GARNET DAM SLOPE STABILITY ASSESSMENT

Summerland, BC

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Report by **exp** Services Inc., "Garnet Dam Slope Stability Assessment"
dated 2013 January 29

1. Executive Summary

This report provides an amendment to the prior slope stability assessment report by **exp** Services Inc. (**exp**), "Garnet Dam Slope Stability Assessment" ("Attachment") dated 2013 January 29, which forms an attachment to this 2014 Report. Though the final conclusions in the prior **exp** report remain unchanged, this 2014 Report amendment covers the following aspects:

- Extreme Consequences Classification;
- Seismic Hazard Evaluation;
- Seismic Slope Deformation Analysis.

Reference is made to the 2013 **exp** report ("Attachment") for discussion of relevant aspects, and, as appropriate, some revisions are included in the current report for clarity.

The Garnet Lake Dam is located up the Garnet Valley, about 10km north of Summerland, BC. The dam is comprised of a "zoned" earthfill embankment, about 12m high, complete with a left bank open channel spillway (part concrete lining) and concrete pipe low level outlet, plus water works. Under the BC Dam Safety Regulations (2011), the Garnet Lake Dam is classified as "EXTREME" consequence level dam.

The Garnet Lake Dam was built about 30 years ago to replace a former dam. An analysis of the dam in 2010 by Associated Engineering/Golder indicated that a berm on downstream slope may improve seismic stability of the dam to meet present day criteria.

The summary design and construction report dated December 1976 indicated a Factor of Safety of 2.0 for the downstream slope. However, assumptions about internal seepage patterns had to be made for stability analyses purposes.

As requested, **exp** was retained in 2012 by Kerr Wood Leidal Associates Ltd. (KWL) on behalf of the District of Summerland to carry out additional site investigations and review the seismic stability of the dam. Stability of the existing embankments has been evaluated based on available prior site investigation records and the recent 2012 exploration data. The study sections were selected to assess potential failure modes. It has been shown that the existing dam downstream slope is stable enough to withstand effects of a 1/10,000 year seismic event. Therefore, it is considered that any additional stabilization work would be unwarranted for stability purposes.

The following presents some discussion about embankment stability and deformational analyses appropriate for the Extreme consequence dam and provides recommendations for the site based on the current study.

2. Terms of Reference

As requested by the District of Summerland in an email dated January 29, 2014, **exp** has carried out a seismic review of the existing Garnet Lake Dam in the District of Summerland, BC. The study was carried out in accordance with the proposal letter by **exp** dated October 9, 2013.

The scope of the study pertains to the embankment stability under seismic events arising due to natural crustal movements and their potential effects on the dam. In particular, the components considered in the analysis include the upstream and downstream embankment slopes as shown on Table 1 (Amended).

The 2013 **exp** report ("Attachment") presents background information and characterization of the dam. An outline of the seismic parameters is presented, followed by a review of performance expectations. Results of the study and recommendations are presented in Sections 6 and 7 ("Attachment").

The "Interpretation & Use of Study and Report" (Appendix A of the "Attachment") contains instructions to readers and forms an integral part of this report and must be included with any copies of this report.

3. Site Description and Characterization

The Garnet Lake Dam retains water for municipal water supply purposes. The community of the District of Summerland is located within the downstream area below the dam. The Garnet Dam is located in the Eneas Creek watershed.

The lake level is usually near full supply level during most of the year and the spillway flows above El. 632.82m. During the summer, the lake level is typically at about Elevation 632m.

Table 1 (Amended) provides a summary of the current general arrangement of the dam as well as the BC Dam Safety Dam Consequence Classification – "**Extreme**".

The current dam was built to replace the pre-existing dam in about 1975.

Sections 3, 4 and 5 of the 2013 **exp** report ("Attachment") provide discussion of the available site records including the 2012 site visit, construction, subsurface exploration and site characterization. The information provided inputs to the evaluation and analyses presented here. Appendix E1 ("Attachment") shows the dam sections utilized in the slope analyses.

4. Evaluation and Analyses

The 2013 **exp** report ("Attachment") had identified a potential for some slope deformation under severe earthquake effects (i.e., 1/10,000 event). However, the prior evaluations had been done for a 1/5,000 year event, and no explicit deformational analysis was warranted for less severe earthquakes. The deformational analysis shown here provides more insight to failure mechanisms appropriate for extreme consequence dam, as presented below.

Section 5.3 in the 2013 report ("Attachment") outlines the seismic hazard analysis developed for Garnet Dam, including the summary in Table 5A ("Attachment"). Section 6.1 ("Attachment") generally outlines the slope stability limit equilibrium analysis methodology.

The following provides discussion of the site-specific, earthquake ground motions design (EGMD) parameters and the slope deformational analysis results.

4.1 Site Specific Seismic Hazard and Evaluation Parameters

Section 5.3 (“Attachment”) outlines the seismic hazard analysis done for Garnet Dam. Section 6.2 (“Attachment”) outlines the site-specific PGA parameters utilized. The seismic design criteria outlined in Table 6-1 of the CDA 2007 (Canadian Dam Association, Dam Safety Guidelines) were used for “**Extreme**” consequence dam, i.e., AEP 1/10,000 year event as per 2014 Report.

The site-specific acceleration response spectrum (5% Damped) corresponding to 1/10,000 year event is shown on Table 4.1:

Table 4.1 Spectral Accelerations, Garnet Dam (Site Class D) (1/10,000)

Period (seconds)	0.2	0.5	0.8	1.0	1.5	2.0
Acceleration (g)	0.6	0.42	0.3	0.26	0.21	0.16

PGA – Peak Ground Acceleration 0.28g

The site-specific spectral accelerations were derived by directly scaling method to produce the 1/10,000 year event, because there is no spectral analysis available for Garnet Dam under design earthquake events. The scaling method is considered reasonable given the anticipated fundamental period of the Garnet Dam, generally in the range of 0.1 to 0.5 seconds. The spectral accelerations were used in the slope deformation analysis.

4.2 Pseudo-static Slope Deformation Analysis

The slope deformation analysis provided an estimation of the displacement along a potential slip or rupture surface. The various slip surfaces analyses were determined by pseudo-static limit equilibrium analyses using commercially available computer software, SlopeW. Some of the pseudo-static limit equilibrium analysis results are also discussed in Section 6 in the 2013 report (“Attachment”).

The pseudo-static analysis method is considered valid because the liquefaction assessment for 1/10,000 year event generally indicated no liquefaction and specifically a limited effect at depth. Table 6.3.1 in the 2013 report (“Attachment”) shows essentially no liquefaction, except at 8.5m depth, and < 0.7m thickness in one of the 2012 test holes. Based on post-earthquake stability analysis (Table 6, “Attachment”), a mass movement or flow slide is considered very unlikely.

The inputs to the pseudo-static slope deformation analysis done in accordance with Bray and Travasarou (2007) method included the following:

- Pseudo-static limit equilibrium analysis;
- Spectral acceleration values;
- Fundamental period of potential slide or movement mass.

The calculations provide a prediction of displacements within a probabilistic context. The method utilizes a database of ground motions to capture the primary source of uncertainty in seismic performance evaluations. The median and double the median (50% and 16% exceedance levels) quantify the anticipated seismic performance. The summary of the displacement estimates is shown in Table 2 of this 2014 Report.

The pseudo-static method is also recognized in the APEGBC Guidelines for Legislated Landslide Assessment for Proposed Residential Development. The seismic slope analysis method used here is described in detail in Appendix E of the APEGBC publication.

The analysis has considered the pseudo-static limit equilibrium results similar to that shown on Table 6 and Appendix F2 ("Attachment"). Appendix A in this 2014 Report shows select stability sections. For the upstream slope (Fps=0.93), the median and double median slope displacements are less than 50mm and 100mm respectively, as shown on Table 2. The estimated displacements are also relatively small for the downstream slope, consistent with yield coefficients $k_y > 0.25$ (Table 2). However, to simulate dynamic effects on the ground, lower bound yield coefficients were also considered in the evaluation. The initial dynamic effects on the deep liquefiable zone was simulated as an average excess pore water pressure applicable to initial seismic loading effects in order to determine k_y. In particular, higher average excess pore water pressures gave lower yield coefficients, and estimated displacements due to excess pore water pressure effects were somewhat greater than the estimates which ignored induced pore water pressure effects (Table 2).

5. Conclusions and Recommendations

The 2013 report ("Attachment") had identified a potential for displacement on the upstream slopes. The slope deformation analysis shows that the slope displacements are relatively small (Table 2), for both the upstream and downstream slopes.

The seismic failure mechanisms as outlined in the prior **exp** report remain valid under the current report.

The seismic slope displacement analysis has provided an opportunity to:

- Update seismic hazard analyses and identify EDGM parameters for Extreme Consequence dam;
- Carry out embankment stability analyses and estimate earthquake induced displacement based on inferred material parameters and seepage conditions detailed in the 2013 **exp** report (attachment).

The following summarizes the findings of the seismic slope deformation analysis:

- The slopes generally meet traditional standards-based stability criteria.
- The slope displacement along potential slip surfaces is within the range anticipated for favourable dam performance under severe earthquake effects.
- For the embankment dam, the anticipated seismic induced embankment settlements are significantly less than the freeboard at full supply lake level.
- The slope displacements and internal straining of the embankment under severe earthquake effects is consistent with failure mechanism discussion in the 2013 report, Section 5 ("Attachment").

- The post-seismic response including limited liquefaction outlined in the **exp** 2013 report Section 6.5 ("Attachment") remains valid.

5.1 Toe Buttress – Downstream Slope

It is considered that the existing downstream slope complete with benches is stable enough for seismic design considerations. Therefore, provision of an additional berm/toe buttress is considered inappropriate and unwarranted for stability purposes.

6. Closure

We trust that the information provided herein is sufficient for your current needs. This report was prepared for the exclusive use of the District of Summerland, Engineering & Public Works, and their designated consultants/agents, and may not be used by other parties without written consent of **exp** Services Inc.

Should you have any questions or concerns, please do not hesitate to contact the undersigned at your convenience.

Yours truly,

exp Services Inc.

Don Sargent, P.Eng.
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Reviewed by:

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Tables

Table 1 – Summary of Current General Arrangement (Amended)

Dam Component	Descriptions
Earth Embankment Crest Level	12m high, on Earth and Rock Foundation El. 634.5m
Impervious Blanket	Upstream, from toe of dam to former dam
Upstream Slope	Impervious Earth, 2.5H:1V
Gate Tower (Vertical)	Situated Upstream of dam crest c/w dry well
Downstream Slope	Upper slope – Impervious Earth, 2H:1V Bench – El. 628m Lower slope – Sandy Gravel, 2H:1V Erosion Protection – Rockfill/Rip Rap (450 thick)
Downstream Drainage Layer	Filter Layers, c/w two 150 dia. drain pipes outfall at outlet structure
Downstream Toe	Seepage Collection, Measurements (V-notch weir)
Spillway	Left Abutment – Free Overflow, Concrete Channel, c/w concrete lining segment above Rip Rap Lining on lower segment
Low Level Outlet	600 dia. pipe, c/w gate tower control valve Intake and outlet concrete structures
Water Works	• 450mm dia. pipe, control valve at gate tower (1975) • Waterworks meter chamber; situated downstream of dam

Special Note:

Dam Consequence Classification (BC Dam Safety Regulation, 2011)	"**Extreme**"

Table 2 – Embankment Seismic Slope Stability and Pseudo-static Displacements (1/10,000 year event)

Slope	Section	$\Delta\frac{Pw}{N}$	F_{ps}	K_y	D_{50} mm	D_{16} mm
Downstream Slope, Overall	BB	0	1.12	0.35	< 50	< 50
		0.5	< 1.0	0.21	< 50	< 100
		0.6	< 1.0	0.15	< 100	140
	-	-	-	-	-	-
Upstream Slope, Overall	AA, BB	0	0.93	0.26	< 50	< 100

$\Delta\frac{Pw}{N'}$ = Increase in initial pore water pressure at depth in potential liquefiable layer, where N' is the initial effective stress.

F = Factor of Safety

F_{ps} = Pseudo-static Factor of Safety

K_y = Yield coefficient, Fps = 1.0

D_{50} = Displacement, 50% Probability of Exceedance or Median

D_{16} = Displacement, 16% Probability of Exceedance or Double Median

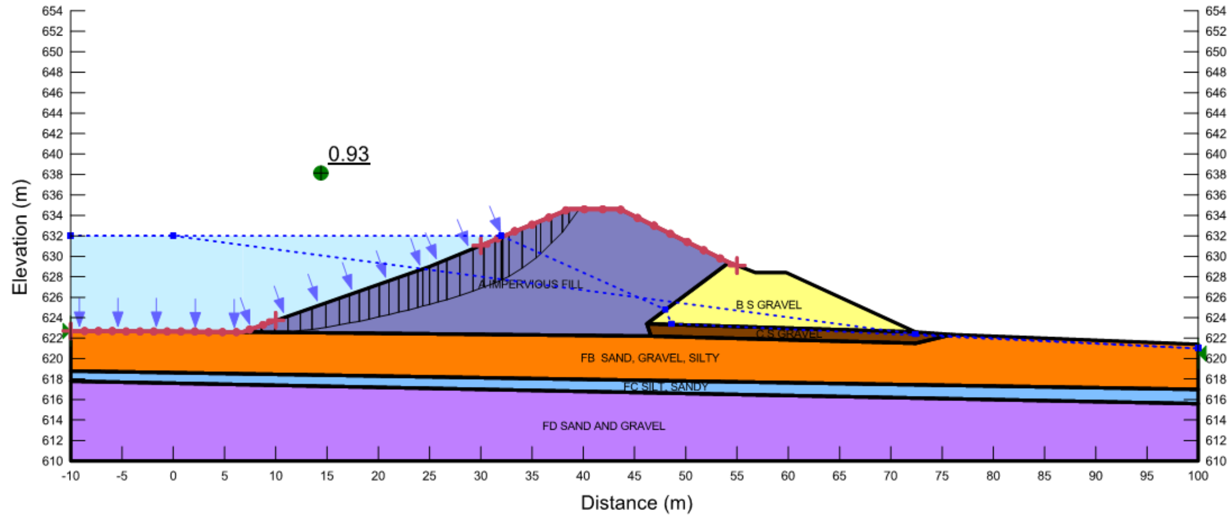
Appendix A

Pseudo-static (Earthquake) Stability

Selected Sections

Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 1
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 1

Horz Seismic Load: 0.286



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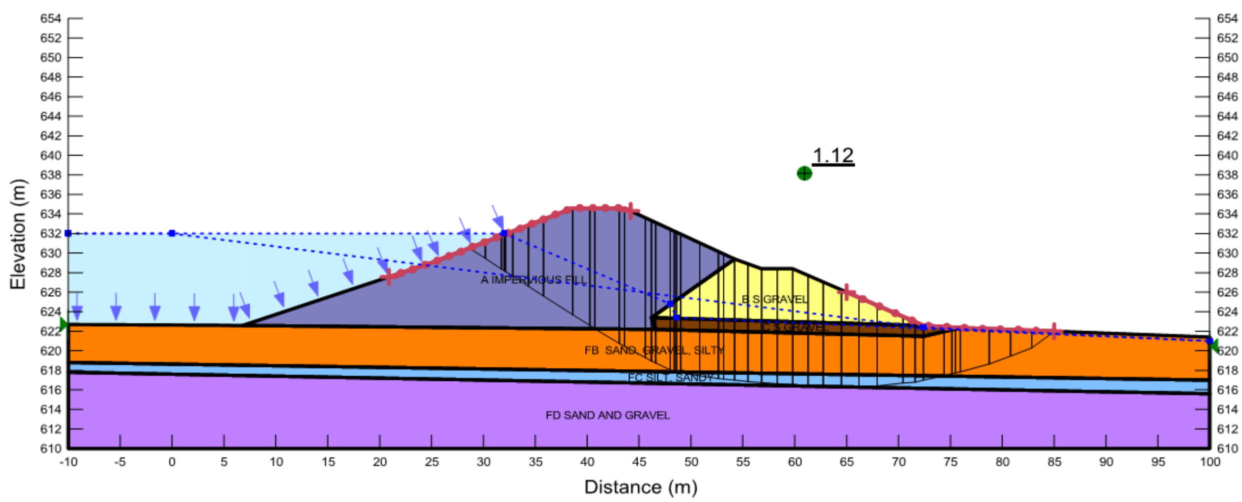
Name: A IMPERVIOUS FILL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 35 ° Piezometric Line: 1
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2

Horz Seismic Load: 0.286



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Attachment

exp Services Inc.

"Garnet Dam Slope Stability Assessment" Report
dated 29 January 2013

Garnet Dam Slope Stability Assessment

Summerland, BC

Project Number:

VAN-00209167-A0

Prepared For:

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Tables

Appendices

1. Executive Summary

The Garnet Lake Dam is located up the Garnet Valley, about 10km north of Summerland, BC. The dam is comprised of a “zoned” earthfill embankment, about 12m high, complete with a left bank open channel spillway (part concrete lining) and concrete pipe low level outlet, plus water works. Under the BC Dam Safety Regulations (2011), the Garnet Lake Dam is classified as a “Very High” consequence level dam.

The Garnet Lake Dam was built about 30 years ago to replace a former dam. An analysis of the dam in 2010 by Associated Engineering/Golder indicated that a berm on downstream slope may improve seismic stability of the dam to meet present day criteria.

The summary design and construction report dated December 1976 indicated a factor of safety of 2.0 for the downstream slope. However, assumptions about internal seepage patterns had to be made for stability analyses purposes.

As requested, **exp** Services Inc. (**exp**) was retained to carryout additional site investigations and review the seismic stability of the dam. Stability of the existing embankments has been evaluated based on available prior site investigation records and the recent 2012 exploration data. The study sections were selected to assess potential failure modes. It has been shown that the existing dam downstream slope is stable enough to withstand effects of a 1/10,000 year seismic event. Therefore, it is considered that any additional stabilization work would be unwarranted for stability purposes.

The following presents the subsurface exploration findings, site characterization and embankment stability analyses and provides discussion and recommendations for the site based on the current study.

2. Terms of Reference

As requested by Kerr Wood Leidal Associates Ltd. (KWL), **exp** Services Inc. (**exp**) has carried out a seismic review of the existing Garnet Lake Dam in the District of Summerland, BC. The study was carried out in accordance with the proposal letter addressed to KWL by **exp** dated 2012 September 17 which had modified the prior **exp** proposal dated 2012 May 15.

The scope of the study pertains to the embankment stability under seismic events arising due to natural crustal movements and their potential effects on the dam. In particular, the components considered in the analysis include the upstream and downstream embankment slopes as shown on Table 1.

The following discussions present background information and characterization of the dam. An outline of the seismic parameters is presented, followed by a review of performance expectations. Results of the study and recommendations are presented in Sections 6 and 7.

The “Interpretation & Use of Study and Report” (Appendix A) contains instructions to readers and forms an integral part of this report and must be included with any copies of this report.

3. Site Description / Record Information

The Garnet Lake Dam retains water for municipal water supply purposes. The community of the District of Summerland is located within the downstream area below the dam. The Garnet Dam is located in the Eneas Creek watershed.

The lake level is usually near full supply level during most of the year and the spillway flows above El. 632.82m. During the summer, the lake level is typically at about Elevation 632m.

Table 1 provides a summary of the current general arrangement of the dam as well as the BC Dam Safety Dam Consequence Classification – "**Very High**".

The current dam was built to replace the pre-existing dam in about 1975. Table 2 provides a summary of the design and construction history of Garnet Lake Dam.

3.1 Site Visit

For site characterization and slope analysis purposes, a site visit was undertaken by the **exp** Senior Geotechnical Engineer on 2012 September 26 (Appendix D contains site visit notes).

3.2 Geological Setting

At the latter stages of the last glacial retreat, the Eneas Valley at Garnet Lake Dam was initially occupied by ice and then subsequently by glacial melt water channels (Nasmith, 1962, BC MMPR Bulletin No. 46). Due to stagnant ice situated to the north and east along Garnet Lake, melt water was deflected to the west side (right side of dam) of Eneas Valley which could explain the terrace above the right abutment of the present day dam on Garnet Lake. The water volumes associated with glacial melting and runoff would be orders of magnitude greater than present day creek flows.

The melt water channel erosion and downcutting has formed the present day valley. The valley thalweg gradient upstream of the dam is essentially flat (under lake) whereas the valley thalweg gradient is steeper, in the order of 5 to 6%, under the dam, consistent with a higher energy deposition environment. It is anticipated that bedrock outcrops in proximity of the left and right sides of the dam also had influences on the outwash channel gradients, and the associated valley bottom profile. As the depth to bedrock is greater than 15m (Appendix C4, DH # 2), some meltwater channel deposited soils intermixed with local deposits of talus (rock slopes) and alluvium/colluvium (minor alluvial fan possibly due to localized meltwater runoff, left side) could be found below the valley bottom.

3.3 Dam Design and Construction Records

The following records are available regarding the dam design and construction:

- Report Part II, Proposed Garnet Lake Dam, H. Fellhauer (1974)
- Summary Report – Design and Construction of Garnet Lake Dam, H. Fellhauer (1976)

Appendix B2 contains the 1975 As-built drawings. Appendix B3 contains the 2012 dam topography survey plan and sections. Appendix C2 provides a synopsis of the above noted records as pertains to embankment stability. Appendices C3 and C4 include test pit and drill hole records given in the above-noted records and Appendix C5 contains a summary of laboratory test reports.

4. 2012 Subsurface Exploration

4.1 Subsurface Exploration

The geotechnical exploration was conducted on 2012 October and included:

- three (3) Cone Penetration Testholes (CPT) using a truck-mounted rig;
- two (2) Shear Wave Velocity Cone Penetration (SCPT) profiles using a truck-mounted rig; and,
- due to resistant ground condition, drill-outs were required at each testhole location.

The CPT and SCPT met refusals at depths of about 2.2 to 9m. The locations of the test holes are shown on Test Hole Location Plan, in Appendix B1. Soil descriptions of each test hole advanced at the site are included in the testhole logs in Appendix C1.

Upon completion of CPT, SCPT probing, the holes were backfilled with grout. Upon completion of auger drill out, the holes were backfilled with the auger cuttings as per the groundwater protection regulations. The geotechnical exploration was undertaken by ConeTec, who located the test holes and obtained probe logs of the subsurface conditions, based on the exploration plan prepared by **exp**.

Static and Seismic Cone Penetration Test (CPT & SCPT)

Exploration included the Piezocone Penetration Test (CPT) to provide continuous readings of tip and sleeve resistance as well as pore pressure. Dissipation tests provided estimates of soil material properties, such as, hydraulic conductivity, as well as measurements of watertable and hydraulic heads vs. depths. Seismic Cone Penetrometer (SCPT) is the same as CPT, but shear wave velocity subsoil profiles are also obtained.

The test holes indicated subsurface conditions only at the locations of test holes. The precision of the subsurface conditions indicated depends on the methods used, frequency of sampling, and the uniformity of the subsurface conditions. The spacing of the test holes, frequency of sampling, and the method of exploration have been selected to meet the needs of the project within constraints of the budget and schedule for geotechnical exploration purposes.

5. Site Characterizations

Subsurface exploration was needed to assess the material types and subsoil strength and groundwater profiles.

5.1 Dam Characterization

The characterization of the dam is based on existing record review combined with 2012 exploration and site reconnaissance by **exp**.

Garnet Dam and Spillway

The Garnet Dam site plan (Appendix B2) shows the main dam situated in a relatively narrow valley with a left bank spillway. The current dam was built in 1975 by construction of conventional earth fills. The Dam features include a zoned earthfill embankment with upstream impervious zone and downstream granular zones, according to 1975 drawings (Appendix B2).

Table 3 provides a summary of the typical characteristics of the Garnet Dam. In particular, Table 3 shows the feature/units (e.g., embankment, etc.) on left column together with the information sources utilized in the site characterization activities. The source information has been utilized to develop embankment and foundation strength parameters for stability purposes.

Observations and measurements considered in the stability assessments include the following:

- Site reconnaissance/characterizations by the Senior Geotechnical Engineer (Appendix D);
- Intrusive investigations including piezocone measurement of hydraulic head and estimates of hydraulic conductivity (Appendix C).

In general, observation of seepage and watertable conditions under high lake levels offered an opportunity for seepage characterization purposes. Seepage conditions as well as material strength properties are considered in the stability assessments.

Prior Site Characterizations

Available information shows the depth to bedrock varies widely, from outcrops at right abutment to greater than 34 to 55ft at drill holes No's 2 and 3 (Appendix C4) located on the valley bottom. The subsoil varied from silt and sand to sand and gravels, with generally compact to very dense soil consistency. The zoned earthfill embankment was constructed by compacting materials in thin lifts to 98% Standard Proctor density. Appendix C2, Synopsis of Foundation and Embankment records provides additional comments as pertains to dam stability.

Embankment Dam and Foundations

The zoned earth embankment dam sections (Appendix B2) combined with site investigation and construction records were used to develop two embankment stability Sections, AA and BB, as shown in Appendix E1. The section topography conforms with that determined in the 2012 Site Survey (Appendix B3). The downstream slope height is slightly greater in Section BB as compared to Section AA. The foundation conditions are also shown on Sections AA and BB (Appendix E1). Section AA shows the seepage cutoff under the upstream slope.

Seepage and Groundwater

The most salient dam features controlling seepage and groundwater regimes are the impervious dam zone (Unit A), the seepage cutoff (Section AA, Appendix E1) and the impervious upstream blanket (Appendix B2, as built drawings). The seepage through the embankment and foundation was evaluated using "Casagrande" seepage theory for the embankment and "Bennett" seepage theory for the foundation. The seepage pattern within the embankment is controlled by the downstream drainage zone, as shown by Peizo # 1 in Sections AA and BB (Appendix E1). The foundation and embankment seepage is controlled by the following:

- Recharge above the upstream blanket;
- Discharge to downstream drainage zones within the embankment;
- Seepage pathways (i.e., more pervious foundation strata), and seepage by-passing the cutoff.

The SCPT 12-1 dissipation testing has confirmed an essentially hydrostatic groundwater profile in the foundation materials under the downstream toe of the dam. The foundation seepage is modeled as piezometric surface Peizo 2 in the Sections AA and BB (Appendix E1).

5.2 Material Strength Properties

The material strength and unit weights shown on Table 4 are based on the following records:

- construction records;
- 1974 and 1975 pits and drills;
- 2012 in-situ testing.

The impervious fill is comprised of gravelly sand, some silt with in-place density of 98.2% SPD. An equivalent SPT N > 50 blows was indicated at CPT 12-3. The downstream embankment section is comprised of sandy gravel placed to 98% SPD.

The foundation soils generally consisted of sand and gravel with varying silt mixtures of dense to very dense consistency. However, some silty sand was encountered in test pits located in the creek, upstream of the dam axis. Finally, a silt layer was identified in SCPT 12-1 (7.5m depth) and DH 2 (4.5m depth). It was assumed that the silt layer could be continuous between the testholes, and it is shown as Unit FC in Sections AA and BB (Appendix E1).

The subsoil strength properties were selected based on typical correlations considering material gradations and densities indicated in the available records. In general, the greater the gravel content and the greater the density indicated by equivalent SPT N values, the greater the materials strength as indicated in Table 4. In general, the materials are dense. However, as fines contents increase the equivalent N values decrease as indicated for the silt and silty sand units, which have the least strengths (Table 4).

It is considered that Sections AA and BB (Appendix E1) represent the worst case ground conditions, at the vicinity of the creek and right side of the dam. The ground located on the left side of the dam is considered to be somewhat stronger based on available records.

5.3 Seismic Hazard Analysis

General

The seismic liquefaction susceptibility and embankment stability depend on the ground strength and shaking conditions due to the earthquake. The shaking effect relates to external seismic hazard and site-specific response.

The seismic hazard analysis is based on a review of information available as follows:

- Seismic Hazard Calculations – Pacific Geoscience Center (PGC), Appendix E2
- Seismic Hazard – Coursier Dam Seismic Information

The available information provided a basis for selecting earthquake hazard and evaluating the earthquake design ground motion parameters for purposes of seismic analyses.

Dam Classification and Seismic Design Criteria

The purpose of the seismic analyses is to provide a perspective on dam safety issues under seismic conditions. In accordance with CDA 2007 dam stability guidelines, an earthquake having a 1/5000 and 1/10,000 AEP (Annual Exceedance probability) was considered for seismic evaluation purposes (**VERY HIGH** consequence dam classification).

Earthquake Hazard Evaluation

A site-specific earthquake hazard analysis at 1/5000 and 1/10,000 AEP is unavailable for the Garnet Lake Dam. Therefore, an estimate was obtained based on a review of records from PGC and BC Hydro – Coursier Dam.

Pacific Geoscience Centre (PGC)

The Pacific Geoscience Centre provides a seismic hazard calculation used for seismic design based on BC Building Code. The calculation is currently provided on a site-specific basis (Appendix E2).

Table 5a shows the PGC calculations for both Garnet Lake and Coursier Dam near Revelstoke, BC.

Coursier Dam – Seismic Hazard

The studies for Coursier Dam near Revelstoke, BC were undertaken about 15 years ago by BC Hydro. The analysis included seismic hazard source analyses and disaggregation analysis appropriate to the dam. Table 5a shows the parameters for comparison purposes.

Garnet Lake Dam – Seismic Hazard

Table 5a shows the Garnet dam seismic hazard selected for this seismic analysis. A site-specific hazard analysis may yield slightly different values. Secondly, any analysis may be subject to some future changes, as the science evolves.

6. Evaluation and Analysis

6.1 General

The available subsurface exploration information, site reconnaissance and records reviews provide a basis for slope stability analyses of the existing embankment slope configuration.

Stability of the downstream slopes and upstream slopes under full supply reservoir were done for long-term, seismic (pseudo-static) conditions, and post-earthquake conditions. Table 6 summarizes the results of slope stability analyses.

The global stability analysis utilizes ground strength and groundwater conditions to calculate factors of safety. In the Limiting Equilibrium Method (LEM), the factor of safety is generally defined as the factor by which shear strengths on a slip surface may be reduced in order to bring the slope into a state of limiting equilibrium along a given slip surface. The computer program SLOPE W has been used to undertake global stability analyses. The ground strengths used in the analyses are shown on Table 4. Appendices E1 and F contain selected stability sections showing stratigraphy, groundwater conditions, slope configuration and slip or rupture surface in the stability analysis.

6.2 Seismic Evaluation Parameters and Criteria

As outlined above, earthquake peak firm ground acceleration (PGA) values were obtained as shown on Table 5a. The related ground motions, in the form of response spectrum are shown in Table 5b.

Note that the above-noted response spectra are applicable for site conditions where the "firm ground" is at or near the surface. The firm ground is defined by the Geological Survey of Canada as soils with an average shear wave velocity in the range of 350 to 760 m/s. Very dense soils or soft bedrock would be classified as "firm ground" based on the above-noted shear wave velocity criteria.

The site-specific ground motions would be altered (amplified and/or attenuated) as the earthquake induced shear waves propagate through the subject site soils. To develop site-specific design ground motions, the following parameters were used:

- Based on 2012 SCPT results, the Site Class D was used, i.e., shear wave velocity > 180 m/s but less than 350 m/s;
- An amplified acceleration or PGA was used based on Fa = 1.3 as follows:

Earthquake Design Ground Motion (EDGM) Parameters

Site Class D:

AEP	1/2,500	1/5,000	1/10,000
PGA	0.18g	0.23g	0.286g

Earthquake Design Criteria

The seismic design criteria outlined on Table 6-1 of the CDA 2007 were used for a "Very High" consequence dam, i.e., AEP – 1/5,000 year event. The 1/10,000 AEP earthquake event was used for comparison purposes.

6.3 Seismic Liquefaction Assessment

The triggering of liquefaction was considered for 1/2,500, 1/5,000 and 1/10,000 AEP events.

Liquefaction assessment was carried out using the procedure outlined in Youd et al. (2001). Post-liquefaction settlements were assessed using the procedure outlined in Zhang et al. (2002).

Both Youd's and Zhang's procedures to assess liquefaction and post-liquefaction settlement are based on the calculated cyclic stress ratio (CSR) taken for design seismic events and the cyclic resistance ratio (CRR) obtained through cone penetration testing (CPT) and Standard Penetration Testing (SPT).

Determination of CSR

The liquefaction assessment of the Garnet Dam site was performed by a simplified approach. Peak ground acceleration (PGA) or EDGM parameters were obtained for the site by interpreting appropriate factors as outlined above. Cyclic stress ratio's (CSR) for the liquefaction assessment were obtained using Seed's Simplified method.

Table 6.3.1 shows the results of liquefaction analyses based on SCPT 12-1 and DH 2. The other testholes generally indicated equal or better ground as compared to results shown on Table 6.3.1.

Table 6.3.1 – Selected Liquefaction Analyses Results

Testhole	AEP	Depth m	SPT N(2) Blows/ft	Liquefy?	Liquefied Layer Thickness, m
SCPT 12-1 Downstream	1/2500	0 to 8	25 to 45	No	-
		8.5	12 to 20	Yes	<0.5
	1/5000	0 to 8	25 to 45	No	-
		8.5	12 to 20	Yes	<0.7
	1/10,000	0 to 8	25 to 45	No	-
		8.5	12 to 20	Yes	<0.7
DH 2 Upstream	1/2500	0 to 4.5(1)	> 15	No	-
		4.5 to 12	> 24	No	-
	1/5000	0 to 4.5(1)	> 15	No	-
		4.5 to 12	> 24	No	-
	1/10,000	0 to 4.5(1)	> 15	No	-
		4.5 to 12	> 24	No	-

(1) Fines Content > 30%.

(2) SPT N values in blows/foot as measured or inferred by correlations.

Based on the results of the liquefaction assessment, the potential of liquefaction and the impact of liquefaction are considered to be generally limited. Calculated settlements due to liquefaction were about 30mm with the calculated thickness of liquefied soils at 8.5m depth in SCPT 12-1.

As some liquefaction was indicated in very localized zones (i.e., no well-defined liquefying layer), a liquefaction induced mass flow slide is considered very unlikely.

6.4 Slope Stability

The stability analyses of the Garnet Dam meet traditional standards-based factors of safety (CDA 2007) for stability (Table 6 and Appendix F).

6.4.1 Long-term Stability (Steady State)

The long-term stability for the embankment slopes exceeds the standards based criteria for dams (Table 6). Note that rapid drawdown effect on the upstream slope is beyond the scope here, but the original dam design records indicated favourable stability.

Appendix F1 shows the selected stability sections including factor of safety and associated slip surfaces (non-circular).

6.4.2 Pseudo-Static Stability (Earthquake)

The downstream slope meets criteria for pseudo-static stability, including the 1/10,000 AEP event (Table 6). Appendix F2 shows the selected stability section for the 1/10,000 AEP event.

The upstream slope meets criteria for 1/5000 AEP event (Table 6), but criteria are slightly exceeded for the 1/10,000 AEP event. However, based on F > 0.9 for the 1/10,000 AEP event, it is anticipated that "Newmark" type ground displacement would be relatively small and much less than the available freeboards. Appendix F2 shows the stability Section AA and BB for AEP 1/5000 event.

The pseudo-static analyses indicated that after a major earthquake, the upstream slope could be prone to some ground movements as compared to little or no movement on the downstream slope.

6.4.3 Post-Earthquake Stability

The upstream slope exceeds post-earthquake stability criteria (Table 6) mainly because no liquefaction was indicated based on available records. Some stability checks were done for assumed limited liquefaction scenarios and stability factors of F > 1.3 were indicated.

The downstream slope indicated compliance with the typical criteria, depending on the assumed post-liquefaction strength (Table 6). However, the liquefiable layer is less than 1m thick at depths of 8 to 9m and the worst case F> 1.1. Therefore, the mass movement or flow slide potential is considered very unlikely. Appendix F3 shows the selected stability sections.

6.5 Seismic Failure Mechanisms

6.5.1 Criteria Review – Seismic Conditions

In general, the indicated stability factors meet the seismic criteria. However, current information available indicates that the upstream slope may suffer more movements than the downstream slope, under earthquake loadings. In particular, the pseudo-static factors of safety for the upstream slope are slightly less than that for the downstream slope (Table 6).

6.5.2 Failure Scenarios

Table 7 outlines failure mode and scenario for the principle components of the Garnet Lake Dam.

Overtopping

There appears to be a very limited potential for overtopping under earthquake scenarios, because the estimated crest settlement is much less than the typical freeboard.

Fissures and Cracking

Minor settlement of the embankment is anticipated under design seismic conditions. The following may be anticipated:

- Relatively uniform settlements of spillway, relatively minimal magnitudes;
- Some non-uniform settlement at main dam, increasing toward right abutment;
- Non-uniform settlement/deformation pattern along the crest and upstream slope related to:
  - Potential liquefaction within very localized zones;
  - Seismic induced downslope movements.
- Uncertain potential piping and internal erosion related to:
  - Potential increased seepage related to ground movements;
  - Potential piping issue due to rock/earth interface at right abutment.

The evaluation of piping potential due to current right abutment design and construction (no "slush" grout on rock surfaces) is beyond the scope here.

6.5.3 Post-Seismic Response

The dam may be expected to leak more after moderate to large earthquakes, at least temporarily. The dam safety management plan should detail appropriate action in response to earthquakes. These may include significant review after major earthquakes. A response for earthquakes may also include:

- immediate inspection, based on an appropriate inspection checklist;
- testing of outlet works to evaluate integrity, etc.

The response plan for earthquake inspections should be updated and provided with the OMS/EPP documents.

7. Conclusions and Recommendations

The seismic slope stability analysis has provided an opportunity to:

- Update seismic hazard analyses and select EDGM parameters (Table 5);
- Carry out simplified liquefaction analyses based on 2012 exploration data and 1975 Records;
- Carry out embankment stability analyses and estimate seismic stability based on inferred material parameters and seepage conditions (Table 6);

- Comment on earthquake failure mechanisms.

The following summarizes the findings of the seismic slope stability analysis:

- The downstream slope generally meets traditional standards-based stability criteria.
- For the upstream slope, the anticipated seismic induced embankment settlements are significantly less than the freeboard at full supply lake level.
- The liquefaction analyses indicated that liquefaction potential is very limited (based on 2012 in-situ tests) and therefore, flow slide instability is considered very unlikely.

7.1 Toe Buttress – Downstream Slope

It is considered that the existing downstream slope complete with benches is stable enough for seismic design considerations. Therefore, provision of an additional berm/toe buttress is considered inappropriate and unwarranted for stability purposes.

8. Closure

We trust that the information provided herein is sufficient for your current needs. This report was prepared for the exclusive use of the District of Summerland, Engineering & Public Works, and their designated consultants/agents, and may not be used by other parties without written consent of **exp** Services Inc.

Should you have any questions or concerns, please do not hesitate to contact the undersigned at your convenience.

Yours truly,

exp Services Inc.

Don Sargent, P.Eng.
Senior Engineer

Reviewed by:

Trevor Lumb, P.Eng.
Senior Discipline Manager

Tables

Table 1 – Summary of Current General Arrangement

Dam Component	Descriptions
Earth Embankment Crest Level	12m high, on Earth and Rock Foundation El. 634.5m
Impervious Blanket	Upstream, from toe of dam to former dam
Upstream Slope	Impervious Earth, 2.5H:1V
Gate Tower (Vertical)	Situated Upstream of dam crest c/w dry well
Downstream Slope	Upper slope – Impervious Earth, 2H:1V Bench – El. 628m Lower slope – Sandy Gravel, 2H:1V Erosion Protection – Rockfill/Rip Rap (450 thick)
Downstream Drainage Layer	Filter Layers, c/w two 150 dia. drain pipes outfall at outlet structure
Downstream Toe	Seepage Collection, Measurements (V-notch weir)
Spillway	Left Abutment – Free Overflow, Concrete Channel, c/w concrete lining segment above Rip Rap Lining on lower segment
Low Level Outlet	600 dia. pipe, c/w gate tower control valve Intake and outlet concrete structures
Water Works	• 450mm dia. pipe, control valve at gate tower (1975) • Waterworks meter chamber; situated downstream of dam

Special Note:

Dam Consequence Classification (BC Dam Safety Regulation, 2011)	"**Very High**"

Table 2 – Summary of Garnet Lake Dam Design and Safety History

Year	Design
1974 [1]	Design Report; Drawings and Specifications (H. Fellhauer) - Test pits in foundation areas
1975 December[1]	As-built drawings
1976 December[1]	Summary Report – Design and Construction (H. Fellhauer) - Includes two drill records within the upstream slope foundation area
1976[1]	Inspection & maintenance outline (Summary Report)
DOS Files	Inspection, maintenance and surveillance record
1998	Dam Safety Review by Golder Associates
2010	Dam Safety Review by Associated Engineering (Draft Report)

DOS – District of Summerland

(1) H. Fellhauer Engineering Consultant

Table 3 – Characterization Summary

Feature/Unit	Drills and Pits	Design and Index Lab Tests	Construction Test Records	In-situ Testing 2012
Embankment Units				
A Impervious Fill	-	Sieve	IPD[2] Sieve	CPT 12-3
B,C Sandy Gravel	-	Sieve	IPD[2]	CPT 12-2
Groundwater[1]	1974 Pits	-	-	CPT 12-1, 12-2 Porewater Dissipation Test (2012) Post-Construction Monitoring [1]
Foundation Strata				
FA SAND, Silty	1975 TH 2, 3 1974 Test pits	- -	- -	CPT 12-1 SCPT 12-1
FB SAND & GRAVEL, Silty	As Above	-	-	As Above
FC SILT, Sandy	As Above	-	-	As Above
FD SAND & GRAVEL	As Above	-	-	As Above
Groundwater Profile[1]	-	-	-	CPT 12-1, 12-2 Porewater Dissipation Test (2012) Post-Construction Monitoring [1]

(1) Seepage Monitoring 1975/1976 (1976 H. Fellhauer)

(2) IPD In-place Density Test, Standard Proctor Density Reference Testing by Interior Testing Services

Table 4 – Soil Parameters

Strata Feature/Unit		Soil Type	Long-term (Steady State)		Pseudo-Static	Post-Earthquake
Embankment						
A	Impervious Fill 98% SPD	GM/SM	c = 5 kPa UW = 20	ϕ= 35º	√	√
B	Sandy GRAVEL Fill $N_{(1)60}$ > 40 Blows/ft	GW	c = 0 UW = 20	ϕ = 41º	√	√
C	Sandy GRAVEL Fill	GW	as above			-
Foundations						
FA	SAND, Silty N > 15 Blows/ft?	SM	c = 5 kPa UW = 18	ϕ = 31º	√	√
FB	SAND & GRAVEL, Silty $N_{(1)60}$ > 25 to 50 Blows/ft	GM/SM	c = 5 kPa UW = 21	ϕ = 37º	√	√
FC	SILT, Sandy to SAND, Silty	ML to SM	c = 10 kPa UW = 18	ϕ = 31º	√	Su/p' =0.1, but Su >7 to 14 kPa (SCPT 12-1 only)
FD	SAND & GRAVEL N > 40 Blows/ft	GP - GM	c = 5 kPa G= 21	ϕ = 39º	√	√

Legend :

C – Cohesion intercept on the Mohr Coulomb Failure Criteria

Φ – Friction Angle on the Mohr Coulomb Failure Criteria

UW – Material Unit weight, kN/m2

√ Same as Long-term Values

Table 5a –Seismic Hazard Parameters

Source	PGA			
AEP	**1/1000**	**1/2500**	**1/5000**	**1/10,000**
National Building Code, 2010				
Garnet Lake Dam	0.099	0.139	-	-
Revelstoke Dam	0.095	0.135	-	-
Coursier Dam, Revelstoke, BC				
Golder, 1998(1)	0.065	-	-	0.22
Garnet Dam, Earthquake Firm Ground Peak Acceleration				
Selected Garnet Lake Dam	-	0.14	0.18	0.22

(1) Obtained from Golder 1998, i.e., BC Hydro Seismic Hazard Review of British Columbia

Table 5b – Seismic Response Spectra and Site Coefficient

Period(s)	Acceleration			
AEP:	**1/1000(1)**	**1/2500(1)**	**1/5000**	**1/10,000**
0.2	0.195	0.282	0.40(3)	0.59(3)
0.5	0.131	0.182	-	-
1.0	0.1082	0.114	-	-
2.0	0.048	0.066	-	-
PGA	0.099	0.139	0.18(2)	0.22(2)

Site Class	Fa – Acceleration Based Site Coefficient			
AEP:	**1/1000**	**1/2500**	**1/5000**	**1/10,000**
C(4)	1	1	1	1
D(4)	1.3	1.29	1.24	1.17

(1) Seismic Hazard Calculation, Appendix E2

(2) Table 5a

(3) Estimated for analyses purposes

(4) As per 2010 National Building Code

Table 6 – Embankment Slope Stability

Condition	Typical Criteria[3]	Slope Section	Factor of Safety
Long-term (Static, Steady State Seepage)			
Downstream Slope, Full supply level	F ≥ 1.5	BB	F = 2.27
Upstream Slope, Full supply level	F ≥ 1.5	BB	F = 2.41
		AA	F = 2.43
Seismic (Pseudo-Static) – PHGA = 0.18g (1/2500 AEP), 0.23g (1/5000 AEP), 0.28g (1/10,000 AEP)			
Pseudo-static – Downstream Slope	F≥ 1.0	BB 1/2500	F = 1.41
		BB 1/5000	F = 1.25
		BB 1/10,000	F = 1.12
Pseudo-static – Upstream Slope	F≥ 1.0	AA, BB AEP = 1/2500	F = 1.24
		AA, BB AEP = 1/5000	F = 1.06
		AA BB 1/10,000	F = 0.93
Post – Earthquake, Liquefaction Very Limited			
Downstream Slope	F≥ 1.2 to 1.3	BB	F = 1.07 to 1.51[1]
Upstream Slope	F≥ 1.2 to 1.3	BB AA	F = 1.72[2] F = 1.95[2]

(1) Lower value shown for minimum Su = 7kPa, Upper value shown for minimum Su = 14 kPa on Unit FC.

(2) Value shown for minimum Su = 7 kPa on Unit FC.

(3) The typical criteria were taken from Table 6-1 in the 2007 Canadian Dam Safety Guidelines, utilizing Standards Based Design criteria for dams.

Table 7 – Seismic Hazard and Failure Mode Summary

Global Failure Mode	Element	Dam Downstream Slope	Dam Upstream Slope	Outlets (1)
Overtopping (Crest Elevation too low Freeboard Lost)	Slope instability	Very Unlikely	Very Unlikely	-
	Settlement, i.e., crest	Most Favourable	Slightly Less Favourable(2)	-
Collapse	Liquefaction flow slide (gross deformations)	Very Unlikely	Very Unlikely	-
	Seismic stability	Good enough	Good enough	-
	Water barrier	Good	Good	-
	Durability/cracking resistance (Earth Embankment)	Good	Good	-
	Fissures/internal erosion and piping	Unlikely	Unlikely(3)	-

Notes:

(1) The effect of failure mechanisms on outlet is beyond scope here.

(2) Provided the dam is resistant to liquefaction effects, the estimated deformations would be substantially less than the typical available freeboard.

(3) Note that right abutment may be a concern for potential piping because there is no concrete seal on the rock/earth embankment interface.

Appendix A

Interpretation & Use of Study and Report

INTERPRETATION & USE OF STUDY AND REPORT

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering consulting practices in this area. No other warranty, expressed or implied, is made. Engineering studies and reports do not include environmental consulting unless specifically stated in the engineering report.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF THE REPORT

The Report has been prepared for the specific site, development, building, design or building assessment objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS “APPROVED USERS”. The contents of the Report remain our copyright property and we authorise only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorised use of the Report.

5. INTERPRETATION OF THE REPORT

a. Nature and Exactness of Descriptions: Classification and identification of soils, rocks, geological units, contaminant materials, building envelopment assessments, and engineering estimates have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations, or building envelope descriptions, utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarising such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b. Reliance on Provided information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the report as a result of misstatements, omissions, misrepresentations or fraudulent acts of persons providing information.

c. To avoid misunderstandings, **exp** Services Inc. (**exp**) should be retained to work with the other design professionals to explain relevant engineering findings and to review their plans, drawings, and specifications relative to engineering issues pertaining to consulting services provided by **exp**. Further, **exp** should be retained to provide field reviews during the construction, consistent with building codes guidelines and generally accepted practices. Where applicable, the field services recommended for the project are the minimum necessary to ascertain that the Contractor’s work is being carried out in general conformity with **exp**’s recommendations. Any reduction from the level of services normally recommended will result in **exp** providing qualified opinions regarding adequacy of the work.

6.0 ALTERNATE REPORT FORMAT

When **exp** submits both electronic file and hard copies of reports, drawings and other documents and deliverables (**exp**’s instruments of professional service), the Client agrees that only the signed and sealed hard copy versions shall be considered final and legally binding. The hard copy versions submitted by **exp** shall be the original documents for record and working purposes, and, in the event of a dispute or discrepancy, the hard copy versions shall govern over the electronic versions. Furthermore, the Client agrees and waives all future right of dispute that the original hard copy signed version archived by **exp** shall be deemed to be the overall original for the Project.

The Client agrees that both electronic file and hard copy versions of **exp**’s instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except **exp**. The Client warrants that **exp**’s instruments of professional service will be used only and exactly as submitted by **exp**.

The Client recognizes and agrees that electronic files submitted by **exp** have been prepared and submitted using specific software and hardware systems. **Exp** makes no representation about the compatibility of these files with the Client’s current or future software and hardware systems.

Appendix B

Figures

2012 Testhole Location Plan – B1

Garnet Dam Plans – B2

2012 Site Topographic Survey Plan and Sections– B3

Appendix B1

2012 Testhole Location Plan


CPT12-03
SCPT12-02
SCPT12-01
2012 Testhole Location Plan
VAN-0209167-A0
- SCPT / CPT Location
CONETEC
123 ft
© 2012 Google
Image © 2012 GeoEye
Google

Appendix B2

Garnet Dam Plans

As-builts by H. Fellhauer Engineering:

1975 Embankment & Spillway General Arrangement, 118-20-R1

1975 Embankment Plan & Sections, 118-21-R2 and 118-27-R1

Relocated Road
Slope down 12%
El. 2073.8
Sta 1+79
20'
25'
El 2074.3
Sta 0+79
El 2078.8
El. 2071
El 2076.0
Sta 2+03 E
=Sta 0+34
Spillway Axis
El. 2075
Sta 0+00
Sta 0+22
40'
El.2080.3
El. 2076
El 2082
El. 2080
2082
2070
2060
2050
A
B
C
Access Road
to crest of dam
R=160'
Δ=67-00
Road to
Summerland
Spillway invert El 2037.2
Concrete encasement
Invert
El. 2032
2040
2030
Channel of creek
paved with
large boulders
El. 2082
Crest of dam
El. 2082+
Axis of Dam
8.7'
9.4'
12.0'
Slope 2½:1
Slope 2:1
2:1
Berm El. 2063.5
CHLORINATOR
STATION
Top of slab
El. 2049.0
18"ø A.C. Pipe
18"ø Ductile
iron pipe
39'
METER CHAMBER
4" Valve for fishwater release
Inv. El. 2040.42
OUTLET STILLING BASIN
Inv. El. 2039.4
V-Notch weir
Eneas Creek
GATE WELL
Sta 0+00
El. 2047.0
El.2042
INTAKE
W. FISH SCREENS
Access bridge
24"ø Concrete
culvert
Outlet of
Gate well drain
Outlets of Foundation Drains 6"ø
Inv. El. 2036.85
Berm
El. 2046.5
From outlet of Old Dam
3:1
Impervious blanket
2080
40.0'
El. 2082.0
El. 2076.0
El. 2075.0
1½:1
Cut-off wall
8" reinf. concrete
SECTION A-A
25.0'
2.5'
2' Riprap
handplaced
8" Reinf. concrete
8" Sand
SECTION B-B
35.0'
20.0'
5.0'
2' Riprap
(50%-1'-size or larger)
SECTION C-C
Revision R1 - "As built" Dec. 1975
THE CORPORATION OF THE DISTRICT
OF SUMMERLAND
GARNET LAKE DAM
EMBANKMENT & SPILLWAY
ARRANGEMENT
H. FELLHAUER, ENGINEERING CONSULTANT
DATE
July 1975
Scale: 1" = 20'
1" = 10'
DRAWING No.
118-20 R1

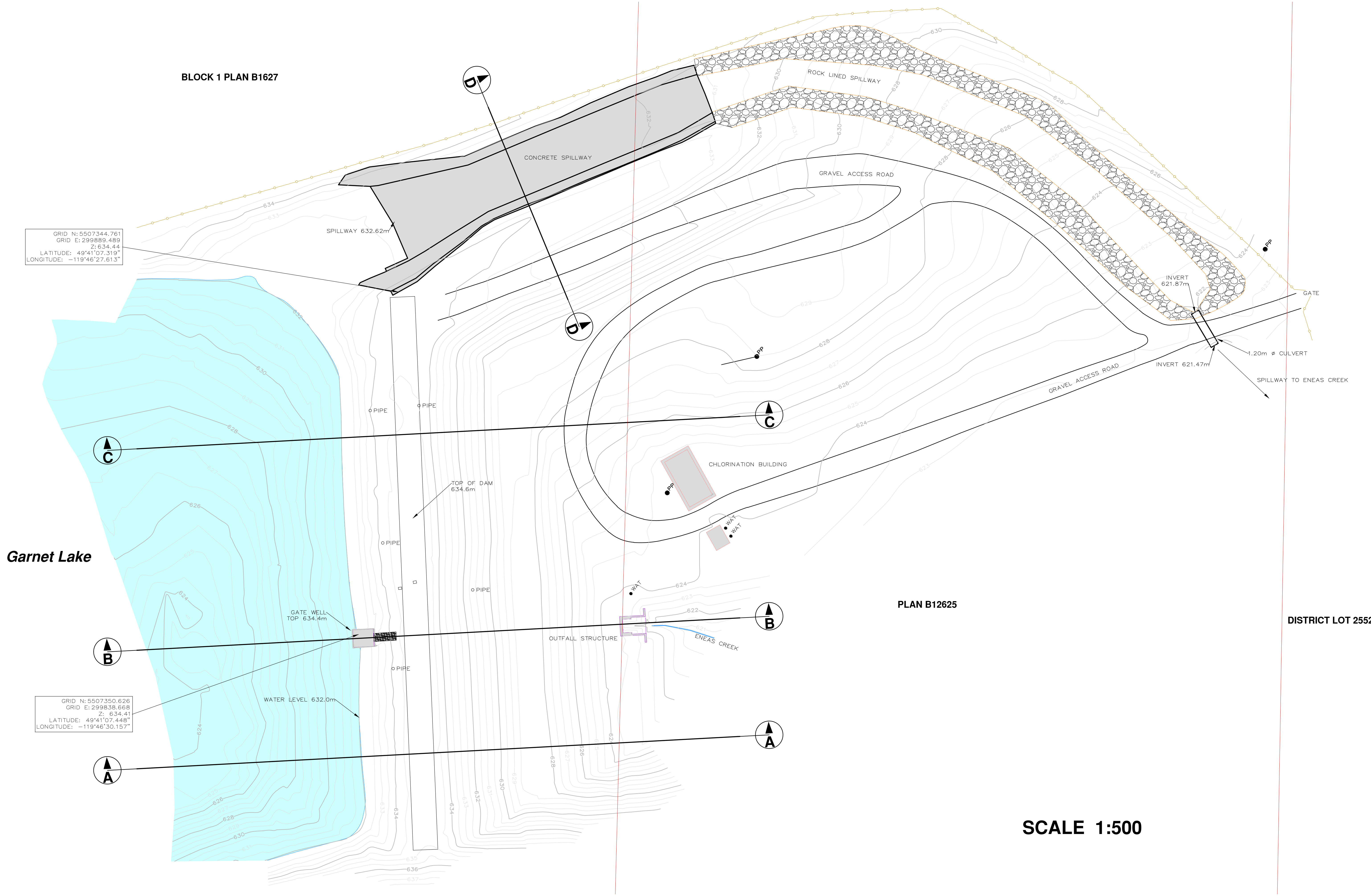

Survey Mon. M-2
Station 3+44.40
El. 2113.41
Prior Axis of Dam
Spillway see Dwg 118-20
Impervious fill
Sta 1+70 E
Crest El. 2082.0
Sandy gravel
Bottom filter 2.0' thick
Upstream toe of dam
B
F
Upstream toe of bottom filter
Bottom filter 3.0' thick
Downstream toe of dam (Rockfill facing not shown)
Upstream toe of sloping filter
Axis of Dam S88-00 W
Impervious fill
Bottom filter 3.0' thick
D
6" Drain
Outlet stilling basin
Intake w. fishscreens
Top El. 2082.0
20.0'
Crest El. 2082.5
Sta 0+00
Drain outlets
Creek bed
Gate well
Survey Mon. M-3
El. 2082.795
Low level outlet
4"ø Gate well drain
E
A
Impervious blanket not shown see Dwg. 118-27
Crest El. 2082.5
2+60
6" Drain
Foundation cutoff see Dwg. 118-27 R1
Bottom filter - 3.0' thick
Bottom filter - 2.0' thick
6.0'
Crest El. 2082.0
PLAN
1" = 20'
Prior Axis of Dam
Survey Mon. M-1
El. 2121.28
Sta. 1+67.70
Axis of Dam
12.0'
El. varies, see Plan
El. 2082.0
2.0
1.6
2.5
Sandy gravel layer 12" to 18" thick
For upstream section see Dwg. 118-27
Upstream limit of bottom filter
Rock fill
18" min.
12.0'
El. 2062.0
Slope down
Impervious fill
1.4
Pervious fill (Sandy gravel)
Sloping filter, 3.0' thick
Rock fill
20.0'
El. 2045.0
Bottom filter 3.0' thick
Rockfill
Creek bed
Graded gravel
6" Perforated drain pipe
SECTION A-A
1" = 10'
Crest of dam
Axis of Dam
El. varies
Terminate filter at El. 2077
Impervious fill
Bottom filter 2.0' thick
Sandy gravel
SECTION B-B
1" = 10'
Pervious fill
Bottom filter
Graded gravel
Sandy gravel
3.5"
6" Perforated drain pipe
Outlet structure
12"
6"
SECTION D-D
1" = 4'
Bottom filter, 3.0' thick
Pervious fill
Stripped surface of creek bed
6" Perforated drain pipe
Graded gravel
Sandy gravel
12"
3.0'
SECTION E-E
1" = 4'
15'
1.5
Impervious fill
12' min.
H/2
H
Sandy gravel
SECTION C-C
1" = 10'
For description of filter and embankment material see Construction Specification.
Crest of dam
Impervious fill
Axis of Dam
Rockfill
18"
Upstream toe of bottom filter
El. 2062.0
Pervious fill
Rock fill 18" thick
Bottom filter 3.0' thick
Sloping filter 3.0' thick
SECTION F-F
1" = 10'
Revision R2 - "As built" Dec. 1975
THE CORPORATION OF THE DISTRICT OF SUMMERLAND
GARNET LAKE DAM
EMBANKMENT
PLAN & SECTIONS
SHEET 1
H. FELLHAUER ENGINEERING CONSULTANT
DATE July 1975
SCALE As shown
DRAWING NO. 118-21 R2

Revision R1 - "As built" Dec. 1975

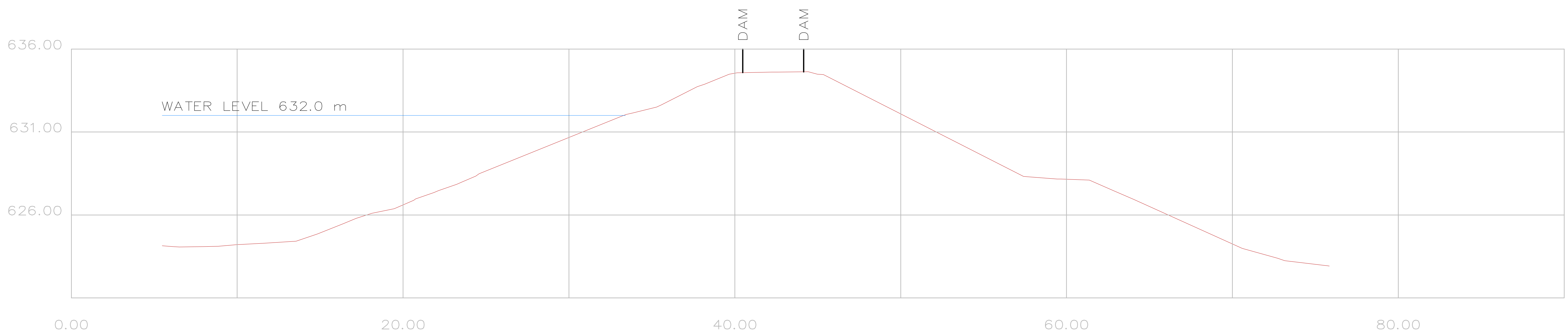
THE CORPORATION OF THE DISTRICT OF SUMMERLAND		
GARNET LAKE DAM EMBANKMENT PLAN & SECTIONS SHEET 2		
H. FELLHAUER, ENGINEERING CONSULTANT		
DATE Dec. 1975	SCALE 1" = 20'	DRAWING NO. 118-27 R1

Appendix B3

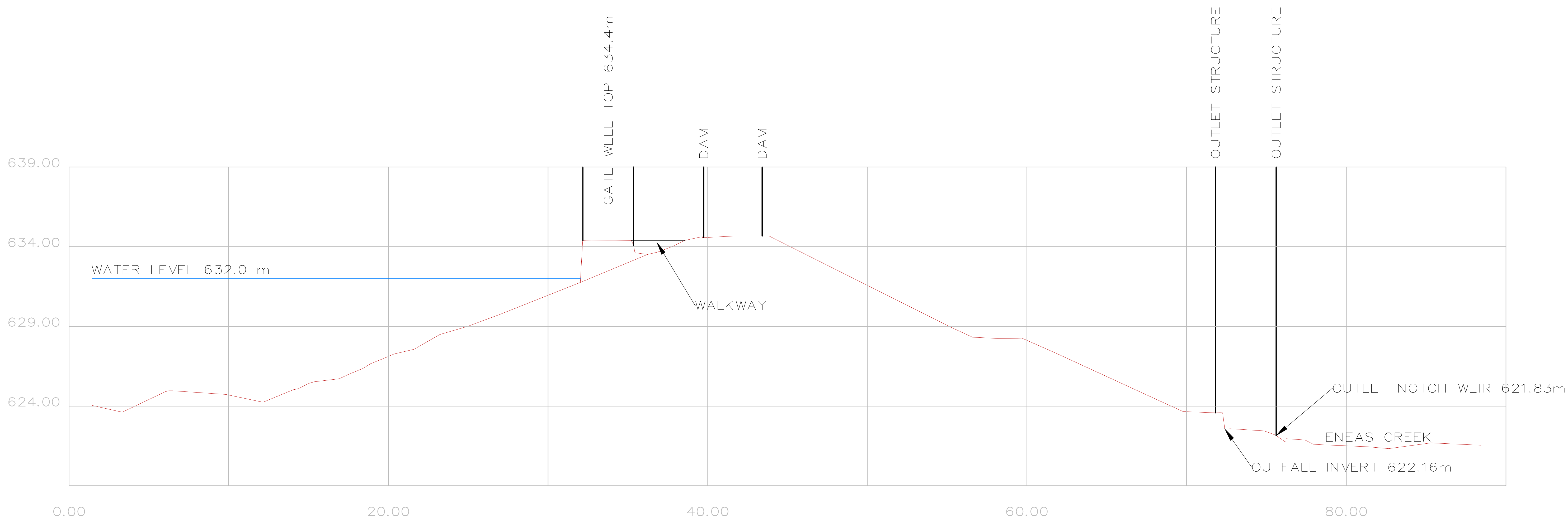
2012 Site Topographic Survey Plan and Sections



BLOCK 1 PLAN B1627
CONCRETE SPILLWAY
ROCK LINED SPILLWAY
GRAVEL ACCESS ROAD
SPILLWAY 632.62m
GRID N: 5507344.761
GRID E: 299889.489
Z: 634.44
LATITUDE: 49°41'07.319"
LONGITUDE: −119°46'27.613"
INVERT 621.87m
GATE
1.20m ø CULVERT
INVERT 621.47m
SPILLWAY TO ENEAS CREEK
GRAVEL ACCESS ROAD
PIPE
CHLORINATION BUILDING
TOP OF DAM 634.6m
Garnet Lake
GATE WELL TOP 634.4m
OUTFALL STRUCTURE
ENEAS CREEK
PLAN B12625
DISTRICT LOT 2552
GRID N: 5507350.626
GRID E: 299838.668
Z: 634.41
LATITUDE: 49°41'07.448"
LONGITUDE: −119°46'30.157"
WATER LEVEL 632.0m
SCALE 1:500

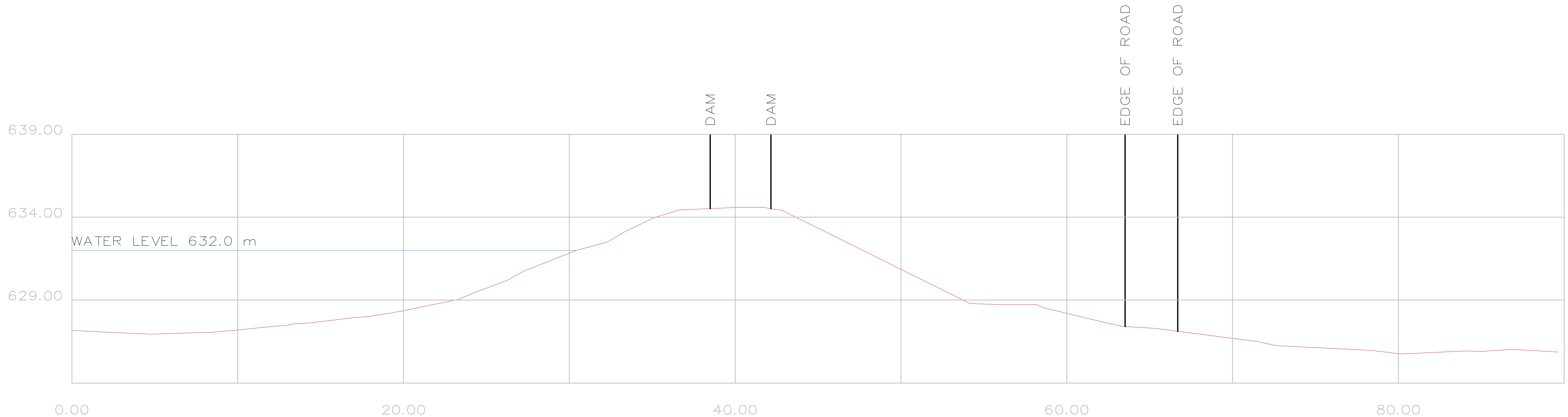


SECTION A-A

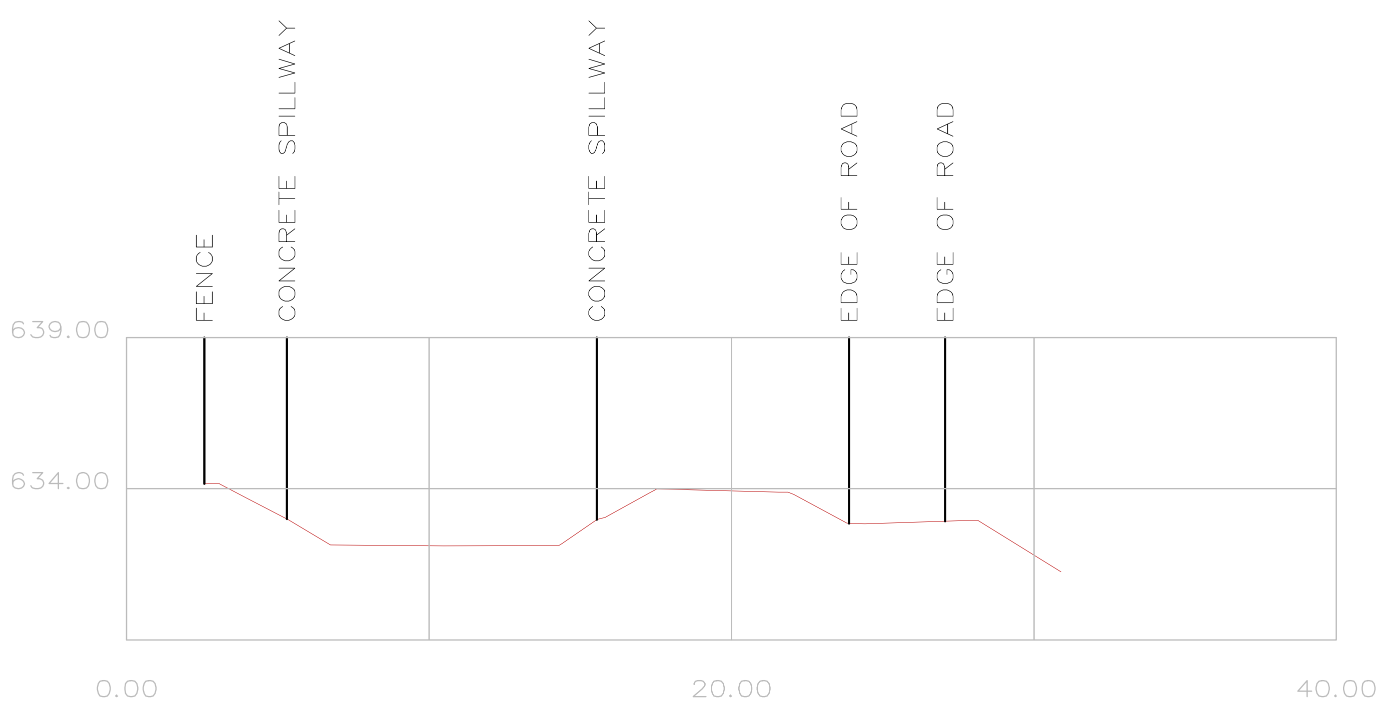


SECTION B-B

SCALE 1:250



SECTION C-C



SCALE 1:250

SECTION D-D

Appendix C

Selected Site Investigation Records

2012 Exploration Testhole Logs – C1

Synopsis – Dam Foundation and Embankment Records Review – C2

Test Pit Logs plus Location Plan, 1974 – C3

Drill Hole Logs, 1975 – C4

Summary of Laboratory Testing (1970's) – C5

Appendix C1

2012 Exploration Testhole Logs

SCPT, CPT Summary

SCPT 12-1 and 12-2, CPT 12-3 Standard Format

SCPT 12-1 and 12-2, CPT 12-3 Shear Wave Format

Dissipation Summary

CONETEC	Job No:	12-252
	Client:	EXP Services Inc.
	Project:	Garnet Lake, Summerland, BC
	Date:	October 2nd, 2012

CPT SUMMARY

CPT Sounding	File Name	Date	Cone	Assumed Phreatic Surface (m)	Final Depth (m)	Handheld GPS UTM Northing (m)	Handheld GPS UTM Easting (m)
CPT12-03	252CP03	10/02/12	342:T1500F15U500	> 2.2	2.20	5507341	299828
SCPT12-01	252SP01	10/02/12	342:T1500F15U500	2.1	9.00	5507313	299851
SCPT12-02	252SP02	10/02/12	342:T1500F15U500	> 3.15	3.15	5507328	299866

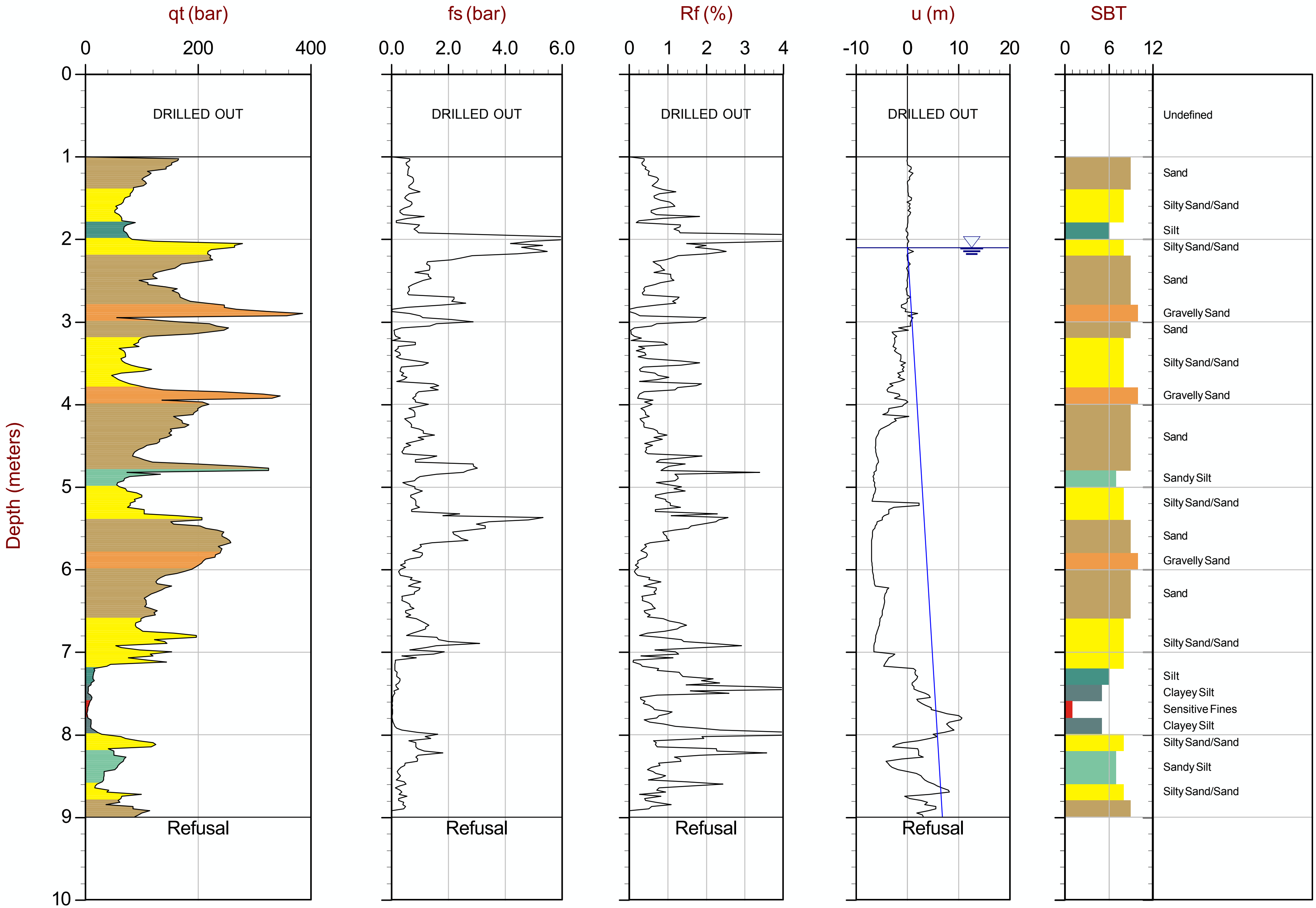
Note: Assumed phreatic surface based on pore pressure dissipations unless otherwise noted, assumed hydrostatic conditions for interpretation tables.
Datum: WGS 84 / UTM Zone 11 North.

CONETEC

EXP

Job No: 12-252
Date: 10:02:12 11:28
Site: Garnet Lake, Summerland, BC

Sounding: SCPT12-01
Cone: 342:T1500F15U500



Max Depth: 9.000 m / 29.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 252SP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Coords: UTM 11N N: 5507313m E: 299851m

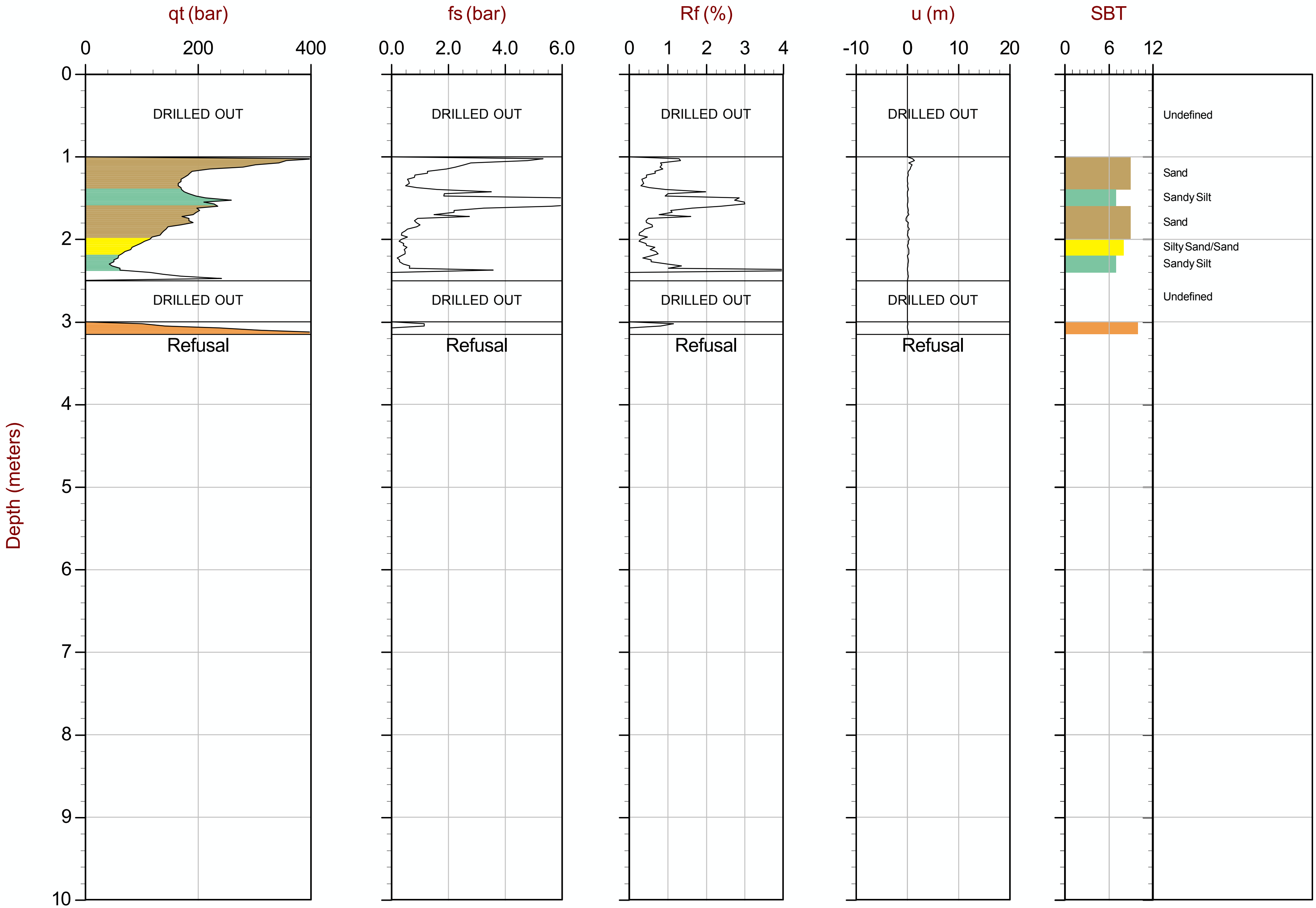
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

EXP

Job No: 12-252
Date: 10:02:12 13:02
Site: Garnet Lake, Summerland, BC

Sounding: SCPT12-02
Cone: 342:T1500F15U500



Max Depth: 3.150 m / 10.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 252SP02.COR
Unit Wt: SBT Chart Soil Zones

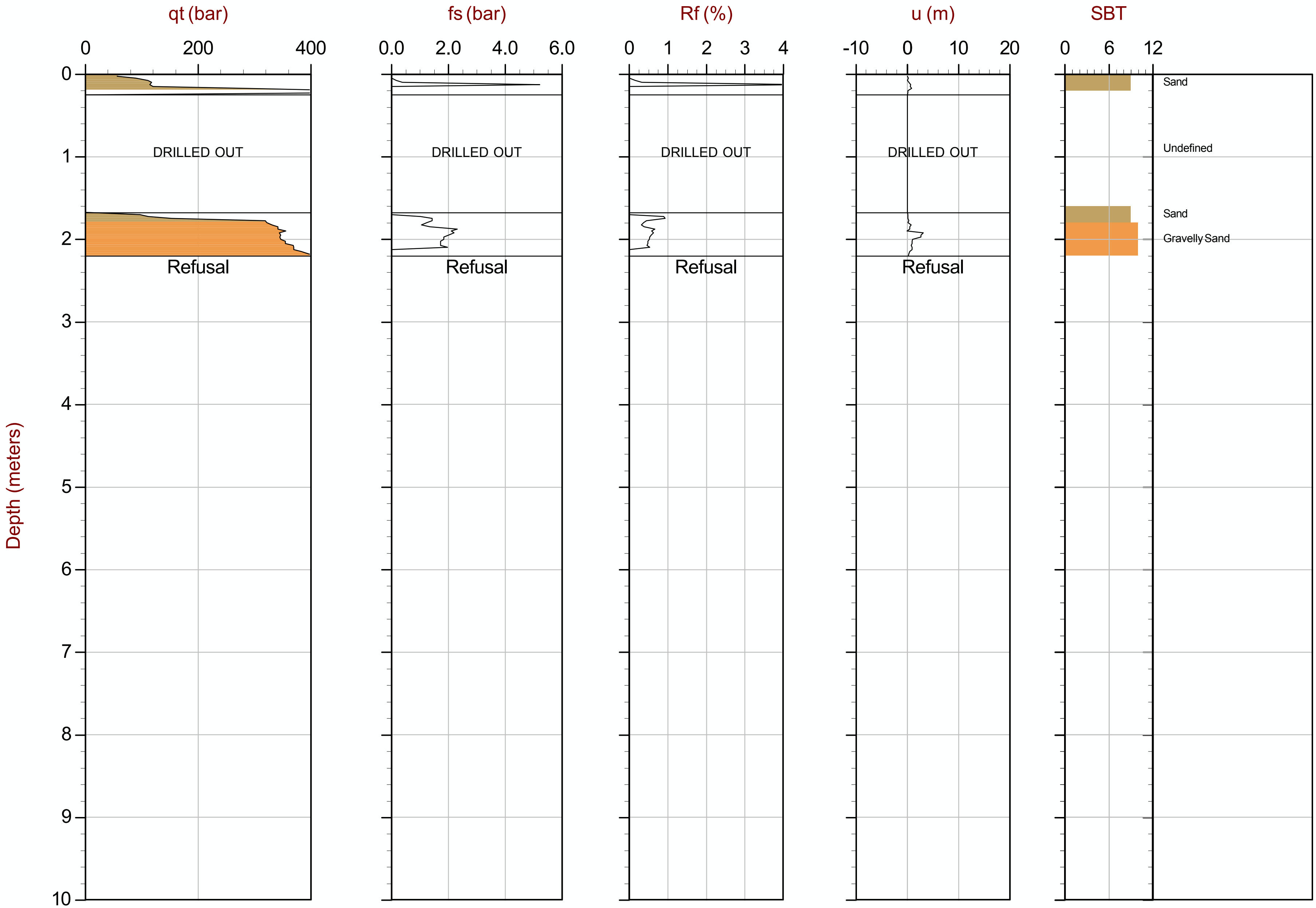
SBT: Lunne, Robertson and Powell, 1997
Coords: UTM 11N N: 5507328m E: 299866m

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

EXP

Job No: 12-252
Date: 10:02:12 15:11
Site: Garnet Lake, Summerland, BC
Sounding: CPT12-03
Cone: 342:T1500F15U500



Max Depth: 2.200 m / 7.22 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 252CP03.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Coords: UTM 11N N: 5507341m E: 299828m

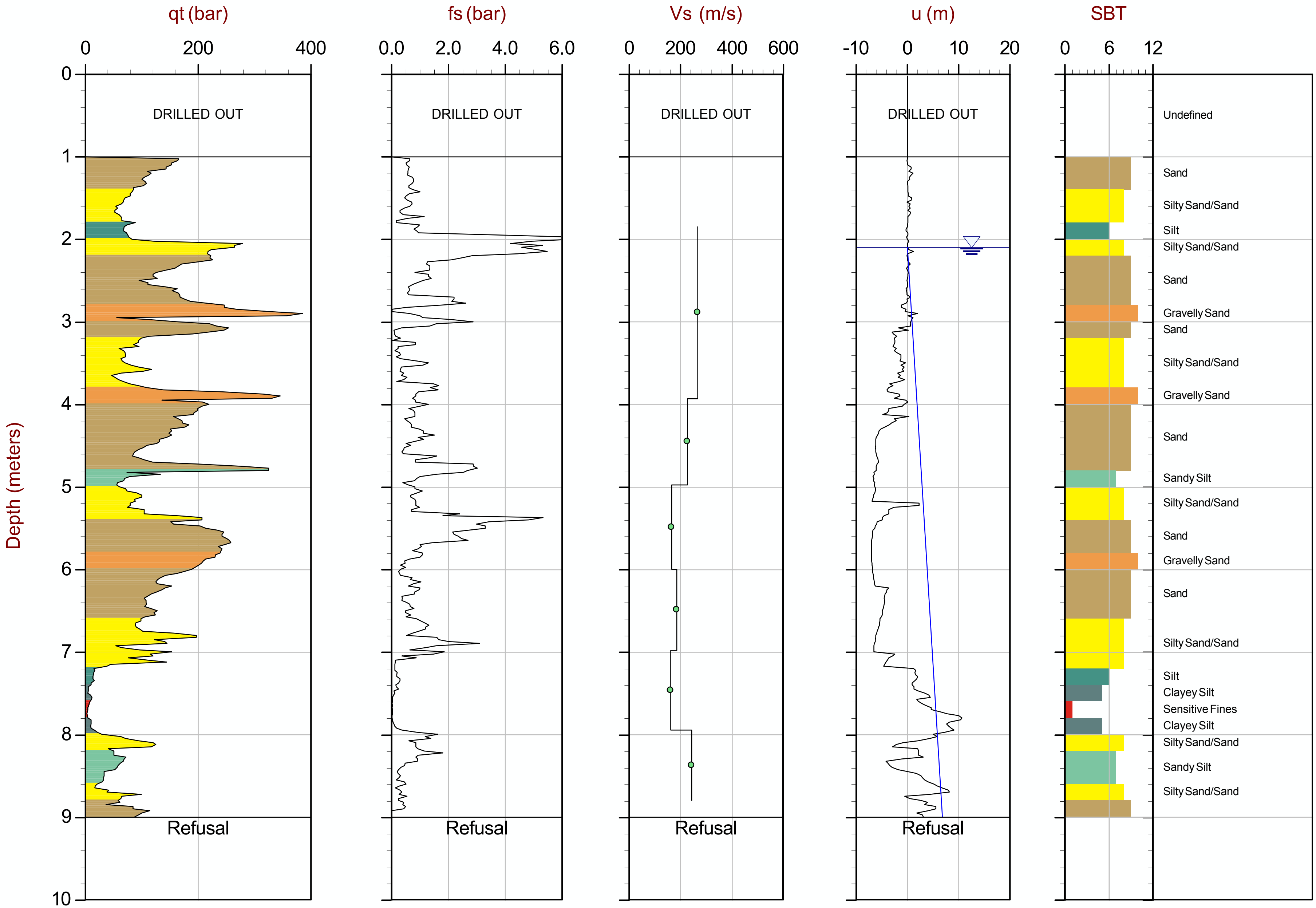
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

EXP

Job No: 12-252
Date: 10:02:12 11:28
Site: Garnet Lake, Summerland, BC

Sounding: SCPT12-01
Cone: 342:T1500F15U500



Max Depth: 9.000 m / 29.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 252SP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Coords: UTM 11N N: 5507313m E: 299851m

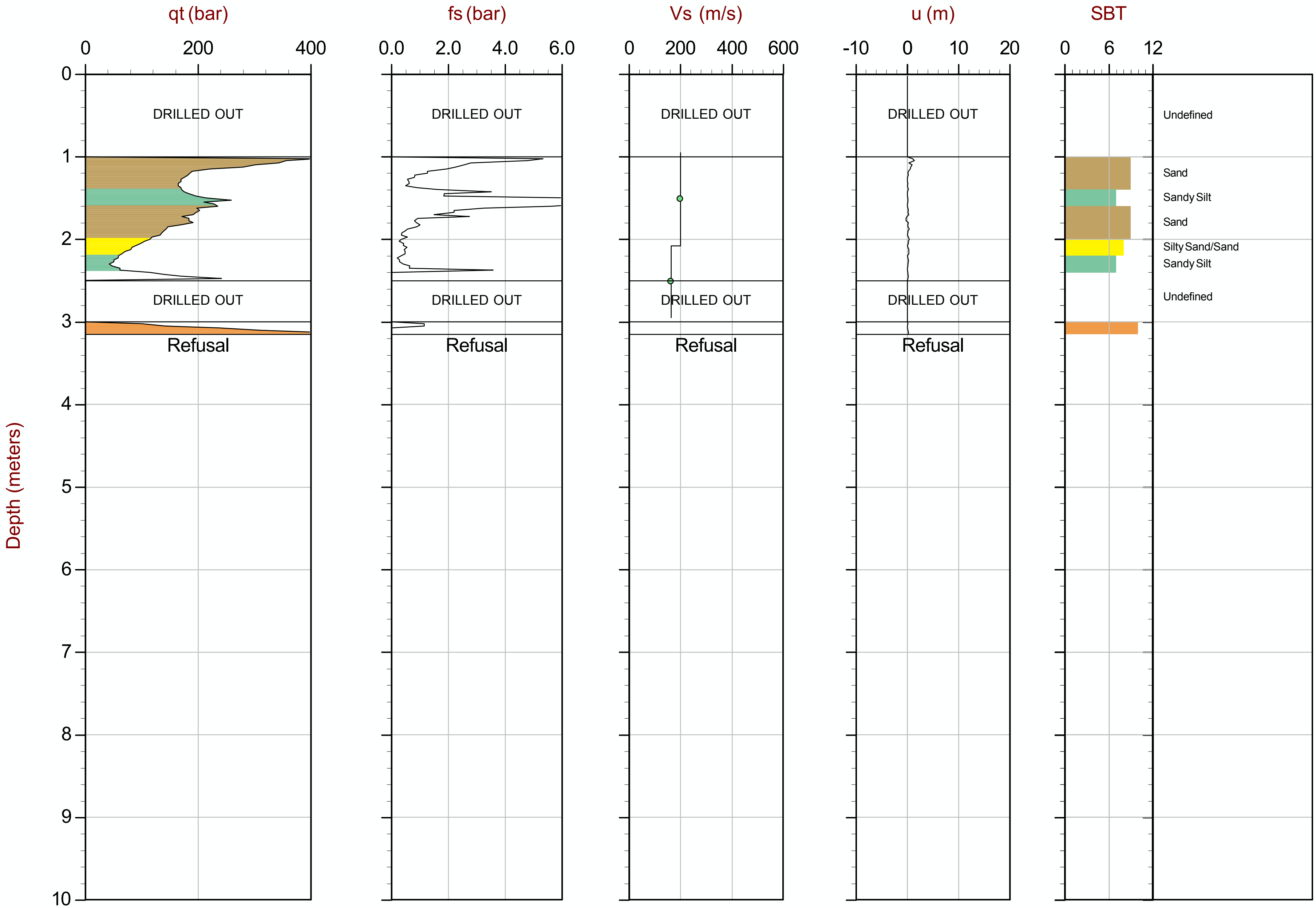
The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec

EXP

Job No: 12-252
Date: 10:02:12 13:02
Site: Garnet Lake, Summerland, BC

Sounding: SCPT12-02
Cone: 342:T1500F15U500



Max Depth: 3.150 m / 10.33 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: 0.200 m

File: 252SP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Coords: UTM 11N N: 5507328m E: 299866m

The reported coordinates were acquired from hand-held GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Job No: 12-252
Client: EXP Services Inc.
Project: Garnet Lake, Summerland, BC
Date: October 2nd, 2012

PPD SUMMARY

CPT Sounding	Duration (s)	Test Depth (m)	Equilibrium Pore Pressure U_{eq} (m)*	Calculated Phreatic Surface (m)
CPT12-03	390	2.000	0.0	
SCPT12-01	70	2.025	0.0	
SCPT12-01	515	2.025	0.0	
SCPT12-01	130	3.950	1.8	2.1
SCPT12-01	300	5.175	2.9	2.3
SCPT12-01	300	7.000	4.8	2.2
SCPT12-01	300	9.000	7.0	2.0
SCPT12-02	380	2.500	0.0	
SCPT12-02	180	3.150	0.0	

* Equilibrium pore pressure estimated from dissipation tests.

CONETEC INTERPRETATION METHODS

A Detailed Description of the Methods Used in ConeTec's CPT Interpretation and Plotting Software

Revision SZW-Rev 05A
April 8, 2011

Prepared by Jim Greig

ConeTec Interpretations as of April 8, 2011

ConeTec's interpretation routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. The interpreted values are not considered valid for all soil types. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the program and does not assume liability for any use of the results in any design or review. Representative hand calculations should be made for any parameter that is critical for design purposes. The end user of the interpreted output should also be fully aware of the techniques and the limitations of any method used in this program. The purpose of this document is to inform the user as to which methods were used and what the appropriate papers and/or publications are for further reference.

The CPT interpretations are based on values of tip, sleeve friction and pore pressure averaged over a user specified interval (e.g. 0.20m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. Since all ConeTec cones have equal end area friction sleeves, pore pressure corrections to sleeve friction, f_s, are not required.

The tip correction is: $q_t = q_c + (1\text{-}a) \cdot u_2$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weights that have been assigned to the Soil Behavior Type zones, from a user defined unit weight profile or by using a single value throughout the profile.

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (this can be obtained from CPT dissipation tests). For over water projects the effects of the column of water have been taken into account as has the appropriate unit weight of water. How this is done depends on where the instruments were zeroed (i.e. on deck or at mud line).

Details regarding the interpretation methods for all of the interpreted parameters are provided in Table 1. The appropriate references cited in Table 1 are listed in Table 2. Where methods are based on charts or techniques that are too complex to describe in this summary the user should refer to the cited material.

The Soil Behavior Type classification charts (normalized and non-normalized) shown in Figures 1 and 2 are based on the charts developed by Dr. Robertson and Dr. Campanella at the University of British Columbia. These charts appear in many publications, most notably: Robertson, Campanella, Gillespie and Greig (1986); Robertson (1990) and Lunne, Robertson and Powell (1997). The Bq classification charts shown in Figures 3a and 3b are based on those described in Robertson (1990) and Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on that discussed in Jefferies and Davies, 1993.

Where the results of a calculation/interpretation are declared *'invalid'* the value will be represented by the text strings "-9999" or "-9999.0". In some cases the value 0 will be used. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g. drilled out section or data gap).

2. Where the interpretation method is inappropriate, for example, drained parameters in an undrained material (and vice versa).

3. Where interpretation input values are beyond the range of the referenced charts or specified limitations of the interpretation method.

4. Where pre-requisite or intermediate interpretation calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the interpreted parameters listed in Table 1 may be included in the output files delivered with this report.

The output files are provided in Microsoft Excel XLS format. The ConeTec software has several options for output depending on the number or types of interpreted parameters desired. Each output file will be named using the original COR file basename followed by a three or four letter indicator of the interpretation set selected (e.g. BSC, TBL, NLI or IFI) and possibly followed by an operator selected suffix identifying the characteristics of the particular interpretation run.

Table 1
CPT Interpretation Methods

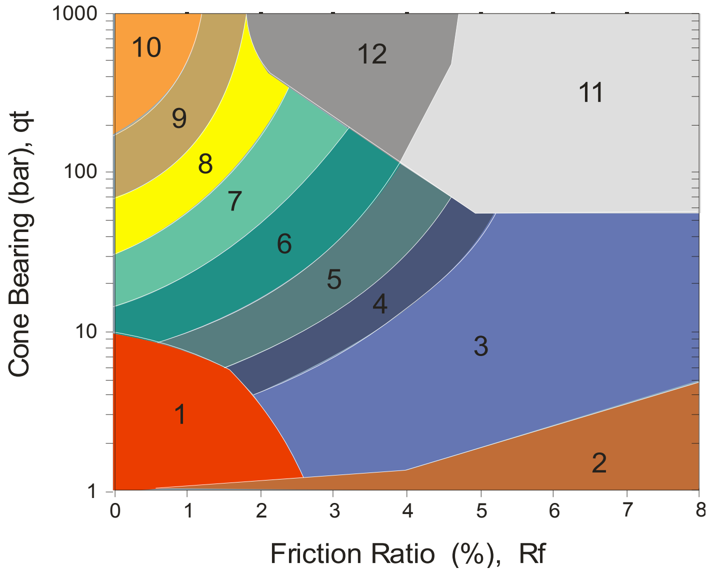
Interpreted Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where interpretations are done at each point then Mid Layer Depth = Recorded Depth)*	*Depth (Layer Top) + Depth (Layer Bottom) / 2.0*	
Elevation	Elevation of Mid Layer based on sounding collar elevation supplied by client	Elevation = Collar Elevation - Depth	
Avgqc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when interpretations are done at each point*	
Avgqt	Averaged corrected tip (q_t) where: $q_t = q_c + (1-a) \bullet u$	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when interpretations are done at each point*	
Avgfs	Averaged sleeve friction (f_s)	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when interpretations are done at each point*	
AvgRf	Averaged friction ratio (Rf) where friction ratio is defined as: $Rf = 100\% \bullet \frac{fs}{qt}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *n=1 when interpretations are done at each point*	
Avgu	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when interpretations are done at each point*	
AvgRes	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} RESISTIVITY_i$ *n=1 when interpretations are done at each point*	
AvgUVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when interpretations are done at each point*	
AvgTemp	Averaged Temperature (this data is not always available since it is a specialized test)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} TEMPERATURE_i$ *n=1 when interpretations are done at each point*	

Interpreted Parameter	Description	Equation	Ref
AvgGamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$Avgu = \frac{1}{n}\sum_{i=1}^{n} GAMMA_i$ *n=1 when interpretations are done at each point*	
SBT	Soil Behavior Type as defined by Robertson and Campanella	See Figure 1	2, 5
U.Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) user supplied unit weight profile	See references	5
T. Stress σ_v	Total vertical overburden stress at Mid Layer Depth. *A layer is defined as the averaging interval specified by the user. For data interpreted at each point the Mid Layer Depth is the same as the recorded depth.*	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ *where* γ_i is layer unit weight h_i is layer thickness	
E. Stress σ_v'	Effective vertical overburden stress at Mid Layer Depth	*Estress = Tstress -* u_{eq}	
Ueq	Equilibrium pore pressure determined from one of the following user selectable options: 1) hydrostatic from water table depth 2) user supplied profile	For hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is unit weight of water D is the current depth D_{wt} is the depth to the water table	
Cn	SPT N_{60} overburden correction factor	$Cn=(\sigma_v')^{-0.5}$ where σ_v' *is in tsf* $0.5 < C_n < 2.0$	
N_{60}	SPT N value at 60% energy calculated from qt/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	4, 5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure	$(N_1)_{60} = Cn \cdot N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the Ic parameter	$(qt/pa)/ N_{60} = 8.5 (1 – Ic/4.6)$	5
$(N_1)_{60}Ic$	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 2 options.	1) $(N_1)_{60}Ic = Cn \cdot (N_{60}\, Ic)$ 2) $q_{c1n}/ (N_1)_{60}Ic = 8.5 (1 – Ic/4.6)$	4 5
$(N_1)_{60cs}Ic$	Clean sand equivalent SPT $(N_1)_{60}Ic$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}Ic)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}Ic)$ 3) $q_{c1ncs})/ (N_1)_{60cs}Ic = 8.5 (1 – Ic/4.6)$ FC ≤ 5%: α = 0, β=1.0 FC ≥ 35% α = 5.0, β=1.2 5% < FC < 35% $\alpha = \exp[1.76 – (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
Su	Undrained shear strength based on q_t Su factor N_{kt} is user selectable	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
Su	Undrained shear strength based on pore pressure Su factor $N_{\Delta u}$ is user selectable	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
k	Coefficient of permeability (assigned to each SBT zone)		5

Interpreted Parameter	Description	Equation	Ref
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ *where:* $\Delta u = u - u_{eq}$ *and* u *= dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 5
Q_t	Normalized q_t for Soil Behavior Type classification as defined by Robertson, 1990	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5
F_r	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson, 1990	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Net qt	Net tip resistance	$qt - \sigma_v$	
qe	Effective tip resistance	$qt - u_2$	
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	
SBTn	Normalized Soil Behavior Type as defined by Robertson and Campanella	See Figure 2	2, 5
SBT-BQ	Non-normalized Soil Behavior type based on the Bq parameter	See Figure 3	2, 5
SBT-BQn	Normalized Soil Behavior based on the Bq parameter	See Figure 3	2, 5
SBT-JandD	Soil Behaviour Type as defined by Jeffries and Davies	See Figure 3	7
SBT-BQn	Normalized Soil Behavior base on the Bq parameter	See Figure 3	2, 5
I_c	Soil index for estimating grain characteristics	$Ic = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_{a2}}\right)\left(\frac{P_a}{\sigma'_v}\right)^n$ *And* *Fr is in percent* P_a *= atmospheric pressure* P_{a2} *= atmospheric pressure* *n varies from 0.5 to 1.0 and is selected in an iterative manner based on the resulting* I_c	3, 8
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ *for* $Ic > 3.5$ $FC=0$ *for* $Ic < 1.26$ $FC = 5\%$ *if* $1.64 < Ic < 2.6$ *AND* $F_r<0.5$	3
Ic Zone	This parameter is the Soil Behavior Type zone based on the Ic parameter (valid for zones 2 through 7 on SBTn chart)	Ic < 1.31 Zone = 7 1.31 < Ic < 2.05 Zone = 6 2.05 < Ic < 2.60 Zone = 5 2.60 < Ic < 2.95 Zone = 4 2.95 < Ic < 3.60 Zone = 3 Ic > 3.60 Zone = 2	3
PHI ϕ	Friction Angle determined from one of the following user selectable options: a) Campanella and Robertson b) Durgunoglu and Mitchel c) Janbu d) Kulhawy and Mayne	See reference	5 5 5 11

Interpreted Parameter	Description	Equation	Ref
Dr	Relative Density determined from one of the following user selectable options: a) Ticino Sand b) Hokksund Sand c) Schmertmann 1976 d) Jamiolkowski - All Sands	See reference	5
OCR	Over Consolidation Ratio	a) Based on Schmertmann's method involving a plot of S_u/σ_v' /($S_u/\sigma_v')_{NC}$ and OCR where the Su/p' ratio for NC clay is user selectable	9
State Parameter	The state parameter is used to describe whether a soil is contractive (SP is positive) or dilative (SP is negative) at large strains based on the work by Been and Jefferies	See reference	8, 6, 5
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart.	Based on Figure 5.59 in the reference	5
Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the Es/qt chart.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)^3$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress and $\sigma_h = K_o \cdot \sigma_v'$ with Ko assumed to be 0.5	5
q_{c1}	q_t normalized for overburden stress used for seismic analysis	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atm. Pressure q_t is in MPa	3
q_{c1n}	q_{c1} in dimensionless form used for seismic analysis	$q_{c1n} = (q_{c1} / Pa)(Pa/\sigma_v')^n$ where: Pa = atm. Pressure and n ranges from 0.5 to 0.75 based on Ic.	3
K_{SPT}	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT}	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference)	10
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \cdot K_{cpt}$	3
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833 [(q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93 [(q_{c1ncs}/1000]^3 + 0.08$	10

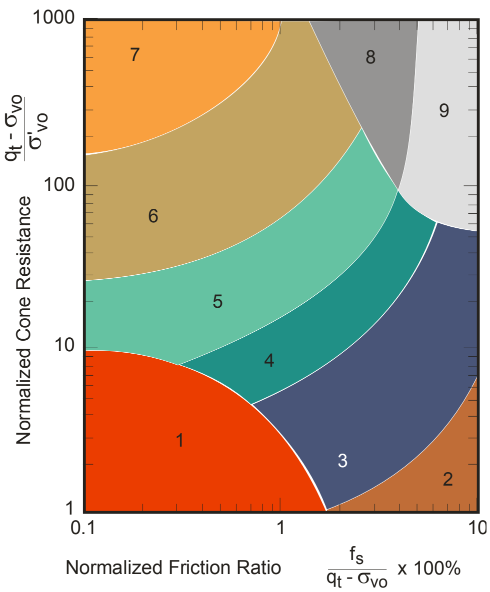
Interpreted Parameter	Description	Equation	Ref
CSR	Cyclic Stress Ratio	$CSR = (\tau_{av}/\sigma_v') = 0.65\ (a_{max} / g)\ (\sigma_v / \sigma_v')\ r_d$ $r_d = 1.0 - 0.00765\ z$ $z \leq 9.15m$ $r_d = 1.174 - 0.0267\ z$ $9.15 < z \leq 23m$ $r_d = 0.744 - 0.008\ z$ $23 < z \leq 30m$ $r_d = 0.50$ $z > 30m$	10
MSF	Magnitude Scaling Factor	See Reference	10
FofS	Factor of Safety against Liquefaction	$FS = (CRR_{7.5} / CSR)\ MSF$	10
Liquefaction Status	Statement indicating possible liquefaction	Takes into account FofS and limitations based on I_c and q_{c1ncs}.	10
Cont/Dilat Tip	Contractive / Dilative qc1 Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ qc1 is calculated from specified qt(MPa)/N ratio	13
Cq	Normalizing Factor	$Cq = 1.8 / (0.8 + ((\sigma_v'/Pa))$	12
qc1 (Cq)	Normalized tip resistance based on Cq	$q_{c1} = C_q * q_t$ (some papers use q_c)	12
Su(Liq)/s'v	Liquefied Shear Strength Ratio	$\frac{Su(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$	13



Zone	qt / N	Soil Behavior Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

Figure 1 Non-Normalized Behavior Type Classification Chart



Zone	Normalized Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay to silty clay
4	clayey silt to silty clay
5	silty sand to sandy silt
6	clean sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

Figure 2 Normalized Behavior Type Classification Chart

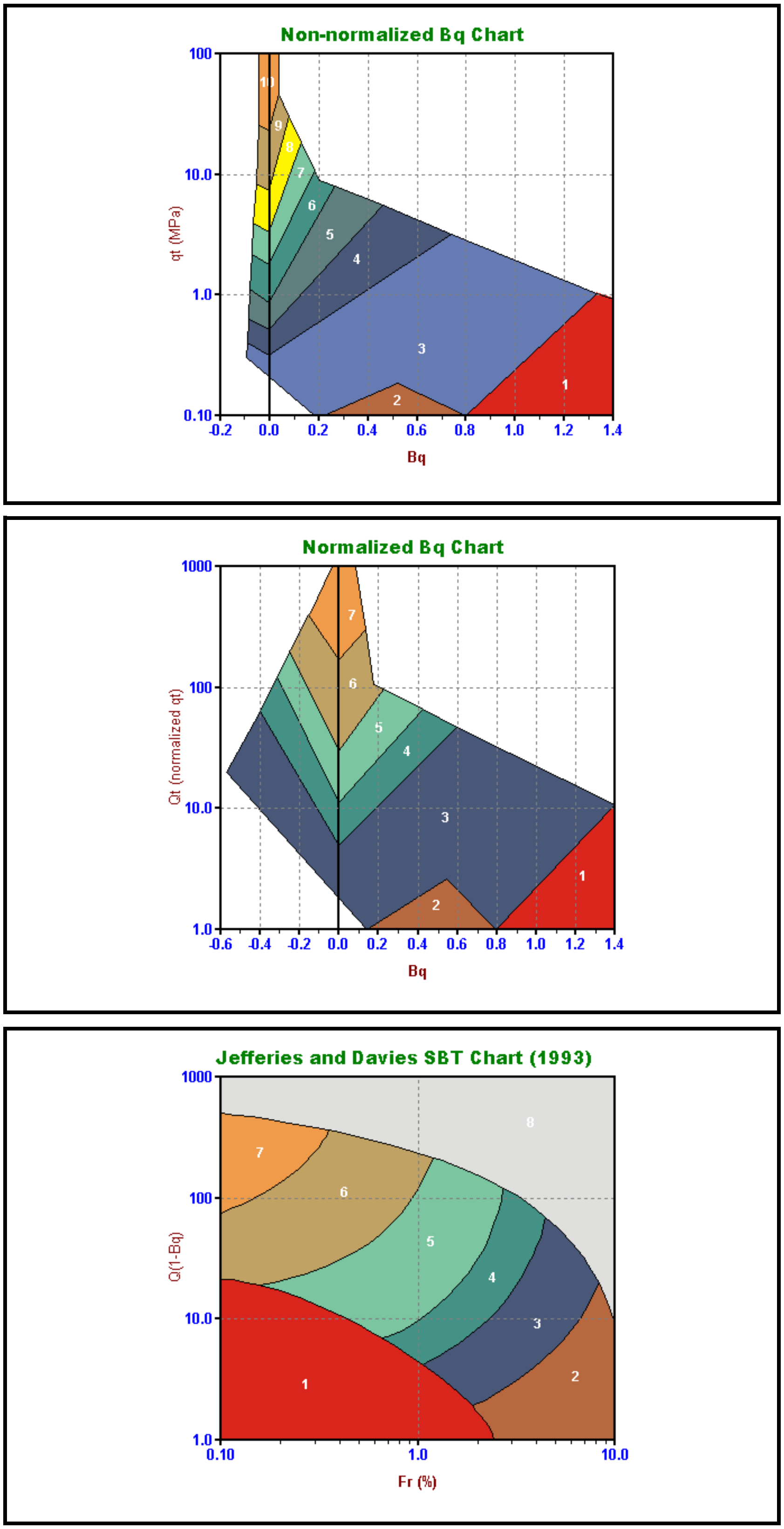


Figure 3 – Alternate Soil Behaviour Type Charts

Table 2 References

No.	References
1	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
2	Robertson, P.K., 1990, “Soil Classification Using the Cone Penetration Test”, Canadian Geotechnical Journal, Volume 27.
3	Robertson, P.K. and Fear, C.E., 1998, “Evaluating cyclic liquefaction potential using the cone penetration test”, Canadian Geotechnical Journal, 35: 442-459.
4	Robertson, P.K. and Wride, C.E., 1998, “Cyclic Liquefaction and its Evaluation Based on SPT and CPT”, NCEER Workshop Paper, January 22, 1997
5	Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, “ Cone Penetration Testing in Geotechnical Practice,” Blackie Academic and Professional.
6	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, “CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility”, 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.
7	Jefferies, M.G. and Davies, M.P., 1993. “Use of CPTu to Estimate equivalent N_{60}”, Geotechnical Testing Journal, 16(4): 458-467.
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9	Schmertmann, 1977, “Guidelines for Cone Penetration Test Performance and Design”, Federal Highway Administration Report FHWA-TS-78-209, U.S. Department of Transportation
10	Proceedings of theNCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt LakeCity, 1996. Chaired by Leslie Youd. 11
11	Kulhawy, F.H. and Mayne, P.W. ,1990, “Manual on Estimating Soil Properties for Foundation Design, *Report No. EL-6800”*, Electric Power Research Institute, Palo Alto, CA, August 1990, 306 p.
12	Olson, S.M. and Stark, T.D., 2002, “Liquefied strength ratio from liquefied flow filaure case histories”, Canadian Geotechnical Journal, 39: 951-966.
13	Oslon, Scott M. and Stark, Timothy D., 2003, “Yield Strength Ratio and Liquefaction Analysis of Slopes and Embankments”, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, August 2003.

Appendix C2

Synopsis – Dam Foundation and Embankment Records Review

Appendix C2

Synopsis – Dam Foundation and Embankment Records Review

Foundation Conditions

The report on the Proposed Dam (H. Fellhauer Engineering Consultant, 1974) described the foundation conditions as a "well graded mixture of silty sand and gravel with a few cobbles". Near the creek the deposits consist of "a 5 to 10 ft. thick layer of medium fine sand and silt, slightly clayey and dense". The test pit locations are shown on Figure 18-3 in Appendix C3. The laboratory test results are shown on Table C5-1 in Appendix C5.

In 1975, two additional exploratory holes were drilled DH # 2 and DH # 3 as outlined in the Summary Design and Construction Report (H. Fellhauer, 1976). The DH # 2 was located in the creek, at the upstream toe of the dam, and DH # 3 was located some 34 ft. (10.3 m) left of the creek, midway between the dam upstream toe and the gatewell. The logs of DH # 2 and DH # 3 are shown in Appendix C4.

Dam Embankment

A part of the embankment material testing program (H. Fellhauer, 1976), the following field and laboratory test records were obtained:

- In-place Density Tests
- Standard Proctor Density
- Atterberg Limits
- Sieve Gradations
- Triaxial CU Test (1)
- Permeability Tests (1)

(1) Material passing # 4 sieve compacted to 90 to 95% Standard Proctor Density.

However, the material in the triaxial test consisted of silty sand. The material placed in the embankment generally consisted of silty sand and gravel (H. Fellhauer, 1976). The embankment materials were compacted to an average in place density (28 construction records) of 98.2% Standard Proctor Density (SPD), with Standard Deviation of 3.5%. Therefore, the triaxial test was ignored because the material tested bears little resemblance to the sand and gravel, some silt and the triaxial test density (90 to 95% SPD) was substantially less than the achieved in-place density of 98.2% SPD.

Table C5-1 in Appendix C5 includes a summary of the laboratory test records.

In-Situ Testing – Dam Foundation

It is assumed that the 1975 drill hole sampling was done using the Standard Penetration Test (SPT) Method, although no expedite statement was found (Appendix C4). However, the testholes appear to have been logged by Interior Testing Services Ltd. and the blow count is shown in a format typical of the SPT test. In particular, the sum of the blows between ½ ft. to 1 ½ ft. depth increment is taken as SPT N value. The SPT values range from 15 to > 100 blows per foot (0.3m), consistent with compact to very dense soil.

Appendix C3

Test Pit Logs, 1974

Test Pit # 1 to # 10, Inclusive plus Pit Location Plan

LOG OF TEST PITS

For location of test pits see drawing 118-3.

Test Pit No. 1

Ground surface El. 2066.

0' - 0.5' Brown topsoil, sandy, with little organic matter, a few roots, dry, loose

0.5' - 7' Light brown well graded silty sand-gravel with cobbles 6" max. size, dry, compact, increasing to dense towards the bottom of pit. (Sample No. 1).

Test Pit No. 2

Ground surface El. 2060

0' - 0.5' Topsoil, loose

0.5' - 7' Light brown well graded silty sand-gravel with cobbles, dry, compact. (Sample No. 2).

Test Pit No. 3

In Creek bed, surface El. 2041.5.

0' - 2' Creek wash gravel with a few cobbles, wet

2' - 6' Grey and brown sand-silt, soft to compact (Sample No. 3)

6' - 7' Cobbles and rocks subangular mixed with sand and clayey silt, hard to dig (probably close to bedrock).

Test Pit No. 4

Ground Surface El. 2048

0' - 1' Topsoil

1' - 13' Light brown, well graded mixture of clayey silt-sand-gravel, with a few cobbles and rocks, dry, compact to dense. (Sample No. 4)

Test Pit No. 5

Ground Surface El. 2035

0' - 2' Creek wash gravel

3' Bedrock

Test Pit No. 6

Ground surface El. 2047

0' - 2' Sand and gravel, dry

2' - 12' Silty fine sand, colour alternating between blue-grey and light brown in well distinguished layers and seams, loose to compact. Water from nearby creek flowing into pit when depth of 12' reached, water level rising to 2' below surface. (Sample No. 6)

12' - 14' Sandy gravel, a few cobbles

Test Pit No. 7

Ground surface El. 2050

0' - 0.5' Topsoil, sandy

0.5'-12' Light brown, well graded silty sand-gravel, with 3" size angular cobbles, dry, compact

12' - 17' Light brown silty sand with round to subangular cobbles 8" max. size, dry, becoming damp near bottom of pit, water seeping into pit, compact to dense with increasing depth. (Sample No. 7)

Test Pit No. 8

Ground surface El. 2047

0' - 0.5' Topsoil

0.5' - 8' Cobbles, angular to subangular, to 1' max. size, embedded in light-brown sand and gravel poorly graded, dry; hard to dig

8' - 15' Light brown fine sand, moist, with a few subangular cobbles; near bottom of pit large boulders, less cobbles, wet, very hard to dig.

Test Pit No. 8

Ground surface El. 2068

0' - 0.5' Topsoil, few large boulders on surface

0.5 - 9' Cobbles, angular to subangular, 12" max. size, with light brown silty sand not filling all voids of the cobbles, very dry, very stoney near bottom of pit, very hard to dig.

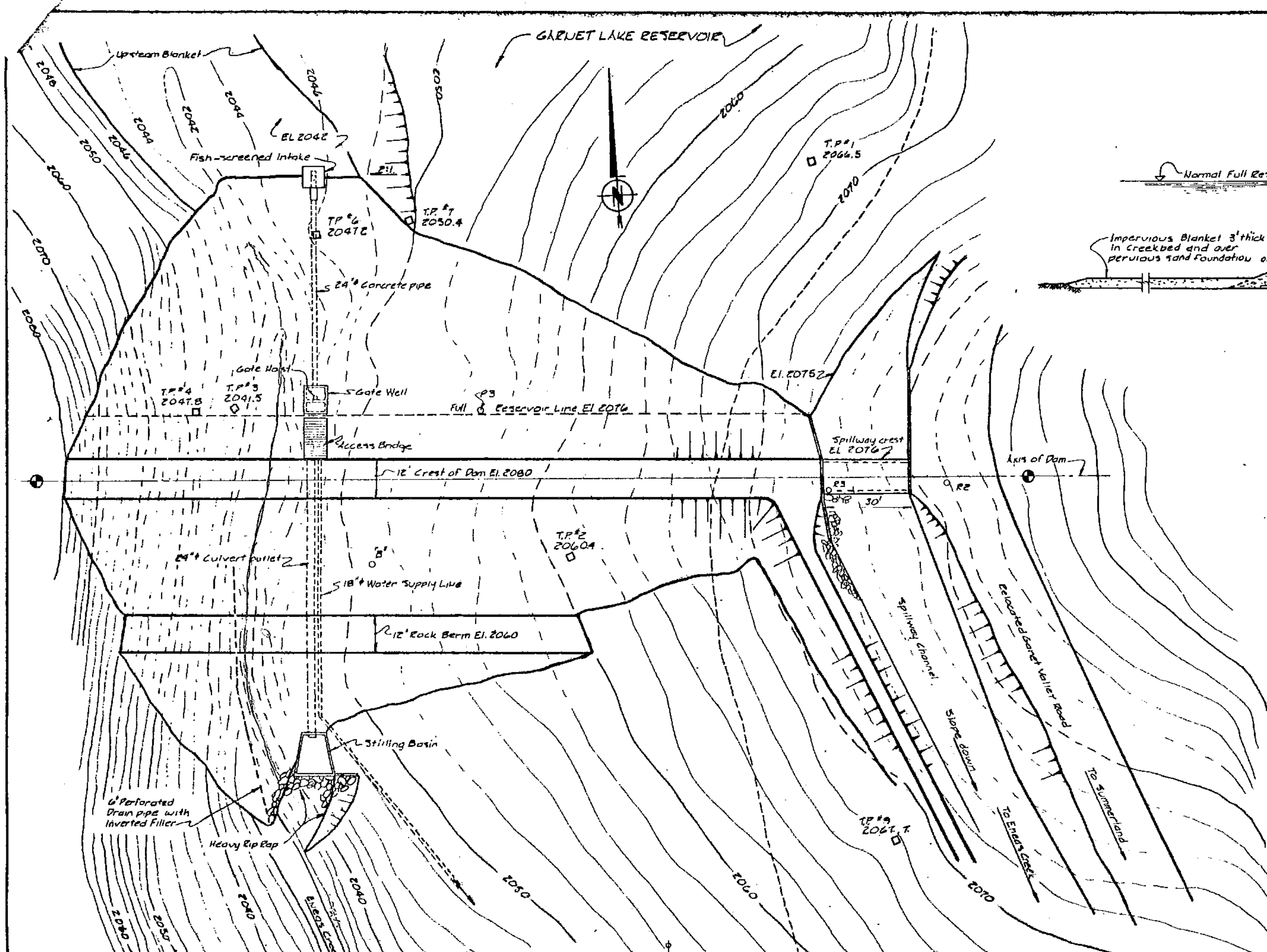
Test Pit No. 10

Bulldozer cut on terrace above right abutment

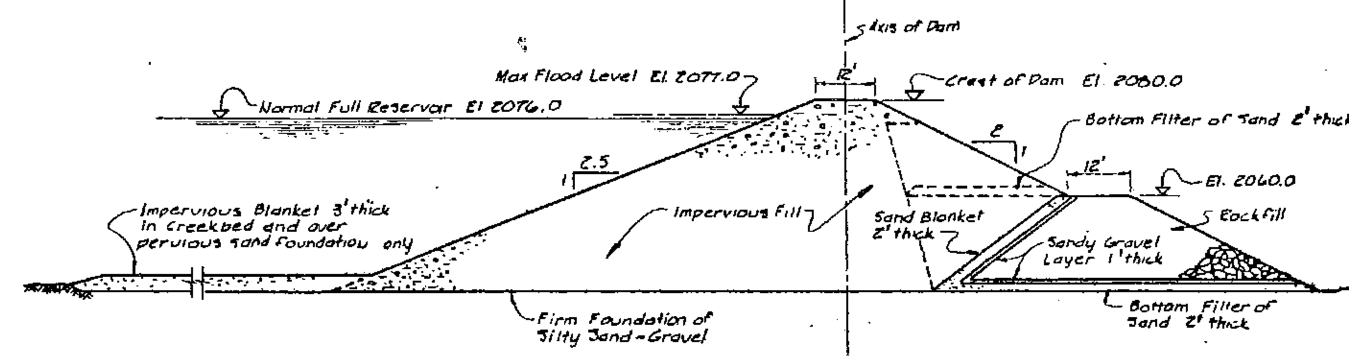
0' - 0.5' Topsoil

0.5'-1.5' Brown silty gravel, subangular

1.5'- 6' Brown to grey, clean sand and gravel, very dry (Sample No. 10, taken at depth 4' - 6')



PLAN
Scale 1" = 20'



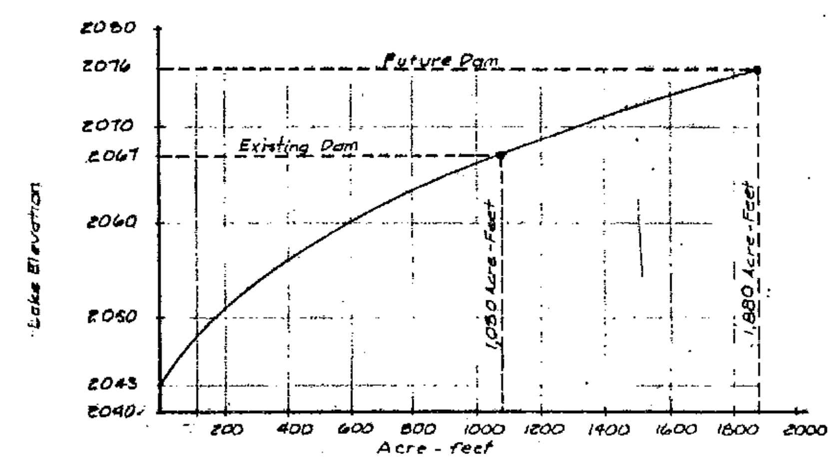
TYPICAL SECTION OF DAM
Scale 1" = 20'

Legend

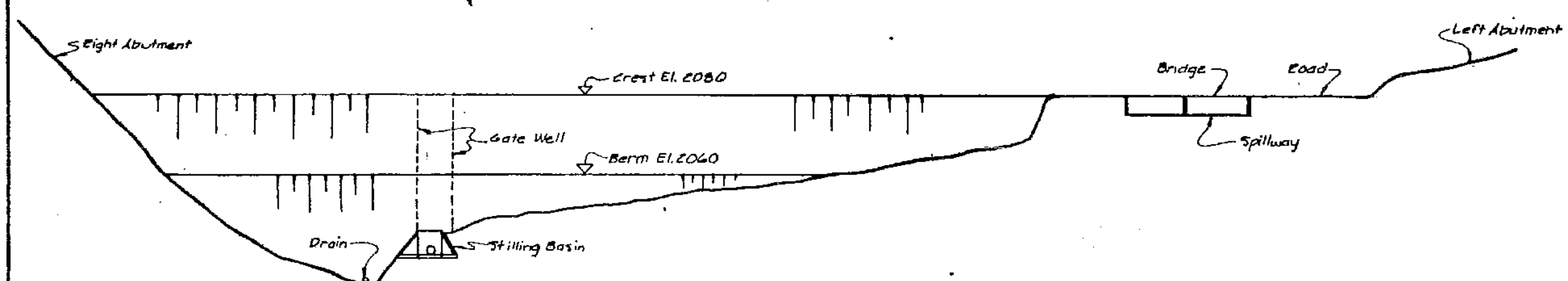
□ T.P. #2 2060.4 — Test Pit No 2, Ground El 2060.4 (Excavated in October 1974)

○ P3 — Survey Pegs

Note:
Ground contours and location of Test Pits from survey by Water Investigation Branch – ARDA Division (Dwg # 21, Nov. 18, 1974.)



STORAGE CURVE
(Information from Survey by Water Investigation Branch)



DOWNSTREAM ELEVATION
Scale 1" = 20'

THE CORPORATION OF THE DIST. OF SUMMERLAND

FUTURE
GARNET LAKE DAM
EMBANKMENT

H. FELLHAUER ENGINEERING CONSULTANT

DATE	NUMBER
NOVEMBER 1974	118-3

Appendix C4

Drill Hole Logs, 1975

Drill Hole # 2 and # 3

A 1

DRILLED TEST HOLES AT GARNET VALLEY DAM SITE, SUMMERLAND, B. C.

TEST HOLE NO. 2 - 23-4-75

0	- Creek bottom.
0 - 5'	- Gravel. Sandy. Silt layers.
5 - 8'	- Sand. Silty.
8 - 12'	- Gravel. Pea gravel. Some silt.
12 - 16'	- Silt. Grey. Clayey. Blow count 3-5-10 (15 - 16.5 feet).
16 - 35'	- Sand. Fine. Grey. Silty
	- No Recovery - pushing a rock in penetration test. Blow count 12-30-52 (20 - 21.5 feet)
	- 23 - 23.5' - Clay. Blue silty layer. Blow count 4-9-15 (25 - 26.5 feet).
	- Thin layers of brown clay. Silty. Blow count 7-13-19 (30 - 31.5 feet).
35 - 38'	- Silt. Grey. Clayey. Blow count 3-7-10 (35 - 36.5 feet).
38 - 40'	- Gravel. Blow count 17-42-61 (40 - 41.5 feet).
40 - 45'	- Gravel. Sand layers.
45 - 55'	- Gravel. Coarse. Lost circulation.
	- Unable to drill further.
	- Lost approximately 90 gals. of mud and water at approximately 15 gals/min.

TEST HOLE NO. 3 - 23-4-75

0 - 0.5'	- Topsoil.
0.5'	- Silt. Light brown. Sandy with cobbles and Boulders.
- 15'	- Blow count 32-65-107 (15 to 16.5 feet)
- 20'	- 25% Recovery. Gravel. Blow count 26-69 (20 - 21 feet).
20 - 28'	- Gravel. Coarse. Very hard drilling.
28 - 30'	- Sand. Coarse. Dense.
30 - 36'	- Gravel. Coarse. Dense.
36 - 40'	- Sand. Grey. Dense.
	- Hole caved. Unable to sample.

Appendix C5

Summary of Laboratory Testing (1970's)

Table C5-1

Table C5 - Garnet Dam Laboratory Tests Records (1976) Summary

VAN-00209167-A0
2013 January

Material	Sample #	Sieve % Passing				Limits			Std. Proctor WC		Comment
		37	4.75	2	0.075	LL	PL	PI	Opt	%	
Impervious Fill	Borrow	96	61	47	14	21	16	5	132.8	7.9	Silty Sand
	101	100	66	56	16						
	106					13	12	1			
	108	100	92	84	51	18	14	4			18% finer than 0.005mm
Triaxial Sample	-	90	43	34	12						Sand & Gravel, some silt
Triaxial Test CU	-	Minus #4	100	77	27						Silty Sand, γ = 119-121 pcf, c=320psf, phi=28 degrees
Spec. Imp. Fill (1974)		92	62	46	17						
Insitu Fdtn	107										
Sandy Gravel	A13	83	36	29	2						
Spec Add#1	102	100	52	33	1						
Foundation, 1974	TP2, No. 2	96	50	40	9						Silty Sand & Gravel, Cobbles
	TP3, No. 3A	100	86	82	38	30	15	15			Creek at 4ft - Silt and Sand, Shelby tube
	TP4, No. 4	89	51	47	19				129.1	7.3	Clayey, Silt, Sand and Gravel, Cobbles
	TP6, No. 6	100	100	98	51						Silty Fine Sand
	TP7, No. 7	100	100	92	46						Silty Sand, Cobbles
Insitu Fdtn, 1975	107					16	12	4			

Appendix D

Site Visit

exp Checklist 2012 September 26

exp Services Inc.
DAM INSPECTION CHECKLIST

Our File: VAN-00209167-A0

GARNET LAKE DAM 2012 September 26

OWNER District of Summerland DATE 2012 September 26

DESCRIPTION Zoned Earthfill,

DAM ______ USE Water Storage /Water Works

LENGTH 61m CREST WIDTH 5.2 m HEIGHT 12m

LICENSEE District of Summerland

CWL 16415, 16416 FILE NO. 075851 DAM NO. ______

DATE INSPECTED 2012 September 26 LAST INSPECTED ______

TYPE OFINSPECTION: FORMAL ____ INCIDENT-RELATED ______ FOLLOW-UP _

WATERSHED AND RESERVOIR CONDITIONS

1.	Saturated	______	6.	Outlet Release	______ m^3/s
2.	Wet	______	7.	Spillway Overflow	0 m^3/s
3.	Dry	✓	8.	Reservoir Debris	H M L
4.	Freeboard	2.9m (9.4ft)	9.	Reservoir Bank Stability	✓
5.	Water Level	29.5ft ± Geodetic (approx.)			

EMBANKMENTS - EARTH			**OUTLET WORKS**		
10.	Growth	✓	30.	Gate	Fish Gate Open
11.	Upstream Slope	X Beaching	31.	Sluice	600mm Dia.
12.	Crest	✓	32.	Submerged	✓
13.	Downstream Slope	✓ Rip Rap	33.	Walls	---
14.	Downstream Toe	✓	34.	Stilling Basin	✓
15.	Rip Rap	✓	35.	Toe Drain	X Seepage
16.	Seepage	X Foundation drains	36.	Channel	✓
17.	Erosion	None Seen	37.	Weir	✓
18.	Sloughing	None Seen	38.	Erosion	None Seen
19.	Boils	None Seen	39.	Seepage	X

GATE WORKS – SLUICE AND WATERWORKS			**SPILLWAY**		
20.	Accessibility	✓ Wallway	40.	Boom	✓ ---
21.	Wheel	X	41.	Entrance	✓ 40ft wide
22.	Threads	✓	42.	Walls	✓ Concrete
23.	Pedestal	✓	43.	Sill	✓ Concrete
24.	Stem Guides	X	44.	Apron	✓ Gravel Apron
25.	Stem	X	45.	Channel	X
26.	Gate	Sluice Gate Closed	46.	Growth	✓
27.	Grill	---	47.	Erosion	✓
28.	Boom	✓	48.	Seepage	None Seen
29.	Gauge	✓	49.	Debris	✓

✓ Inspected, satisfactory
X Inspected, requiring attention (see remarks page(s))

INSPECTED WITH Shawn Hughes (DOS) SIGNED Don Sargent, P.Eng.

11.	Beaching evident on upstream slope of dam, see photo 3.
16, 35, 39	Seepage Observations: • Six inch diameter drain pipes, left and right sides of outlet stilling basin (photos 7 and 8). • A toe seepage collection training works consist of concrete wall around downstream right side of outlet stilling basin, complete with V-notch weir (photos 5 and 6).
24, 25	Reinforced concrete Gatewell (Gate Tower); metal plate cover was locked. Arrangement consists of wet well and drywell, complete with 4 inch diameter drain pipe. Grate cover over drywell. Sluice gateworks mounted on Gate Tower. Waterworks consist of pressurized pipe to downstream metering chambers.
45	Spillway channel comprised of concrete entrance (40ft wide) and sloping channel (25ft wide) above a rip rap-lined channel. The dam access road crosses the rip rap channel just above return to creek (Photo 14).

Photo 1 – View from right bank showing upstream area, including upstream slope, spillway entrance on left bank.

Photo 2 – View from right bank showing downstream dam slope; benches.

Photo 3 – View of upstream slope, above waterline, showing beaching on slope, Gate Tower in background.

Photo 4 – View looking upstream showing outlet stilling basin. Note water release for fish purposes (fish gate).

Photo 5 – View looking upstream showing concrete wall for toe seepage collection works in foreground; outlet stilling basin, timber stairs in background.

Photo 6 –View looking upstream showing V-notch weir in toe seepage collection works. A six inch diameter drain pipe outlet is situated in the background.

Photo 7 – View looking at right side of creek bank, showing six inch diameter drain pipe outlet.

Photo 8 – View looking at left side of creek bank below outlet, showing six inch diameter drain pipe outlet, near timber stairs.

Photo 9 – View looking from left side looking upstream showing right abutment and downstream dam slope.

Photo 10 – View looking upstream showing dam and spillway interface.

Photo 11 – View looking downstream showing spillway apron and channel.

Photo 12 – View looking downstream showing concrete spillway sill and channel lining.

Photo 13 – View looking downstream showing spillway rip rap channel.

Photo 14 – View looking downstream slowing access road / culvert crossing spillway channel.

Appendix E

Site Characteristics

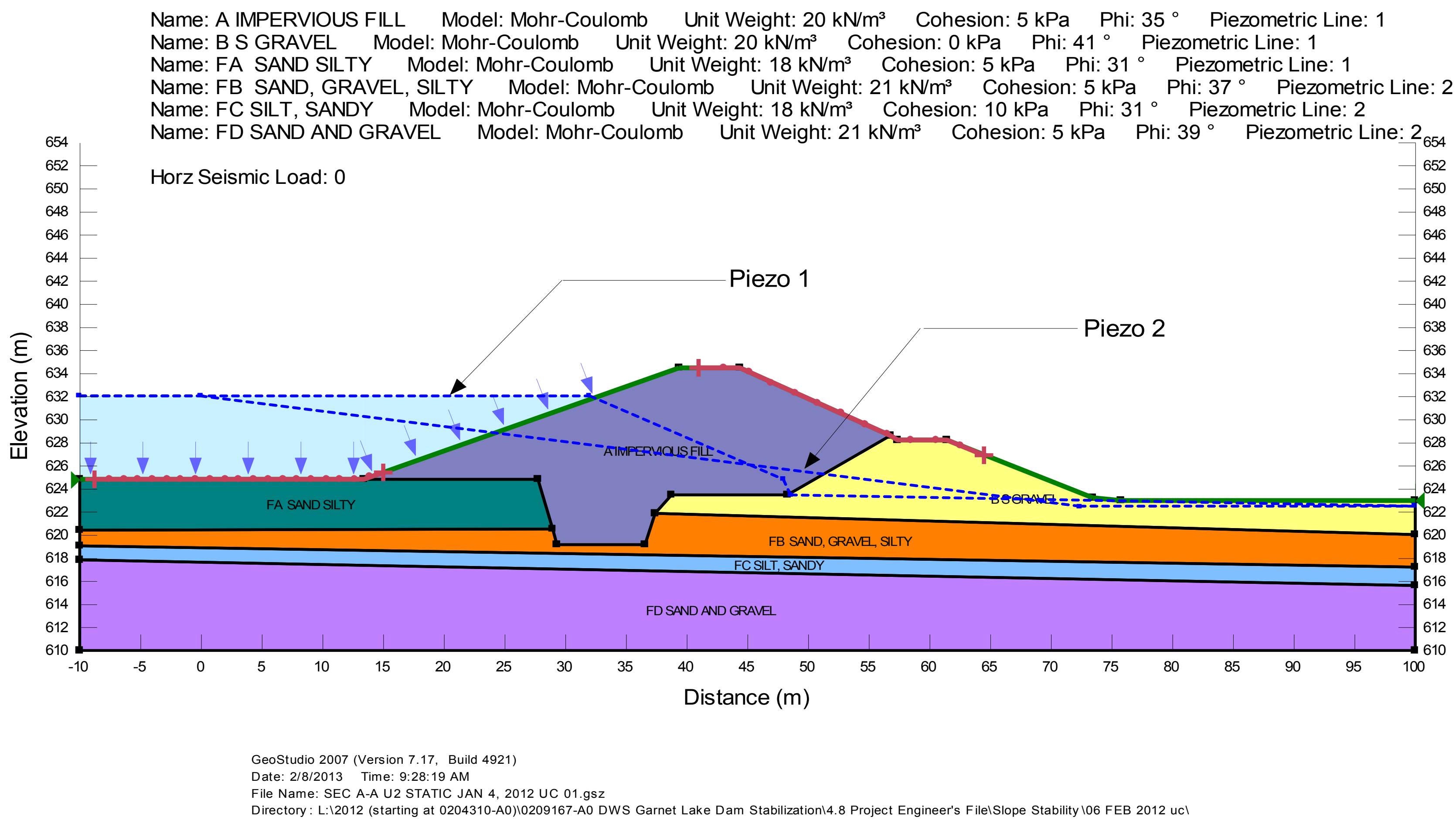
Selected Dam Sections – E1

Seismic Hazard Calculations – E2

Appendix E1

Selected Dam Sections

Sections AA and BB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Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FA SAND SILTY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 31 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 2
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2
Horz Seismic Load: 0
Piezo 1
Piezo 2
A IMPERVIOUS FILL
B S GRAVEL
FA SAND SILTY
FB SAND, GRAVEL, SILTY
FC SILT, SANDY
FD SAND AND GRAVEL
Elevation (m)
Distance (m)
GeoStudio 2007 (Version 7.17, Build 4921)
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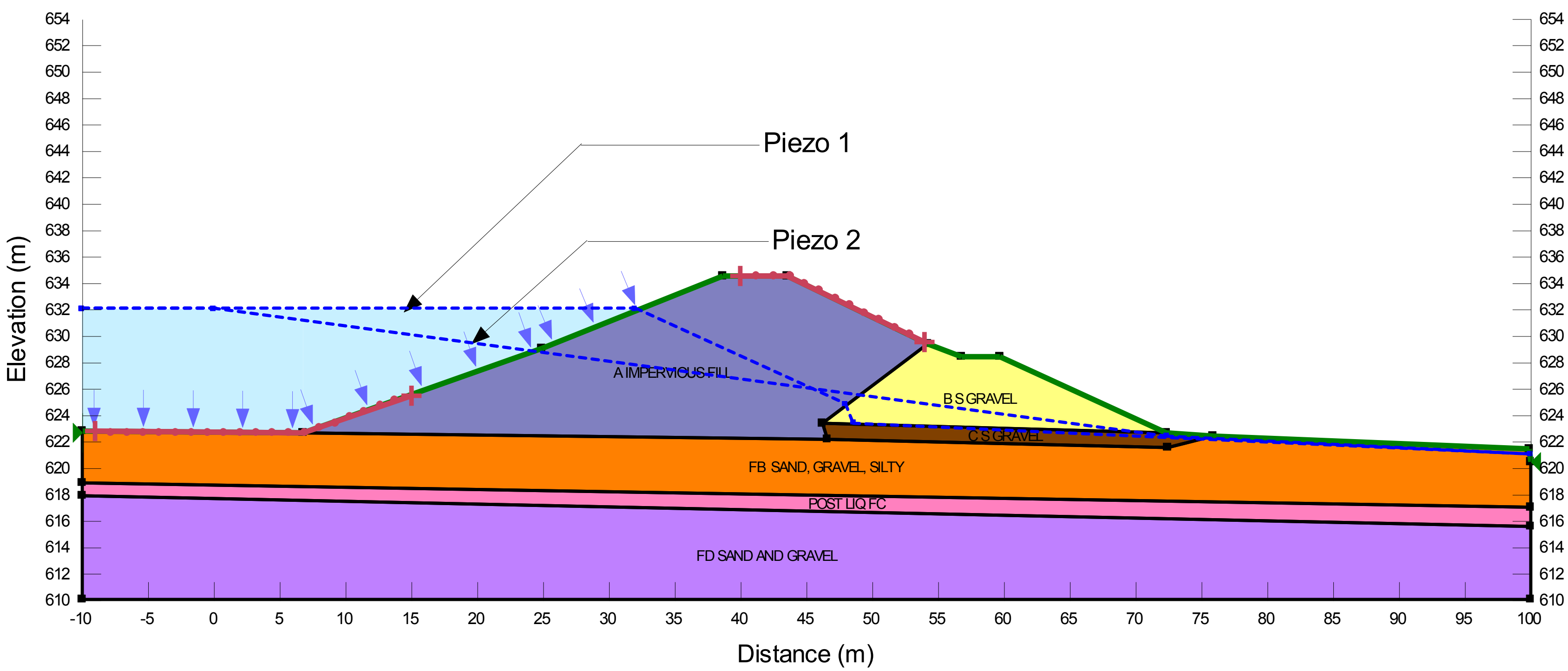
Directory: L:\2012 (starting at 0204310-A0)\0209167-A0 DWS Garnet Lake Dam Stabilization\4.8 Project Engineer's File\Slope Stability\06 FEB 2012 uc\

Section AA

Name: A IMPERVIOUS FILL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 35 ° Piezometric Line: 1
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 1
Name: POST LIQ FC Model: S=f(overburden) Unit Weight: 18 kN/m³ Tau/Sigma Ratio: 0.1 Minimum Strength: 7 Piezometric Line: 1

Horz Seismic Load: 0



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Section BB

Appendix E2

Seismic Hazard Calculations

Garnet Lake Dam, Summerland, BC

Revelstoke, BC

2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: , exp Services Inc — October 01, 2012

Site Coordinates: 49.685 North 119.7745 West

User File Reference: Garnet Lake Dam

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.282	**0.182**	**0.114**	**0.066**	**0.139**

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.065	0.141	0.195
Sa(0.5)	0.049	0.099	0.131
Sa(1.0)	0.029	0.061	0.082
Sa(2.0)	0.017	0.035	0.048
PGA	0.036	0.074	0.099

References

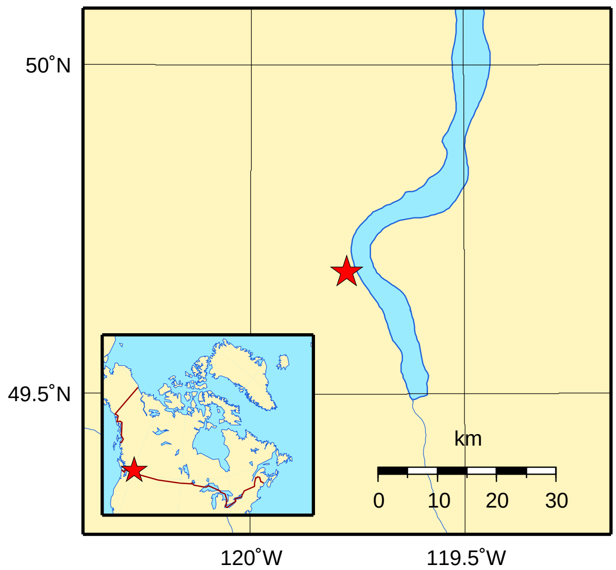
National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



2010 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: , exp Services Inc October 01, 2012

Site Coordinates: 51 North 118.197 West

User File Reference: Revelstoke

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.271	**0.162**	**0.080**	**0.045**	**0.135**

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2010 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.053	0.128	0.182
Sa(0.5)	0.033	0.077	0.109
Sa(1.0)	0.016	0.037	0.053
Sa(2.0)	0.009	0.021	0.030
PGA	0.031	0.069	0.095

References

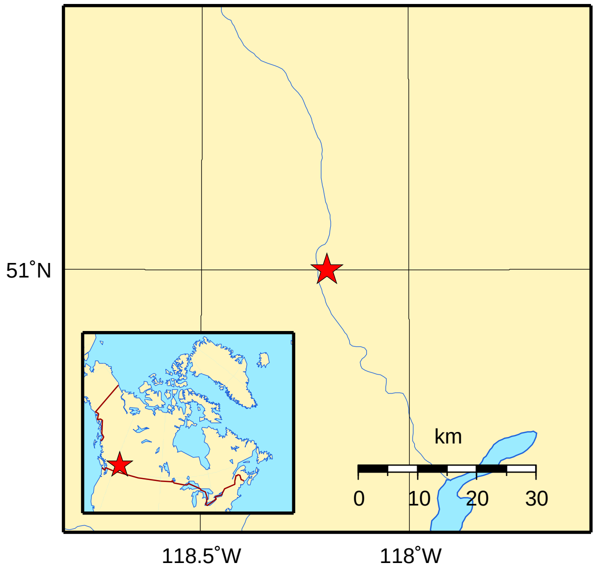
National Building Code of Canada 2010 NRCC no. 53301; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2010, Structural Commentaries NRCC no. 53543 (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Maps and grid values to be used with the 2010 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information

Aussi disponible en français



Appendix F

Slope Stability

Long-term Stability – Selected Sections – F1

Pseudo-static (Earthquake) Stability – Selected Sections – F2

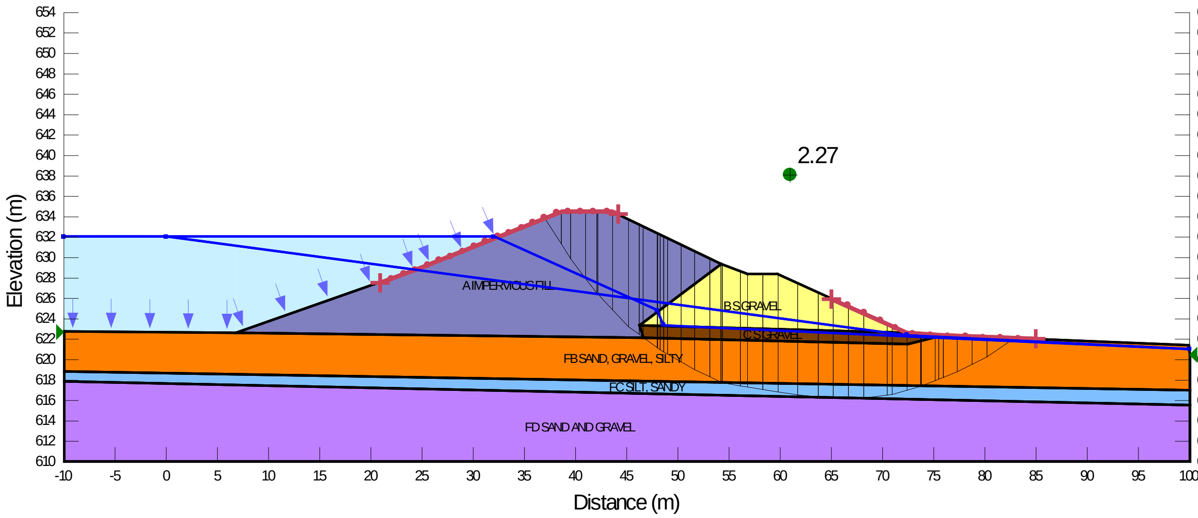
Post-Earthquake Stability – Selected Sections – F3

Appendix F1

Long-term Stability – Selected Sections

Name: A IMPERVIOUS FILL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 35 ° Piezometric Line: 1
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2

Horz Seismic Load: 0



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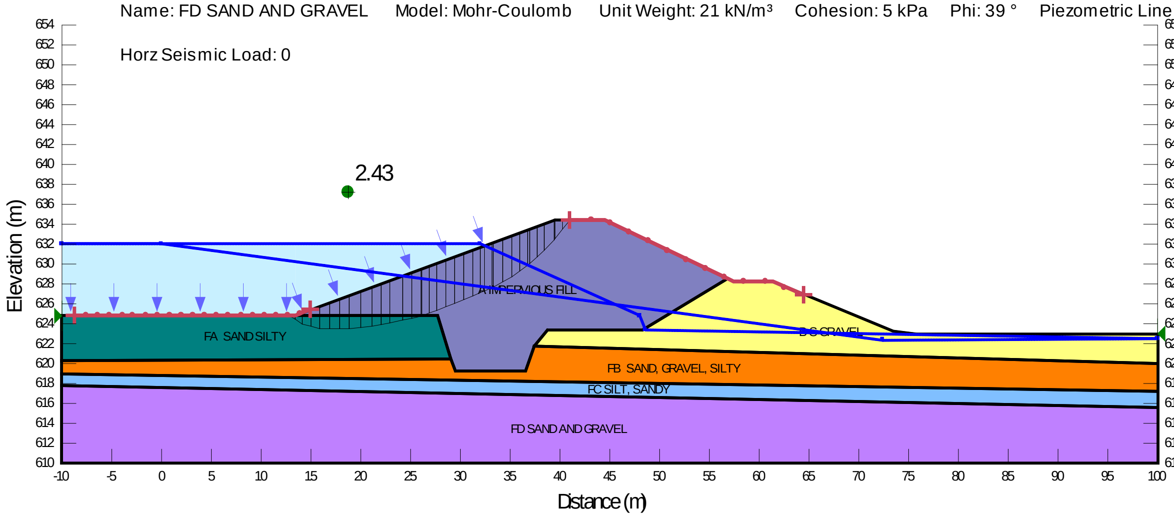
Name: FA SAND SILTY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 31 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 2
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2

Horz Seismic Load: 0



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Appendix F2

Pseudo-static (Earthquake) Stability – Selected Sections

Name: A IMPERVIOUS FILL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 35 ° Piezometric Line: 1
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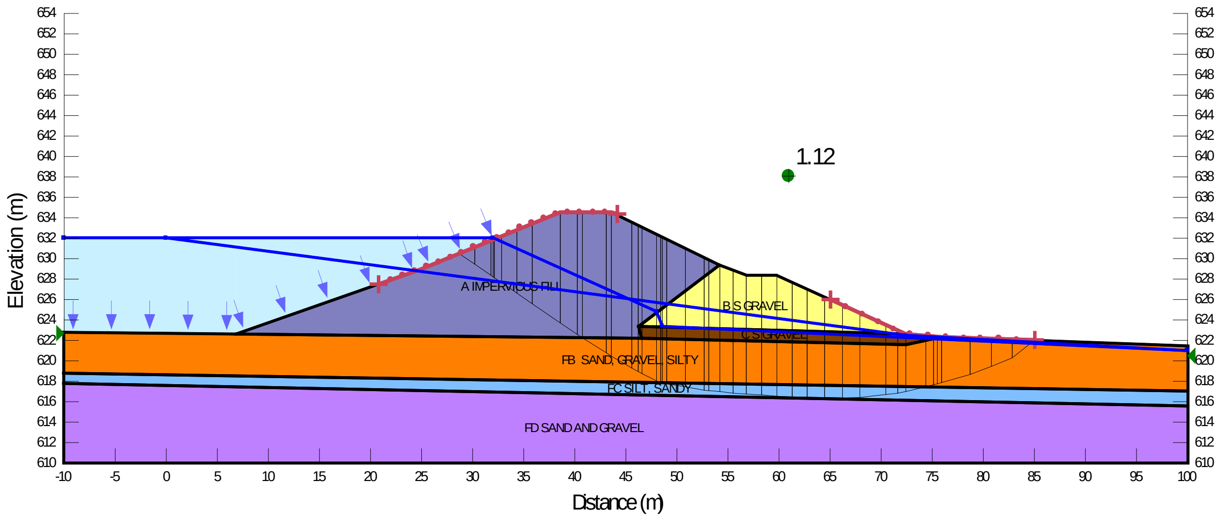
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2

Horz Seismic Load: 0.286



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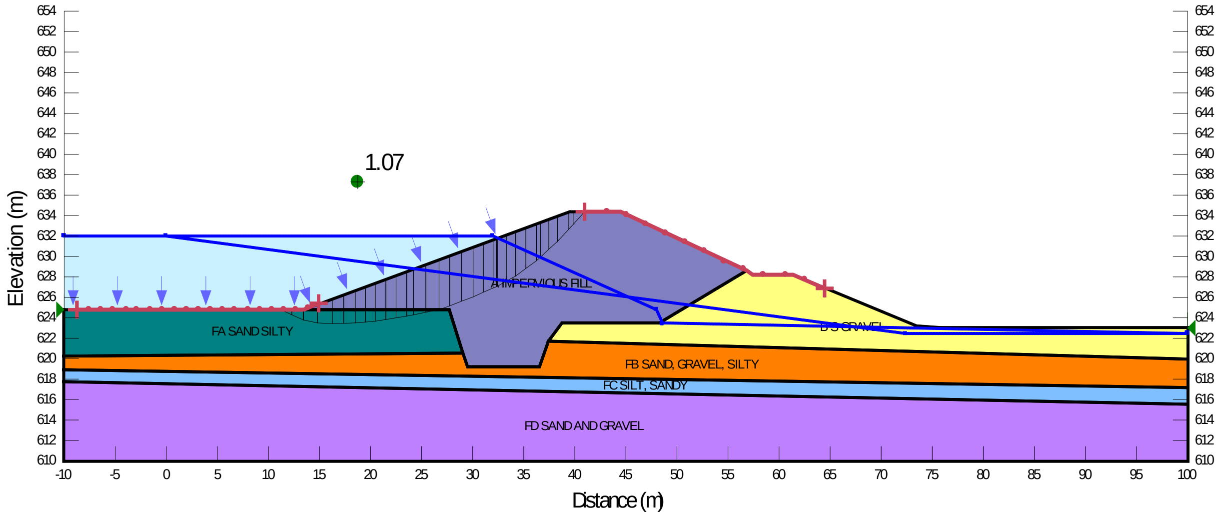
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FA SAND SILTY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 31 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 2
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2

Horz Seismic Load: 0.236



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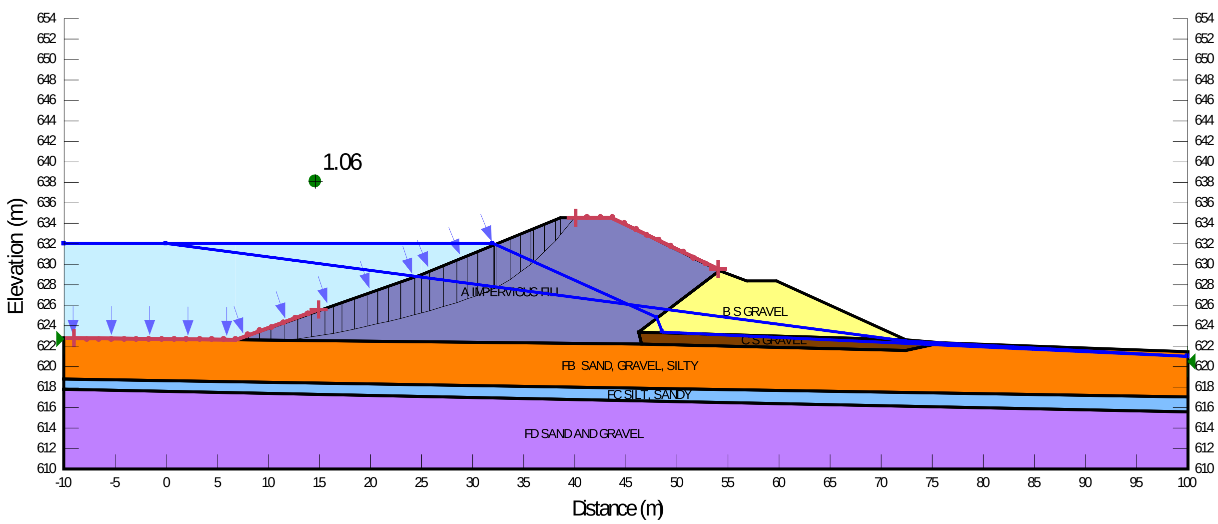
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FC SILT, SANDY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 10 kPa Phi: 31 ° Piezometric Line: 1
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 1

Horz Seismic Load: 0.236



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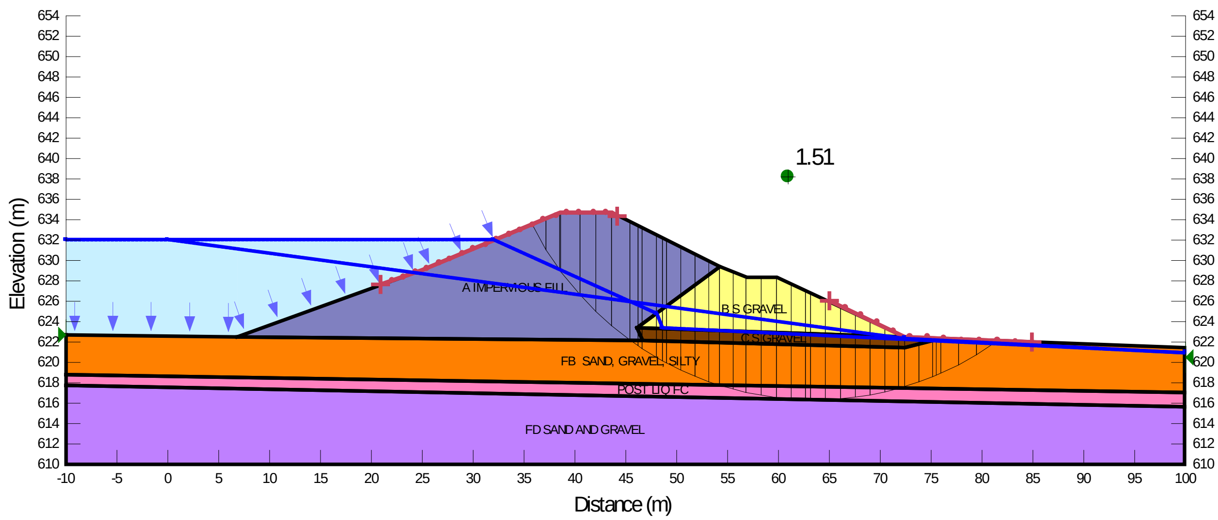
Appendix F3

Post-Earthquake Stability – Selected Sections

Name: A IMPERVIOUS FILL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 5 kPa Phi: 35 ° Piezometric Line: 1
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: C S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 1
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2
Name: POST LIQ FC Model: S=f(overburden) Unit Weight: 18 kN/m³ Tau/Sigma Ratio: 0.1 Minimum Strength: 14 Piezometric Line

Horz Seismic Load: 0



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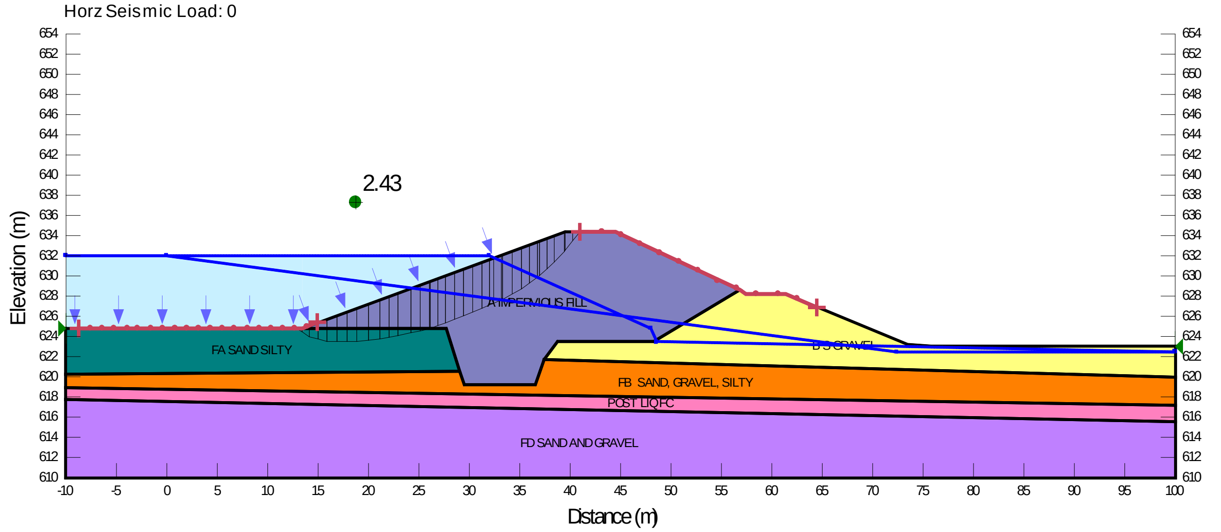
Name: B S GRAVEL Model: Mohr-Coulomb Unit Weight: 20 kN/m³ Cohesion: 0 kPa Phi: 41 ° Piezometric Line: 1
Name: FA SAND SILTY Model: Mohr-Coulomb Unit Weight: 18 kN/m³ Cohesion: 5 kPa Phi: 31 ° Piezometric Line: 1
Name: FB SAND, GRAVEL, SILTY Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 37 ° Piezometric Line: 2
Name: FD SAND AND GRAVEL Model: Mohr-Coulomb Unit Weight: 21 kN/m³ Cohesion: 5 kPa Phi: 39 ° Piezometric Line: 2
Name: POST LIQ FC Model: S=f(overburden) Unit Weight: 18 kN/m³ Tau/Sigma Ratio: 0.1 Minimum Strength: 7 Piezometric Line: 2

Horz Seismic Load: 0



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