediate Dam
(in order from South to North along the valley)

	$(N_1)_{60}$		
Elevation	**Borehole No.**		
(m)	**BH 80-37**	**BH 79-33**	**BH 80-46**
1053	-	-	-
1052	-	-	154
1051	-	-	-
1050	-	-	-
1049	-	-	84
1048	11	-	-
1047	-	-	-
1046	-	-	22
1045	18	-	-
1044	-	-	-
1043	-	-	53
1042	-	12	-
1041	-	-	-
1040	-	-	40

Intermediate Dam
Borehole Data - SPT

Borehole # : BH 79-33
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 3.05 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1054.07	0.00	0.30		forzen sandy silt	16.7							
1053.77	0.30	2.74		sand and gravel	18.6							
1042.07	2.74	12.80	12.00	sandy silt	16.7	19.6	14	1.00	14	143.45	12	0

*Bedrock at depth = 20.12 m

Intermediate Dam
Borehole Data - SPT

Borehole # : BH 80-37
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 2.70 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m^3)	Saturated Unit Weight (kN/m^3)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m^2)	$(N_1)_{60}$	Gravel Content (%)
1053.41	0.00	0.30		organic silt	10.8							
1048.66	0.30		4.75	silty sand & till	20.6	22.6	10	0.97	10	78.82	11	1
1045.66		8.10	7.75				19	1.00	19	117.08	18	N/A

*Bedrock at depth = Not Encountered

Intermediate Dam
Borehole Data - SPT

Borehole # : BH 80-46
Type of hammer used: Safety Hammer
Estimated Rod Energy: 60 %

Water Level: 2.25 m depth

Elevation (m)	Beginning Depth of Soil (m)	End Depth of Soil (m)	SPT Test Depth (m)	Soil Description	Dry Unit Weight (kN/m³)	Saturated Unit Weight (kN/m³)	N (No. of blows)	Rod Length Factor (C_R)	N_{60}	Vertical Effective Stress σ_z' (kN/m²)	$(N_1)_{60}$	Gravel Content (%)
1054.11	0.00	0.90		peat & orgainc	10.8							
1053.21	0.90	1.50		silty sand	16.7							
1052.24	1.50	4.30	1.88	sand & gravel	18.6	20.6	88	0.91	80	26.71	154	N/A
1049.30	4.30		4.81	fine silty sand & fine to medium sand		19.6	67	0.98	65	60.85	84	N/A
1046.30			7.81				21	1.00	21	90.28	22	N/A
1043.24			10.88				58	1.00	58	120.32	53	[1] 36
1040.30		14.20	13.81				49	1.00	49	149.14	40	N/A

[1]additional 10% cobble was present as well

*Bedrock at depth = Not Encountered

APPENDIX D

LIQUEFACTION ASSESSMENT OF THE INTERMEDIATE DAM, ROSE CREEK TAILINGS IMPOUNDMENT YUKON TERRITORY

A Report to:

SRK Consulting, Engineers and Scientists

Suite 800 – 1066 West Hastings Street
Vancouver, B.C. V6E 3X2, Canada
Tel: 604.681.4196
Fax: 604.687.5532

by

Peter M. Byrne, Ph.D., P. Eng. and Mahmood Seid-Karbasi, M.Sc.
Peter M. Byrne Engineering Ltd.
4669 Osler Street, Vancouver, BC Canada V6H 2Y3
Phone: 604-822-2516, Fax: 604-822-6901
Email: pmb@civil.ubc.ca

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EXECUTIVE SUMMARY

The Rose Creek Tailings Impoundment is located in the Yukon. Cross Valley and Intermediate Dams retain sludge/water and tailings/water, respectively. Retention of the tailings solids depends on the satisfactory performance of these two dams. The impoundment is located in a region of moderate to high seismicity and a concern arose regarding stability of the Intermediate Dam under earthquake loading. The concern is whether liquefaction could be triggered in the foundation soil beneath the dam resulting in a failure of the dam and release of tailings into the Cross Valley Dam reservoir immediately downstream. A screening level study has been carried out to assess the potential for liquefaction and is reported herein.

As-built records indicated that the fill materials within the dam were generally well compacted and are therefore assumed to be dense. Available geotechnical information within the foundation soils, including Standard Penetration Test data, are more related to the Cross Valley Dam. Due to the limited data beneath the Intermediate Dam, soil information from beneath the Cross Valley Dam was also used to estimate materials properties for liquefaction assessment. Six different soil profiles representing different dam foundation conditions were analyzed for the design earthquake.

The design earthquake loading at this site has been addressed by Atkinson (2003). She recommended 6 earthquake time histories of acceleration be considered and these were applied as base input motions to compute the dynamic response and assess the potential for liquefaction. The results indicate that for the deepest section, near the left (south) abutment, liquefaction is predicted in one zone based on a single Standard Penetration Test value. On the right shoulder where the depth of soil is less, widespread liquefaction is predicted based on soil information from beneath the Cross Valley Dam.

We recommend that additional penetration testing be carried out at the site to verify the density / penetration resistance of all materials below the water table.

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1. INTRODUCTION

As requested by Mr. Cam Scott of SRK Consulting in October 2003, we have carried out an assessment of the potential for liquefaction of the foundation of the Intermediate Dam at Rose Creek Tailings Impoundment in the event of the design earthquake. The impoundment is located in a region of high seismicity in the Yukon and the dam is underlain by foundations soils that are generally dense but do contain some looser granular soils that could liquefy. Such liquefaction could cause a loss in strength and stiffness of the foundation soils and result in significant displacement or a possible flow failure of the dam.

The potential for liquefaction was examined by comparing the dynamic stresses caused by the design earthquake with the dynamic resistance of the foundation soil as a consequence of its density or penetration resistance. The design earthquake loading was provided by Atkinson (2003) and used to compute dynamic stresses. Standard Penetration Test data provided by Scott (2003) were used to estimate the dynamic resistance. The dynamic analyses were carried out using the commercially available PROSHAKE computer Program. This program simulates vertically propagating shear wave motion taking into account the nonlinear shear stress-strain and damping characteristics of the soil. PROSHAKE is essentially a newer and proven version of the older SHAKE program that has been a standard for many years.

This report is based on information provided by SRK Consulting including:

- Meeting with Mr. Cam Scott on Oct. 7, 2003
- Report by Joe Pun, SRK Consulting, Dec. 2003, "SPT Data from the Main Dams at the Rose Creek Tailings Facility".
- Report by Dr. Gail Atkinson, Dec. 2003, "Seismic Hazard Assessment for Faro, YK".
- E-mail communications from Mr. Cam Scott dated Jan.14, and Jan. 23, 2004.

The geotechnical conditions at the site, soil properties used in analyses, the design earthquake records, the ground response analyses, and the results of the liquefaction assessment are presented in this report.

2. IMPOUNDMENT PLAN AND DAM CROSS SECTION

A plan of the Rose Creek Tailings Impoundment showing the Cross Valley and Intermediate Dams is shown in Fig. 1. Fig. 2 shows a typical maximum cross section of the Intermediate Dam. Fig. 3 shows a section and borehole locations at the Intermediate Dam.

3. GEOTECHNICAL CONDITIONS

A series of boreholes was completed near the Cross Valley and Intermediate Dam (Fig. 1) during the 1979 and 1980 field investigations, as reported by Pun, 2003. Standard penetration tests (SPT's) were undertaken in ten boreholes at the Cross Valley Dam and three boreholes at the Intermediate Dam. The locations of the holes in which SPT's were undertaken are shown on Fig. 1, Fig. 3 and Fig. 4, which were adapted from figures TM-1 to TM-3 in the Pun memorandum (2003).

Fig. 3 indicates that most of the SPT data at the Intermediate Dam is confined to the soils in the channel on the south (left) side of the valley. There are no SPT data in the shallow soils that comprise the "bench" immediately north of the channel. Fig. 4 indicates that the SPT data cover much more of the soil profile at the Cross Valley Dam, although SPT's were not conducted at several boreholes in the mid-region of the valley. The available geotechnical data represent foundation conditions at the Cross Valley Dam foundation to a much greater extent than at the Intermediate Dam. The distance between the two dams is approximately 480 m.

Results of field tests in terms of $(N_1)_{60}$ values as reported by Pun, 2003 are summarized in Table 1 and Table 2 for the Intermediate and Cross Valley Dam respectively. As may be seen, borehole BH 80-46 is the only borehole in the vicinity of the center of the valley with lowest bedrock elevation (around El. 1011) where the dam cross section is deepest. Note that elevations in this report are based on local datum, known as Down Valley datum, which can be converted to mean sea level datum by subtracting 32.4 m. A soil profile with properties based on BH 80-46 data was used to represent the maximum cross section with deepest bedrock. Information from borehole BH 79-21 from the Cross Valley Dam section was used for elevations lower than that of the bottom of BH 80-46 and the assumed bedrock elevation.

Because of the limited data available at the Intermediate Dam location, two other soil profiles representing dam cross sections founded on shallower bedrock at El. 1028 and El. 1038 were developed based on BH 79-21 and BH 79-16 from the Cross Valley section to investigate foundation thickness effects on dam response.

The effects of the presence of the dam itself were examined by considering conditions both with and without the added dam material.

4. 1-D MODEL OF DAM-FOUNDATION

One dimensional models or soil columns representing soil conditions near the dam crest as well as at the toe are shown in Figs. 5 and 6. Monitoring of the piezometric level indicates that phreatic line in the downstream shell of the dam is at the drain layer elevation (El. 1064). Fig. 6-a shows the 1-D model of soil profile with top elevation at El. 1080 (for dam-foundation system) and bedrock elevation at 1011 for the maximum section. In Fig. 6-b the corresponding model for free field conditions (without dam) is depicted.

5. SOILS PROPERTIES AND PROFILES

Soil shear modulus at small strain (G_{max}) and its variation with shear strain and damping are the main materials parameters required for ground response analysis. G_{max} is best obtained from direct field measurements of shear wave velocity which are related as follows:

$$\mathbf{G_{max} = \rho . V_s^{\,2}} \qquad \textbf{Eq. 1}$$

Where:
ρ is soil bulk density.
V_s is shear wave velocity.

However, such tests were not carried out at this site.

Many investigators have correlated G_{max} with other in situ tests results e.g. SPT, N value. Equation 2 has been suggested by Seed et al. (1986) for sandy soils.

$$\mathbf{G_{maxv} = 434\ (N_1)_{60}\ )^{1/3} .\ Pa.\ (\sigma'_m/Pa)^{0.5}} \qquad \textbf{Eq. 2}$$

Where:
$(N_1)_{60}$ is the normalized SPT value for 60% energy at 100kpa overburden pressure.
Pa is atmospheric pressure (100 kPa).
σ'_m is mean effective stress.

Coarse materials e.g. gravels exhibit higher shear modulus values (50% on average) than sandy soils as reported by Stoke et al. (2004) and Ohta and Goto, 1978 (quoted in PIANC, 2001). Therefore G_{max} values based on $(N_1)_{60}$ were increased by 50% where applicable.

Soil profiles used in the analyses were based on three boreholes; BH 80-46, at the Intermediate Dam location and BH 78-21 and BH 79-16 at the Cross Valley Dam location. Materials properties were based on soil types and $(N_1)_{60}$ and are shown in Fig. 7 to Fig. 12 for profiles with and without the dam, respectively. Corresponding materials properties are tabulated in Table 3 to Table 8 for the six profiles.

6. EARTHQUAKE RECORDS

The earthquake records used in the analyses were recommended and provided by Atkinson (2003). Six earthquake records, including four from the 1989 Loma Prieta earthquake (recorded at Station Gilroy #3 and Lick Lab Station with two components each), and two records from the 1994 Northridge earthquake (recorded at Pacoima Dam Station) were used. The time histories of the six earthquakes are shown in Fig. 13 to Fig. 18. The peak ground acceleration of these records varies from 0.37g to 0.55g. Dr. Atkinson estimated the appropriate design earthquake magnitude at the site to be M7.2.

7. GROUND RESPONSE ANALYSES

Response analyses were carried out for the six soil profiles and for the six input motions using PROSHAKE (EduPro Civil Systems, Inc.) computer code. This program follows the original SHAKE approach and uses a frequency domain equivalent linear method to solve the ground response problem. In simple terms, the input motion is represented as the sum of a series of sine waves of different amplitudes, frequencies, and phase angles. The response of the soil profile is obtained for each frequency. The overall response is obtained by summing the individual responses for each input frequency. To approximate the nonlinear, inelastic response of soil, an equivalent linear approach is utilized in which, linear analyses are performed with soil properties shear modulus and damping values that are iteratively adjusted to be consistent with an effective level of shear strain induced in the soil.

The equivalent linear approach involves shear modulus and damping values that vary with the level of strain induced. The curves used are shown in Fig. 19 to Fig. 21.

8. LIQUEFACTION ASSESSMENT

Triggering of liquefaction is assessed by comparing the cyclic stress ratio, CSR, caused by the design earthquake with the capacity or cyclic resistance ratio, CRR, that the soil possesses due to density or $(N_1)_{60}$ value. A factor of safety against triggering liquefaction is defined as

$$F_{TRIG} = CRR / CSR \qquad \text{Eq. 3}$$

In the analysis carried out in this report the CSR was computed as follows;

$$CSR = 0.65\ (\tau_{max} / \sigma'_{vo}) \qquad \text{Eq. 4}$$

Where:

τ_{max} is the maximum dynamic shear stress computed from the PROSHAKE analyses at various depths,
σ'_{vo} is the vertical effective stresses prior to earthquake loading,
0.65 is a factor to approximate an equivalent uniform cyclic stress ratio.

CRR can be estimated from $(N_1)_{60}$ values as shown in Fig. 22 (NCEER workshop, Youd et al. 2001). Fig. 22 provides soil resistance, CRR_1, under reference conditions corresponding with M7.5, σ'_{vo} =100 kpa and level ground. CRR under general conditions can be estimated from Eq.5 (Youd et al. 2001).

$$\mathbf{CRR = K_\sigma . K_\alpha . K_m . CRR_1} \qquad \textbf{Eq. 5}$$

Where
K_σ is a factor to account for effective confining stress;
K_α is a factor to account for ground slope;
K_m is an earthquake magnitude scaling factor.

Fig 23 to Fig 24 show figures from NCEER used to determine K_σ and K_m. ground slope factor, K_α is assumed equal to1.

Effects of fine content on N value are accounted for as Eq. 6 (Youd et al. 2001).

$$\mathbf{(N_1)_{60cs} = \alpha + \beta\, (N_1)_{60}} \qquad \textbf{Eq. 6}$$

Where α and β are coefficients determined from the following relationships:

$\alpha = 0$ for FC < 5%
α = exp[1.76 2 (190/FC)] for 5% < FC < 35%
$\alpha = 5.0$ for FC > 35%
$\beta = 1.0$ for FC < 5%
β = [0.99 1 (FC /1,000)] for 5% < FC < 35%
$\beta = 1.2$ for FC > 35%

9. RESULTS OF ANALYSES

The results of the ground response analyses are shown in Fig. 25 to Fig. 30 in terms of predicted CSR versus depth. The CSR's are shown for each of the six earthquakes. The CRR is also shown on each of the same figures and allows the liquefaction potential to be assessed. Liquefaction is predicted if CSR > CRR.

Conditions Beneath the Crest of the Dam

Conditions beneath the crest of the dam are shown in Figs. 25, 26 and 27. Fig. 25 shows conditions near the deepest section of the Intermediate Dam, which are represented by BH 80-46. The results show that CRR is significantly greater than CSR except for one SPT value at about El. 1046. Fig. 26 shows results for soil conditions corresponding to BH 79-21 located near the deepest section of the Cross Valley Dam. It may be seen that conditions are similar to Fig. 25 with CRR > CSR except for a single reading at about El. 1056 where triggering of liquefaction is predicted. This suggests that conditions in the valley beneath the Intermediate and Cross Valley Dams may be reasonably similar. Fig. 27 shows conditions on the right shoulder. Since SPT data at the Intermediate Dam were not available, soil conditions at BH 79-16 beneath the Cross Valley Dam were used. The results indicate that significant zones below El 1064 could be triggered to liquefy by the design earthquake.

Conditions Beneath the toe of the Dam

Predicted CSR and CRR values versus depth for conditions at the downstream toe of the dam (without dam material) are shown in Figs. 28 to 30. Fig. 28 shows conditions corresponding to BH 80-46, and it may be seen that liquefaction is predicted in the depth range 0-10 m. Fig. 29 shows conditions corresponding to BH 79-21 and liquefaction is predicted in the depth range 0-10 m. Fig. 30 represents conditions on the right shoulder and indicates liquefaction is predicted in the depth range 0-10 m and also at depth, in particular at depth 20 m.

10. SUMMARY

A liquefaction assessment of the foundation soils beneath the Intermediate Dam at the Rose Creek Tailings Impoundment has been carried out. Triggering of liquefaction was assessed by comparing the Cyclic Stress Ratio, CSR, caused by the design earthquake with the Cyclic Resistance ratio, CRR, derived from standard penetration test, SPT, values from the site. The design earthquake was supplied by Dr. Atkinson and comprised of six acceleration records. The SPT data were supplied by SRK, and the dynamic analyses were carried out using the commercially available computer program PROSHAKE.

Six soil profiles (soil columns) were analyzed by applying all six records to each profile, and comparing CSR and CRR versus depth. At the deepest section, corresponding to conditions at BH 80-46, liquefaction is predicted to occur at El. 1046. This prediction is based one low SPT values at this elevation. However, conditions for BH 79-21 located beneath the Cross Valley Dam 480 m away show a similar trend. On the right shoulder, where the depth of soil is less and the soil conditions based on BH 79-16 at the Cross Valley Dam location, appear to be looser, widespread liquefaction is predicted for the design earthquake.

The available soil data are sparse at the Intermediate Dam location and were determined prior to construction. Consideration should be given to obtaining more information such as shear wave velocity and penetration data at this location, particularly in the shoulder area towards the right abutment. The analyses assumed that the fill material placed beneath the dam, which is currently beneath the water table, is dense. Consideration should be given to verifying its density.

This screening level study indicates that significant liquefaction could be triggered in the foundation beneath the Intermediate Dam. Such liquefaction could cause significant displacements of the dam and perhaps a flow slide and release of tailings. Further study of the liquefaction potential is, therefore, warranted.

Peter M. Byrne, Ph.D, P. Eng

Mahmood Seid-Karbasi, M.Sc.

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TABLES

Table 1: $(N_1)_{60}$ values for boreholes at the Intermediate Dam foundation

Elevation (m)	$(N_1)_{60}$ Values by Borehole		
	BH 80-37	BH 79-33	BH 80-46
1053	-	-	-
1052	-	-	154
1051	-	-	-
1050	-	-	-
1049	-	-	84
1048	11	-	-
1047	-	-	-
1046	-	-	22
1045	18	-	-
1044	-	-	-
1043	-	-	53
1042	-	12	-
1041	-	-	-
1040	-	-	40

Table 2: $(N_1)_{60}$ values for boreholes at the Cross Valley Dam foundation

Elevation (m)	$(N_1)_{60}$ Values by Borehole									
	BH 79-6	BH 79-21	BH 79-7	BH 80-41	BH 79-18	BH 80-38A	BH 79-19	BH 79-16	BH 79-1	BH 79-15
1052	-	-	-	-	-	-	-	-	-	14
1051	-	-	-	-	-	-	-	-	-	16
1050	-	-	-	-	-	-	-	-	-	-
1049	-	-	-	-	-	-	-	-	-	22
1048	33	-	-	-	-	-	-	-	-	30
1047	-	-	-	54	-	-	-	-	49	-
1046	14	11	113	50	-	-	-	22	73	19
1045	-	36	-	-	154*	-	-	-	-	18
1044	-	-	-	-	-	32	-	20	59	-
1043	33	116	74	-	-	-	21	18	31	42
1042	27	40	-	-	131*	-	-	-	-	-
1041	-	-	33	-	117*	-	-	20	24	38
1040	33	51	14	-	-	-	-	20	32	-
1039	-	-	-	-	105*	-	-	-	-	-
1038	-	-	21	-	11	-	-	28	-	-
1037	18	37	11	-	-	-	-	51	48	56
1036	-	-	-	-	11	-	-	-	-	-
1035	-	-	19	-	11	-	-	31	22	-
1034	-	74	35	-	-	-	-	23	53	24
1033	-	-	-	-	16	-	-	-	-	-
1032	-	-	13	-	10	-	-	29	-	-
1031	-	67	21	-	-	-	-	7	-	5
1030	-	-	-	-	11	-	-	-	-	-
1029	-	-	-	-	14	-	-	18	-	-
1028	-	62	-	-	-	-	-	-	-	10
1027	-	-	41	-	-	-	-	23	-	-
1026	-	-	-	-	14	-	-	-	-	-
1025	-	58	-	-	-	-	-	-	-	46
1024	-	-	-	-	-	-	-	23	-	-
1023	-	-	58	-	-	-	-	58	-	-
1022	56	-	-	-	31	-	-	-	-	-
1021	-	-	-	-	-	-	-	-	-	43
1020	-	-	-	-	-	-	-	-	-	-
1019	-	-	-	-	22	-	-	-	-	-
1018	-	-	-	-	-	-	-	-	-	-
1017	-	50	-	-	-	-	-	-	-	-
1016	-	-	-	-	-	-	-	-	-	-
1015	-	-	-	-	41	-	-	-	-	-

Table 3: Materials properties for BH 80 – 46 profile analysis with dam

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve
1	Dam Body (Gravel)	2	20	73	189.2	Gravel (Seed et al.)	Gravel (Seed et al.)
2	Dam Body (Gravel)	2	20	126.15	248.71	Gravel (Seed et al.)	Gravel (Seed et al.)
3	Dam Body (Gravel)	4	20	192.3	307.07	Gravel (Seed et al.)	Gravel (Seed et al.)
4	Dam Body (Gravel)	4	20	241.5	344.12	Gravel (Seed et al.)	Gravel (Seed et al.)
5	Dam Body (Gravel)	4	20	282	371.85	Gravel (Seed et al.)	Gravel (Seed et al.)
6	Fill (Gravel)	5	21	323.79	388.85	Gravel (Seed et al.)	Gravel (Seed et al.)
7	Fill (Gravel)	5	21	348.35	402.17	Gravel (Seed et al.)	Gravel (Seed et al.)
8	Fill (Gravel)	3	21	348.55	402.28	Gravel (Seed et al.)	Gravel (Seed et al.)
9	Foundation (Gravel)	2	20.6	488.85	482.41	Gravel (Seed et al.)	Gravel (Seed et al.)
10	Fine Silty Sand	2	19.6	332.2	407.69	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
11	Fine Silty Sand	3	19.6	227	337.01	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
12	Sand & Gravel	3	19.6	468	483.9	Gravel (Seed et al.)	Gravel (Seed et al.)
13	Sand & Gravel	3	19.6	437.25	467.74	Gravel (Seed et al.)	Gravel (Seed et al.)
14	Sand & Till	3.54	22.6	558.9	492.47	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
15	Sand & Till	3	22.6	556.35	491.34	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
16	Sand & Till	3	22.6	556.95	491.61	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
17	Sand & Till	8	22.6	579.3	501.37	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
18	Sand & Till	1.5	22.6	558	492.07	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
19	Sand & Till	8	22.6	590.4	506.15	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
20	Bed Rock	infinite	23	5,276.95	1,500.00	Linear	Linear

Table 4: Materials properties for BH 79 – 21 profile analysis with dam

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve
1	Dam Body (Gravel)	2	20	72.85	189	Gravel (Seed et al.)	Gravel (Seed et al.)
2	Dam Body (Gravel)	2	20	126.15	248.71	Gravel (Seed et al.)	Gravel (Seed et al.)
3	Dam Body (Gravel)	4	20	192.75	307.43	Gravel (Seed et al.)	Gravel (Seed et al.)
4	Dam Body (Gravel)	4	20	241.5	344.12	Gravel (Seed et al.)	Gravel (Seed et al.)
5	Dam Body (Gravel)	4	20	282.15	371.95	Gravel (Seed et al.)	Gravel (Seed et al.)
6	Fill (Gravel)	3	21	303.9	376.72	Gravel (Seed et al.)	Gravel (Seed et al.)
7	Foundation (Cobble & Boulder)	2	19.6	312.3	395.3	Gravel (Seed et al.)	Gravel (Seed et al.)
8	Foundation (Silty Sand)	2	18.6	152	283.09	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
9	Foundation (Sand & Gravel)	1.5	21.6	346.95	396.89	Gravel (Seed et al.)	Gravel (Seed et al.)
10	Foundation (Sand & Gravel)	1.5	21.6	450	452	Gravel (Seed et al.)	Gravel (Seed et al.)
11	Fine Silty Sand	2.5	21.6	253	338.92	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
12	Fine Silty Sand	3	21.6	284.8	359.59	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
13	Fine Silty Sand	2	21.6	262	344.9	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
14	Sand &Till	3.5	22.6	516.15	473.26	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
15	Sand &Till	3	22.6	516.22	473.29	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
16	Sand &Till	3	22.6	519	474.56	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
17	Sand &Till	8	22.6	416.4	425.07	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
18	Sand & Till	1	22.6	525	477.3	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
19	Bed Rock	infinite	23	5,276.95	1,500.00	Linear	Linear

Table 5: Materials properties for BH 79 – 16 profile analysis with dam

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve
1	Dam Body (Gravel)	2	20	72.85	189	Gravel (Seed et al.)	Gravel (Seed et al.)
2	Dam Body (Gravel)	2	20	126.2	248.76	Gravel (Seed et al.)	Gravel (Seed et al.)
3	Dam Body (Gravel)	4	20	192.8	307.47	Gravel (Seed et al.)	Gravel (Seed et al.)
4	Dam Body (Gravel)	4	20	241.65	344.22	Gravel (Seed et al.)	Gravel (Seed et al.)
5	Dam Body (Gravel)	4	20	282	371.85	Gravel (Seed et al.)	Gravel (Seed et al.)
6	Foundation (Gravel)	1	22	285	356.43	Gravel (Seed et al.)	Gravel (Seed et al.)
7	Foundation (Gravel)	1.5	22	293.7	361.83	Gravel (Seed et al.)	Gravel (Seed et al.)
8	Foundation (Sand & Gravel)	2	21.6	265.5	347.19	Gravel (Seed et al.)	Gravel (Seed et al.)
9	Foundation (Sand & Gravel)	2	21.6	264.9	346.8	Gravel (Seed et al.)	Gravel (Seed et al.)
10	Foundation (Sand & Gravel)	2.5	21.6	284.5	359.4	Gravel (Seed et al.)	Gravel (Seed et al.)
11	Sand & Gravel	1.5	21.6	290.85	363.39	Gravel (Seed et al.)	Gravel (Seed et al.)
12	Sand & Gravel	1.5	21.6	332.1	388.3	Gravel (Seed et al.)	Gravel (Seed et al.)
13	Sand & Gravel	1.5	21.6	413.6	433.34	Gravel (Seed et al.)	Gravel (Seed et al.)
14	Sand & Gravel	1.5	21.8	357	402.6	Gravel (Seed et al.)	Gravel (Seed et al.)
15	Sand & Gravel	1.5	21.6	329.2	386.6	Gravel (Seed et al.)	Gravel (Seed et al.)
16	Sand & Gravel	1.5	21.6	361.95	405.38	Gravel (Seed et al.)	Gravel (Seed et al.)
17	Sand & Gravel	1.5	21.6	229.4	322.73	Gravel (Seed et al.)	Gravel (Seed et al.)
18	Sand & Gravel	1	22.6	317.6	371.24	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
19	Sandy Silt	3.5	20.6	200	308.56	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
20	Sandy Silt	1	20.6	210	318.18	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
21	Silty Sand	1	20.8	396	434.19	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
22	Bed Rock	infinite	23	5,276.95	1,500.00	Linear	Linear

Table 6: Materials properties for BH 80-46 profile analysis without dam (free field)

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve
1	Foundation (Gravel)	2	20.6	72	185.14	Gravel (Seed et al.)	Gravel (Seed et al.)
2	Fine Silty Sand	2	19.6	80.4	200.57	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
3	Fine Silty Sand	3	19.6	75	193.72	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
4	Sand & Gravel	3	19.6	184	303.42	Gravel (Seed et al.)	Gravel (Seed et al.)
5	Fine Silty Sand	3	19.6	128.8	253.88	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
6	Sand & Till	3.54	22.6	278.6	347.7	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
7	Sand & Till	3	22.6	299.6	360.56	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
8	Sand & Till	3	22.6	318.6	371.82	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
9	Sand & Till	8	22.6	372.3	401.93	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
10	Sand & Till	1.5	22.6	364.1	397.48	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
11	Sand & Till	8	22.6	412.5	423.08	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
12	Bed Rock	infinite	23	5,276.95	1,500.00	Linear	Linear

Table 7: Materials properties for BH 79-21 profile analysis without dam (free field)

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve
1	Foundation (Cobble & Boulder)	2	19.6	50.47	158.91	Gravel (Seed et al.)	Gravel (Seed et al.)
2	Foundation (Silty Sand)	2	18.6	40.26	145.69	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
3	Foundation (Sand & Gravel)	1.5	21.6	115.1	228.6	Gravel (Seed et al.)	Gravel (Seed et al.)
4	Foundation (Sand & Gravel)	1.5	21.6	172.7	280.02	Gravel (Seed et al.)	Gravel (Seed et al.)
5	Fine Silty Sand	2.5	21.6	113.8	227.3	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
6	Fine Silty Sand	3	21.6	145.1	256.67	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
7	Fine Silty Sand	2	21.6	142	253.91	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
8	Sand &Till	3.5	22.6	305.4	364.04	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
9	Sand &Till	3	22.6	323.2	374.49	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
10	Sand &Till	3	22.6	339.8	383.99	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
11	Sand&Till	8	22.6	371	401.23	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
12	Sand &Till	1	22.6	377.3	404.62	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
13	Bed Rock	infinite	23	5,276.95	1,500.00	Linear	Linear

Table 8: Materials properties for BH 79-16 profile analysis without dam (free field)

Layer Number	Material Name	Thickness (m)	Unit Weight (kN/m^3)	Gmax (MPa)	Vs (m/sec)	Modulus Curve	Damping Curve
1	Foundation (Gravel)	1	22	40	133.53	Gravel (Seed et al.)	Gravel (Seed et al.)
2	Foundation (Gravel)	1.5	22	61	164.9	Gravel (Seed et al.)	Gravel (Seed et al.)
3	Foundation (Sand & Gravel)	2	21.6	86.7	198.4	Gravel (Seed et al.)	Gravel (Seed et al.)
4	Foundation (Sand & Gravel)	2	21.6	106.8	220.2	Gravel (Seed et al.)	Gravel (Seed et al.)
5	Foundation (Sand & Gravel)	2.5	21.6	134.6	247.21	Gravel (Seed et al.)	Gravel (Seed et al.)
6	Sand & Gravel	1.5	21.6	147.1	258.43	Gravel (Seed et al.)	Gravel (Seed et al.)
7	Sand & Gravel	1.5	21.6	177.4	283.8	Gravel (Seed et al.)	Gravel (Seed et al.)
8	Sand & Gravel	1.5	21.6	231.3	324.06	Gravel (Seed et al.)	Gravel (Seed et al.)
9	Sand & Gravel	1.5	21.6	207.7	307.08	Gravel (Seed et al.)	Gravel (Seed et al.)
10	Sand & Gravel	1.5	21.6	198	299.83	Gravel (Seed et al.)	Gravel (Seed et al.)
11	Sand & Gravel	1.5	21.6	224.3	319.12	Gravel (Seed et al.)	Gravel (Seed et al.)
12	Sand & Gravel	1.5	21.6	145.8	257.29	Gravel (Seed et al.)	Gravel (Seed et al.)
13	Sand & Gravel	1	21.6	205.1	305.15	Sand (Seed & Idriss) - Lower Bound	Sand (Seed & Idriss) - Lower Bound
14	Sandy Silt	3.5	20.6	160.25	276.2	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
15	Sandy Silt	1	20.6	183.5	278.99	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
16	Silty Sand	1	20.6	230.8	331.47	Sand (Seed & Idriss) - Upper Bound	Sand (Seed & Idriss) - Upper Bound
17	Bed Rock	infinite	23	5,276.95	1,500.00	Linear	Linear

FIGURES

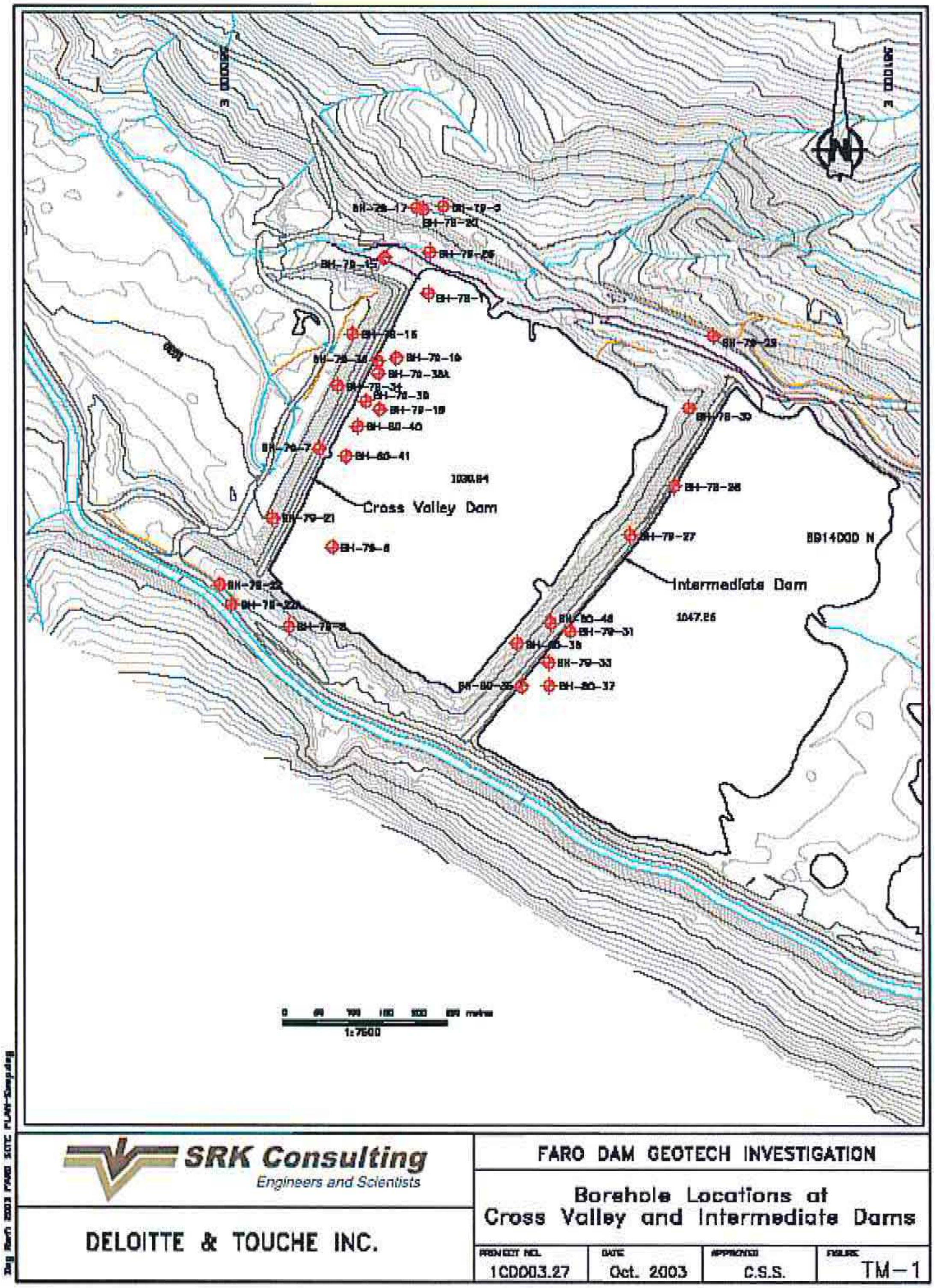

Fig. 1: Impoundment plan with drilled boreholes locations

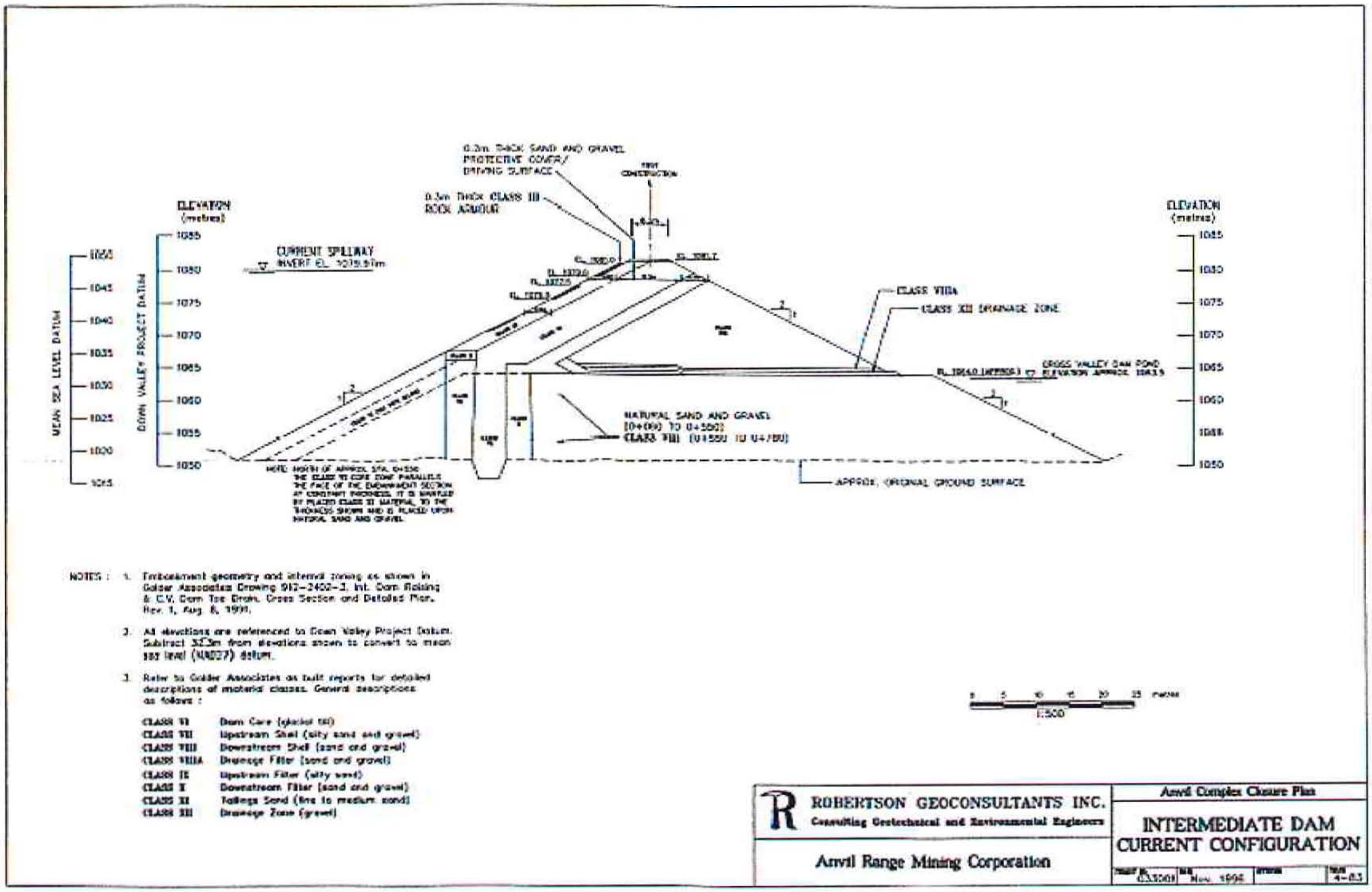

FIG. 2: Maximum typical cross section of the Intermediate Dam

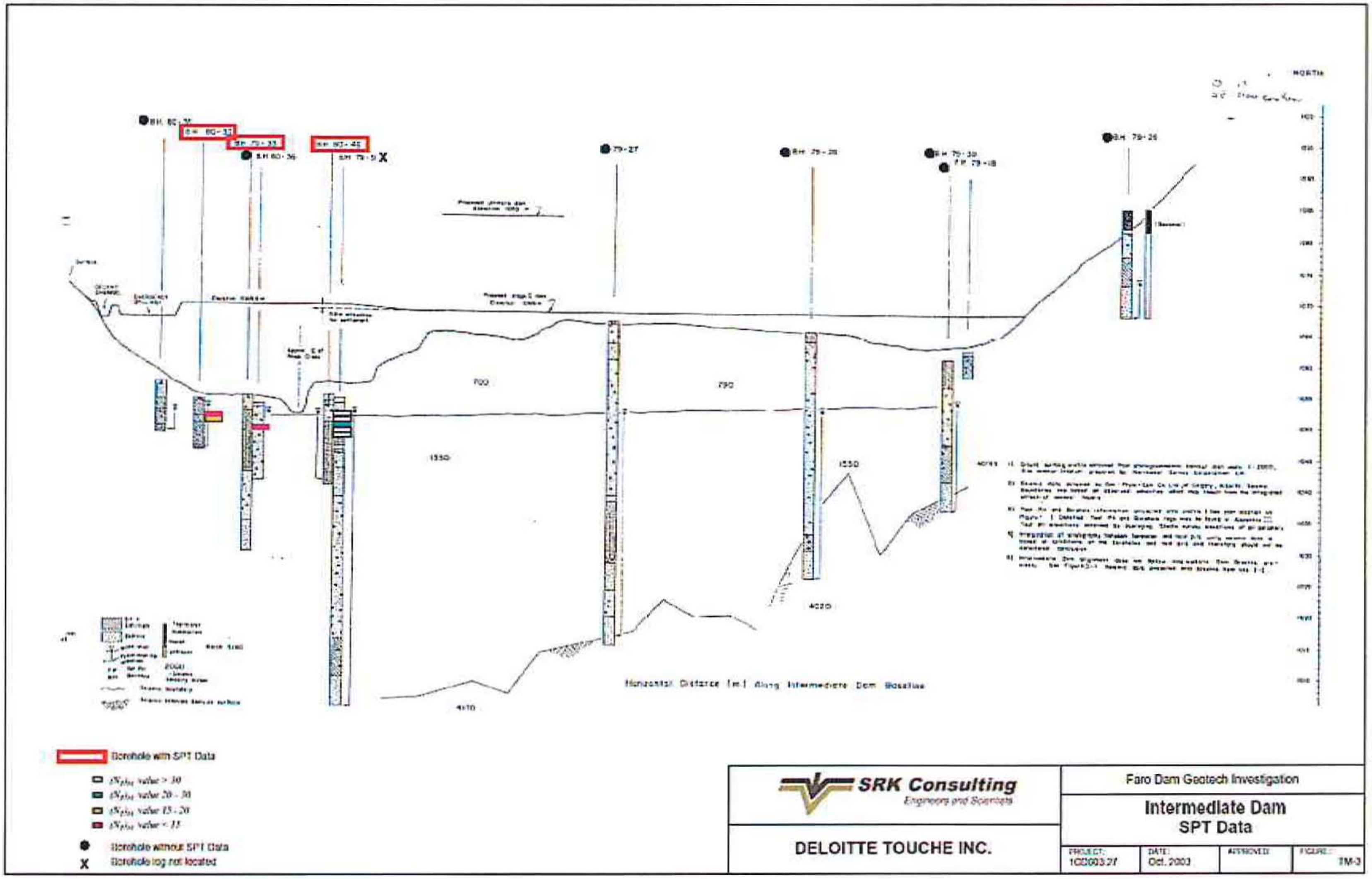

Fig. 3: Longitudinal section of Intermediate Dam foundation along with drilled boreholes positions

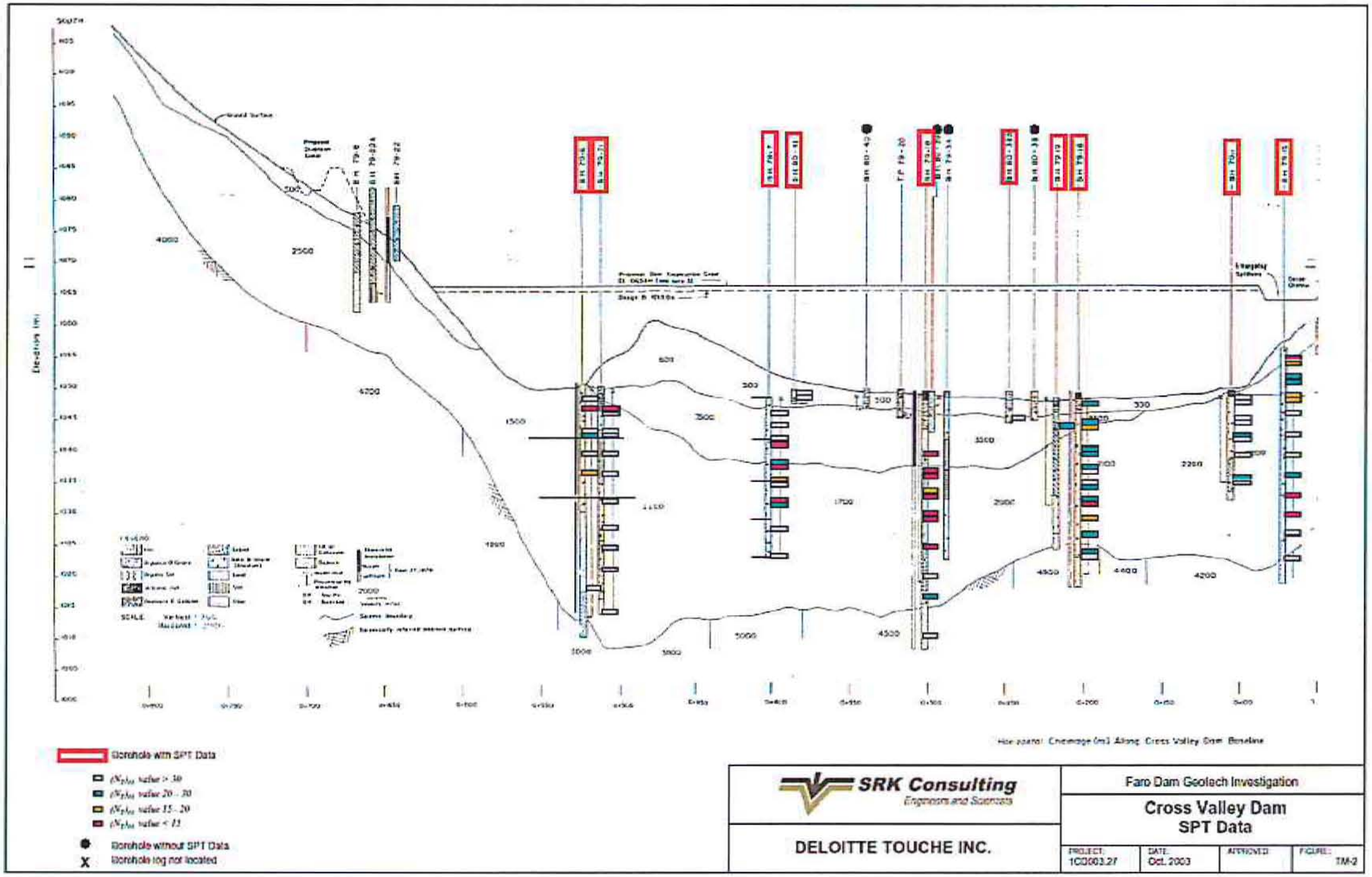

Fig. 4: Longitudinal section of Cross Valley Dam foundation along with drilled boreholes position

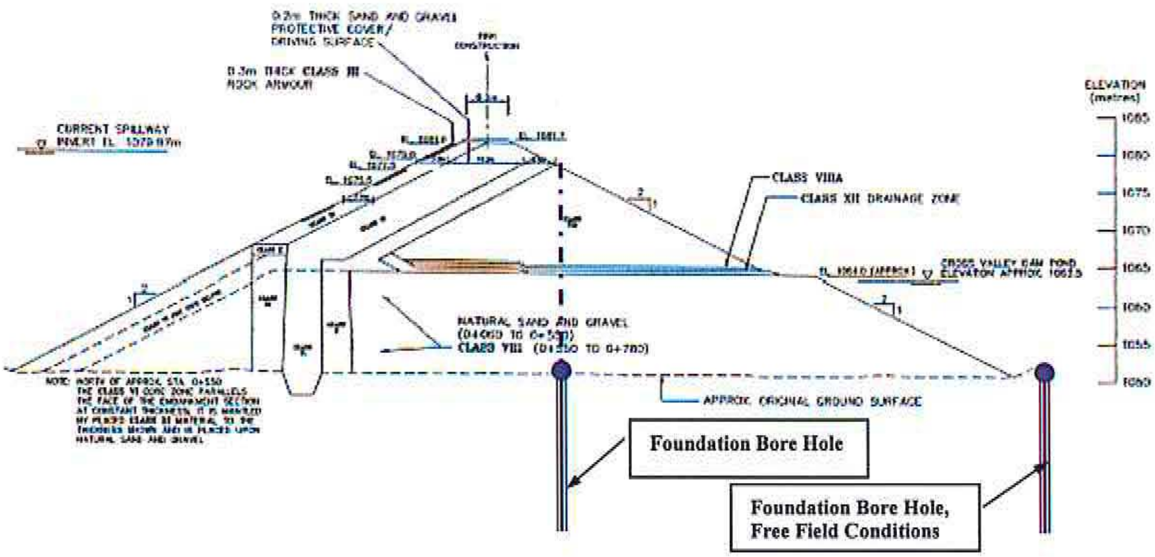

Fig. 5: Relative position of borehole in the dam cross section

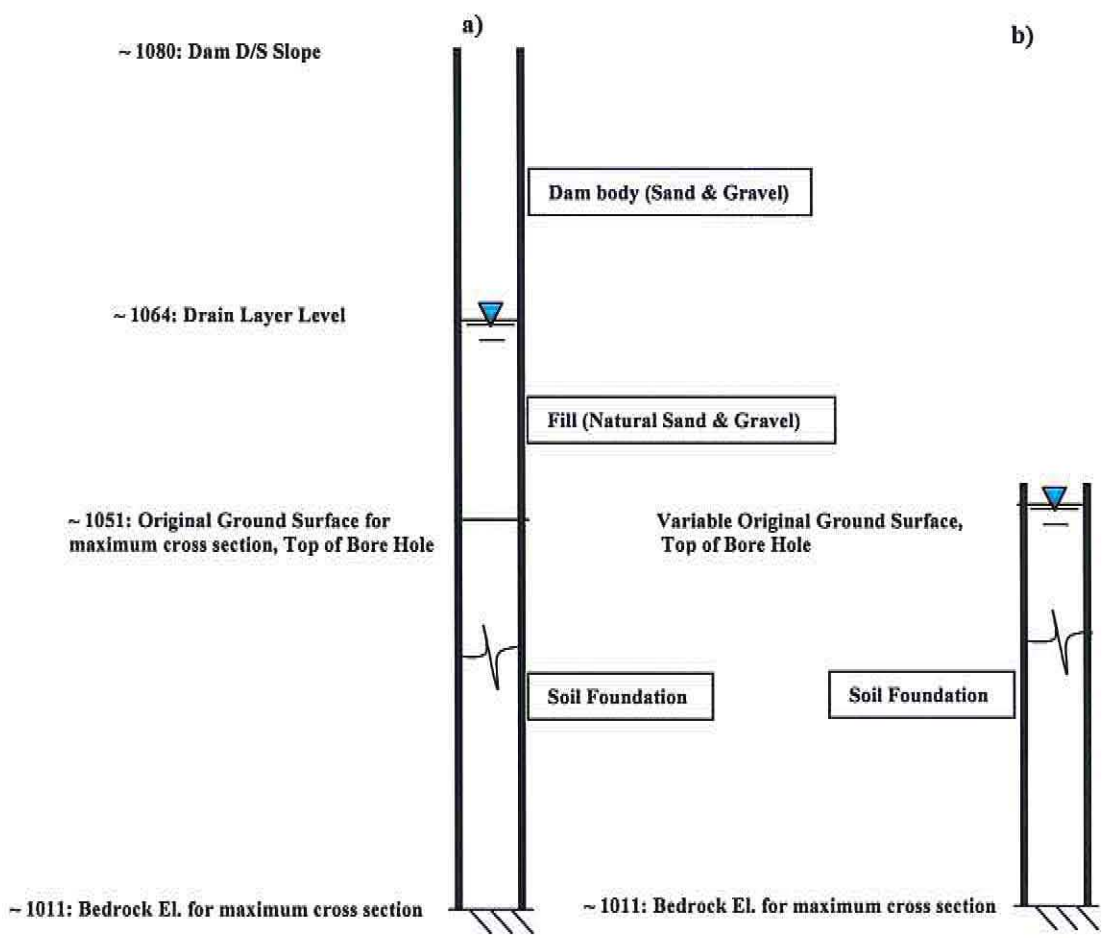

Fig. 6: 1-D model for ground response analysis, a) with dam, and b) without dam

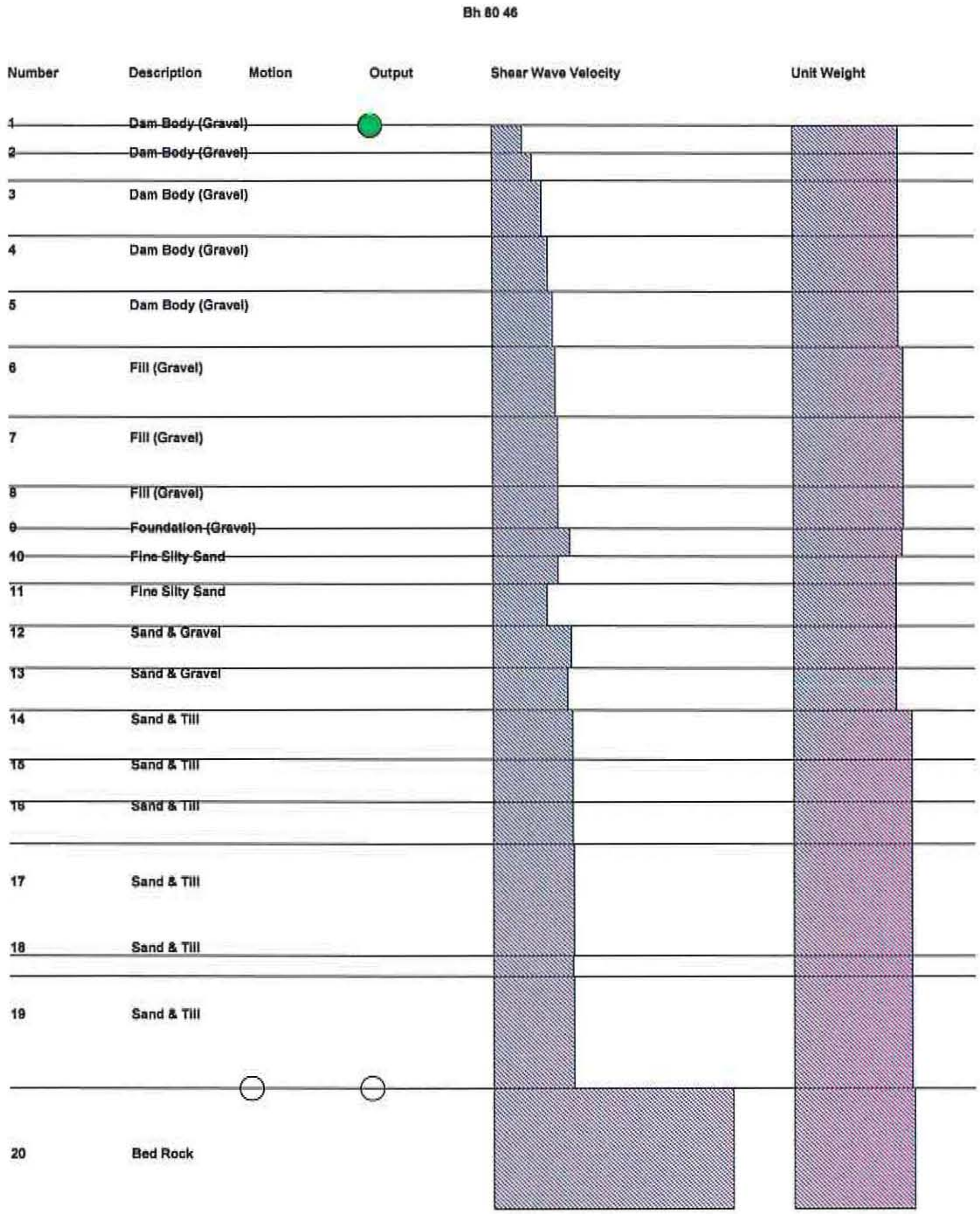

Fig. 7: PROSHAKE model for dam-foundation system using borehole BH 80-46 data

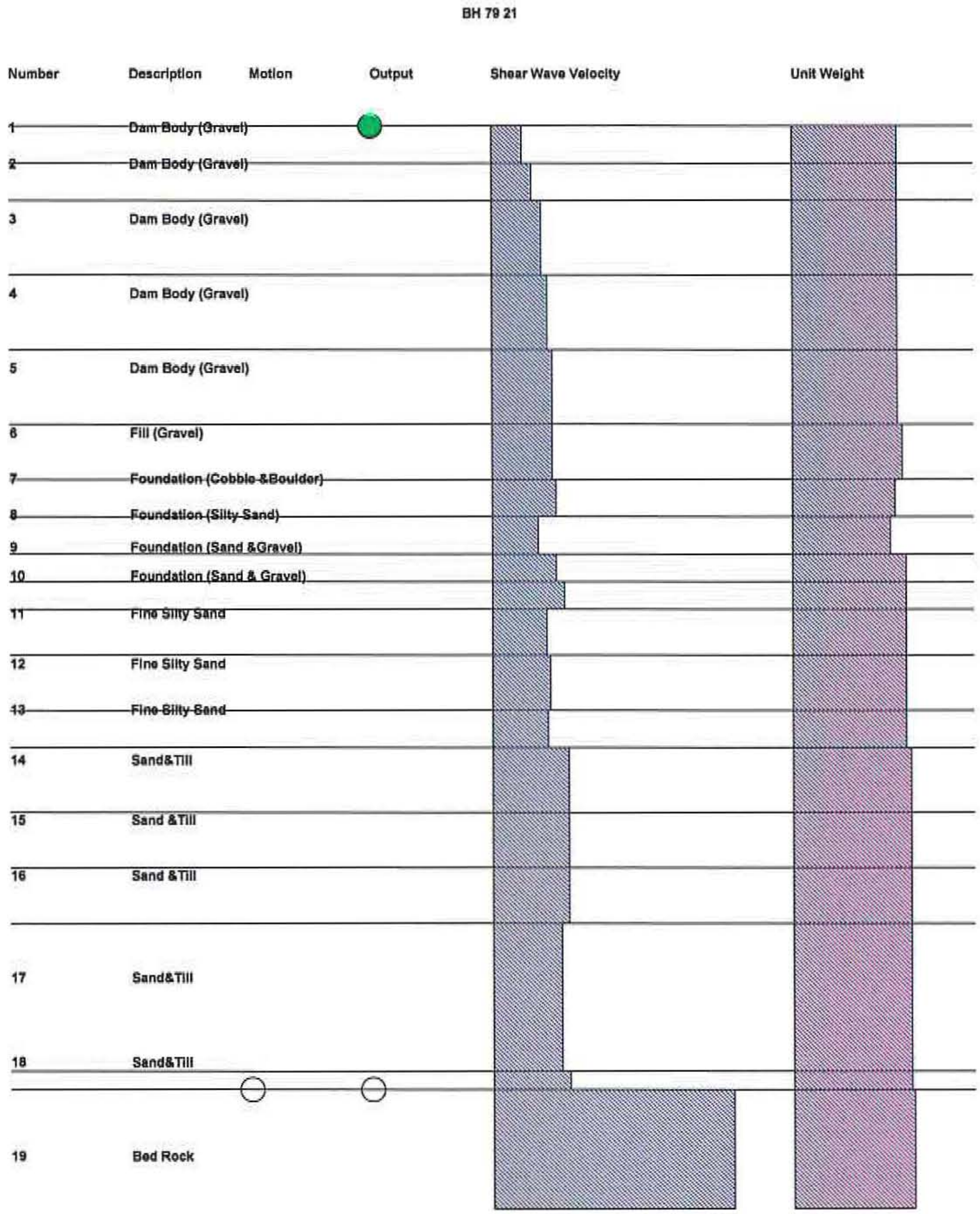

Fig. 8: PROSHAKE model for dam-foundation system using borehole BH 79-21 data

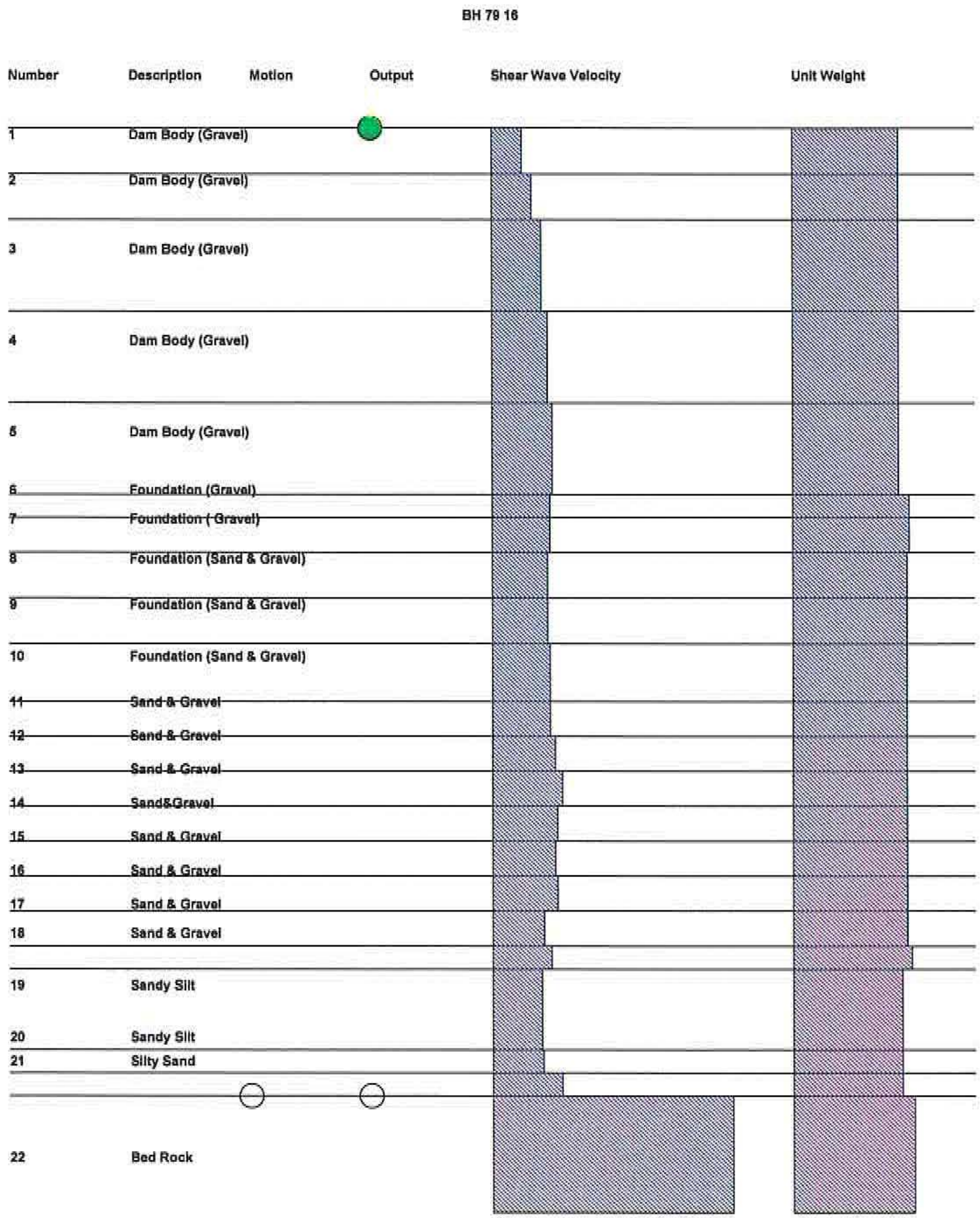

Fig. 9: PROSHAKE model for dam-foundation system using borehole BH 79-16 data

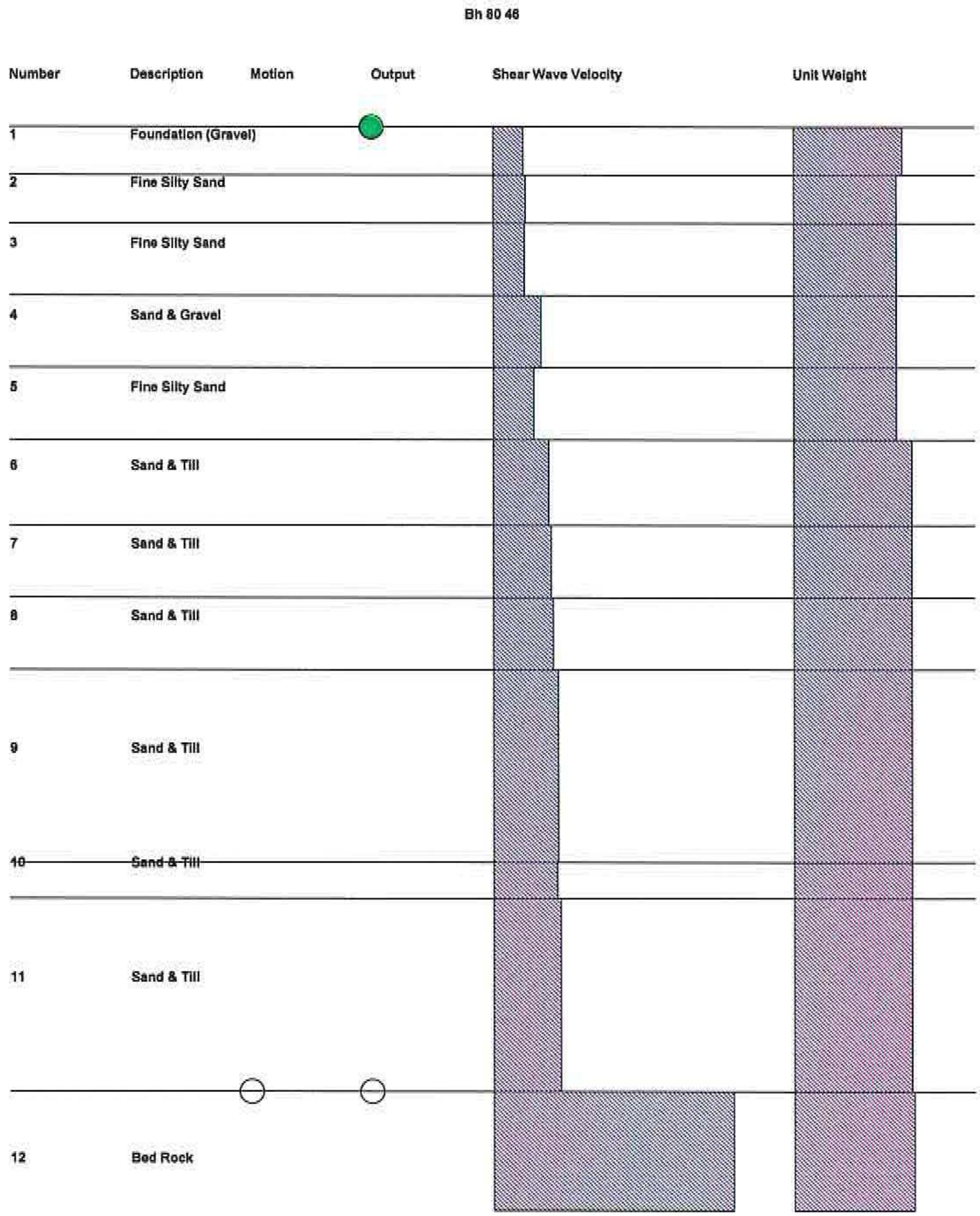

Fig. 10: PROSHAKE model for foundation system using borehole BH 80-46 data

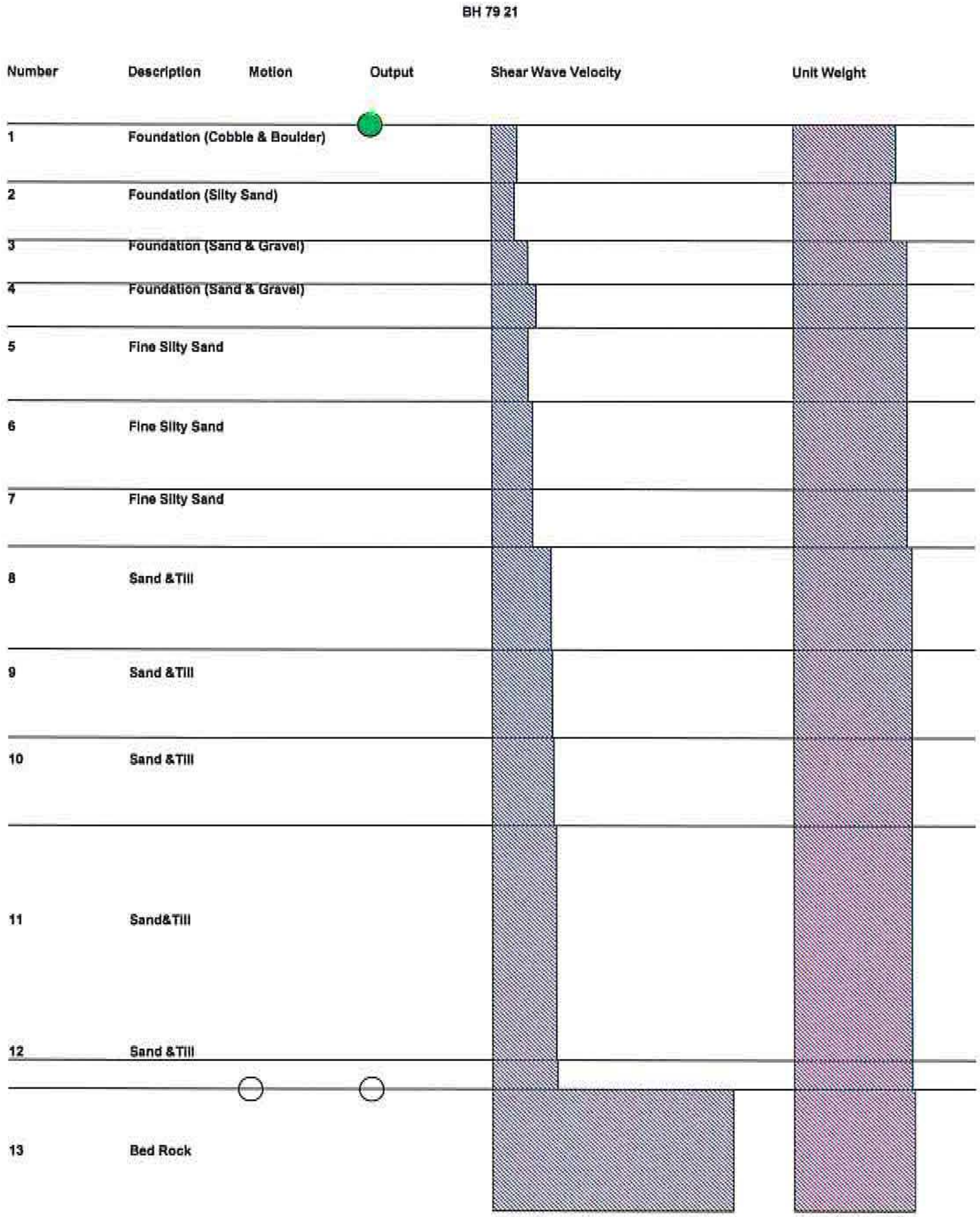

Fig. 11: PROSHAKE model for foundation system using borehole BH 79-21 data

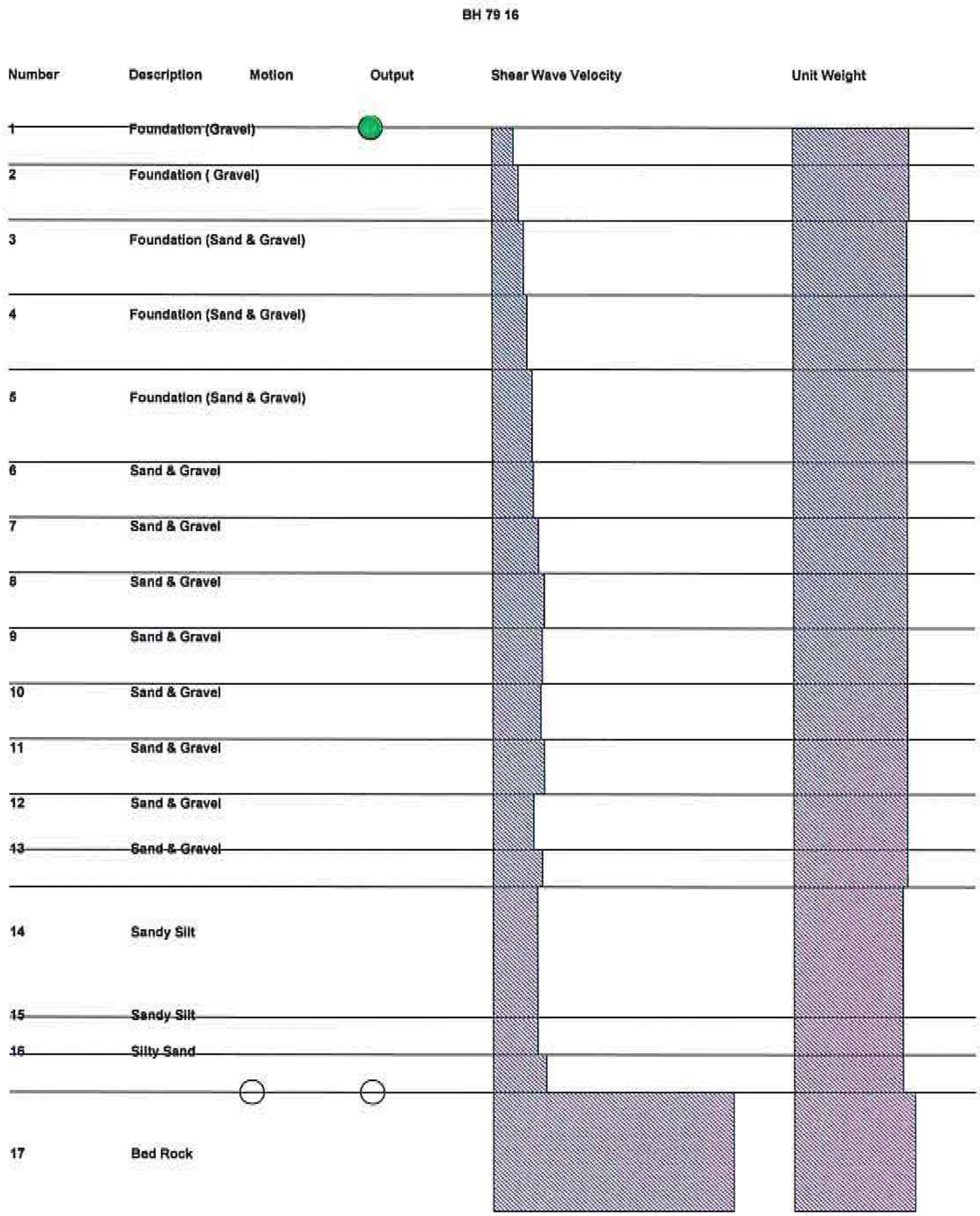

Fig. 12: PROSHAKE model for foundation system using borehole BH 79-16 data

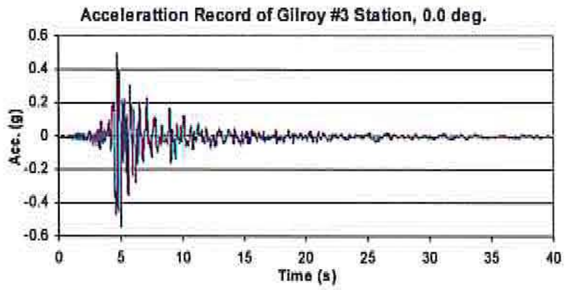

Fig. 13: Loma Prieta Earthquake record at Gilroy St. #3 at 0.0 deg.

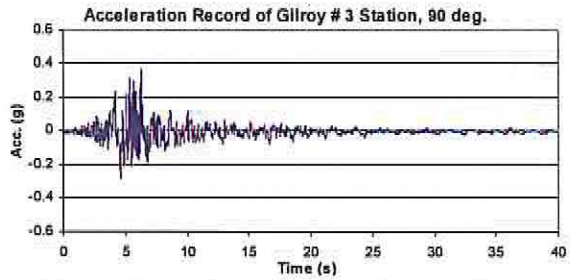

Fig. 14: Loma Prieta Earthquake record at Gilroy St. #3 at 90 deg.

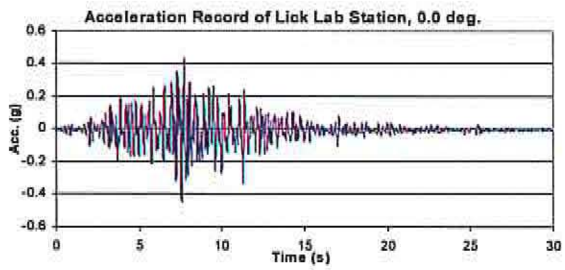

Fig. 15: Loma Prieta Earthquake record at Lick Lab St. at 0.0 deg.

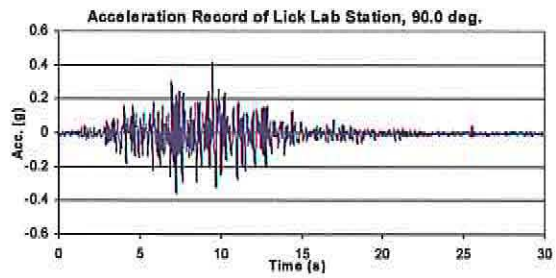

Fig. 16: Loma Prieta Earthquake record at Lick Lab St. at 90 deg.

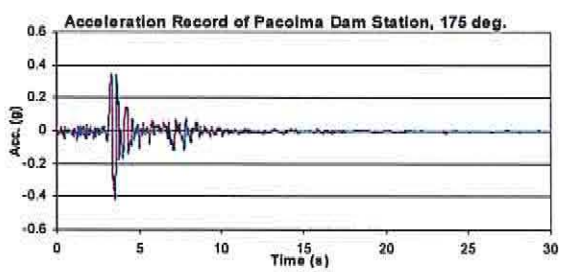

Fig. 17: Northridge Earthquake record at Pacoima Dam St. at 175 deg.

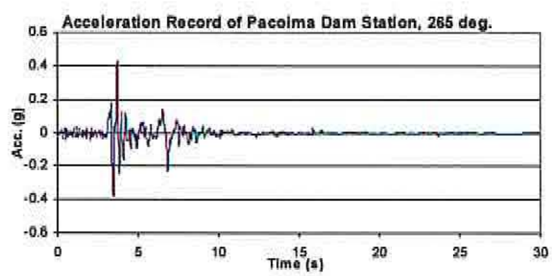

Fig. 18: Northridge Earthquake record at Pacoima Dam St. at 265 deg.

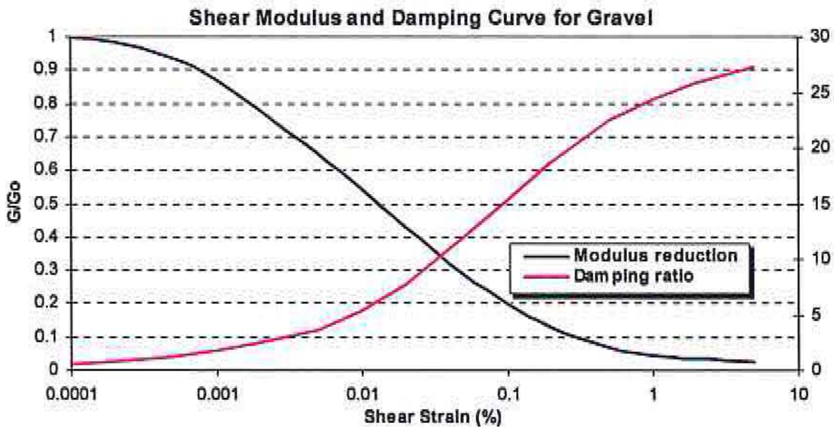

Fig. 19: G/Go and Damping ratio vs. shear strain for gravel (Seed et al. 1986)

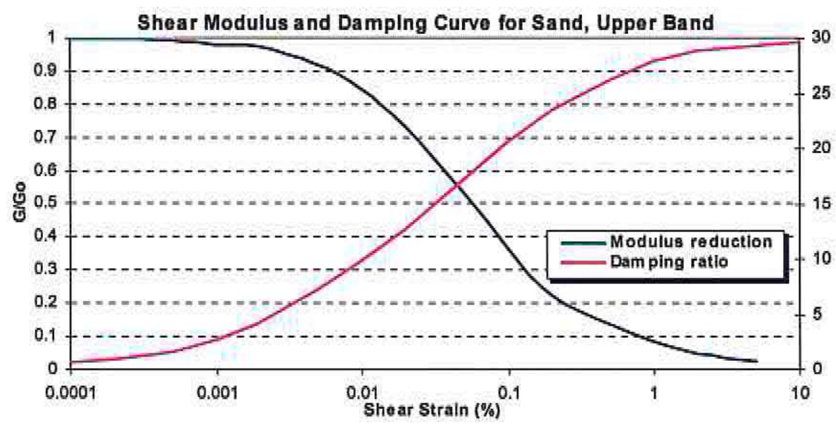

Fig. 20: G/Go and Damping ratio vs. shear strain for sand, upper band (Seed et al. 1986)

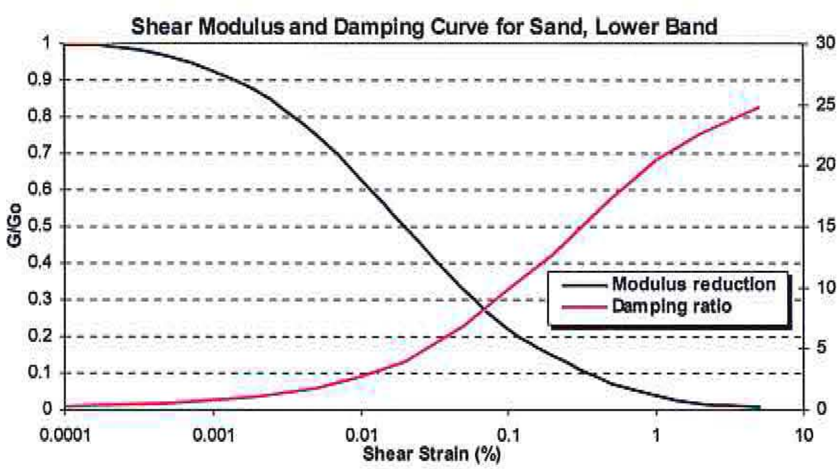

Fig. 21: G/Go and Damping ratio vs. shear strain for sand, lower band (Seed et al. 1986)

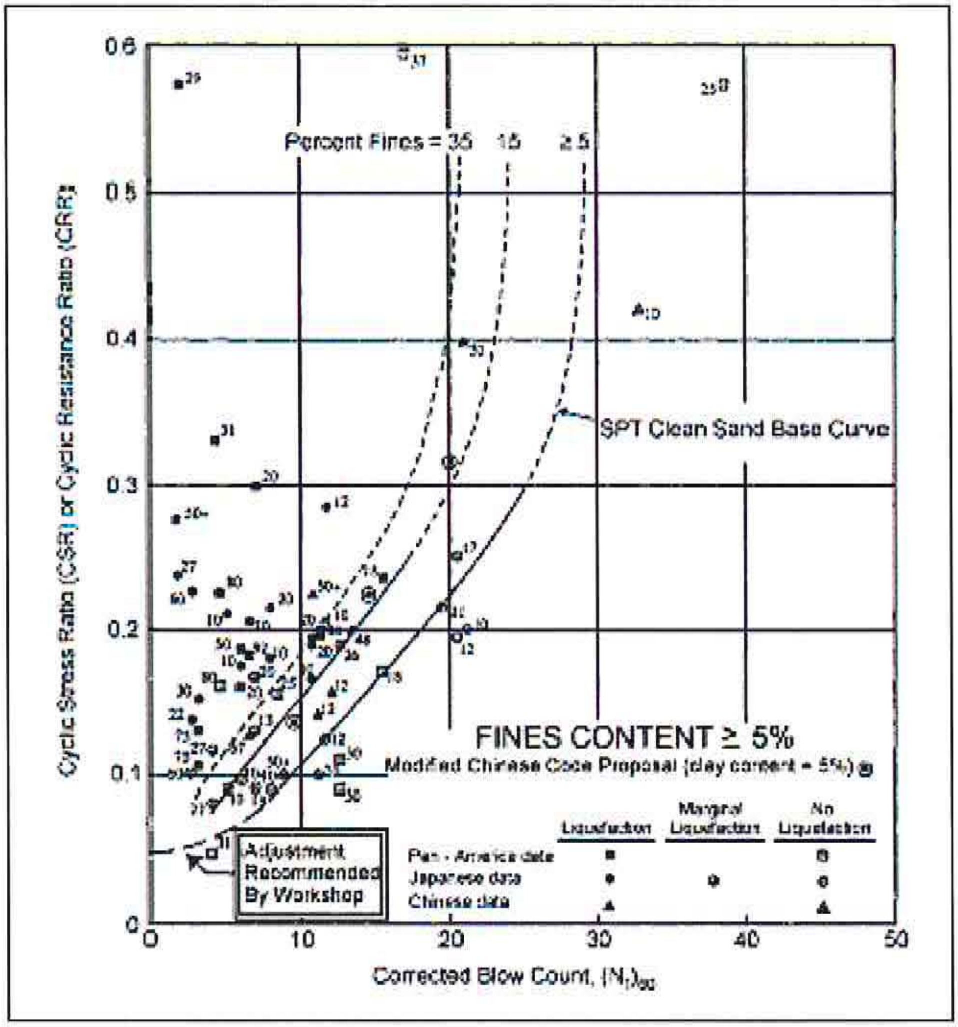

Fig. 22: CRR vs. $(N1)_{60}$, Base Curve for Magnitude 7.5 Earthquakes (NCEER, Youd et al. 2001)

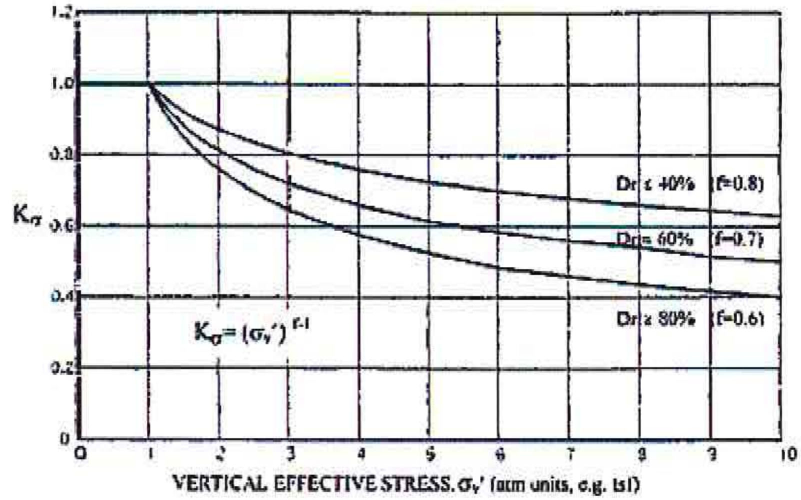

Fig. 23: Vertical effective correction factor

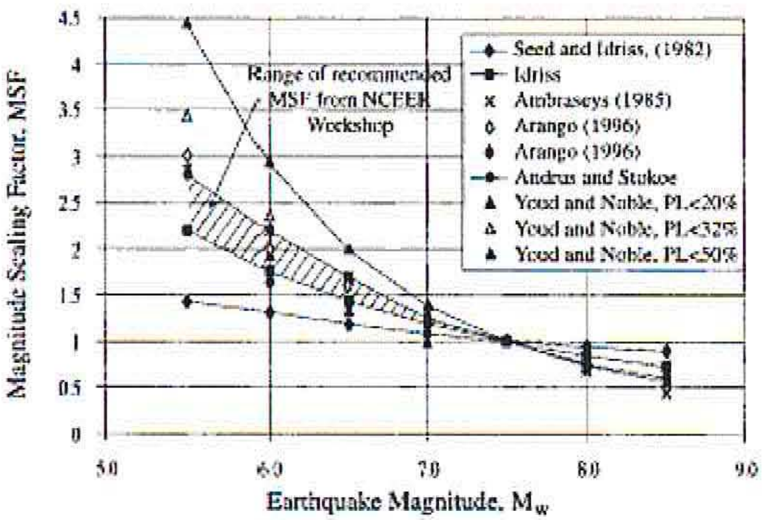

Fig. 24: Correction factor for earthquake magnitude

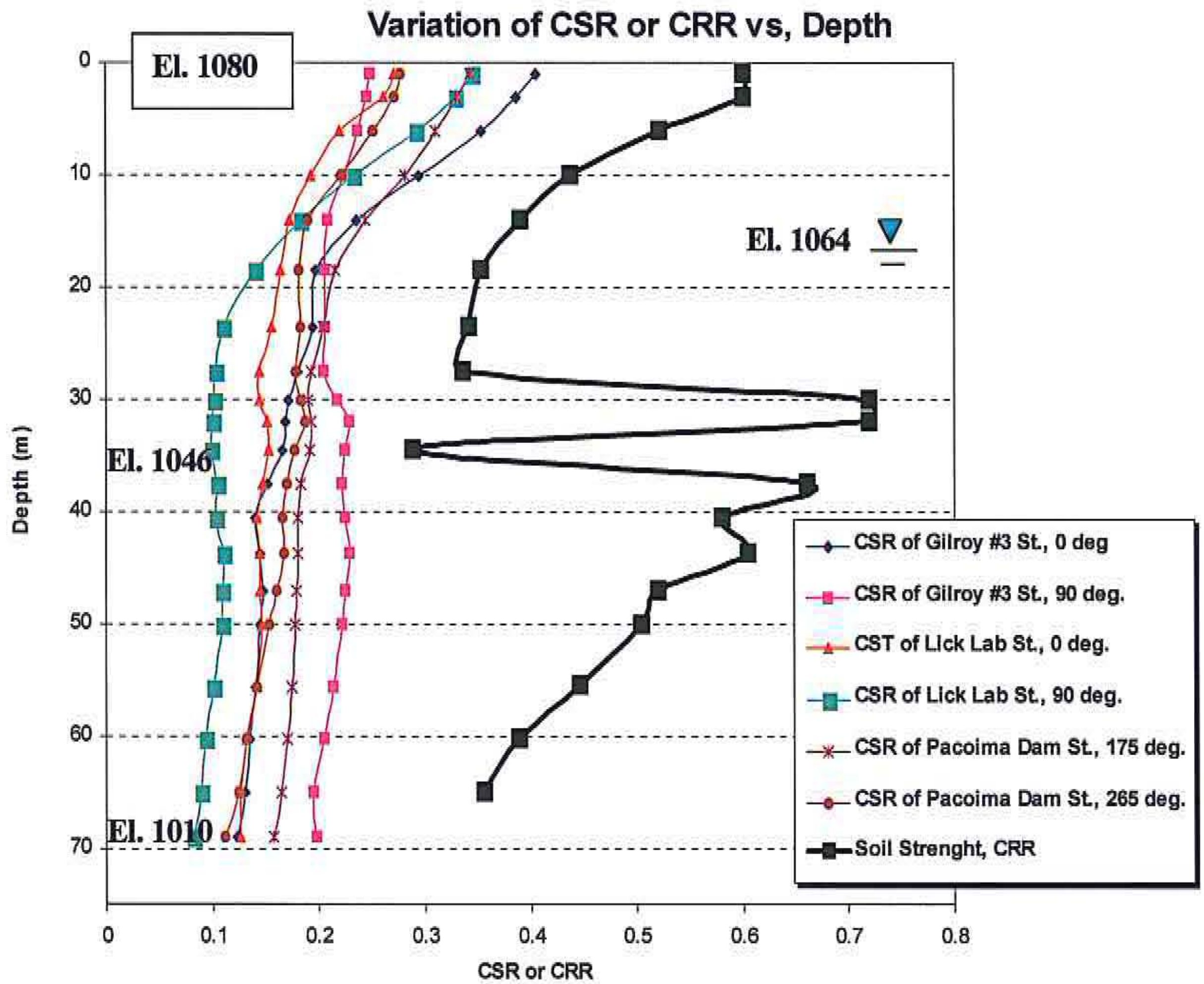

Fig. 25: Variation of CSR and CRR along depth of BH 80-46 profile with dam for different input motions

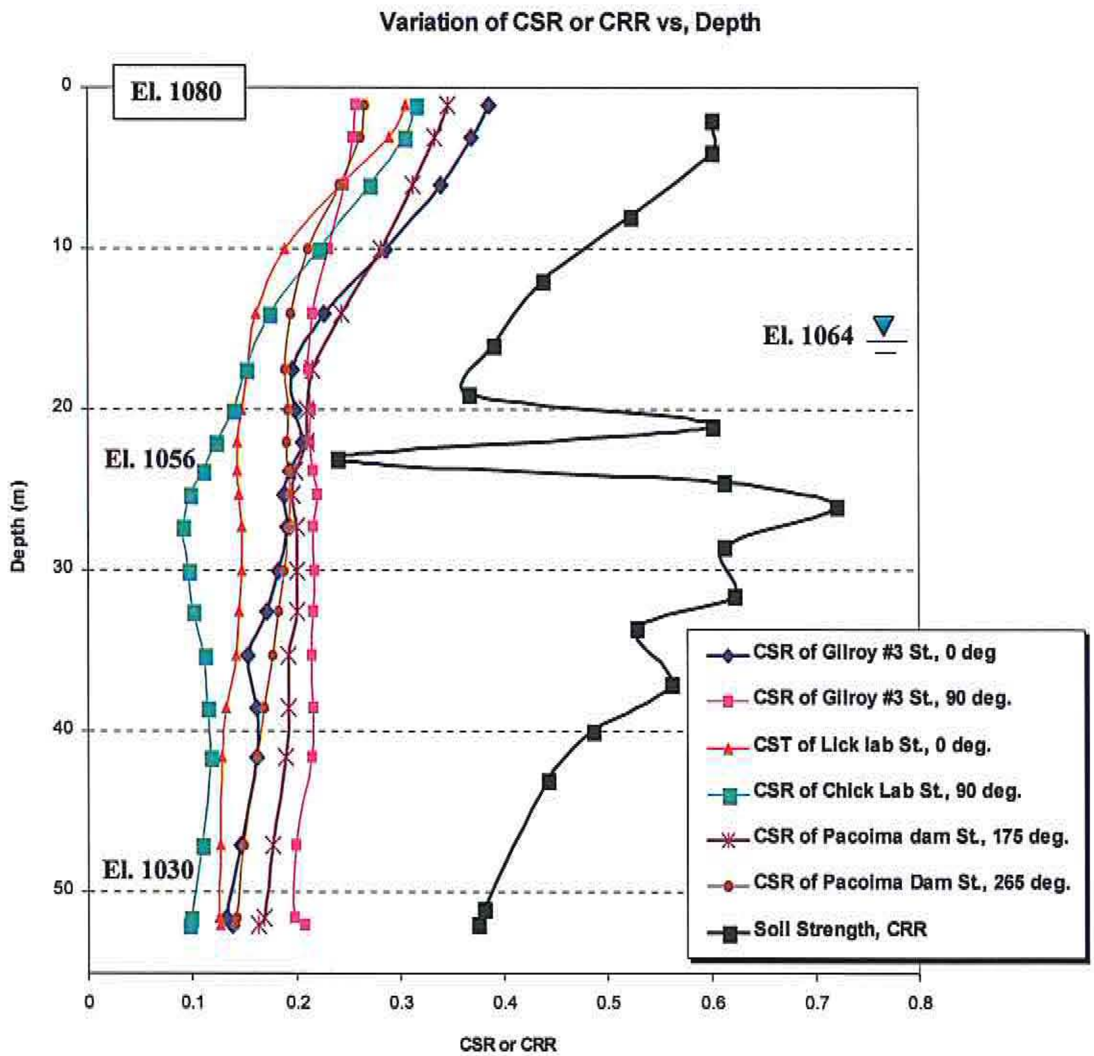

Fig. 26: Variation of CSR and CRR along depth of BH 79-21 profile with dam for different input motions

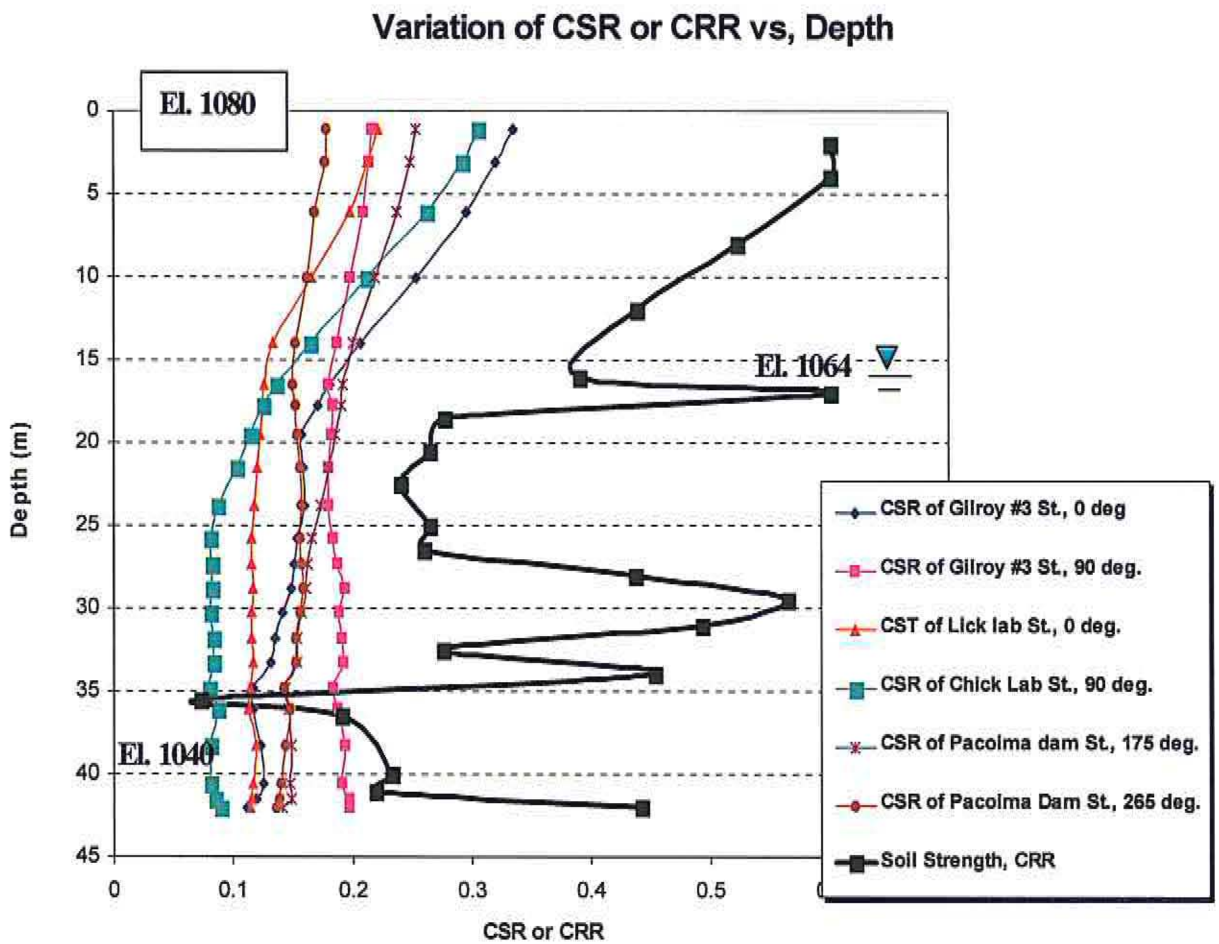

Fig. 27: Variation of CSR and CRR along depth of BH 79-16 profile with dam for different input motions

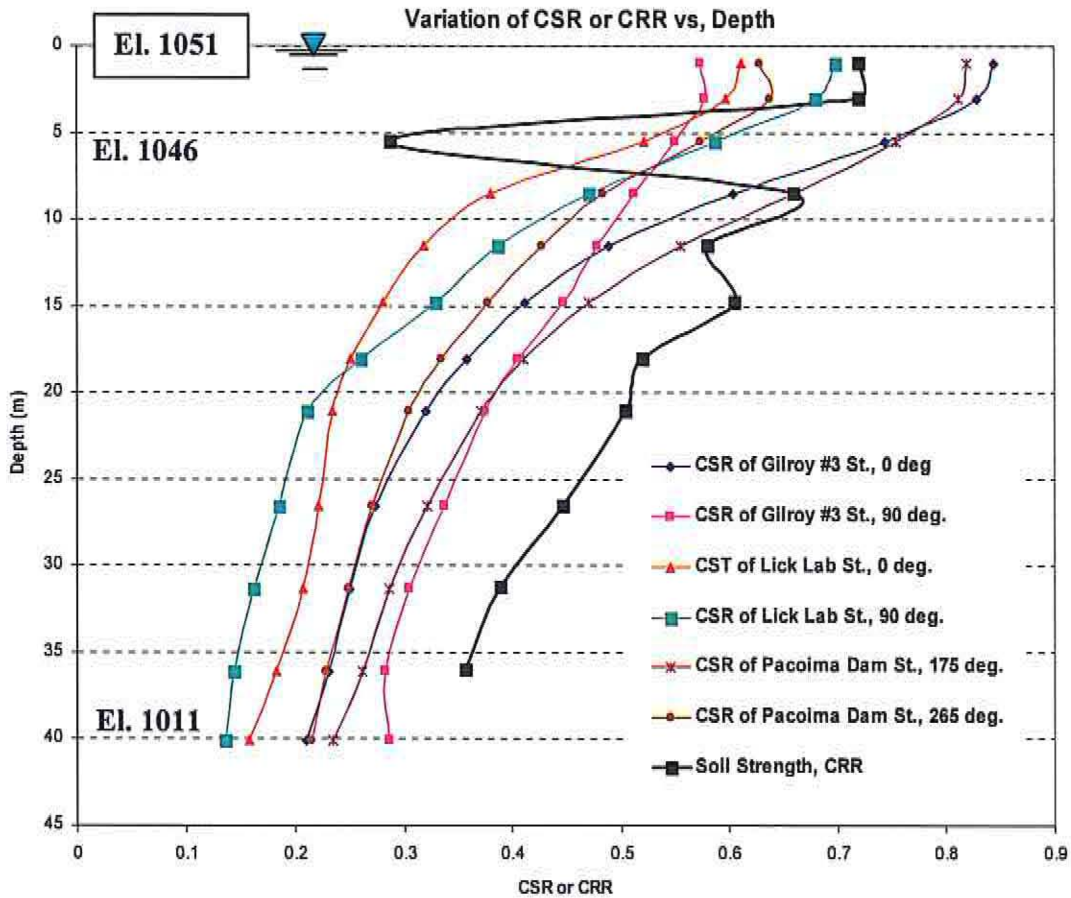

Fig. 28: Variation of CSR and CRR along depth of BH 80-46 profile without dam for different input motions

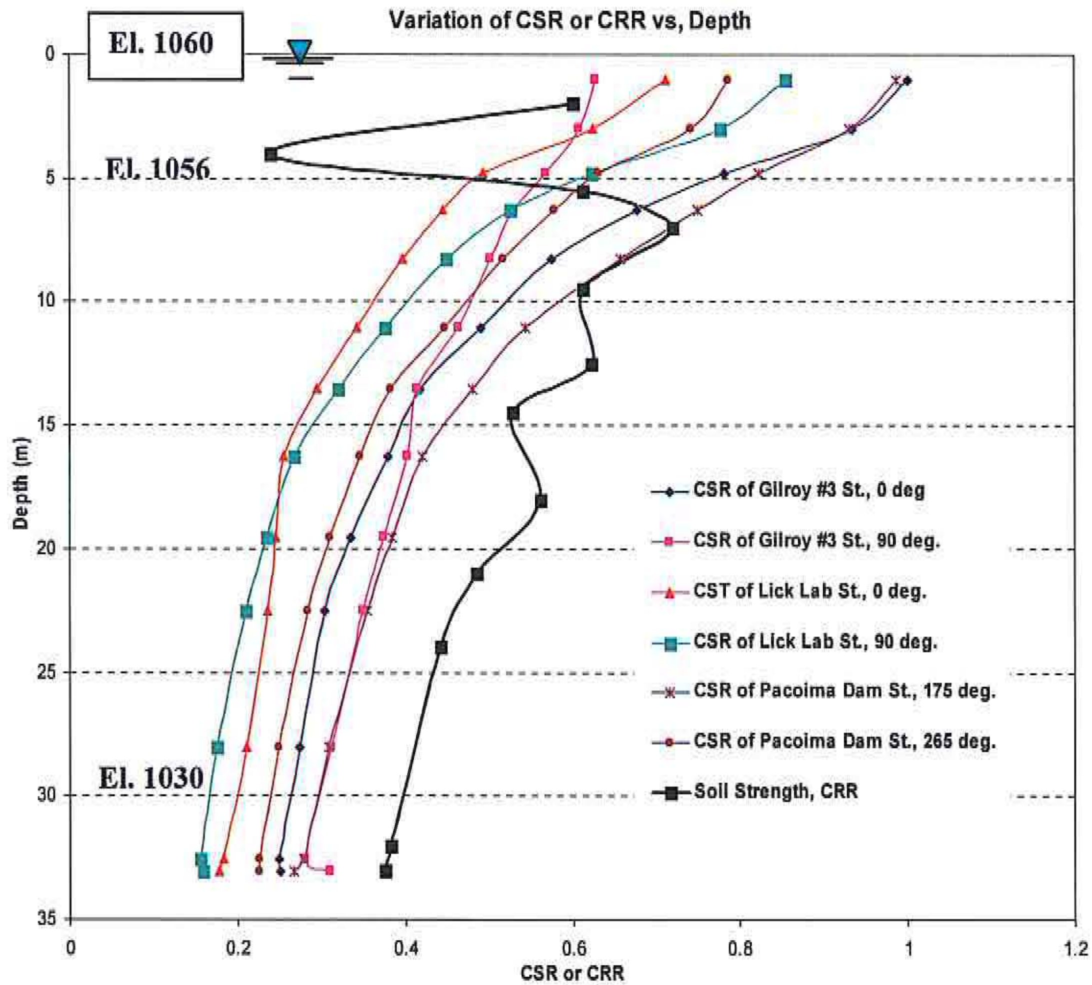

Fig. 29: Variation of CSR and CRR along depth of BH 79-21 profile without dam for different input motions

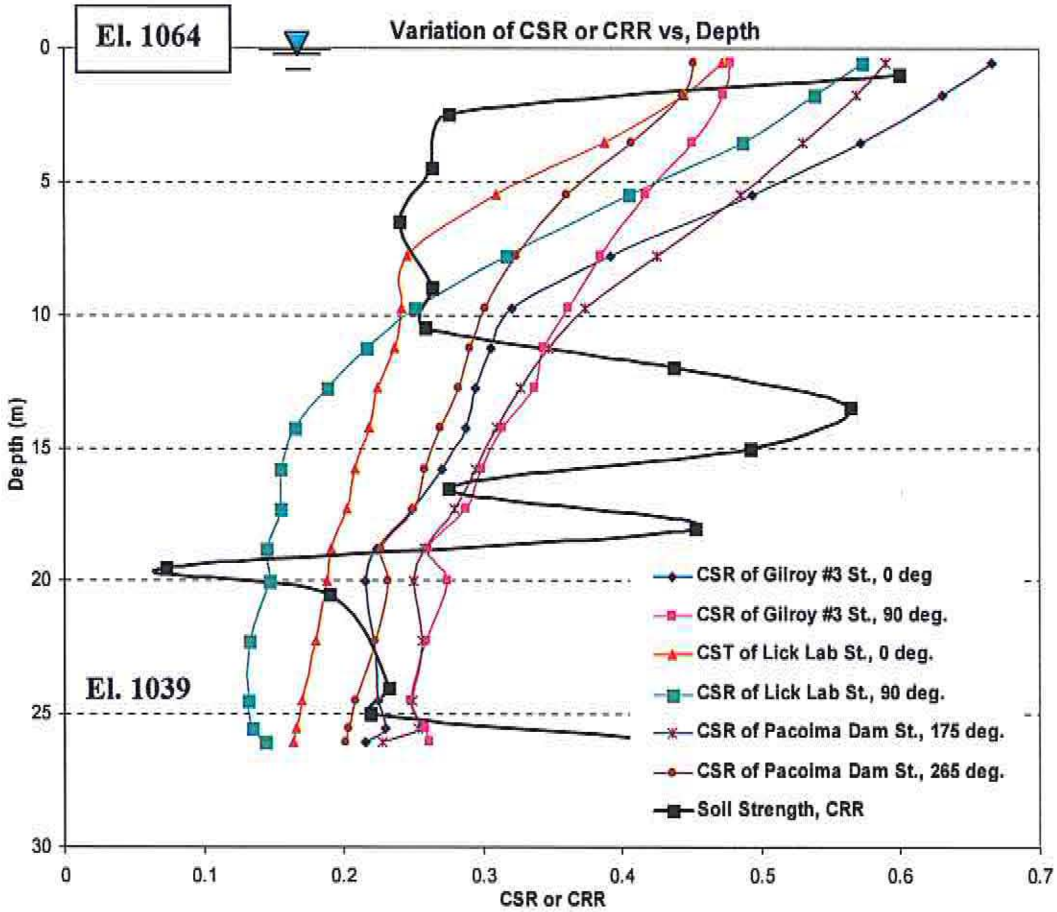

Fig. 30: Variation of CSR and CRR along depth of BH 79-16 profile without dam for different input motions