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Dam Safety Review and Risk Assessment of Stocking Lake Dam

Presented To:

CVRD

Dated: November 2018

Ecora File No.: GK-18-020-CVD

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Version Control and Revision History

Version	Date	Prepared By	Reviewed By	Notes/Revisions
0	2018-11-28	MJL/AG/CE	AGC	Issued for Use

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Ecora's General Conditions are provided in Appendix K of this report.

Executive Summary

The Cowichan Valley Regional District (CVRD) engaged Ecora Engineering & Resource Group Ltd. (Ecora) to undertake a comprehensive Dam Safety Review (DSR) and risk assessment of Stocking Lake Dam located west of Ladysmith, BC. A summary of key dam and reservoir attributes are included in Table i below:

Table i Summary of Key Dam Attributes

Stocking Lake Dam:		
Provincial Dam File Number:	D720127-00	
Stream Name:	Stocking Creek	
Current Consequences Classification:	Significant (Recommended: High)	
Dam Type:	Earth-fill Embankment	
Location:	Latitude: 48° 57’ 25” N	Longitude: 123° 49’ 08” W
Height:	3.3 m	
Length:	30 m	
Crest Width:	8.5 m	
Spillway Capacity:	9.3 m^3/s	
Live Storage Capacity:	1,074,400	
Potential Storage Volume:	1,379,000	
Drainage Area:	1.90 km^2	
Peak of Inflow Design Flood (IDF):	14.5 m^3/s (Significant)	20.3 m^3/s (High)
Peak Outflow During IDF	2.7 m^3/s (Significant)	3.8 m^3/s (High)

The DSR was undertaken in general accordance with the requirements of the BC Water Sustainability Act including all amendments up to BC Reg. 301/2016 (December 7, 2016), the BC Dam Safety Regulation BC Reg. 40/2016 (February 29, 2016), The Association of Professional Engineers and Geoscientists of BC (APEGBC) Professional Practice Guidelines – Legislated Dam Safety Reviews in BC V3.0 (October 2016), and the Canadian Dam Association (CDA) Dam Safety Guidelines (DSG) 2007 (2013 Edition).

The scope of the comprehensive DSR included the following tasks:

- Background review;
- Site reconnaissance;
- Geotechnical investigation of the dam;
- Review of consequence classification;
- Geotechnical assessment, including embankment stability and seepage;
- Hydrotechnical analysis including dam break analysis, flood routing and hydraulics;
- Review of any existing Operation, Maintenance & Surveillance Manual, Dam Emergency Plans (Emergency Response Plan and/or Emergency Preparedness Plan) and/or public safety management strategies;
- Risk assessment as per the National Disaster Mitigation Program (NDMP) framework;
- Assessment of compliance with CDA design criteria; and,
- Development of conclusions and recommendations.

A summary of the hydrotechnical & geotechnical analyses undertaken within the scope of the DSR are summarized Table ii below.

Table ii **Summary of Results from Engineering Analyses**

Does the dam meet the applicable design criteria?	Yes/No	Comments
Is the current consequences classification considered appropriate for this dam in accordance with the BC Dam Safety Regulation, BC Reg. 40/2016?	No	See Section 8
Does the dam meet CDA static stability requirements?	No	See Section 10.5
Does the dam meet CDA pseudo-static stability requirements?	No	See Section 10.5
Does the dam meet CDA post-earthquake stability requirements?	Yes	See Section 10.5
Does the strength and/or characteristics of the dam materials and/or it's foundation provide sufficient resistance to liquefaction or cyclic softening during seismic (cyclic) loading due to application of the EDGM?	No	See Section 10.4
Is sufficient freeboard maintained following post-earthquake deformation?	No	See Section 10.6
Does the design of the dam and/or characteristics of the dam materials and/or it's foundations provide sufficient resistance to and/or control of seepage to prevent internal erosion?	No	See Section 10.7
Does the spillway have sufficient hydraulic capacity to safely pass the Inflow Design Flood (IDF)?	Yes	See Section 11.5
Does the dam meet CDA freeboard requirements including the effects of wind and wave action?	Yes	See Section 11.5

Based on the results of the investigation, analyses and assessment of the dam, a number of observations, conclusions and recommendations were developed as summarized in Table ii below. Priorities (Low, Medium, High or Very High) are given in parentheses. Low, Medium, High and Very High priority recommendations should be addressed within 5, 3, 1 and 0.5 year(s) respectively.

Table iii Dam Safety Review of Stocking Lake Dam — Observations, Conclusions and Recommendations

Task	Observations & Conclusions	Recommendations
Background Review	▪ The dam was originally constructed in 1902 and last modified in 1966. ▪ No obvious signs of historical or current slope instability of the reservoir sides slopes were observed in the review of available aerial photographs.	▪ There are no recommendations in this area of review.
Site Reconnaissance	▪ The dam access road is in poor condition and is only suitable for four-wheel drive vehicles. ▪ Seepage has been observed exiting the downstream face of the dam and a sinkhole is present near the right abutment of the dam. ▪ The saturated backfill above the water main trench downstream of the dam suggests that preferential seepage is occurring through the dam along the low level outlet conduit. ▪ The log boom is constructed out of encapsulated foam which has been reported by the CVRD to be ineffective at preventing debris from entering the spillway during storm events.	▪ The condition of the dam access road should be improved in accordance with the BC Ministry of Forests, Lands, Natural Resource Operation & Rural Development (MFLNRORD) Engineering Manual (2018) minimum specifications or similar standard to allow two-wheel drive vehicle access in the event of an emergency, or alternatively all emergency responders should be advised that they will require four- wheel drive vehicles (High). ▪ The preferential seepage of water through the dam along the low level outlet should be addressed during the remediation or replacement of the dam (High). ▪ The current log boom should be replaced with one that is effective at capturing debris under both normal and storm conditions (High). ▪ The importance of regular monitoring of the seepage clarity and rate of seepage when the risk of piping exists is underlined by Foster et. Al (2000b) study. Weekly documented monitoring of the “sinkhole” present near the right abutment of the dam noting observations of any leakage and turbidity of the water along the toe of the dam should be undertaken during site surveillance activities until remedial works have been constructed. This should include estimating rates and clarity of seepage, along with taking photographs as comparisons may need to be made between future and past conditions (Very High). ▪ A filter buttress should be design and placed over the “sinkhole” present near the right abutment until remediation of the existing dam or the construction of a new dam has been completed (High).
Consequence Classification	▪ The dam breach inundation mapping indicates that a total area of approximately 1.05 km2 would be flooded in the event of a dam breach, potentially impacting S Watts Rd, Highway 1, and water mains servicing Saltair and the Town of Ladysmith. ▪ Dam breach analysis and inundation mapping results confirmed that Stocking Lake Dam should have a "High" consequence classification. The CDA guidelines recommend an Inflow Design Flood (IDF) for a "High" consequence dam of ⅓ of the way between a 1,000-year flood and a Probable Maximum Flood (PMF).	▪ Based on the estimated potential loss of life and economic losses within the dam break flood inundation area it is recommended that the consequence classification of Stocking Lake Dam be increased from “Significant” to "High". However, any decision to modify the consequence classification rating must be confirmed by the BC MFLNRORD Dam Safety Section (Very High).
Failure Mode Assessment	▪ The plausible failure modes of the dam are; overtopping as the spillway may become blocked with debris, embankment failure due to earthquake loading and internal erosion through the embankment, its foundation or along the low level outlet conduit.	▪ There are no recommendations in this area of review.
Geotechnical Assessment	▪ The boreholes advanced as part of the geotechnical investigation of the dam indicated that the dam is founded on till-like material and bedrock. ▪ Results of the static stability analysis indicated that the embankment meets CDA criteria for normal loading conditions for any potential slip surfaces that would impact the dam freeboard. ▪ Preliminary post seismic liquefaction deformation analyses of the dam indicate that sufficient freeboard would be lost resulting in an overtopping of the dam due to a 1 in 475-year earthquake corresponding to a NDMP likelihood rating of 3. ▪ A probabilistic piping risk assessment was conducted using the UNSW method, which resulted in a calculated probability of piping failure of 1.4 x 10-2 (1 in 70 years) corresponding to a NDMP likelihood rating of 3.	▪ CVRD should commission a design study to address the major deficiencies in the Stocking Lake Dam, namely its susceptibility to liquefaction under the design seismic event and its susceptibility to piping. It is envisioned this would result in a recommendation to either substantially remediate the existing dam or the construction of a new dam immediately downstream (Medium).
Hydrotechnical Assessment	▪ The peak inflow to Stocking Lake Dam during the IDF for a “High” consequence dam was determined to be 20.2 m3/s. ▪ The spillway has adequate capacity to pass the routed inflow design flood. ▪ The dam should have freeboard such that 95% of the waves do not overtop the dam crest during a 1,000-year wind event under normal lake level conditions or during a 2-year wind event under inflow design flood conditions (IDF). Stocking Lake Dam has been shown to exceed these requirements in both scenarios with freeboards of 0.73 m and 0.26 m respectively, in excess of what is required.	▪ There are no recommendations in this area of review.
Dam Safety Management	▪ An Operation, Maintenance and Surveillance Manual and a Dam Emergency Plan need to be prepared for Stocking Lake Dam.	▪ An Operation, Maintenance and Surveillance Manual and a Dam Emergency Plan need to be prepared for Stocking Lake Dam (High).
Risk Assessment	▪ Preliminary post seismic liquefaction deformation analyses of the dam indicate that sufficient freeboard would be lost resulting in an overtopping of the dam due to a 1 in 475-year earthquake corresponding to a NDMP likelihood rating of 3. ▪ A probabilistic piping risk assessment was conducted using the UNSW method, which resulted in a calculated probability of piping failure of 1.4 x 10-2 (1 in 70 years) corresponding to a NDMP likelihood rating of 3. ▪ A wind-wave analysis indicated that an event greater than the IDF coupled with extreme wind could lead to dam failure by overtopping corresponding to a NDMP likelihood rating of 1. ▪ A preliminary estimate of reconstruction costs as a result of a dam breach is between $3 million and $30 million based on the scope of the infrastructure impacted.	▪ Should the CVRD wish to proceed with a NDMP funding application to remediate or replace Stocking Lake Dam they should undertake a more detailed cost estimate of infrastructure that would be impacted in the event of a dam breach (High).

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Acronyms and Abbreviations

AEP	Annual Exceedance Probability
APEGBC	Association of Professional Engineers and Geoscientists of British Columbia
BC	British Columbia
CCIL	Canadian Council of Independent Laboratories
CDA	Canadian Dam Association
CSR	Cyclic Stress Ratio
CVRD	Cowichan Valley Regional District
DBE	Dam Breach Elevation
DEP	Dam Emergency Plan
DSG	Dam Safety Guidelines, Canadian Dam Association 2007
DSR	Dam Safety Review
EDGM	Earthquake Design Ground Motion
EPP	Emergency Preparedness Plan
ERP	Emergency Response Plan
FEA	Finite Element Analysis
FoS	Factor of Safety
FSR	Forestry Service Road
GPS	Global Positioning System
GSC	Geological Survey of Canada
HEC-HMS	Hydrologic Modeling System
ICOLD	International Congress on Large Dams
IDF	Inflow Design Flood
IDF	Intensity-Duration-Frequency
LOL	Loss of Life
LSE	Limit State Equilibrium
MASW	Multichannel Analysis of Surface Waves
mbgl	Metres Below Ground Level
MFLNRORD	Ministry of Forests, Lands, Natural Resource Operations & Rural Development
MSC	Meteorological Service of Canada
NAD	North American Datum

NBCC	National Building Code of Canada
NDMP	National Disaster Mitigation Program
OMS	Operations, Maintenance and Surveillance
PAR	Population at Risk
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PSP	Public Safety Plan
PVC	Polyvinyl Chloride
RAIT	Risk Assessment Information Template
RFP	Request for Proposal
Sa(T)	Spectral Accelerations
SCS	Soil Conservation Service
SPT	Standard Penetration Test
SWL	Standing Water Level
ToL	Town of Ladysmith
TRIM	Terrain Resource Information Management
TT-EBA	Tetra Tech EBA
UBC	University of British Columbia
UNSW	University of New South Wales
US	United States
USBR	United States Bureau of Reclamation
UTM	Universal Transverse Mercator
V_s	Shear Wave Velocity

1. Introduction

1.1 General

The Cowichan Valley Regional District (CVRD) engaged Ecora Engineering & Resource Group Ltd. (Ecora) to undertake a comprehensive Dam Safety Review (DSR) and risk assessment of the Stocking Lake Dam located south of Ladysmith, BC. A geotechnical investigation of the dam was also undertaken in conjunction with the DSR.

The dam functions as a part of both the CVRD-managed Saltair water distribution system, and the Town of Ladysmith (ToL) water supply.

This report presents the technical findings of the Stocking Lake Dam DSR and it is understood that this is the first comprehensive DSR of this facility.

A DSR is considered to be a “snapshot in time” and the observations, conclusions, and recommendations provided in this report are deemed to be valid until the next scheduled DSR, which should be conducted in 10 years (2028) for the Stocking Lake Dam. However, if conditions (e.g. loading, reservoir level, etc.) change, the results of this DSR may no longer be considered valid and/or current, and a reassessment may be required.

Stocking Lake Dam is catalogued in the BC Ministry of Forests, Lands, Natural Resource Operations & Rural Development (MFLNRORD) Dam Safety Section, Dam File No. D720127-00. The BC MFLNRORD has currently assigned the dam a consequence classification rating of “Significant” in terms of the BC Dam Safety Regulation (BC Reg. 40/2016), and the Canadian Dam Association (CDA) DSR Guidelines 2007 (2013 Edition).

The DSR was undertaken in general accordance with the requirements of the British Columbia Water Sustainability Act including all amendments up to BC Reg. 301/2016 (December 7, 2016), the BC Dam Safety Regulation BC Reg. 40/2016 (February 29, 2016), The Association of Professional Engineers and Geoscientists of BC (APEGBC) Professional Practice Guidelines – Legislated Dam Safety Reviews in BC V3.0 (October, 2016), and the Canadian Dam Association (CDA) Dam Safety Guidelines (DSG) 2007 (2013 Edition).

The objective of the British Columbia Dam Safety Regulation (BC Reg. 40/2016) is to mitigate loss of life and damage to property and the environment from a dam breach. This Regulation requires dam owners to:

- Operate the dam in a safe manner in accordance with any terms and conditions;
- Inspect their dams;
- Undertake proper maintenance;
- Report incidents and take remedial action; and,
- Undertake periodic Dam Safety Reviews.

The risk assessment of the Stocking Lake Dam was undertaken in general accordance with the National Disaster Mitigation Program (NDMP) framework

1.2 Dam Description and Access

Stocking Lake Dam is a homogeneous earthfill embankment dam situated along Stocking Creek approximately 3.8 km south of Ladysmith, BC, at Map Grid (NAD 83) co-ordinates E440052, N5422991 (Zone 10). The dam is oriented northeast to southwest and is situated in a shallow northwest to southeast trending valley. The dam impounds approximately 1,074,400 m³ of water at the spillway level, with a watershed area of approximately 1.90 km² upstream of the dam.

Stocking Lake Dam is approximately 3 m high and 30 m long according to the MFLNRORD dam database. The spillway comprises a 3.70 m wide concrete sill spillway located at the right abutment with a crest elevation of approximately 360.78 m as identified in the survey completed April 17, 2018 by Ecora. This survey also determined that the embankment height was approximately 3.3 m with the dam crest at approximately 362.08 m elevation and approximately 8.5 m wide. The upstream and downstream slopes are approximately 2.5H:1V and 2H:1V (horizontal: vertical) respectively and the upstream slope is armoured with riprap.

Stored water is discharged via an inlet controlled low level outlet pipe of unknown diameter and material, which was installed in a trench which runs perpendicular to the dam axis, exiting through the dam foundation at approximately the centre of the dam. The intake structure is located approximately 21.5 m upstream of the upstream edge of the dam crest, approximately 6.7 m below the crest elevation. There is a small concrete foundation on the dam crest which historically housed a recording device

Public access to the dam is provided from Ladysmith, BC, via Highway 1 with directions as follows. From Ladysmith travel south on Highway 1 and turn west onto Thicke Road near the southern end of Ladysmith. Turn left to stay on Thicke Road at 90 m, and right onto South Watts Road at 200 m. Keep right at 2.6 km, and turn right onto the Stocking Lake Dam access road at 4.0 km. Continue along the access road for approximately 400 m to the parking area with a locked gate, and continue north along a footpath for approximately 350 m to arrive at Stocking Lake Dam. A map showing the location of the dam and access routes is shown on Figure 1.2.

1.3 Operation, Maintenance and Surveillance

Operations at Stocking Lake Dam are regulated under several conditional and final water licences summarized in Table 1.3.

Table 1.3 Summary of Water Licences on Stocking Lake

Licence Type	Licence Number	Purpose	Quantity (m^3/year)	Licence Holder
Conditional	C005333	Waterworks: Local Provider	829662.36	ToL
Conditional	C067481	Waterworks: Local Provider	448017.17	CVRD
Conditional	C067482	Waterworks: Local Provider	477339.064	CVRD
Conditional	C067483	Stream Storage: Non-Power	123348	CVRD
Conditional	C067484	Stream Storage: Non-Power	542731.2	CVRD
Final	F051654	Irrigation: Private	3083	P. Mears & N. Hatherly
Final	F052153	Irrigation: Private	57726.864	C. Porter & D. C.

Copies of individual water licenses can be found at http://a100.gov.bc.ca/pub/wtrwhse/water_licences.input

It is understood that the day to day operation and maintenance of the Stocking Lake Dam is overseen by the Town of Ladysmith.

From discussions with the CVRD, and review of 2016 inspection reports, it is understood that surveillance (inspection) of the dam is generally undertaken weekly, weather permitting, however it is not documented. Formal annual inspections are carried out using the MFLNRORD dam site surveillance template.

2. Scope of Work

2.1 Comprehensive Dam Safety Review

Ecora's scope of work for the DSR was developed in accordance with the requirements of the CDA Dam Safety Guidelines 2007 (2013 Edition). In summary, the study included the following tasks:

- Background review;
- Site reconnaissance;
- Geotechnical investigation of the dam;
- Review of consequence classification;
- Geotechnical assessment, including embankment stability and seepage;
- Hydrotechnical analysis including dam break analysis, flood routing and hydraulics;
- Review of any existing Operation, Maintenance & Surveillance Manual;
- Review of any existing Dam Emergency Plans (Emergency Response Plan and/or Emergency Preparedness Plan);
- Review of any public safety management strategies;
- Risk assessment as per the NDMP framework;
- Assessment of compliance with CDA design criteria; and,
- Development of conclusions and recommendations.

The results of each task are detailed in the following sections.

2.2 NDMP Risk Assessment

The NDMP Risk Assessment Information Template (RAIT) provides a likelihood rating scale for a specific risk event and the likelihood that this event will occur based on conditions expected over a certain timeframe (Table 2.2). As the consequences of a dam failure (break) are the same, the event for this assessment is defined as any embankment overtopping, internal erosion, slope instability and/or earthquake induced condition(s) that cause failure of Stocking Lake Dam. The NDMP RAIT is discussed in more detail in Section 13.

Table 2.2 Likelihood Rating Scale

Likelihood Rating	Definition
5	The event is expected and may be triggered by conditions expected over a 30-year period.
4	The event is expected and may be triggered by conditions expected over a period of 30 – 50-year period
3	The event is expected and may be triggered by conditions expected over a period of 50 – 500-year period
2	The event is expected and may be triggered by conditions expected over a period of 500 – 5,000-year period
1	The event is possible and may be triggered by conditions exceeding a period of 5,000 years

3. Background Review

3.1 Sources of Information

The following sources of background information were reviewed during the DSR:

- Historic aerial photographs;
- Readily available published sources of geological data;
- Past Dam Safety Reviews, inspections and other reports; and,
- MFLNRORD Dam Safety Branch files.

A detailed list of the various documents reviewed from these sources is provided in Appendix A.

3.2 Design, Construction and Modification

It is understood that Stocking Lake Dam was initially constructed as an approximately 1.7 m high homogeneous earthfill dam in 1902 by the Wellington Colliery Company for supply to its coal washer and newly-located townsite of Ladysmith. The Village of Ladysmith acquired the Stocking Lake Dam following closure of the coal mines.

The dam was reconstructed and raised to the current height of approximately 3.3 m in 1965 using similar earthfill material to the original dam structure with the low level outlet pipe replaced and lowered in elevation by excavating a trench through the dam foundation to increase the volume of live storage. The low level outlet pipe is directly connected into the water distribution system and outflows are controlled by a valve in the water distribution line. The available design and record drawings of the dam are reproduced in Appendix B.

3.3 Historical Aerial Photographs

A review was conducted of available historical aerial photographs of the Stocking Lake area held by the Geography Department of the University of British Columbia (UBC) as summarized in Table 3.3 below.

Table 3.3 Summary of Reviewed Aerial Photographs of the Stocking Lake Dam Area

Year	Aerial Photo No.	Type
1936	BC1 NO. 22-23	Black and white
1938	AR100 NO.63	Black and white
1950	BC1053 NO. 22-23	Black and white
1957	BC2086 NO. 72-73, BC5047 NO. 77-78	Black and white
1962	BC5047 NO. 77-78	Black and white
1968	BC7076 NO. 153-155,	Black and white
1972	BC7407 NO. 195-197	Black and white
1975	BC7751 NO. 119-121	Black and white
1981	15BC81053 NO. 070-074, 15BC81053 NO. 087-092	Black and white
1986	30BCC393 NO. 194-196	Black and white
1989	15BC89020 NO. 010-012	Black and white

Year	Aerial Photo No.	Type
1993	30BC93026 NO.020-022	Black and white
1998	30BCC98037 NO. 032-034	Colour

The review of the available historical aerial photographs included the historical condition of the dam and reservoir side slopes, noting the following:

- Modifications to the dam and access road were undertaken between the 1962 and 1968 historical aerial photographs, which is consistent with review of the dam modification records (Section 3.2);
- The forest service road (FSR) south of Stocking Lake was constructed between the 1986 and 1989 historical aerial photographs and a large area on the side slope approximately 550 m west of the lake was deforested;
- A large area approximately 400 m south of Stocking Lake Dam was deforested between the 1993 and 1998 aerial photographs; and,
- No signs of instability or erosional changes of the reservoir side slopes can be observed from 1936 onwards.

A review of historical aerial imagery on Google Earth shows that periodic clearing and the development of access roads has occurred in areas of dense forest on the reservoir side slopes between 2005 and 2010.

In all historical aerial photographs reviewed there appears to be an area on the northern side slope, approximately 380 m north of the dam, of what appears to be boggy ground which shows signs of potential surficial erosion likely associated with storm water runoff. Signs of erosion do not extend down to the lake and the land making up the side slope between the boggy area and the lake is densely vegetated.

3.4 Surficial Geology

Reference to Province of British Columbia Ministry of Energy, Mines and Petroleum Resources 1:50,000 scale map “Surficial Geology of the Duncan Area” indicates that the site is underlain by silty diamicton deposits, likely overlying glaciofluvial sand and gravel deposits, overlying bedrock (Blyth & Rutter, 1992). Limited surficial geology information is available for the site, so a surficial geology figure is not included in this report.

3.5 Bedrock Geology

The Geological Survey of Canada (GSC) 1:5,000,000 scale map “Geological Map of Canada” indicates that the site is close to the boundary of two geological units, namely Coeval with Karmutsen Formation comprising gabbro, diabase, feldspar diabase, glomeroporphyritic diabase and gabbro, minor diorite and Pyroxine-feldspar phyric agglomerate, breccia, lapilli tuff, massive and pillowed flows, massive tuffite, laminated tuff, jasper and chert. The bedrock geology for the site is presented on Figure 3.5.

3.6 Seismicity

The GSC has developed a new probabilistic (5th Generation) seismic hazard model (Halchuk, Adams and Allen, 2015) that forms the basis of the seismic design provisions of the 2015 National Building Code of Canada (NBCC, 2015).

Based on the surficial geology of the area, which indicates shallow bedrock, the site classification for seismic response for the Stocking Lake Dam is considered to be Site Class C (very dense soil and soft rock). Peak Ground Accelerations (PGA) and Spectral Accelerations (Sa(T)) for a reference "Site Class C" (very dense soil and soft rock) can be obtained from Earthquakes Canada for various return periods, with the reference values for the Stocking Lake Dam summarized in Table 3.6.a below.

Table 3.6.a Site Class C Design PGA and Sa for Stocking Lake Dam, Ladysmith, BC

Annual Exceedance Probability (AEP)	PGA (g)	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
1/100 year	0.111	0.258	0.213	0.102	0.053
1/475 year	0.257	0.591	0.526	0.275	0.153
1/1,000 year	0.350	0.805	0.735	0.402	0.233
1/2,475 year	0.484	1.109	1.032	0.594	0.355

For seismic hazards with very low probabilities (i.e. return periods greater than 2,475 years) the GSC recommends plotting the annual probability versus acceleration of the 1/475 year and 1/2,475 year values on a log-log scale and extrapolating the line to the required return period. Extrapolated Site Class C PGA and Sa(T) reference values for the Stocking Lake Dam are summarized in Table 3.6.b.

Table 3.6.b Extrapolated Site Class C Design PGA and Sa for Stocking Lake Dam, Ladysmith, BC

Annual Exceedance Probability (AEP)	PGA (g)	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
1/5,000 year	0.700	0.913	0.847	0.471	0.278
1/10,000 year	0.823	1.039	0.974	0.552	0.332

With respect to selection of earthquake design magnitudes the CDA Technical Bulletin, Seismic Hazard Considerations for Dam Safety recommends utilising the greatest of the mean magnitude, modal magnitude or the 84th percentile of the total magnitude contributions when considering multiple seismogenic probabilistic seismic hazards.

The relative contribution of the earthquake sources to the seismic hazard in terms of distance and magnitude can be obtained by deaggregation of the seismic hazard result. The deaggregation data for the NBCC 2015 design model has been obtained from Earthquakes Canada, which provides the mean and modal magnitude of the seismic hazard for the Stocking Lake Dam for the 1/2,475 year event as summarized in Table 3.6.c below.

Table 3.6.c Design Earthquake Magnitudes for Stocking Lake Dam, Ladysmith, BC

Magnitude Contributions	PGA	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
Mean	7.43	7.37	7.67	8.14	8.39
Modal	7.15	7.15	7.45	8.95	8.95
84th Percentile	8.83	8.85	9.00	9.05	9.05

3.7 Existing Drawings

A review of the existing documentation for the Stocking Lake Dam indicates that there are several series of drawings available for the dam, namely:

- 1986 – Stocking Lake Reservoir General Plan of Dam Drawing No. 4984-12A – BC Ministry of Environment Water Management Branch
- 1986 – Stocking Lake Reservoir Plan of Reservoir Drawing No. 4984-12 – BC Ministry of Environment Water Management Branch

- 1988 – Stocking Lake Dam Fig. 25 – BC Ministry of Environment

All existing drawings for Stocking Lake Dam are presented in Appendix B.

3.8 Instrumentation

Stocking Lake Dam has a metric staff level gauge in the reservoir installed near the spillway inlet channel.

3.9 Previous Dam Safety Reviews

It is understood that this DSR is the first for this facility and no previous DSR is available for review.

3.10 Other Reports

A review was undertaken of other available reports associated with the dam (listed in Appendix A) including a geotechnical assessment prepared by TT-EBA (2016).

Key points from Ecora's review of the 2016 dam inspection report are as follows:

- The site inspection was undertaken to investigate the increased rate of seepage noted by the dam owner.
- Seepage was observed near both abutments and at the toe of the embankment. Water was noted as appearing clear. At the right abutment, the majority of this seepage appeared to be coming from an isolated area 10 to 20 cm in diameter, halfway down the slope.
- A seepage measuring device was previously installed at base of abutment with the estimated flow rate being 1 L/s.
- There appears to have been significant disturbance and material loss in vicinity of the 'ATV' trail at the right abutment, halfway down the downstream slope due to recreational vehicle traffic.
- A recommendation to install a temporary short-term filter near the left abutment, where the bulk of the seepage was noted, was made during the inspection.

4. Site Reconnaissance

4.1 General

Ecora conducted a site reconnaissance of the Stocking Lake Dam on two occasions, as part of the Request for Proposal (RFP) on January 17, 2018 and as part of a scheduled site inspection on March 29, 2018. Ecora's site representatives in March were Michael J. Laws, P.Eng, Caleb Pomeroy, P.Eng., Dr. Adrian Chantler, P.Eng. and Bram Hobuti, P.Eng.

The site reconnaissance comprised three components, namely:

- A visual inspection of the exposed section of the dam, underwater pole camera inspection of the submerged upstream slope of the dam and tour of some of the area in the vicinity of Stocking Lake;

- A simple level survey of the dam crest; and,
- Staff interviews.

A summary of the site reconnaissance notes is provided as Appendix C.

4.2 Visual Inspection

Ecora inspected the crest, upstream slope, downstream slope, spillway structure, downstream toe, and outlet (creek downstream) of the dam. Photographs 1 through 12 show the Stocking Lake Dam at the time of the two site visits undertaken on January 17, 2018 and March 29, 2018. The observations made through this inspection are presented in the Photo Log following the text of this report.

Key observations from the site inspection are as follows:

- Seepage was noted at the left abutment toe of the dam. It appeared clear (Photo 3).
- A sinkhole located at the right abutment was observed to be discharging clear water during the site reconnaissance on January 17, 2018, however was no longer flowing during the site reconnaissance on March 29, 2018 (Photo 4).
- Debris was noted in the spillway outlet channel (Photos 5-7).
- A sign was in place at the base of the dam in the spillway channel (Photo 8).
- The spillway approach channel had a log boom in place (Photo 9).
- The backfilled materials above the section of the water distribution main immediately down stream of the dam were saturated (Photo 10).
- No vehicle access is currently available to the dam, however there is a trail that provides access for all terrain vehicles.
- The dam was noted to be wider than the design drawings indicate (Photo 11).
- Riprap 0.4 m – 0.7 m was observed to extend to the upstream toe of the dam (Photo 12).
- Encapsulated foam log boom that is in front of both the upstream face of the dam and the spillway.

4.3 Basic Topographical Survey

A basic topographical survey was completed as part of the March 29 site inspection with additional survey conducted during the invasive geotechnical investigation of the dam on April 17, 2018. The purpose of the surveys was to compare the current minimum elevation of the dam crest to historical surveys of the dam (361.21 m) and confirm critical dimensions of the dam.

The water level at the time of the investigation was surveyed at an average elevation of 360.86 m.

The result of the survey as presented on Figure 4.3 indicates that the lowest crest elevation of the dam is 361.85 m, and that the current available freeboard above the spillway crest is approximately 1.06 m.

4.4 Staff Interviews

Following completion of the site reconnaissance, an interview with David Parker (CVRD) was carried out regarding the operations, maintenance and surveillance of the dam.

Key points from discussions with the CVRD are as follows:

- Surveillance (inspection) of the dam is predominantly undertaken by the ToL weekly, weather permitting;
- Debris passing under the log boom in a storm event has been identified as an issue. This has been attributed to the buoyancy and lack of submerged depth of the log boom; and
- Debris boom alignment was identified to be deficient by the CVRD.

5. Geotechnical Investigation

5.1 Geophysical Investigation

A geophysical investigation was conducted by ConeTec Investigations Ltd. (ConeTec) of Richmond, BC on March 16, 2018. The investigation consisted of two two-dimensional (2D) Multichannel Analysis of Surface Waves (MASW) tests and three Magnetometer tests. The purpose of the investigation was to provide shear wave velocity profiles for the materials beneath the embankment crest.

The MASW data was acquired along two lines on the upstream and downstream sides of the dam crest. A 48 geophone static array with station spacing of 0.5 m was used with a roll along method to survey the full line lengths. The source (baleen hammer) position was moved through the static array at a spacing of 1 m. The roll along method was organized such that at least 24 channels were maintained behind the source location with a 1 m offset from the nearest geophone. A summary of the MASW tests is presented in Table 5.1.a.

Table 5.1.a Summary of MASW Tests

Section ID	Section Length (m)	Array Length (m)	Start of Section		End of	Section
			Northing (m)	Easting (m)	Northing (m)	Easting (m)
MASW18-01	29	35.5	5422970	440044	5422985	440069
MASW18-02	8	16	5422966	440050	5422970	440057

The magnetometer data was collected along the same two lines as the MASW tests. Readings were taken every 0.5 m at the same positions as the geophones. Along the upstream side of the dam crest, magnetic field data was measured at two different vertical sensor locations (2.27 m and 0.9 m above ground) at every survey position. A summary of the magnetometer tests is presented in Table 5.1.b.

The ConeTec report is presented in Appendix D. The start, midpoint and end coordinates of the tests were measured with a hand-held GPS and checked against aerial imagery.

Table 5.1.b Summary of Magnetometer Tests

Section ID	Section Length (m)	Start of Section		End of Section	
		Northing (m)	Easting (m)	Northing (m)	Easting (m)
MAG18-01 (2.27 m)	32	5422969	440041	5422985	440069
MAG18-01 (0.9 m)	32	5422969	440041	5422985	440069
MAG18-02 (2.27 m)	16	5422964	440047	5422972	440061

5.2 Invasive Investigation

Ecora conducted an invasive geotechnical investigation of the dam on April 17, 2018 comprising the advancement of two boreholes, one on the upstream and one on the downstream side of the dam crest, using sonic drilling methodology. A track mounted sonic drill rig owned and operated by Drillwell Enterprises of Duncan, BC was used to advance the boreholes, which were both terminated in bedrock at depths of 4.6 m and 5.2 m below ground level (mbgl). The drilling investigation was supervised by Mr. Peter Wittstock, E.I.T., who logged the encountered materials and collected representative soil samples for laboratory testing.

Standard Penetration Testing (SPT) using a split spoon sampler was carried out at regular intervals within the depth zone investigated by the boreholes. The SPT is an in-situ dynamic penetration test design to provide information on the geotechnical properties of the soils. The split spoon sampler comprises a thick-walled sample tube, with an outside diameter of 51 mm and an inside diameter of 36 mm, and a length of approximately 650 mm. This is driven into the ground at the bottom of a borehole by blows from a drop hammer with a weight of 63.5 kg (140 lb.) falling through a distance of 760 mm (30 in.). The sample tube is driven 75 mm into the ground then the number of blows needed for the tube to penetrate each 75 mm (3 in.) up to a depth of 600 mm (24 in.) is recorded.

Standpipe piezometers were installed in each of the boreholes upon completion. The standpipe piezometers comprised a 50 mm (2 in.) diameter schedule 80 PVC standpipe, including a machine slotted length installed between approximately 1.2 mbgl and 3.0 mbgl within each of the boreholes. The boreholes were then backfilled with filter sand around the screened portion with bentonite plugs above and below the screen. Details of the standpipe installation is presented on the logs in Appendix D.

Table 5.2 summarizes the details of the invasive investigation. Coordinates were taken during the topographic survey and are reported in NAD 1983 UTM Zone 10. An investigation location plan is provided in Figure 5.1 and the logs are presented in Appendix D.

Table 5.2 Summary of Invasive Investigation

Borehole ID	Location	Northing (m)	Easting (m)	Termination Depth (mbgl)	Termination Reason
BH18-01	Upstream	5422984.87	440055.85	5.18	Target depth reached (bedrock encountered)
BH18-02	Downstream	5422979.28	440058.37	4.57	

6. Encountered Subsurface Conditions

6.1 Encountered Materials

The subsurface conditions that were encountered in the boreholes are summarized below, with detailed logs provided in Appendix D.

- **Embankment Fill,** comprising loose to compact sand, some gravel to gravelly, some silt to silty, with rootlets in the upper 0.2 mbgl. The fill was typically moist to wet, brown to grey with sub-angular to sub-rounded gravel. SPT N-values within this unit ranged between 6 and 20, encountered to a depth of 2.4 mbgl and 1.2 mbgl within the upstream and downstream boreholes respectively, underlain by;
- **Alluvial Deposits,** comprising loose to compact silty sand, trace gravel to gravelly with occasional organic material. The alluvial deposits were typically wet, grey to brown/black, encountered to 3.0 mbgl to 3.2 mbgl within the upstream and downstream boreholes respectively, underlain by;
- **Till-like Deposits,** comprising compact to dense silty sand and gravel. The till-like deposits were typically moist to wet and grey. SPT N-values within this unit were 50+, encountered to 3.4 mbgl to 4.0 mbgl within the upstream and downstream boreholes respectively, underlain by;
- **Bedrock** comprising moderately fractured medium grained igneous rock encountered to the extent of the depths investigated 5.2 mbgl and 4.6 mbgl within the upstream and downstream boreholes respectively.

6.2 Groundwater

The standing water level (SWL) within each of the standpipe piezometers was dipped on August 16, 2018 and recorded at 2.8 mbgl for both the upstream and downstream boreholes (Table 6.2), corresponding to the approximate level of the toe of the dam. It should be noted that at the time period during which the levels were dipped, groundwater elevations are generally low. Groundwater levels are likely higher during certain times of year, especially periods of heavy rainfall and/or snow-melt.

Table 6.2 Summary of Measured Standing Water Levels

Borehole ID	Location	Northing (m)	Easting (m)	Screen Interval (mbgl)	SWL (mbgl) 08/16/2018
BH18-01	Upstream	5422984.87	440055.85	1.2-3.0	2.8
BH18-02	Downstream	5422979.28	440058.37	1.2-3.0	2.8

6.3 Laboratory Testing

Laboratory testing was conducted at Ecora's Canadian Council of Independent Laboratories (CCIL) certified laboratory in Penticton on selected soil samples obtained from the invasive investigation to confirm field observations and their physical characteristics. The testing program consisted of four grain size analyses and four moisture content tests.

A summary of the laboratory test results is presented in Table 6.3 with gradation curves in Appendix D.

Table 6.3 Summary of Laboratory Test Results

Borehole ID	Sample Depth (m)	Moisture Content (%)	Particle Size Distribution (%)		
			Fines	Sands	Gravel
BH18-01	1.4 – 1.8	16.7	21	60	19
BH18-02	0.4 – 0.8	-	18	64	18
BH18-02	2.7 – 3.4	12.5	30	41	29
BH18-02	3.4 – 3.7	-	23	27	50

6.4 Shear Wave Velocities

Shear wave velocity (V_s) test results are included in the appendices of the ConeTec report (Appendix D).

The V_s values for the upstream side of the dam crest ranged between approximately 92 m/s and 1,000 m/s, typically in the range of 120 m/s to 250 m/s. The V_s profile along the upstream side of the dam crest shows a deeper soil profile towards the west with shallow bedrock inferred at approximately 1 mbgl at the eastern end of the embankment. A localized area of lower density material was picked up in the MASW survey at approximately 5 m to 9 m along the upstream profile at approximately 1 m to 3 m depth possibly indicative of internal erosion (piping). This corresponds to the area of seepage observed during the site reconnaissance of January 17, 2018.

The V_s values for the downstream side of the dam crest ranged between approximately 76 m/s and 365 m/s and show a more consistent profile with V_s increasing with depth. V_s values within the upper 2 m corresponding to the earthfill material were typically in the range of 76 m/s to 120 m/s. The native soils below the embankment were typically in the range of 120 m/s and 165 m/s with inferred bedrock ranging between V_s of 200 m/s and 365 m/s.

7. Dam Break Analysis

The consequence classification of a dam depends on the incremental consequences of a dam failure, and this can be the result of overtopping, a piping failure, or an earthquake, for example. A dam break analysis, including characterization of a hypothetical dam breach, flood wave routing, and inundation mapping, was carried out as part of this review.

The characterization of the dam breach and initial flood hydrograph was conducted using the US National Weather Service Breach Erosion Model (BREACH). The BREACH model was used to evaluate breach opening, time of dam failure and the subsequent breach flow into the downstream creek.

As overtopping of the Stocking Lake Dam could occur due to blockage of the spillway, only this, more conservative, failure mechanism was considered in the dam break analysis.

A summary of the overall dam breach parameters is provided in Table 7.0.a.

Table 7.0.a Summary of Dam Breach Parameters

Dam Breach Parameter	Stocking Lake Dam
Type of Dam	Earthfill
Peak Inflow to Reservoir	10.0 m^3/s (100-year flood)
Dam Breach Elevation (DBE)	362.08 m (overtopping failure)
Final Breach Elevation	359.02 m (dam toe)
Volume of Reservoir Between Breach Elevations	306,000 m^3
Reservoir Surface Area at Breach Elevation	250,000 m^2
Width of Crest	8.0 m
Length of Crest	30 m
Upstream Dam Face Slope	1V:2.25H
Downstream Dam Face Slope	1V:2H
D_{50} Grain Size	0.47 mm
Porosity Ratio	0.35
Unit Weight	184 kN/m^3
Internal Friction	35°
Cohesive Strength	0

Dam Breach Parameter	Stocking Lake Dam
Final Breach Width[1.]	12.8 m
Time at Which Peak Outflow Occurs[1.,2.]	3.05 hrs
Peak Breach Flow[2]	73.2 m^3/s

[1.] Evaluated using BREACH.

[2.] From commencement of inflow to the reservoir

The resulting dam breach hydrographs were routed using a 2-dimensional volume conservation flood routing model, FLO-2D, with the flood wave simulation run for 24 hours. Topographical inputs for the model were developed from the BC Terrain Resource Information Management (TRIM) Program data.

It should be noted that in the FLO-2D model, the ground surface is represented by a grid. The grid size utilized for this project is 20 m × 20 m. This is considered adequate to represent the rough terrain that accounts for the majority of the study area. Sudden changes in topographic relief, such as channels, roads and river dykes, may not be accurately characterized at this resolution, as elevation variations are averaged out within a grid area and therefore some localised variation in flow depths from those modelled is anticipated.

The model assumed that any hydraulic structures such as culverts were blocked by debris picked up by the flood wave and therefore their effect on routing the flood wave was ignored.

Changes in the Manning's roughness coefficients in the FLO-2D model due to variations in the flood wave depth, velocity and flow regime are automatically calculated by assigning a limiting Froude number. The Froude number represents the relationship between the kinematic flow forces, gravitational forces and the threshold between subcritical and supercritical flow. Limiting Froude numbers assigned to the grid cells in the analysis are based on the suggested values summarized in Table 7.0.b for various terrain characteristics.

Table 7.0.b Suggested Limiting Froude (Fr) Numbers[1]

Terrain Characteristics	Flat or Mild Slope (large rivers and floodplains)	Steep Slope (alluvial fans and watersheds)
Channels	0.4 – 0.6	0.7 – 1.05
Overland	0.5 – 0.8	0.7 – 1.5
Streets	0.9 – 1.2	1.1 – 1.5

[1.] From FLO-2D Reference Manual, September 1996.

Figure 7.0a and Figure 7.0b presents the results of the flood extents and maximum depth of flooding, indicating a total inundation area of 1.05 km^2. The flow travels along Stocking Creek for approximately 6 km where it enters the Strait of Georgia at Davis Lagoon.

Figure 7.0c and Figure 7.0d shows the delay time between the initial dam breach and the time at which flooding reaches a depth of 0.6 m.

Areas of interest impacted by the dam breach and flooding are summarized below.

- Transportation Infrastructure:
  - Forestry Service Roads;
  - S. Watts Road;
  - Highway 1 – Island Highway, and:
  - Chemainus Road Bridge over Stocking Creek

- Residences:
    - Minor flooding of downstream structures.
- Other Potential Impacts:
    - Natural gas transmission line.
    - Water main to Saltair and the Town of Ladysmith;
    - Stocking Lake Creek Park.

Flood hazard maps are presented on Figure 7.0e and Figure 7.0f., using the method of Garcia et al. (2003 and 2005). The flood hazard level at a specific location is a function of flood intensity (flow depth and velocity) and probability. The map uses three colours to define high (red), medium (orange) and low (yellow) hazard levels. Definitions of each flood hazard level are provided in the legend of the map and in Table 7.0c below.

Table 7.0.c Definition of Water Flood Intensity

Flood Intensity	Maximum depth h (m)		Product of maximum depth h times maximum velocity v (m^2/s)
High	h > 1.5 m	OR	V h > 1.5 m^2/s
Medium	0.5 m < h < 1.5 m	OR	0.5 m^2/s < v h < 1.5 m^2/s
Low	h < 0.5 m	AND	V h < 0.5 m^2/s

8. Consequences Classification

8.1 General

A consequences classification system has been developed by the Canadian Dam Association (CDA, 2007) to categorize the consequences of dam failure in terms of potential loss of life; environmental and cultural losses; and infrastructure and economic losses. The consequences classification of a dam should be selected using the highest rating based on these types of loss. Note that the consequences are incremental to those that would have occurred in the same event without failure of the dam. The CDA (2007) defines incremental consequence of failure as:

"The incremental consequences or damage that a dam failure might inflict on upstream areas, downstream areas or on the dam itself, over and above any losses or damage that may have occurred in the same event or conditions had the dam not failed".

These consequences categories are applied to establish guidelines for some of the design parameters for a dam, such as the Inflow Design Flood (IDF) and the Earthquake Design Ground Motion (EDGM), and the standard of care expected of owners. The BC Dam Safety Regulation and CDA describes five consequence categories: "Low", "Significant", "High", "Very High" and "Extreme".

The BC Dam Safety Regulation 40/2016 (February 29, 2016), and the 2007 CDA Dam Safety Review Guidelines (2013 Edition), provide consequences classification criteria as well as suggested design flood and earthquake levels as a function of dam consequence classification as reproduced as Table 8.1 below. It is noted that the BC Dam Safety Regulation was amended in 2011 so that consequence classifications are now in alignment with those provided in the 2007 CDA guidelines and care must be taken in the interpretation of engineering reports dated prior to November 2011.

Table 8.1 BC Regulation 40/2016 & CDA Consequences Classification Criteria and Design Earthquake and Flood

Dam Classification from BC Reg. 40/2016 & CDA 2007	Population at Risk (BC Reg. 40/2016)	Loss of Life (BC Reg. 40/2016)	Infrastructure and Economics (BC Reg. 40/2016)	Environmental and Cultural Losses (BC Reg. 40/2016)	Annual Exceedance Probability Level	
					EQ Design Ground Motion (CDA 2007)	Inflow Design Flood (CDA 2007)
Extreme	Permanent[3]	>100	Extremely high economic losses affecting critical infrastructure, public transportation or services or commercial facilities, or some destruction of or some severe damage to residential areas	Major loss or deterioration of: a) critical fisheries habitat or critical wildlife habitat, b) rare or endangered species, c) unique landscapes, or d) sites having significant cultural value, and restoration or compensation in kind is impossible.	1/10,000	PMF
Very High	Permanent[3]	10-100	Very high economic losses affecting important infrastructure, public transportation or services or commercial facilities, or some destruction of or some severe damage to residential areas	Significant loss or deterioration of: a) critical fisheries habitat or critical wildlife habitat, b) rare or endangered species, c) unique landscapes, or d) (d) sites having significant cultural value, and restoration or compensation in kind is possible but impractical	½ between 1/2,475 and 1,10,000	⅔ between 1/1000 year and PMF
High	Permanent[3]	1-10	High economic losses affecting infrastructure, public transportation or services or commercial facilities, or some destruction of or some severe damage to scattered residential buildings	Significant loss or deterioration of: a) important fisheries habitat or important wildlife habitat, b) rare or endangered species, c) unique landscapes, or d) sites having significant cultural value, and restoration or compensation in kind is highly possible	1/2,475	⅓ between 1/1000 year and PMF

Dam Classification from BC Reg. 40/2016 & CDA 2007	Population at Risk (BC Reg. 40/2016)	Loss of Life (BC Reg. 40/2016)	Infrastructure and Economics (BC Reg. 40/2016)	Environmental and Cultural Losses (BC Reg. 40/2016)	Annual Exceedance Probability Level	
					EQ Design Ground Motion (CDA 2007)	Inflow Design Flood (CDA 2007)
Significant	Temporary Only[2]	Low potential for multiple loss of life	Low economic losses affecting limited infrastructure and residential buildings, public transportation or services or commercial facilities, or some destruction of or damage to locations used occasionally and irregularly for temporary purposes	No significant loss or deterioration of: a) important fisheries habitat or important wildlife habitat, b) rare or endangered species, c) unique landscapes, or d) sites having significant cultural value, and restoration or compensation in kind is highly possible	1/1,000	Between 1/100 and 1/1000 year
Low	None[1]	0	Minimal economic losses mostly limited to the dam owner's property, with virtually no pre-existing potential for development within the dam inundation zone	Minimal short-term loss or deterioration and no long-term loss or deterioration of: a) fisheries habitat or wildlife habitat, b) rare or endangered species, c) unique landscapes, or d) sites having significant cultural value	1/475	1/100 year

1. There is no Identifiable Population at Risk
2. People are only occasionally and irregularly in the dam-breach inundation Zone, for example stopping temporarily, passing through on transportation routes or participating in recreational activities.
3. The population at risk is ordinarily or regularly located in the dam-breach inundation zone, whether to live, work or recreate

The BC MFLNRORD has currently assigned the dam a consequence classification rating of “Signficant” in terms of the BC Dam Safety Regulation (BC Reg. BC Reg. 40/2016). The “Significant” classification suggests that, in the event of a dam failure, no permanent population would be at risk, and at worst, there would be no significant loss or deterioration of important fish, or wildlife habitat, or low economic losses affecting infrastructure, public transportation and commercial facilities.

8.2 Consequences Classification Review

8.2.1 General

Based on the results of the dam break analysis and flood inundation mapping, a review of the consequence classification criteria for the Stocking Lake Dam was conducted as per the CDA 2007 Dam Safety Guidelines considering each of the following loss criteria:

- Loss of life;
- Environmental and cultural losses; and
- Infrastructure and economics.

8.2.2 Loss of Life

There are several factors that affect the severity of the loss of life consequence, such as depth of flow, velocity and advance warning time within the inundated area.

However, the most important factor in estimating the loss of life (LOL) that would result from dam failure is determining when dam failure warnings would be initiated. The United States Bureau of Reclamation (USBR) has compiled data on dam failure warning times from US dam failures that have occurred since 1960, as well as other notable global dam failures as summarized in Table 8.2.a below.

Table 8.2.a Guidance for Estimating when Dam Failure Warning would be Initiated (Dam Type: Earthfill Dam)

Cause of Failure	Special Considerations	Time of Failure	When Would Dam Failure Warning be Initiated	
			Many Observers at Dam	No Observers at Dam
Overtopping	Drainage area of dam less than 260 km²	Day	0.25 h before dam failure	0.25 h after floodwater reaches populated area
	Drainage area of dam less than 260 km²	Night	0.25 h after dam failure	1 h after floodwater reaches populated area
	Drainage area of dam more than 260 km²	Day	2 h before dam failure	1 h before dam failure
	Drainage area of dam more than 260 km²	Night	1 to 2 h before dam failure	0 to 1 h before dam failure
Piping (full reservoir, normal weather)		Day	1 h before dam failure	0.25 h after floodwater reaches populated area
		Night	0.5 h after dam failure	1.0 h after floodwater reaches populated area
Seismic	Immediate Failure	Day	0.25 h after dam failure	0.25 h after floodwater reaches populated area
		Night	0.5 h after dam failure	1.0 h after floodwater reaches populated area
	Delayed Failure	Day	2 h before dam failure	0.5 h before floodwater reaches populated area
		Night	2 h before dam failure	0.5 h before floodwater reaches populated area

Brown and Graham (1988) developed a series of empirical equations for estimating loss of life due to dam failure from analysis of major dam failures and flash floods. Their study concluded that loss of life is much greater in those areas that receive little warning time compared to those areas that receive 90 minutes or more of warning, and three empirical equations were developed as a function of warning time as summarized in Table 8.2.b below.

Table 8.2.b Loss of Life Empirical Equations

Warning Time	Estimated Loss of Life (LOL)
Less than 15 minutes	LOL = 0.5 x PAR
When warning time is between 15 and 90 minutes	LOL $=PAR^{0.6}$
Greater than 90 minutes	LOL = 0.0002 x PAR

PAR = Population at Risk.

The population at risk is considered to be contained within any residences identified within the High Hazard areas shown in Figure 8.0c using Google Earth imagery.

No dwellings were identified within the High Hazard area and therefore no permanent population is considered to be at risk in the event of dam failure. However, it is anticipated that loss of life could occur due to the presence of a transitory population in the inundation zone, for example persons in vehicles on Highway 1 could be impacted by a flood wave in the event of a breach. In the event of a night time inundation of Highway 1 by a flood wave at this location it would be reasonable to expect that the loss of life could be greater than 1 but probably less than 10 and therefore the consequence classification rating for Stocking Lake Dam would equate to “High” based on loss of life.

8.2.3 Environmental and Cultural Losses

It is understood that rainbow trout are present in Stocking Lake and salmon use the lower reaches of the creek for spawning. In the event of a dam breach Stocking Lake would not be completely drained, however erosion would occur along Stocking Creek downstream of the dam. This suggests that potential loss of minor restorable habitat could occur in the event of dam break, equating to a consequence classification rating of “Significant” based on environmental losses.

8.2.4 Infrastructure and Economic Losses

Notable infrastructure within the downstream flood inundation zone includes multiple residential lots along either side of Stocking Creek, multiple logging/access roads, Watt Road, Highway 1, Chemainus Road Bridge over Stocking Creek, the Southern Vancouver Island Railway, walking trails within Stocking Creek Park and multiple utilities. The Chemainus Road Bridge is elevated above Stocking Creek and therefore would likely not be directly impacted in the event of a dam breach and would serve as an alternate transportation route to Highway 1. Water mains feeding Ladysmith and Saltair could be impacted in the event of a dam breach, though the Town of Ladysmith can be alternatively serviced from Holland Lake Dam thru Holland Creek. Other utilities that could be impacted include BC Hydro transmission infrastructure, a gas transmission line running parallel to Highway 1 and fiber optic lines. Any impact to the BC Hydro transmission line is expected to be minor as the flood extents are expected to fit within the gap between towers. Without more detailed information on the gas transmission and the fiber optic lines it is difficult to estimate the impact, however, if the gas transmission line were to be damaged during the breach, service to the greater Victoria area would be greatly disrupted for an unknown length of time.

The BC Dam Safety Regulation 40/2016 (February 29,2016) nor the 2007 CDA Dam Safety Review Guidelines (2013 Edition) provides guidance with respect to the monetary value associated with infrastructure and economic losses associated with each consequences classification. Therefore, reference has been made to the Ontario Ministry of Natural Resources Technical Bulletin on Classification and Inflow Design Flood Criteria (August 2011), which provides suggested monetary values for economic loses. Table 8.2.c includes the estimated property losses from the technical bulletin for each consequence classification in equivalent CDA consequence rating.

It therefore can be reasonably expected that estimated damage from a dam breach of Stocking Lake could be greater than $3 million but is likely to be below $30 million in damages. This suggests that a consequence classification of "High" would be appropriate for Stocking Lake Dam based on infrastructure and economic losses.

Table 8.2.c Property Loss Criteria based on Consequences Classification

Consequence Classification Rating	Economic Losses
Low	Not exceeding $300,000
Significant	Not exceeding $3 million
High	Not exceeding $30 million
Very High & Extreme	In excess of $30 million

1. 2011 Dollars

8.3 Conclusions

Based on the assessment of the three loss criteria summarized in the sections above, it is recommended that the consequences classification rating of Stocking Lake Dam be revised to have a consequences classification of "High" to reflect the potential loss of life and critical infrastructure that could be impacted in the event of a dam breach. For a dam with a "High" consequence classification, the Inflow Design Flood (IDF) is required to be ⅓ between the 1000-year event and the PMF and the design seismic hazard is required to be the 1/2,475-year event, according to the BC Dam Safety Regulation (BC Reg. 40/2016).

9. Failure Modes

Foster et al. (2000a) reviewed a database on dam failures (up to 1986) worldwide prepared by the International Congress on Large Dams (ICOLD) and determined the most common modes of failure for an earthfill dam as presented below, with percentages of total failure in brackets:

a. Embankment overtopping (34%)

b. Piping through the embankment (33%)

c. Piping through the foundation (15%)

d. Downstream and upstream slope instability (4%)

e. Other causes e.g. earthquake (16% total)

The percentages presented above reflect the characteristics of that database, not the likelihood of those failures developing at the Stocking Lake Dam. It is important to note that the database presents cases where multiple modes of failure were believed to have occurred. As such, the percentage total is greater than 100%.

a. Embankment overtopping occurs when the spillway has insufficient capacity to discharge flood flows, either due to inadequate size or due to blockage with debris. Overtopping could also occur due to a flood wave produced by a landslide descending into the reservoir or failure of an upstream dam. Embankment overtopping is addressed in the hydrotechnical assessment presented in Section 11.

b. and c. Piping is the progressive internal erosion of dam fill or foundation materials along preferential seepage paths. The seepage starts to erode finer soil particles at the toe of a dam or at an interface between dissimilar materials that are not compatible from a filtering perspective (such as a silty clay core adjacent to a coarse rockfill shell). With time and continued seepage erosion, "pipes" or voids will be created within the dam that grow in an upstream direction towards the reservoir with acceleration of seepage and rate of erosion. Eventually, collapse of overlying fill, breach of the dam and subsequent uncontrolled discharge of the reservoir will occur. Piping is discussed further in Section 10.7.

d. Slope instability. Gravitational, seepage and seismic forces can cause instability in earthfill dams when they exceed the available shear strength of the soil. Slope stability of the dam is discussed further in Section 10.4.

e. Other causes of dam failure included slope instability due to earthquake forces, liquefaction and failure of the spillway/gate (appurtenant works).

A modified version of the MFLNRORD Hazard and Failures Modes Matrix (HFMM) to consider other negative human/wildlife interactions beyond terroism was utilized in assessing the plausible failure modes for Stocking Lake Dam as presented in Appendix E. The likelihood of each hazard and associated failure mode being applicable to Stocking Lake Dam was assessed as either, high, moderate or low as represented by red, orange and green cells respectively in the matrix. It can be noted that the unmodified version uses ratings of applicable versus non-applicable in place of low, medium or high.

For Stocking Lake Dam, the following failure modes are considered to be the most plausible:

- Overtopping – The spillway may become blocked with debris.
- Internal erosion through the embankment, its foundation or abutments – Based on the age of the dam it is likely not to have a filter that satisfies modern design criteria and therefore is more vulnerable to internal erosion processes.
- Seismic upstream and downstream slope instability – The dam embankment and/or foundation materials may undergo liquefaction and a loss of strength when subjected to the EDGM resulting in embankment instability and/or deformation.

10. Geotechnical Assessment

10.1 General

The geotechnical assessment is based on the results of the geotechnical investigation, observations made during the site reconnaissance, available data on the dam, historical geotechnical reports, published geological data, and Ecora’s engineering judgment.

The geotechnical assessment of the dam considers the maximum embankment height based on topographical data (Section 4.3) and the subsurface conditions encountered in the recent investigation as well as historical data.

The following subjects will be discussed in this section:

- Embankment seepage;
- Embankment stability;
- Liquefaction; and,
- Internal erosion (piping).

10.2 Geotechnical Parameters

Soil parameters for the geotechnical analysis have been estimated using a combination of field observations and published data for similar material types.

Several publications provide typical values for a range of different soil types encountered, such as Craig (1992), which provides typical ranges of hydraulic conductivities in Table 2.1 (reproduced as Table 10.2.a) and Bowles (1988), which provides representative values of angle of internal friction in Table 2-6 (reproduced as Table 10.2.b). However, the hydraulic conductivity of sand and gravel mixtures is highly sensitive to the silt content as discussed in Bandini et al. (2009) with the hydraulic conductivities as a function of silt content presented on Figure 10.2.

Table 10.2.a Coefficient of Permeability (m/s) from Craig (1992)

1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}
Clean gravels	Clean sands and sand gravel mixtures			Very fine sands, silts and clay-silt laminate			Unfissured clays and clay-silts (>20% clay)			
	Desiccated and fissured clays									

Table 10.2.b Representative Values for Angle of Internal Friction ϕ from Bowles (1988)

Soil Type	Angle of Internal Friction, ϕ (°)
Gravel	
Medium Size	40 – 50°
Sandy	35 – 50°
Sand	
Loose	27 – 35°
Dense	43 – 50°
Silt or silty sand	
Loose	27 – 30°
Dense	30 – 35°
Clay	20 – 42°

Based on review of the above references and available existing information on the dam the following geotechnical parameters as summarized in Table 10.2.c below were utilized in the various analyses.

The analyses use the most critical section which is considered to be through the pipe trench, running orthogonal to the dam at a depth of 5.5 m below the dam crest, which was likely cut into bedrock beneath the dam. The pipe trench backfill material is assumed to have used native material and therefore have similar properties to the embankment fill. Hence, only the embankment fill and bedrock units are used in the geotechnical analysis.

Table 10.2.c Summary of Parameters Used in the Geotechnical Analysis

Material	Soil Parameters			
	c' (kPa)	ϕ' (°)	γsat (kN/m³)	ksat (m/s)
Embankment fill	0.5	32	19	10^{-4}
Bedrock	Infinite strength		24	10^{-5}

c' = Effective Cohesion Intercept.

ϕ' = Effective Friction Angle.

γ_{sat} = Saturated Unit Weight of Soil.

k_{sat} = Saturated Hydraulic Conductivity.

10.3 Seepage

Initial pore water pressure conditions in the embankment to provide suitable inputs for stability analyses were determined by undertaking a two-dimensional steady state seepage analysis utilising the built-in Finite Element Analysis (FEA) module within the RocScience Slide v8.017 software. The soil hydraulic conductivity parameters used in the analysis are estimated from the background information and published correlations, and therefore may not be accurate; however, the relative values are considered appropriate.

The seepage analysis considered reservoir levels at both elevation 359.0m and elevation 360.8 m for the upstream and downstream analyses respectively, in order to consider the most conservative operating conditions with respect to embankment stability. The reservoir levels noted above correspond to the approximate level of the toe of the embankment and the spillway sill elevation respectively.

The rates of toe seepage calculated for the dam are summarized in Table 10.3 below. It should be noted that the analyses were undertaken at the dam's maximum height and reduced seepage rates are anticipated where the embankment heights are less. The analysis did not consider potential concentrated sources of seepage such as along the low level outlet conduit.

The flow fields from the steady state analyses of the dam are provided on Figures 10.3a and 10.3b.

Table 10.3 Estimated Rate of Toe Seepage for the Stocking Lake Dam

Reservoir Level (Elev m)	Calculated Toe Seepage	Figure No.
359.0	0.13 m^3/m/day	10.3a
360.8	1.91 m^3/m/day	10.3b

10.4 Liquefaction Assessment

Simplified liquefaction triggering analyses were undertaken for the 1/475-year, 1/1,000-year and 1/2,475-year AEP seismic events corresponding to "Low", "Significant" and "High" consequence classification ratings respectively. The PGA values for the different AEP seismic events are as per Table 3.6.a.

The analyses were undertaken utilizing the method of Idriss & Boulanger (2008) using the Geologismiki liquefaction analysis software LiqSVs v1. The analyses utilized averaged Cyclic Stress Ratio (CSR) adjusted to the 84th percentile 8.83 magnitude event based on the deaggregation data (Table 3.6.c) and the strength profile from available borehole SPT and geophysics (V_s) data.

The strength of the SPT and V_s profiles were adjusted to account for the fines content of the encountered soils in the boreholes; utilizing a Soil Behavior Type Index I_c correlation as recommended by Idriss & Boulanger (2014). Liquefaction analyses were undertaken for the groundwater profiles resulting from the steady state seepage analyses discussed in Section 10.3 for both reservoir levels of 360.8 m and 359.0 m elevation.

The results of the simplified liquefaction triggering analyses indicate that the soils beneath the upstream crest of the dam do not liquefy under the various considered EDGM's. The liquefiable thickness within the soils beneath the downstream crest of the dam ranges between 1.6 m and 3.0 m when subject to the 1/475-year to 1/2,475-year AEP seismic events with liquefaction primarily occurring within the dam foundation.

Summary plots of the simplified liquefaction triggering analyses are presented in Appendix F.

10.5 Embankment Stability Review

10.5.1 Criteria

The CDA Technical Bulletin, Geotechnical Consideration for Dam Safety provides accepted minimum slope stability factors of safety for various static and seismic loading conditions as reproduced in Table 10.5.a and Table 10.5.b.

Table 10.5.a Acceptable Factors of Safety for Embankment Stability – Static Assessment

Loading Conditions	Minimum Factor of Safety	Slope
End of construction before reservoir filling	1.3	Upstream and downstream
Long-term (steady state seepage, normal reservoir level)	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3	Upstream

Table 10.5.b Acceptable Factors of Safety for Embankment Stability – Seismic Assessment

Loading Conditions	Minimum Factor of Safety	Slope
Pseudo-Static	1.0	Upstream and downstream
Post-Earthquake	1.2 to 1.3	Upstream and downstream

10.5.2 Methodology

Static and pseudo-static global stability factors of safety (FoS) for the Stocking Lake Dam were calculated using the RocScience two-dimensional Limit State Equilibrium (LSE) analysis software Slide v8.017. Pseudo-static stability analyses were undertaken for the 1/475-year, 1/1,000-year and 1/2,475-year AEP seismic events corresponding to "Low", "Significant" and "High" consequence classification ratings respectively. The PGA values for the different AEP seismic events are as per Table 3.6.a.

Initial pore water pressure conditions in the embankment were determined by importing the results of the two-dimensional steady state FEA seepage model into the LSE analysis.

With respect to assessing the seismic stability of earthfill dams, the CDA Technical Bulletin, Geotechnical Consideration for Dam Safety, recommends a staged approach, beginning with simplified methods using suitably conservative input assumptions to demonstrate that a dam is safe; progressing to more sophisticated analysis methods should the simplified approach lead to unfavourable results. The first recommended stage of analysis undertaken is the pseudo-static method, in which the effects of an earthquake are applied as constant horizontal load via the use of dimensionless coefficients (k_h) equal to the PGA for the earthquake return period. Should the embankment have a factor of safety in excess of 1.0 for this loading, it is considered not to undergo any significant deformation during the design earthquake and therefore no further analysis is required. Should a factor of safety of less than 1.0 be obtained from the pseudo-static analysis, then it is likely that the embankment will undergo deformation during the design earthquake event and a simplified deformation analysis (e.g., as per Newmark (1965), Bray (2007)) approach is recommended as the second stage of analysis to confirm that the embankment has adequate freeboard post the design earthquake event deformation (Section 10.6). Should the second stage of analysis yield unfavourable results then a series of more sophisticated analysis approaches (e.g., FEA) are recommended.

The thickness of liquefiable material resulting from the simplified liquefaction triggering analyses (Section 10.4) was used for the post-earthquake embankment stability analyses with an undrained residual shear strength. The undrained residual shear strength (s_r) of the soil was estimated in accordance with Figure 88 of Idriss & Boulanger (2008) using the average SPT corrected blow count $(N_1)_{60}$ for the embankment fill material.

The stability review of the dam was considered at its maximum height based on the embankment cross sections from available topographic information and the soil profiles encountered in the borehole logs. The results of the static and seismic stability analyses are summarized in Table 10.5.c and Table 10.5.d respectively and are presented on Figures 10.4a to 10.4j.

Table 10.5.c Factors of Safety for Static Slope Stability – Stocking Lake Dam

Loading Conditions	Minimum Calculated FoS	Slope	Figure No.
Static long term (steady state seepage, 1/1,000-year IDF)	1.48	Downstream	10.5a
Static long-term (steady state seepage, low reservoir level)	1.36	Upstream	10.5b
Full or partial rapid drawdown[1]	-	Upstream	N/A

1 Not considered an applicable loading condition as the dam has limited capability to be drawn down rapidly.

Table 10.5.d Factors of Safety for Seismic Slope Stability – Stocking Lake Dam

Loading Conditions	EQ AEP	Consequence Classification Rating	Minimum Calculated FoS	Slope	Figure No.
Seismic pseudo-static (steady state seepage, reservoir at spillway sill elevation, 360.8 m)	1/475	Low	0.83	Downstream	10.5c
Seismic pseudo-static (steady state seepage, low reservoir level, 359.0 m)	1/475	Low	0.80	Upstream	10.5d
Seismic pseudo-static (steady state seepage, reservoir at spillway sill elevation, 360.8 m)	1/1,000	Significant	0.70	Downstream	10.5e
Seismic pseudo-static (steady state seepage, low reservoir level, 359.0 m)	1/1,000	Significant	0.68	Upstream	10.5f
Seismic pseudo-static (steady state seepage, reservoir at spillway sill elevation, 360.8 m)	1/2,475	High	0.57	Downstream	10.5g
Seismic pseudo-static (steady state seepage, low reservoir level, 359.0 m)	1/2,475	High	0.54	Upstream	10.5h
Post-earthquake embankment foundation at residual shear strength (steady state seepage, reservoir at spillway sill elevation, 360.8 m)	1/2,475[1]	High	1.48	Downstream	10.5i
Post-earthquake embankment foundation at residual shear strength (steady state seepage, reservoir at spillway sill elevation, 360.8 m)	1/2,475	High	N/A	Upstream	10.5b

1 As the minimum FoS calculated for the post-earthquake stability under the "High" consequence classification rating seismic event is greater than the accepted value of 1.2 to 1.3, the post-earthquake stability for lower consequence classification ratings have not been assessed.

2 The results of the simplified liquefaction triggering analyses indicate that the soils beneath the upstream crest of the dam do not liquefy under the various earthquake design ground motions for the groundwater cases considered.

As the results of the pseudo-static stability analyses resulted in calculated factors of safety less than 1 for both the downstream and upstream slopes, simplified seismic deformation analyses were undertaken as discussed in Section 10.6.

10.6 Seismic Slope and Liquefaction Post-Seismic Deformation

As the results of the pseudo-static stability analyses resulted in calculated factors of safety of less than 1 for the upstream and downstream slopes, simplified seismic slope deformation analyses were undertaken based on the method of Bray and Travasarou (2007). The deformation of the dam was estimated for varying earthquake design ground motions and slopes for which a pseudo-static static factor of safety of less than 1 was calculated for a range of slip surfaces.

The 84th percentile 8.83 magnitude event based on the deaggregation data as shown in Table 3.6.c was used in the simplified seismic deformation analyses with the initial fundamental period of the sliding mass (Ts) estimated using the V_s data for the embankment fill material (~100 m/s).

The results of the simplified seismic deformation analyses are summarized in Table 10.6.a below and presented on Figures 10.6a and 10.6b.

Table 10.6.a Summary of Simplified Seismic Deformations Analyses

Slope	Earthquake AEP	Consequence Classification Rating	Calculated Displacement Range (cm)
Downstream	1/475	Low	3.9 – 12.5
	1/1,000	Significant	8.4 – 24.6
	1/2,475	High	17.8 – 47.1
Upstream	1/475	Low	3.3 – 16.0
	1/1,000	Significant	7.2 – 30.8
	1/2,475	High	15.6 – 57.6

The available dam freeboard between the elevation of the dam crest and spillway sill elevation is approximately 1.06 m as determined by the available topographic data. The results of the simplified seismic slope deformation analysis indicate that estimated deformations resulting from application of the design earthquake are anticipated to be less than 1.06 m and therefore sufficient freeboard would be maintained during the various earthquake design ground motions.

The thickness of liquefiable material resulting from the simplified liquefaction triggering analyses for each seismic event (Section 10.4) was used to estimate the post-seismic deformation induced loss of freeboard of the dam based on the method of Rauch et al. (2007). The assessment was undertaken to identify the AEP seismic event that would trigger a deformation large enough to result in failure of the dam due to overtopping. The method used is based on a regression analysis of 20,000 numerical simulations, where equations were developed utilizing geometric inputs to estimate loss of freeboard due to post-earthquake deformation as shown on the attached Figure 10.6c. A summary of the results is presented in Table 10.6.b below.

Table 10.6.b Estimated Loss of Crest Freeboard due to Liquefaction Post-Seismic Deformation

Reservoir Level (Elev m)	Seismic Event AEP	Consequence Classification Rating	Liquefiable Thickness (m)	Estimated Loss of Freeboard[1] (cm)
359.0	1/475 year	Low	1.6	134
	1/1,000 year	Significant	2.7	390
	1/2,475 year	High	2.7	390
360.8	1/475 year	Low	2.9	355
	1/1,000 year	Significant	2.9	355
	1/2,475 year	High	3.0	371

1 At maximum dam height.

The results of the estimated loss of crest freeboard due to liquefaction post-seismic deformation indicate that the estimated loss of freeboard resulting from application of the 1/475-year AEP seismic event is in excess of 1.06 m for the downstream slope for both reservoir levels considered, which is anticipated to be enough to cause overall failure of the dam. The results of the simplified liquefaction triggering analyses indicate that the soils beneath the upstream crest of the dam do not liquefy under the various design conditions considered.

10.7 Internal Erosion (Piping)

10.7.1 Internal Erosion Mechanisms

Internal erosion mechanisms of embankment dams and their foundations are categorized into three general failure modes, namely:

- Internal erosion through the embankment, which includes internal erosion associated with penetrating structures, such as conduits associated with outlet works, spillway walls or adjoining a concrete gravity structure supporting the embankment;
- Internal erosion through the foundation; and,
- Internal erosion of the embankment into the foundation. Including (a) seepage through the embankment eroding material into the foundation, or (b) seepage in the foundation at the embankment contact eroding the embankment material.

The process of internal erosion may be broadly divided into four phases, namely:

- Initiation of erosion;
- Continuation of erosion;
- Progression to form a pipe or occasionally cause surface instability (sloughing); and,
- Initiation of a breach.

The models for the development of embankment failures due to internal erosion are shown on Figure 10.7a.

Erosion can be initiated by four mechanisms, namely:

- Concentrated leaks. Concentrated leaks occur where there is an opening in the soil through which preferential seepage occurs, with the sides of the opening enlarging through continual erosion by the leaking water. Such concentrated leaks may occur through a crack caused by differential settlement during construction of the dam or its operation, hydraulic fracturing due to low stresses around conduits or the upper parts of the dam due to differential settlement, or through desiccation at high levels of fill. Frost action also can create cracks in dam crests. Concentrated leaks can also occur due to collapse settlement of poorly compacted fill in the embankment, around conduits and adjacent to walls. They may also occur due to the action of animals burrowing into levees and small dams and tree roots rotting in dams and forming seepage conduits.
- Backward erosion. There are two types of backward erosion, namely:
  - Backward erosion piping. Backward erosion piping occurs where critically high hydraulic gradients at the toe of a dam erode particles upwards and internal erosion develops backwards below the dam through small erosion conduits and flow velocity can transport the eroded particles. The presence of backward piping erosion is often exhibited by the manifestation of sand boils at the downstream side of the dam. An example of backward erosion piping is shown on Figure 10.7b.

  - Global backward erosion. Global backward erosion occurs in embankments with a narrow or downstream sloping core, which is inadequately protected by the filter or transition zone. The progression of the erosion process is assisted by gravity and there is no need for a cohesive soil layer to form the roof for a pipe and it is one of the causes of sinkholes in dams constructed of glacial tills. An example of global backward erosion is shown on Figure 10.7b.

- Contact erosion. Contact erosion occurs when a coarse soil such as a gravel is in contact with a fine soil and flow parallel to the contact in the coarse soil erodes the fine soil.
- Suffusion. Suffusion occurs when water flows through widely graded or gap graded (internally unstable) non-plastic soils, with the small particles of soil transported by the seepage flow through the pores of the coarse particles. Poorly graded soils such as non-plastic glacial tills are more vulnerable to suffusion. Suffusion results in an increase in permeability, greater seepage velocities, and potentially higher hydraulic gradients, potentially accelerating the rate of suffusion. A filter constructed of internally unstable materials will have a potential for erosion of the finer particles in the filter, rendering the filter coarser and less effective in protecting the core materials from erosion. Segregation of broadly or gap graded non-plastic soils during dam construction may create layers which are internally unstable even though the average grading of the soil is internally stable.

10.7.2 Embankment Susceptibility

Once internal erosion is initiated it will continue unless the eroding forces are reduced or the passage of the eroded particles is impeded in some way. Since the 1950's, dam engineers have known that the most efficient way to stop the erosion process in embankments is to zone the dam and incorporate filters. Based on the statistics of embankment dam failures and incidents (Foster et al 2000b) these can be categorized in regards to their capability of providing control of internal erosion in the embankment as shown in Figure 10.7c and Table 10.7.a below.

Table 10.7.a Susceptibility of Embankment Dams to Internal Erosion by Zoning

Vulnerability to Internal Erosion	Control for Internal Erosion	Dam Zoning and Category Number[1]
A Very Vulnerable	Little or no control.	Homogeneous earthfill (0) Earthfill with rock toe (2)
B Vulnerable	Some control of internal erosion depending on detail of zoning and filter capability.	Zoned earthfill (3) Zoned earth and rockfill (4) Puddle core (8) Hydraulic fill (11)
C Low Vulnerability	Moderate control of internal erosion depending on the filter capacity and details of the core wall or face slab.	Concrete face earthfill (6) Concrete face rockfill (7) Concrete core earthfill (9) Concrete core rockfill (10)
D Very Low Vulnerability	Good control of internal erosion subject to good details of zoning and filter design.	Earthfill with filters (1) Central core earth and rockfill (5)

[1] See Figure 10.7c for Illustrations of Dam Zoning Categories.

Based on the available information on the Stocking Lake Dam, it is considered likely to be a zoning category number 0 embankment and therefore very vulnerable to internal erosion.

10.7.3 Probabilistic Piping Potential Assessment

As presented in Foster et al. (2000b), Ecora has used a probabilistic method (the University of New South Wales (UNSW) method), for assessing the relative likelihood of failure of the dam by piping. This paper is included in Appendix G for reference. The UNSW method is based on a retrospective, critical review of dam failure case histories for piping failures that were included in the ICOLD database of dam failures. As a result of its dependence on judgement in selecting weighting factors and its semi-qualitative nature, the results of this assessment should be viewed as providing a general, high level indication of the likelihood of a piping failure occurring sometime in the future.

Based on Ecora's application of the UNSW method, the total annual likelihood of piping failure under current conditions for the Stocking Lake Dam is 1.4 x 10^{-2} (1 in 70 years) corresponding to a NDMP likelihood rating of 3. This figure is the sum of individual probabilities for piping through the embankment, piping of the embankment into the foundation and piping of the foundation. The selection of the weighting factors for every piping mode is presented in Appendix G and a summary of the results in presented in Table 10.7.b below.

While this figure implies a high degree of accuracy, it is not possible to accurately estimate the likelihood of failure for Stocking Lake Dam given the limited dam records. The implied accuracy is due to the statistics used in the Foster et al. (2000b) study.

A significant seismic event could alter the structure of the dam by cracking the core, for instance, or its foundation. If this were to occur, the field performance of the dam could change, with an increased probability of a piping failure or dam safety incident. The satisfactory time record of dam performance would then start at the day of the significant seismic event (some time in the future), not the date of first filling of the reservoir. The probability of a piping failure developing in the dam in the first five years after an earthquake, is estimated from Foster et al. (2000b) to be more than ten times higher than the currently estimated probability.

Table 10.7.b Summary of Estimated Piping Potential

Likelihood of Piping (based on UNSW method)	Piping Event AEP	NDMP Likelihood Rating
1.4 x 10^{-2}	1/70 year	3

11. Hydrotechnical Assessment

The following sections provide a description of the study watershed, a review of available climatic and hydrometric data, and a summary of the method used to develop the Inflow Design Flood (IDF).

11.1 Watershed

Stocking Lake Dam is situated at the outlet of Stocking Lake approximately 4.5 km south of Ladysmith, BC, and has a drainage area of approximately 1.90 km² based on existing community watershed boundaries. The inflows to the reservoir are rainfall and snowmelt within the catchment area. The median basin elevation of the Stocking Lake watershed was estimated to be approximately 380 m. The lake is surrounded by forested land that is subject to logging and tree canopy and vegetative cover is considered to vary from year to year which can affect the time of concentration and other catchment characteristics. The boundary of the Stocking Lake basin is shown on Figure 11.1.

11.2 Climatic and Snow Course Data

A number of climate stations operated by the Meteorological Service of Canada (MSC) are located within the study region. In view of their proximity to the project site, elevation, and length of record, the following stations were considered to have climatic data that was useful in determining the climate conditions at the project site as summarized in Table 11.2.a with station locations presented on Figure 11.2.

Table 11.2.a Regional Climate Stations

Station Name	Station No.	Elevation (m)	Period of Record	Data Type	Rainfall IDF* Curve	Distance to Site (km)
Nanaimo A	1025370	28	1985 – 2012	Daily	Yes	11.1
North Cowichan	1015628	45	1982 – 2005	Daily	Yes	16.6
Nanaimo City Yard	1025370	114	1980 – 2007	Daily	Yes	30.1

*Intensity – Duration – Frequency data

According to the 1981 to 2010 Climate Normals data on the Environment Canada website, the mean annual precipitation at the Nanaimo A Station, which is located North of Stocking Lake, is 1165.4 mm (1098.2 mm rainfall and 68.7 cm snowfall depth). Rainfall occurs throughout the year with 78% during the cooler half of the year (October to March). Snowfall mainly occurs in winter (November to March). Mean daily temperatures range from -3.1°C in December to 18.2°C in August. The rainfall intensity frequency data for the Nanaimo A, North Cowichan and Nanaimo City Yard stations are shown in Table 11.2.b and the 24-hour rainfall totals for various return periods were obtained from IDF curves available through the MSC. The 500-year, 1,000-year and 5,000-year 24-hour rainfall totals were obtained by extrapolation and adjusted to apply to the project site based on the elevation-rainfall relationship for the regional climate stations in Table 11.2.a. The data for the 24-hour events coupled with return periods are provided in Table 11.2.b.

Table 11.2.b Rainfall Intensity Frequency Data at Regional Climate Stations

Return Period (Years)	24-Hour Rainfall Total (mm)		
	North Cowichan	Nanaimo A	Nanaimo City Yard
2	57.8	55.5	58.1
5	70.8	69.7	73.0
10	79.4	79.0	82.9
25	90.3	90.9	95.4
30	92.2	92.9	97.6
50	98.5	99.8	104.8
100	106.5	108.4	113.9
500	126.9	130.6	137.3
1,000	135.5	139.9	147.1
5,000	155.3	161.5	169.9

The River Forecast Centre of the BC Ministry of Environment has a number of snow course and snow pillow sites available on Vancouver Island. The station closest to the project site, by distance and elevation, is the Jump Creek snow pillow station (at an elevation of 1160 m) located north of the Cowichan Lake. The information for this automatic snow pillow station is presented in Table 11.2.c.

Table 11.2.c Regional Snow Pillow Station

Station Name	Station No.	Elevation	Period of Record	Distance to Site
Jump Creek Snow Pillow Station	3B23P	1160 m	1995-2011	33.0 km

The average snow water equivalents for the period of record at the Jump Creek snow pillow station are summarized in Table 11.2.d.

Table 11.2.d Average Snowpack Data for Jump Creek Snow Pillow

Month	Snow Water Equivalent (mm)
Jan	580.6
Feb	836.1
Mar	1070.2
Apr	1257.5
May	1015.6
June	308.5

The data shows that the peak average snow water equivalent (1257.5 mm) occurs in April. Note that this station is approximately 800 m higher than Stocking Lake Dam, so use of this data is considered conservative.

11.3 Hydrometric Data

There is no long-term streamflow data available within the Stocking Lake watershed. Regional hydrometric data was obtained from the Water Survey of Canada to characterize the hydrology of the study area. The regional hydrometric stations used in this study are listed in Table 11.3 with station locations presented on Figure 11.3.

Table 11.3 Regional Hydrometric Stations

Station ID	Station Name	Drainage Area (km²)	Period of Record	Status
08HA016	Bings Creek Near the Mouth	15.5	1961 – 2018	Active
08HA001	Chemainus River Near Westholme	355	1912 – 2018	Active
08HB002	Englishman River Near Parksville	319	1913 – 2018	Active
08HA003	Koksilah River at Cowichan Station	209	1912 – 2018	Active
08HB032	Millstone River at Nanaimo	86.2	1961 – 2018	Active
08HA011	Cowichan River Near Duncan	826	1912 – 2018	Active

11.4 Determination of Inflow Design Flood

11.4.1 General

Based on a review of dam consequence classification discussed in Sections 6.2.and 6.3, Stocking Lake Dam should be classified as a “High” consequence dam in accordance with the 2007 Canadian Dam Association (CDA) Dam Safety Guidelines (2013 Edition). The CDA guideline for an Inflow Design Flood (IDF) for a “High” consequence dam is 1/3rd between 1,000-year flood and the PMF. For the study watershed, peak runoffs are generated either by major rainstorms alone or by rain-on-snow events.

11.4.2 Determination of the 1,000-Year Flood

Two methods were used to determine the 1000-year flood: a rainfall-runoff approach and a regional analysis. The rainfall-runoff approach refers to the development of a hydrologic model to determine the runoff hydrograph at the site, using precipitation and snowmelt as inputs. The regional analysis involves frequency analyses of regional hydrometric data and determination of the relationship between peak discharge and size of drainage area. The following paragraphs further illustrate the methodology and present the results of the two approaches.

Rainfall-Runoff Approach

The 1000-yr 24-hour rainfall totals were calculated using a regression analysis from available 24-hour rainfall data at the North Cowichan, Nanaimo A and Nanaimo City Yard stations. The elevations and the magnitude of the 1000-year rainfall events are included in Table 11.4.a.

Table 11.4.a 1000-Year 24-Hour Rainfall

Station Name	Elevation (m)	1000-Year 24-Hour Rainfall (mm)
North Cowichan	45	136
Nanaimo A	28	140
Nanaimo City Yard	114	147

A relationship between 1000-year 24-hour rainfall and elevation was developed using the above results to calculate the corresponding rainfall at the project site. The calculated 1000-year 24-hour rainfall at the site was estimated to be 175 mm.

To take into account the snowmelt occurring during a rain-on-snow event, the following equation was applied (Gray, 1973):

For heavily forested regions (60 – 100%)

$$M = (0.074 + 0.007*P)*(Ta - 32) + 0.05$$

where

M = snowmelt (in/day);

P = precipitation (in); and

Ta = temperature (°F).

For the 1000-year flood, the 1000-year 24-hour rainfall and the average daily temperature from January to March was used in estimating the daily snowmelt rate. The average value of the mean daily temperature (4.03°C) at Stocking Lake Dam was calculated by defining a relationship for average temperature based on elevation for the above referenced climate stations and using that relationship to estimate the temperature at the Stocking Lake Dam. The average daily snowmelt during a 1,000-year rainfall event was determined to be 23.8 mm/day. This daily snowmelt is possible when compared to the Jump Creek snow pillow station data because there would be enough snow to supply the calculated amount of snowmelt. The combination of the 1,000-year 24-hour precipitation and snowmelt amount to 199.1 mm.

The hydrologic model used in the runoff analysis was HEC-HMS version 4.0, developed by the U.S. Army Corps of Engineers. The US Soil Conservation Service (SCS) unit hydrograph method was applied to determine the runoff hydrograph from the 1000-year 24-hour rainfall combined with the average daily snowmelt rate. The SCS Type Ia distribution was selected to define the distribution of rainfall over 24 hours. The average daily snowmelt was evenly distributed and combined with the rainfall for the storm of interest. In general, the Stocking Lake catchment area consists of heavily forested areas in good condition with intermittent logging activities taking place within the catchment. Soil Type B, representing soil with a well and moderately well drained infiltration rate, was chosen for the study area. Antecedent moisture condition III (saturated conditions) was assumed. A curve number (CN) of 79 was estimated for the catchment area. Slopes, elevations and channel lengths were taken from GIS maps to estimate the time of concentration for the catchment.

The peak inflow to Stocking Lake during the 1000-year return period flood was estimated to be 14.5 m^3/s.

Regional Analysis

A regional hydrological analysis was carried out to provide an alternative estimate of the 1000-year flood inflow to Stocking Lake. Flood frequency analyses were conducted for the selected regional hydrometric stations using the HYFRAN software Version 2.2. Four different frequency distributions: Gumbel, the Three Parameter Lognormal, Weibull and the Log Pearson Type III distributions, were applied to the data. The maximum instantaneous flows were plotted against drainage area and a logarithmic regression equation was fitted to obtain the 1000-yr flows for each selected hydrometric station. The peak flow estimates for three return periods at the project site are tabulated in Table 11.4.b.

Table 11.4.b Regional Analysis Peak Flood Estimates

Return Period (Years)	Flood Estimate (m³/s)
10	2.6
30	3.0
50	3.1
100	3.4
200	3.6
500	3.8
1,000	4.0
5,000	4.5

1,000-year Flood

The 1,000-year peak flood estimate obtained from the regional analysis is lower than that from the hydrologic model. However, most of the available regional stations with data sets extensive enough for statistical analysis are from larger watersheds than that of Stocking Lake. As larger watersheds have a greatly reduced peaking factor and significantly larger time of concentration, it is likely that this method underestimates flooding within the watershed. Also, the data sets mostly have too short record periods for accurate statistical assessment of a 1,000-year event. The HEC-HMS hydrologic model was based on site specific conditions such as soil type and local climate data, making this the preferred, as well as conservative, method. Therefore, the 1,000-year peak inflow to Stocking Lake was determined as 14.5 m³/s.

11.4.3 Determination of the Probable Maximum Flood

The Probable Maximum Flood was assumed to be the result of the Probable Maximum Precipitation combined with snowmelt.

The rainfall-runoff approach was used in determining the Probable Maximum Flood (PMF) for the Stocking Lake Dam. The 24-hour Probable Maximum Precipitation (PMP) was estimated using the Hershfield method described in the Rainfall Frequency Atlas for Canada (Hogg and Carr, 1985).

$K_{M24} = 19 \text{ x } 10^{-0.000965 \, X_{24}}$

$X_{PMP} = X_{24} + K_{M24} \text{ x } S$

where

K_{M24} = frequency factor for a 24-hour duration rainfall;

X_{24} = mean annual 24-hour extreme rainfall (mm);

X_{PMP} = PMP for a 24-hour duration (mm); and

S = standard deviation for a 24-hour duration rainfall (mm).

The 24-hour PMP determined by this method is 326 mm.

The hydrologic model, HEC-HMS was used to estimate the Probable Maximum Flood. The 24-hour PMP was distributed using the SCS Type Ia rainfall distribution, which included a daily snowmelt rate of 31.5 mm/day, for combining with the 24-hour PMP. The PMF for Stocking Lake was determined to be 31.8 m³/s.

The PMF estimator for British Columbia (Abrahamson, 2010) was further used as a rough check for the results of the hydrologic model. The following equation for Vancouver Island was applied:

$Q_{PMF} = 17.795 \text{ x } A^{0.8156}$

where

Q = probable maximum flood (m³/s); and

A = area of the watershed (km²).

The PMF determined using the PMF estimator for British Columbia is approximately 30.0 m³/s, providing good agreement with the hydrogeological model result. However, as the PMF estimator is based on very few data points and considerable variability can occur based on the physical characteristics of the selected catchments, the hydrologic model result is considered to be more representative and the PMF for Stocking Lake is estimated to be 31.8 m³/s.

11.4.4 Inflow Design Flood

The rainfall-runoff method is considered appropriate for developing the IDF for Stocking Lake as it accounts for site specific conditions such as soil type and local climate data.

As indicated earlier, the 1000-year flood event and the PMF was determined to be 14.5 m³/s and 31.8 m³/s, respectively. The CDA guidelines recommend that the IDF for a “High” consequence dam should be 1/3rd between the 1000-year and the PMF (CDA, 2007).

The peak inflow to Stocking Lake during the IDF was therefore determined to be 20.3 m³/s. The hydrographs for calculated return periods are shown on Figure 11.4.

11.5 Flood Routing and Freeboard Determination

A hydrological model was developed to simulate water levels in Stocking Lake and determine the peak outflow during the IDF. The following sections provide a summary of the methodology and results of this analysis.

11.5.1 Volume-Elevation Relationship

The volume-area-elevation relationship for Stocking Lake was determined utilizing lake bathymetry of Stocking Lake completed for the BC Ministry of Environment dated June 1981. Based on this information, Stocking Lake has a temporary storage capacity of about 1,379,000 m³ and surface area of 250,000 m² at the dam crest. The minimum dam crest level is 1.31 m above the spillway crest. The Area Elevation Storage relationship is illustrated in Figure 11.5a

11.5.2 Rating Curve

As determined from the site survey completed by Ecora, the spillway crest length is approximately 3.7 m with a sill elevation of 360.78 m. The rating curve for the spillway was estimated based on the following equation (Smith, 1995):

For broad crested weir flow:

$Q = CLH^{1.5}$

Where:

Q = Discharge (m^3/s);

C = Discharge coefficient for a broad crested weir;

L = Effective spillway crest length (m); and

H = Head above spillway crest (m).

The earthfill dam crest, with a length of 30 m, will act also as a weir if the flood overtops the dam crest, although it was not designed to do so. The rating curve developed for the Stocking Lake Dam spillway is shown on Figure 11.5b. The capacity of the spillway, to the dam crest, is 9.3 m^3/s.

11.5.3 Flood Routing Results

The flood routing was performed using the HEC-HMS model, which includes a routing component for flows through reservoirs. The starting water surface elevation was assumed to be at the spillway crest elevation (360.78 m) and for conservative results it was assumed that the low level outlet was not operating. The results of the HEC-HMS flood routing during the IDF corresponding to the “Significant” classification as well as other notable flows are summarized in Table 11.5.a. Figure 11.5c represents the results of the flood routing graphically.

Table 11.5.a Results of Flood Routing

Consequence Classification/ Return Period	Spillway Weir Crest Elevation (m)	Initial Lake Level (m)	Peak Lake Level (m)	Peak Storage (1000 m³)	Peak Inflow (m³/s)	Peak Outflow (m³/s)	Dam Crest Elevation (m)	Available Freeboard (m)
30-year	360.78	360.78	361.16	88.3	7.8	1.5	362.08	0.9
50-year	360.78	360.78	361.19	94.9	8.8	1.7	362.08	0.9
100-year	360.78	360.78	361.23	103.3	10.0	1.9	362.08	0.9
500-year	360.78	360.78	361.32	124.5	13.2	2.4	362.08	0.8
Significant (1,000-year)	360.78	360.78	361.35	132.8	14.5	2.7	362.08	0.7
5000-year	360.78	360.78	361.43	152.1	17.7	3.3	362.08	0.6
High (1/3rd between 1,000-year and PMF)	360.78	360.78	361.49	167.2	20.2	3.8	362.08	0.6
Very High (2/3rd between 1,000-year and PMF	360.78	360.78	361.64	201.7	26.0	5.0	362.08	0.4
Extreme (PMF)	360.78	360.78	361.78	236.5	31.8	6.3	362.08	0.3

The results above indicate that for the "High" consequence storm there is no overtopping of the dam. The lake level response to the IDF is plotted in Figure 11.5d. Peak outflows would reach 3.8 m^3/s during the "High" consequence storm. Note that the "Significant", "Very High" and "Extreme" results are included for comparison only, as it has been established that "High" is the appropriate classification.

It is noted that no information was provided on additional discharge features, such as the inlet pipe, and as such these features have not been incorporated into the analysis.

11.5.4 Wind and Wave Analysis

In accordance with the 2007 CDA Guidelines, the freeboard at all dam structures should be evaluated for normal and extreme conditions. In general, the crest level of an embankment structure should be set so that the structure is protected against the most critical of the following cases (CDA 2007):

- No overtopping by 95% of the waves caused by the most critical wind with a frequency of 1/1,000 years when the reservoir is at its maximum normal elevation; and
- No overtopping by 95% of the waves caused by the most critical wind when the reservoir is at its maximum level during the passage of the IDF. For a "High", "Very High" or "Extreme" classification dam the critical wind is one with a 2-year return period.

A wind and wave analysis was performed to determine the freeboard available. A frequency analysis of hourly wind data for the Nanaimo A climate station was conducted, as it is the closest climate station with a suitable record period of wind data. The winds blowing from the north, northeast and northwest were used for the wave analysis, since these winds travel directly towards the upstream face of the dam along the longest possible fetch. An extreme event analysis using the methods described by Goda (1988) was used to calculate the wind speed for various return periods from the 59-year time series data from 1954 to 2013. Goda's extreme event analysis uses a partial duration or peak-over-threshold data series. Primary and secondary thresholds of 6.1 m/s and 8.1 m/s for the Nanaimo A data were selected, and the statistical distributions describing the extreme value analysis were produced. The best fit distributions were chosen to estimate the design event. Table 11.5.b shows the Goda extreme event analysis results. The hourly wind data for Nanaimo A climate station is included on a wind rose plot in Figure 11.5e.

Table 11.5.b Goda Extreme Event Wind Speed Analysis Results

Return Period (y)	Wind Speed (m/s)	Wind Speed (km/h)
2	10.4	37.3
1000	17.3	62.3

Setup and wave height were calculated using the 2-year and 1,000-year wind values. The wind tide or setup is a phenomenon in which the water level at the dam rises due to the effect of wind blowing over the water. The setup was calculated using the following equation (Smith, 1985):

$S = FV^2/63000D$

Where:

S = Wind tide or setup in m

F = Fetch in km

V = Wind velocity in km/h

D = Average reservoir depth in m.

The wave height was calculated using the following equation (Smith, 1985):

$Hw = 0.00513V^{1.06}Fe^{0.56}$

$Fe = KL$

Where:

Hw = Wave height in m

Fe = Effective fetch in km

L = Maximum straight unobstructed water length facing the dam in km

K = Fetch correction factor based on the relation between the average lake width and L.

The wave run-up is the height above the mean water surface at which a crest of a wave will interact with the barrier. The wave run-up is based on the design wave height. A factor of 1.37 was applied to the calculated significant wave height to obtain the design wave height, which is the average of the highest 5% waves, as recommended by the CDA for freeboard calculations. Based on the upstream embankment slope (3H:1V) and the dam surface material, the ratio of run-up to the design wave height was determined to be 1.35 (Smith, 1985).

As discussed earlier, the ‘normal freeboard’ is the difference between the normal elevation (chosen in this case to be the full supply level, i.e. the spillway weir crest elevation) and the dam crest elevation. The ‘minimum freeboard’ is the difference between the peak lake level during the passage of the IDF and the dam crest.

The results of the wave analysis and freeboard assessment for the two design scenarios are summarized in Table 11.5.c.

Table 11.5.c Summary of Setup, Run-up and Freeboard Estimates

Parameter Scenario	Stocking Lake Dam	
	Lake Level at Normal Elevation	Lake Level during Passage of IDF (High)
Wind Frequency	1,000-yr	2-yr
Wind Setup (m)	0.006	0.002
Wave Run-up (m)	0.56	0.33
Required Freeboard for Setup + Run-up (m)	0.57	0.33
Available Freeboard from Still Water Level (m)	1.30	0.59
Available Freeboard During Design Event (m)	0.73	0.26
Acceptable according to CDA Guidelines	Yes	Yes

11.5.5 Freeboard Assessment

The flood routing exercise and the wind and wave analysis described above determine that the available freeboard during the passage of the IDF event is 0.26 m, when wind setup and wave run-up are considered, in other words the dam would not overtop.

12. Dam Safety Management System

12.1 General

Dam safety management can be generally described in terms of five components (CDA Guidelines 2007):

- Owner commitment to safety;
- Regular inspections and Dam Safety Reviews with proper documentation and follow up;
- Implementation of effective Operations, Maintenance and Surveillance (OMS) practices;
- Preparation of effective Emergency Preparedness Plan; and
- Management of Public Safety.

A general schematic of a dam safety management system is presented in Figure 12.1. Ecora has assessed the dam safety management system in place for the Stocking Lake Dam and the results of this assessment are presented in this section.

12.2 Operations, Maintenance and Surveillance Manual

An Operations, Maintenance and Surveillance (OMS) Manual is a means to provide both experienced and new staff with the information they need to support the safe operation of a dam (CDA 2007). It is Ecora's understanding that currently Stocking Lake Dam does not have an Operation, Maintenance and Surveillance Manual.

12.3 Dam Emergency Plan

The objective of a Dam Emergency Plan (DEP) is to establish a formal internal document that operators of a dam should follow in the event of an emergency at the dam. The DEP outlines the key emergency response roles and responsibilities, in order of priority, as well as the required notifications and contact information. The DEP also provides basic information that allows for the planning and coordination by municipalities, Royal Canadian Mounted Police, Provincial agencies, utility owners, transportation companies and other parties that would be affected by a major flood (CDA 2007). The DEP is intended to combine the requirements of both the Emergency Response Plan (ERP) and Emergency Preparedness Plan (EPP) based on the BC Dam Safety Regulation (40/2016).

It is Ecora's understanding that currently Stocking Lake Dam does not have a DEP.

12.4 Public Safety Management

The CDA released Guidelines for Public Safety around Dams in 2011. Public safety around dams is an emerging topic in the dam safety community around the world, which in Canada is led by the CDA.

Dam owners are responsible for managing the public safety risks caused by a dam, as far upstream and downstream as the owner has property rights. Beyond the property the dam owner may have additional responsibilities to assess specific locations where the hazards are known by the owner to result directly from the dam or its operation and to inform the public and other affected property owners of these hazards. In most jurisdictions in Canada, due diligence is the test that the dam owner has taken reasonable and prudent precautions to protect the public. The implementation of a Public Safety Plan (PSP), records of decisions made, and activities performed to manage public safety at the dam, provide evidence of due diligence (CDA 2011).

During Ecora's inspection of Stocking Lake Dam it was noted that there is limited restriction on public interaction with the dam, with some evidence of ground disturbance or vandalism noted.

Currently there is no PSP in place for this facility and given that Stocking Lake is utilised recreationally, public interaction with the dam is anticipated and therefore a PSP should be developed for this facility.

12.5 Dam Safety Expectations Assessment

12.5.1 General

The British Columbia Ministry of Forests, Lands, Natural Resource Operations & Rural Development (MFLNRORD) has developed a sample check sheet of Dam Safety Expectations, Deficiencies and Priorities (May 2010) which is based on the BC Hydro Hazards and Failures Modes Matrix and the 2007 CDA Guidelines. A dam safety expectations assessment has been undertaken for Stocking Lake Dam using the sample check sheet prepared by the MFLNRORD as presented in Appendix H.

The Dam Safety Expectations are divided into five categories:

- Dam Safety Management System
- Operations, Maintenance and Surveillance
- Emergency Preparedness
- Dam Safety Review
- Dam Safety Analysis

A brief summary of the results of the Dam Safety Expectations is discussed below.

12.5.2 Dam Safety Analysis

There is one actual deficiency and two non-conformances, namely:

- Limited inspection and operational records are available;
- Based on potential loss of life and economic consequences in the inundation zone it is recommended to increase the dam consequence classification to "High"; and,
- The dam is homogeneous and therefore susceptible to internal erosion.

12.5.3 Operations, Maintenance and Surveillance

There are no deficiencies in this category.

There are sixteen non-conformances in this category, eight of which could be resolved by preparing an OMS Manual and DEP for this facility. The other non-conformances can be resolved by improving or maintaining documentation of training, and maintenance and testing of equipment on site.

12.5.4 Emergency Preparedness

There are ten non-conformances in this category, all of which could be addressed by preparing an OMS Manual and DEP for this facility and undertaking an emergency exercise and training of staff involved.

12.5.5 Dam Safety Review

There are no deficiencies and non-conformances in this category. By commissioning this Dam Safety Review, the Cowichan Valley Regional District conforms to the dam safety expectations for this category.

12.5.6 Dam Safety Management

There are six non-conformances, all of which could be addressed by preparing an OMS Manual and DEP for this facility.

13. Risk Assessment

13.1 General

As part of the DSR, the NDMP Risk Assessment Information Template (RAIT) was completed in accordance with the NDMP and is attached in Appendix I. The assessment process allows stakeholders to identify and prioritize the risks that are likely to create the most disruption to them. The assessment also helps decision-makers to identify and describe hazards and assess impacts and consequences based upon the vulnerability or exposure of the local area or its functions to that hazard.

The risk assessment approach aims to understand the likely impacts of a range of emergency scenarios upon community assets, values and functions. As such, risk assessments provide an opportunity for multiple impacts and consequences to be considered enabling collaborative risk treatment plans and emergency management measures to be described.

The outputs of the assessment process can be used to better inform emergency management planning and priority setting, introduce risk action plans, and ensure that communities are aware of and better informed about hazards and the associated risks that may affect them.

13.2 Risk Assessment Information

Descriptions of the risk ranking, and definitions associated with the five-point scale used to define the impacts are presented below. The impact risk rating definitions are based on qualitative and quantitative elements referenced from a diverse array of risk and resilience methodologies and external risk management models.

People and Societal Impacts

It is a priority at the municipal, provincial and federal levels to protect the health and safety of Canadians. Impacts on people are considered pertinent in the assessment process given that natural hazards can result in significant societal disruptions such as evacuations and relocations as well as injuries, immediate deaths, and deaths resulting from unattended injuries or displacement. As such, the following impact criteria will be assessed on a 1 to 5 scale:

- number of fatalities;

- ability for local healthcare resources to address injuries; and,
- number of individuals displaced and duration of displacement.

Environmental Impacts

A priority for municipal, provincial and federal governments is to protect Canada's natural environment for current and future generations. As such, environmental impacts were included in the assessment to measure the risk event in relation to the degree of damage and predicted scope of clean-up and restoration needed following an event. The definitions consider the direct and indirect environmental impacts within the defined geographic area on a 1 to 5 scale, and include an assessment of air quality, water quality and availability (exclusive to on land and in-ground water), and various other nature indicators.

Local Economic Impacts

There may be impacts on the local economy that are the result of a risk event occurring. Local economic impacts attempt to capture the value of damages or losses to local economically productive assets, as well as disruptions to the normal functioning of the community/region's local economic system. The definitions consider the local economic impacts within the defined geographic area on a 1 to 5 scale and should consider direct and indirect economic losses (e.g. productivity losses, capital losses, operating costs, financial institutions and other financial losses).

Local Infrastructure Impacts

There are several local infrastructure components, as per a variety of risk assessment and management sources and guidelines that are fundamental to the viability and sustainability of a community/region. Those components that appear most pertinent to assess impacts resulting from natural hazards, such as floods, include: energy and utilities; information and communication technology; transportation; health, food and water; and safety and security. At a minimum, an assessment of the aforementioned components must be completed, defined on a 1 to 5 scale, and should consider both direct and indirect impacts.

Public Sensitivity Impacts

Public sensitivity was included as an impact criterion given that credibility of governments is founded on the public's trust that all levels of government will respond effectively to a disaster event. The definitions consider the impacts on public visibility on a 1 to 5 scale and include an assessment of public perception of government institutions, and trust and confidence in public institutions.

13.3 Risk Assessment Summary

From the impact categories considered, the following was noted:

- The primary risk event is the breach of Stocking Lake Dam due to internal (piping) erosion caused by a 1 in 70-year event.
- In the event of dam breach, significant damage to public infrastructure would occur including damage to the following:
  - The TransCanada Highway;
  - Water mains servicing the Town of Ladysmith and community of Saltair;
  - Forest service roads;
  - S Watts Road;

  - Chemainus Road Bridge over Stocking Creek;
  - Fortis natural gas transmission line;
  - Stocking Lake Creek Park; and
  - Southern Vancouver Island Railway;
- The event would most likely occur in the spring freshet period when the lake levels and hydrostatic pressures within the dam are higher.

13.4 Confidence Levels

The risk assessment process requires confidence levels to be defined, particularly since confidence levels can vary considerable depending on the availability of quality data, availability of relevant expertise to feed the risk assessment process, and the existing Canadian body of knowledge associated with specific natural hazards and natural disaster events.

Confidence levels have been defined using letters ranging from A to E, where ‘A’ is the highest confidence level and ‘E’ is the lowest. This approach was taken to ensure all applicants can determine the confidence in their risk assessment in a simplified, straightforward manner, which also ensures that a more consistent representation of confidence levels is being determined across all submissions.

The level of confidence for this assessment is considered to be “B”, based on the level of assessment completed to date.

14. Observations and Conclusions

The conclusions reached during the DSR of Stocking Lake Dam are presented as follows for each area of review:

14.1 Background Review

- The dam was originally constructed in 1902 and last modified in 1966.
- No obvious signs of historical or current slope instability of the reservoir sides slopes were observed in the review of available aerial photographs.

14.2 Site Reconnaissance

- The dam access road is in poor condition and is only suitable for four-wheel drive vehicles.
- Seepage has been observed exiting the downstream face of the dam and a sinkhole is present near the right abutment of the dam.
- The saturated backfill above the water main trench downstream of the dam suggests that preferential seepage is occurring through the dam along the low level outlet conduit.
- The log boom is constructed out of encapsulated foam which has been reported by the CVRD to be ineffective at preventing debris from entering the spillway during storm events.

14.3 Consequence Classification Review

- The dam breach inundation mapping indicates that a total area of approximately 1.05 km^2 would be flooded in the event of a dam breach, potentially impacting S Watts Rd, Highway 1, and water mains servicing Saltair and the Town of Ladysmith.
- Dam breach analysis and inundation mapping results confirmed that Stocking Lake Dam should have a "High" consequence classification. The CDA guidelines recommend an Inflow Design Flood (IDF) for a "High" consequence dam of ⅓ of the way between a 1,000-year flood and a Probable Maximum Flood (PMF).

14.4 Failure Mode Assessment

- The plausible failure modes of the dam are; overtopping as the spillway may become blocked with debris, embankment failure due to earthquake loading and internal erosion through the embankment, its foundation or along the low level outlet conduit.

14.5 Geotechnical Assessment

- The boreholes advanced as part of the geotechnical investigation of the dam indicated that the dam is founded on till-like material and bedrock.
- Results of the static stability analysis indicated that the embankment meets CDA criteria for normal loading conditions for any potential slip surfaces that would impact the dam freeboard.
- Preliminary post seismic liquefaction deformation analyses of the dam indicate that sufficient freeboard would be lost resulting in an overtopping of the dam due to a 1 in 475-year earthquake corresponding to a NDMP likelihood rating of 3.
- A probabilistic piping risk assessment was conducted using the UNSW method, which resulted in a calculated probability of piping failure of 1.4 x 10-2 (1 in 70 years) corresponding to a NDMP likelihood rating of 3.

14.6 Hydrotechnical Assessment

- The peak inflow to Stocking Lake Dam during the IDF for a “High” consequence dam was determined to be 20.2 m^3/s.
- The spillway has adequate capacity to pass the routed inflow design flood.
- The dam should have freeboard such that 95% of the waves do not overtop the dam crest during a 1,000-year wind event under normal lake level conditions or during a 2-year wind event under inflow design flood conditions (IDF). Stocking Lake Dam has been shown to exceed these requirements in both scenarios with freeboards of 0.73 m and 0.26 m respectively, in excess of what is required.

14.7 Dam Safety Management

- An Operation, Maintenance and Surveillance Manual and a Dam Emergency Plan need to be prepared for Stocking Lake Dam.

14.8 Risk Assessment

- Preliminary post seismic liquefaction deformation analyses of the dam indicate that sufficient freeboard would be lost resulting in an overtopping of the dam due to a 1 in 475-year earthquake corresponding to a NDMP likelihood rating of 3.
- A probabilistic piping risk assessment was conducted using the UNSW method, which resulted in a calculated probability of piping failure of 1.4 x 10-2 (1 in 70 years) corresponding to a NDMP likelihood rating of 3.
- A wind-wave analysis indicated that an event greater than the IDF coupled with extreme wind could lead to dam failure by overtopping corresponding to a NDMP likelihood rating of 1.
- A preliminary estimate of reconstruction costs as a result of a dam breach is between $3 million and $30 million based on the scope of the infrastructure impacted.

15. Recommendations

The recommendations that have been developed during this DSR of Stocking Lake Dam are presented as follows for each area of review. Priorities (Low, Medium, High or Very High) are given in parentheses. Low, medium, high and very high priority recommendations should be addressed within 5, 3, 1 and 0.5 year(s) respectively.

15.1 Background Review

- There are no recommendations in this area of review.

15.2 Site Reconnaissance

- The condition of the dam access road should be improved in accordance with the BC MFLNRORD Engineering Manual (2018) minimum specifications or similar standard to allow two-wheel drive vehicle access in the event of an emergency, or alternatively all emergency responders should be advised that they will require four- wheel drive vehicles (High).
- The preferential seepage of water through the dam along the low level outlet should be addressed during the remediation or replacement of the dam (High).
- The current log boom should be replaced with one that is effective at capturing debris under both normal and storm conditions (High).
- The importance of regular monitoring of the seepage clarity and rate of seepage when the risk of piping exists is underlined by Foster et. Al (2000b) study. Weekly documented monitoring of the “sinkhole” present near the right abutment of the dam noting observations of any leakage and turbidity of the water along the toe of the dam be undertaken during site surveillance activities until remedial works have been constructed. This should include estimating rates and clarity of seepage, along with taking photographs as comparisons may need to be made between future and past conditions (Very High).
- A filter buttress should be design and placed over the “sinkhole” present near the right abutment until remediation of the existing dam or the construction of a new dam has been completed (High).

15.3 Consequence Classification

- Based on the estimated potential loss of life and economic losses within the dam break flood inundation area it is recommended that the consequence classification of Stocking Lake Dam be increased from “Significant” to "High". However, any decision to modify the consequence classification rating must be confirmed by the BC MFLNRORD Dam Safety Section (Very High).

15.4 Failure Mode Assessment

- There are no recommendations in this area of review.

15.5 Geotechnical Assessment

- CVRD should commission a design study to address the major deficiencies in the Stocking Lake Dam, namely its susceptibility to liquefaction under the design seismic event and its susceptibility to piping. It is envisioned this would result in a recommendation to either remediate the existing dam or the construction of a new dam immediately downstream (Medium).

15.6 Hydrotechnical Assessment

- There are no recommendation in this area of review.

15.7 Dam Safety Management

- An Operation, Maintenance and Surveillance Manual and a Dam Emergency Plan need to be prepared for Stocking Lake Dam (High).

15.8 Risk Assessment

- Should the CVRD wish to proceed with a NDMP funding application to remediate or replace Stocking Lake Dam they should undertake a more detailed cost estimate of infrastructure that would be impacted in the event of a dam breach (High).

16. Dam Safety Review Assurance Statement

In accordance The Association of Professional Engineers and Geoscientists of BC (APEGBC) Professional Practice Guidelines – Legislated Dam Safety Reviews in BC V3.0 (October 2016) we have completed a Dam Safety Review Assurance Statement, which is presented in Appendix J.

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Figure 12.1 Dam Safety Management System

SITE LOCATION & ACCESS ROUTE

DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

- 🟥 Dam Locations
- ═ Highways
- — Streams
- — Digital Road Atlas Roads
- — Access Route

LOCATION MAP

1:20,000

0 250 500 750 1,000 Meters

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/09/21
Drawn: JS Check: AG

Figure 1.2

BEDROCK GEOLOGY

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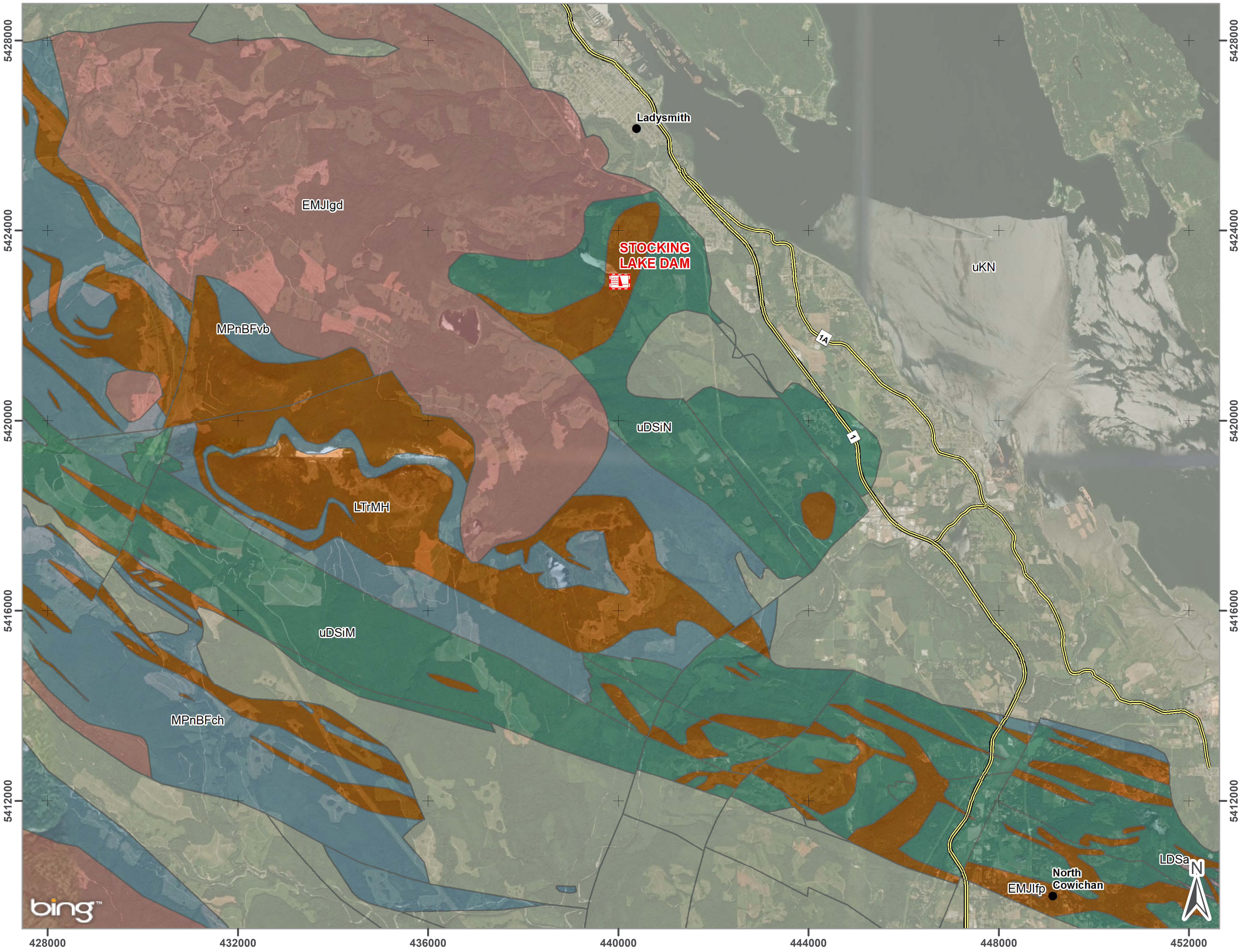
DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC



Legend

- ● Cities
- Dam Locations
- Highways
- Streams

Bedrock Geology

- 🟧 EMJIfp - Feldspar porphyry, hornblende porphyry, augite porphyry, dacite, basalt (92B, C, F).
- 🟥 EMJlgd - Granodiorite, quartz diorite, quartz monzonite, diorite, agmatite, feldspar porphyry, minor gabbro and aplite (170 - 185 Ma).
- ⬜ LDSa - Granodiorite, feldspar porphyry, quartz-feldspar porphyry, coeval with McLaughlin Ridge Formation (365 -360 Ma) (92B).
- 🟧 LTrMH - Gabbro, diabase, feldspar diabase, glomeroporphyritic diabase and gabbro, minor diorite (215 - 230 Ma). Coeval with Karmutsen Formation.
- 🟦 MPnBFch - Ribbon chert, cherty tuff, graphitic argillite, thinly bedded intercalated sandstone-siltstone-argillite, volcanic sandstone and conglomerate, interbedded argillite and crinoidal limestone, massive and pillowed basalt with intercalated cherty sed
- 🟦 MPnBFvb - Mafic volcanics
- 🟩 uDSiM - Thickly bedded tuffite and lithic tuffite, breccia, tuff, feldspar and quartz-feldspar crystal tuff, lapilli tuff, rhyolite, dacite, laminated tuff, jasper, chert, hematite-chert iron formation (92B, C, F).
- 🟩 uDSiN - Pyroxene-feldspar phyric agglomerate, breccia, lapilli tuff, massive and pillowed flows, massive tuffite, laminated tuff, jasper and chert (92B, C, F)
- ⬜ uKN - Boulder, cobble and pebble conglomerate, coarse to fine sandstone, siltstone, shale, coal (Santonian to Maastrichtian).). Includes BENSON, COMOX, HASLAM, EXTENSION, PENDER, PROTECTION, EAST WELLINGTON, TRENT RIVER, CEDAR DISTRICT, DE COURCY, DE

1:80,000

0 1 2 3 4 km

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/09/21
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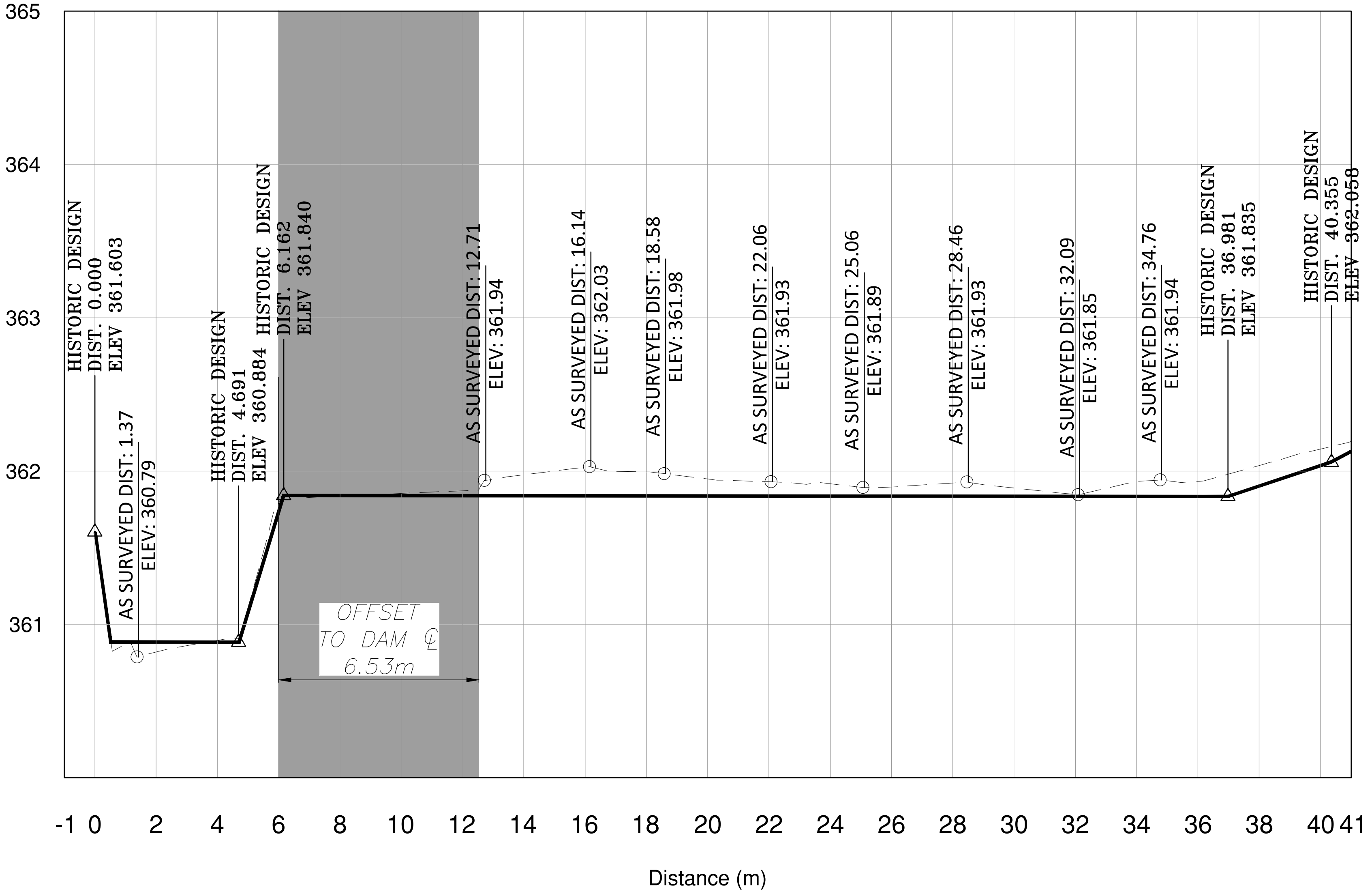
Figure 3.5



Notes:

1. Historic design elevations derived by assuming Design Datum 0.0 m represents Geodetic Datum of 260 m. Adopted from Drawing No. 4984-12A, supplied by client.

DAM SAFETY REVIEW OF STOCKING LAKE DAM

DAM LEVELLING SURVEY PROFILE

Project No.:	GK-18--020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	1:200
Date:	SEPT. 24, 2018
DWN:	RERH CHK: CE

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Figure 4.3

INVESTIGATION LOCATION PLAN

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DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

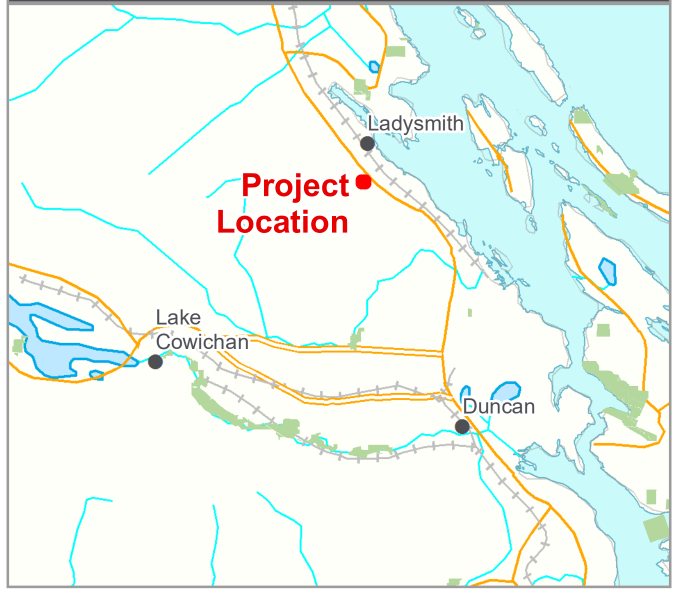
Legend

- ⊕ TT-EBA Boreholes (2008)
- ⊕ Ecora Boreholes (2018)
- — TRIM Roads
- — Fresh Water Atlas Streams
- — 20m TRIM Contours
- — 100m TRIM Contours
- ┼┼┼ Stocking Lake Dam

Geophysics Profiles

- — MASW
- — Magnetometer

LOCATION MAP



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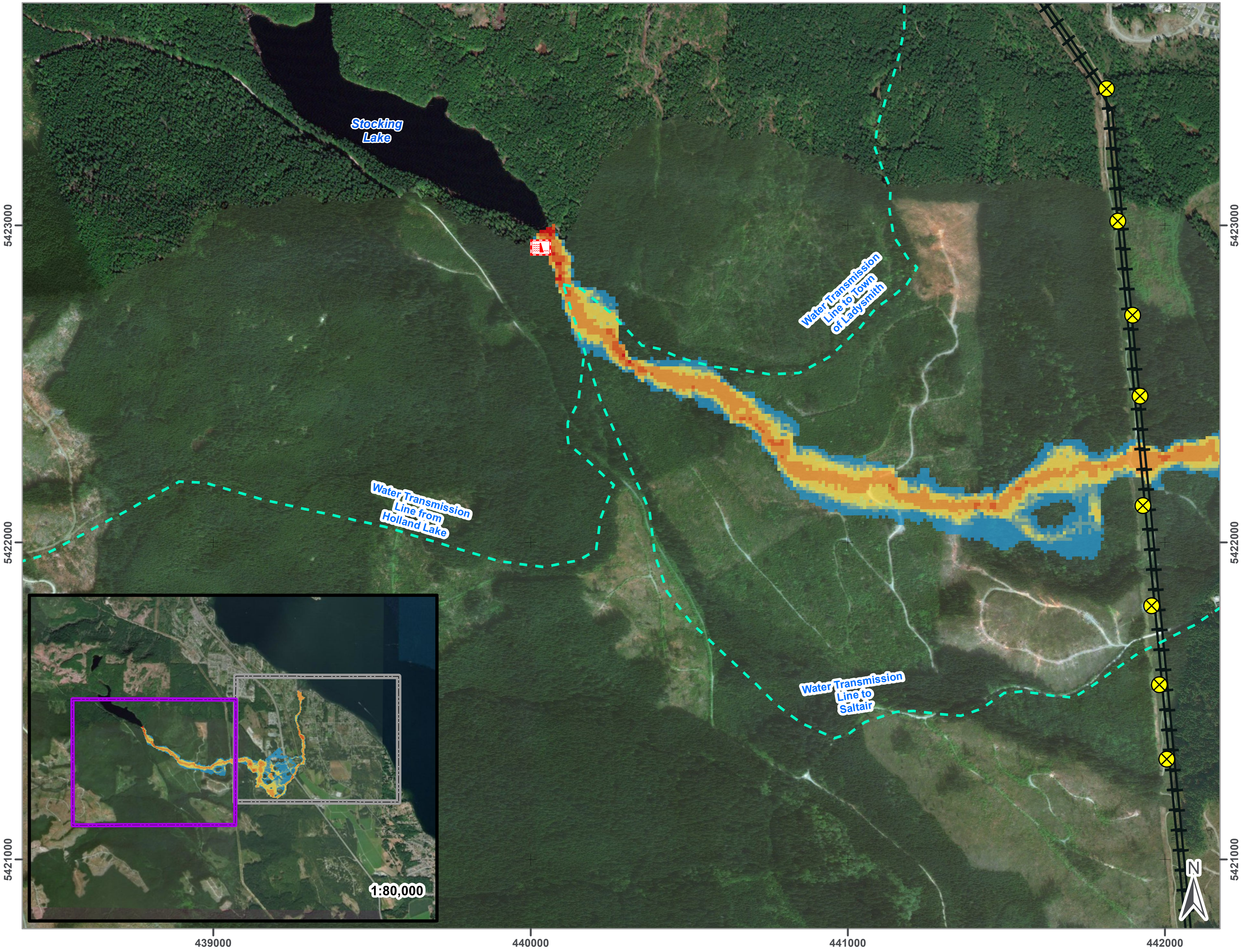
0 20 40 60 Meters

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/09/21
Drawn: JS Check: AG

Figure 5.1

EXTENT OF INUNDATION & MAXIMUM FLOW DEPTH



DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

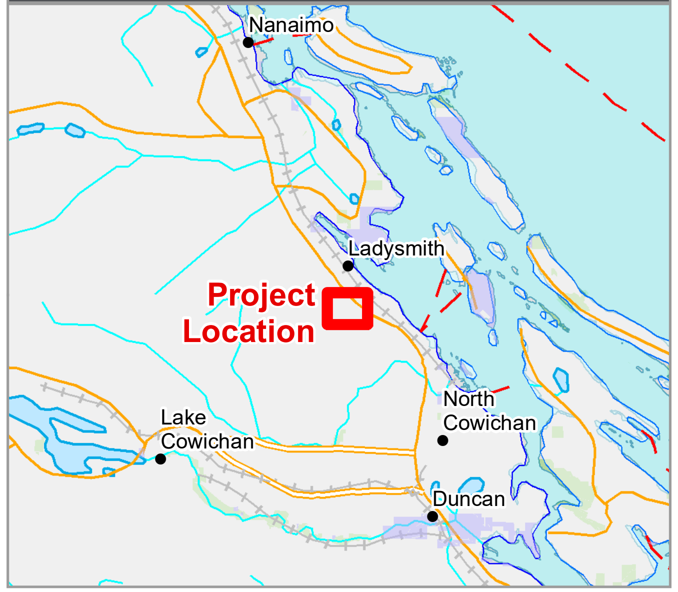
- ▣ Stocking Lake Dam Location
- ⊗ Transmission Towers
- ☗ Assumed Residence
- ☖ Other Building
- - - Water Transmission Line
- ┼ Transmission Lines

Maximum Flow Depth (m)

- 🟦 0.000 - 0.250
- ▨ 0.251 - 0.500
- 🟩 0.501 - 0.750
- ▨ 0.751 - 1.000
- 🟨 1.001 - 2.000
- 🟧 2.001 - 4.000
- 🟧 4.001 - 6.000
- 🟥 6.001 - 8.000

Total Area of Inundation = 1.05 km^2

LOCATION MAP



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Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/08/17
Drawn: MT Check: BF

Figure 7.0a

EXTENT OF INUNDATION & MAXIMUM FLOW DEPTH

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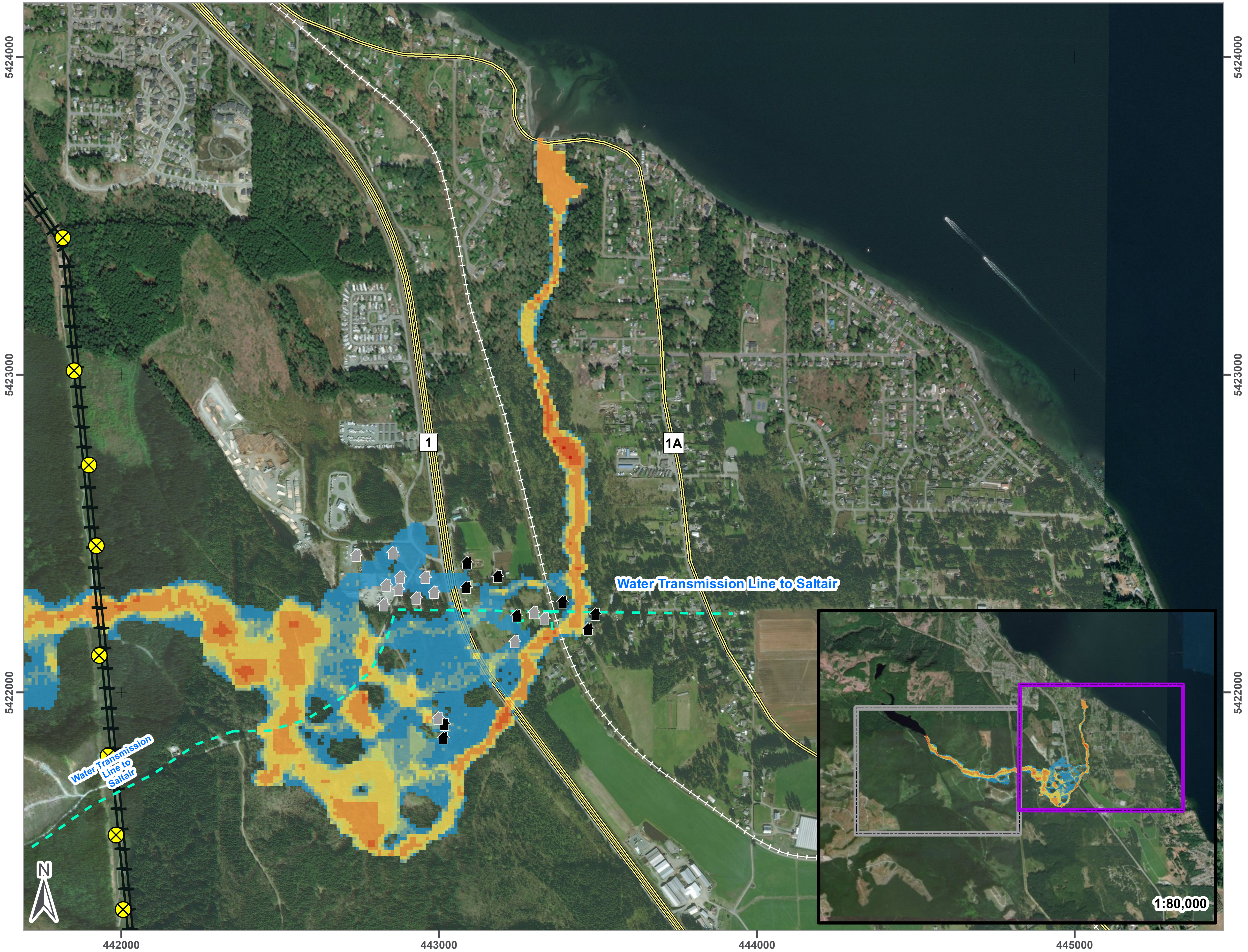


DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

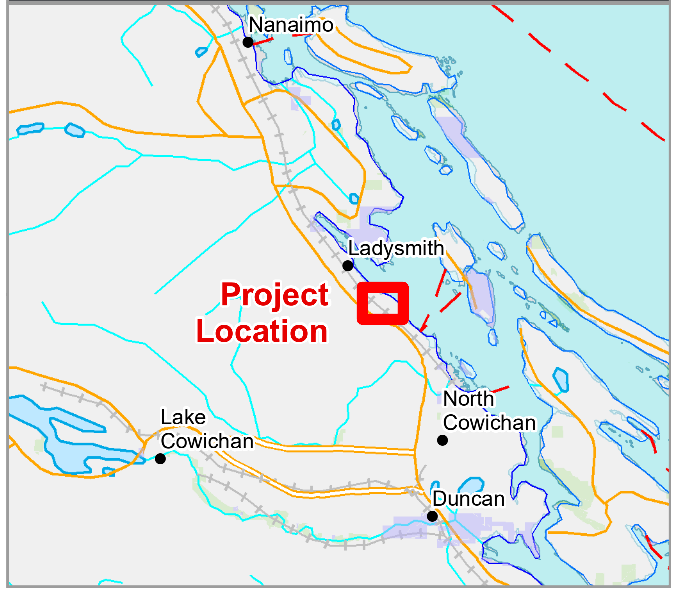
- ⊗ Transmission Towers
- ☗ Assumed Residence
- ☗ Other Building
- - - Water Transmission Line
- ┼ Transmission Lines
- ═ Highways

Maximum Flow Depth (m)

- 🟦 0.000 - 0.250
- ▨ 0.251 - 0.500
- 🟩 0.501 - 0.750
- 🟨 0.751 - 1.000
- 🟨 1.001 - 2.000
- 🟧 2.001 - 4.000
- 🟧 4.001 - 6.000
- 🟥 6.001 - 8.000

Total Area of Inundation = 1.05 km^2

LOCATION MAP



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Project No.: GK-18-020-CVD

Client: Cowichan Valley Regional District

NAD 1983 UTM Zone 10N

Date: 2018/08/17

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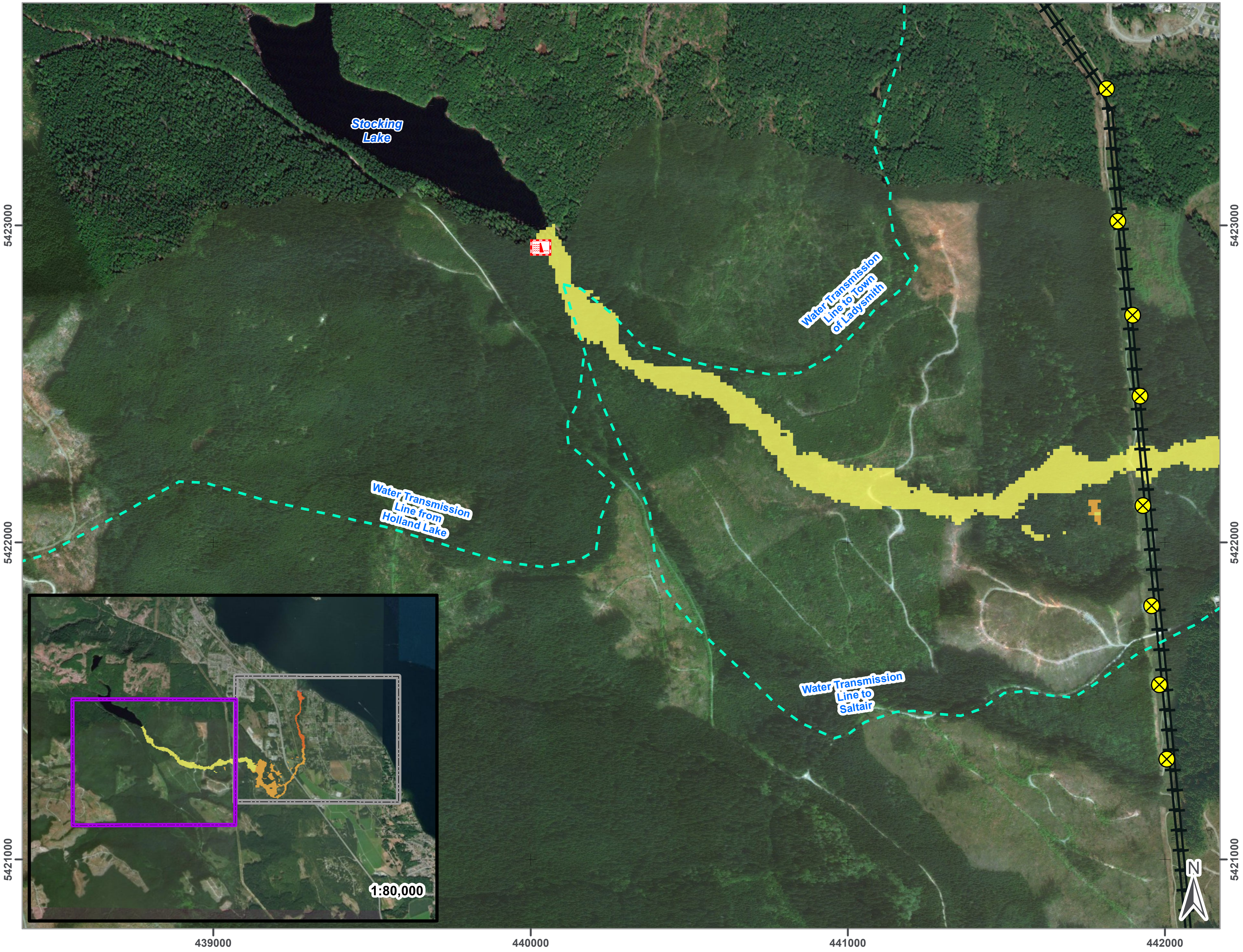
Figure 7.0b

TIME (HRS) FOR 0.6m FLOW DEPTH



DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

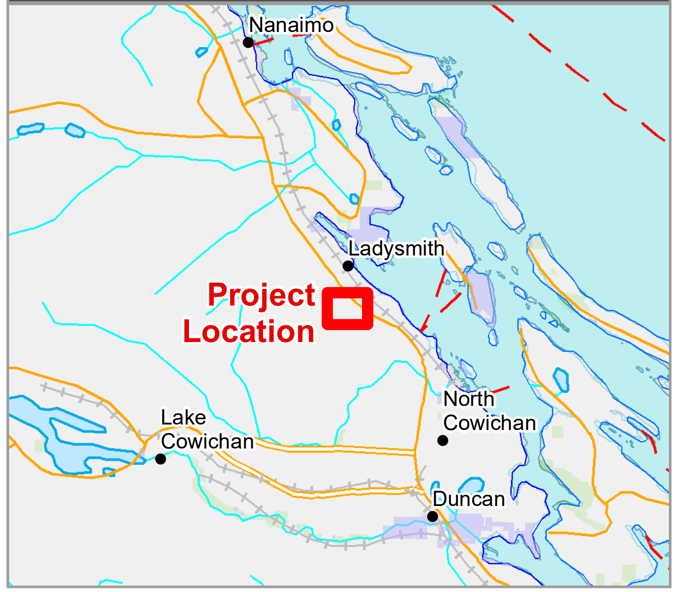
Legend

- ▣ Stocking Lake Dam Location
- ⊗ Transmission Towers
- ☗ Assumed Residence
- ☖ Other Building
- ‐ ‐ Water Transmission Line
- ┼ Transmission Lines

Time (hrs) for 0.6m Flow Depth

- 🟦 0.100 - 1.000
- 🟩 1.001 - 3.000
- 🟩 3.001 - 5.000
- 🟨 5.001 - 7.000
- 🟧 7.001 - 9.000
- 🟧 9.001 - 10.000
- 🟥 10.001 - 15.000

LOCATION MAP



1:12,000

Meters
0 100 200 300 400 500

Project No.: GK-18-020-CVD
Date: 2018/08/17
Client: Cowichan Valley Regional District
Drawn: MT Check: BF
NAD 1983 UTM Zone 10N

Figure 7.0c

TIME (HRS) FOR 0.6m FLOW DEPTH

ecora

DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

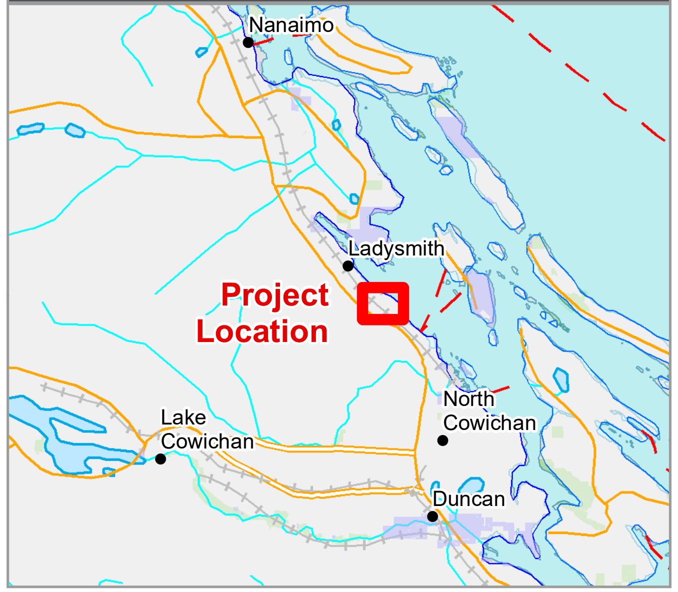
Legend

- ⊗ Transmission Towers
- ☗ Assumed Residence
- ☗ Other Building
- - - Water Transmission Line
- ┼ Transmission Lines
- ═ Highways

Time (hrs) for 0.6m Flow Depth

- 🟦 0.100 - 1.000
- ▨ 1.001 - 3.000
- 🟩 3.001 - 5.000
- 🟨 5.001 - 7.000
- 🟧 7.001 - 9.000
- 🟧 9.001 - 10.000
- 🟥 10.001 - 15.000

LOCATION MAP



1:12,000

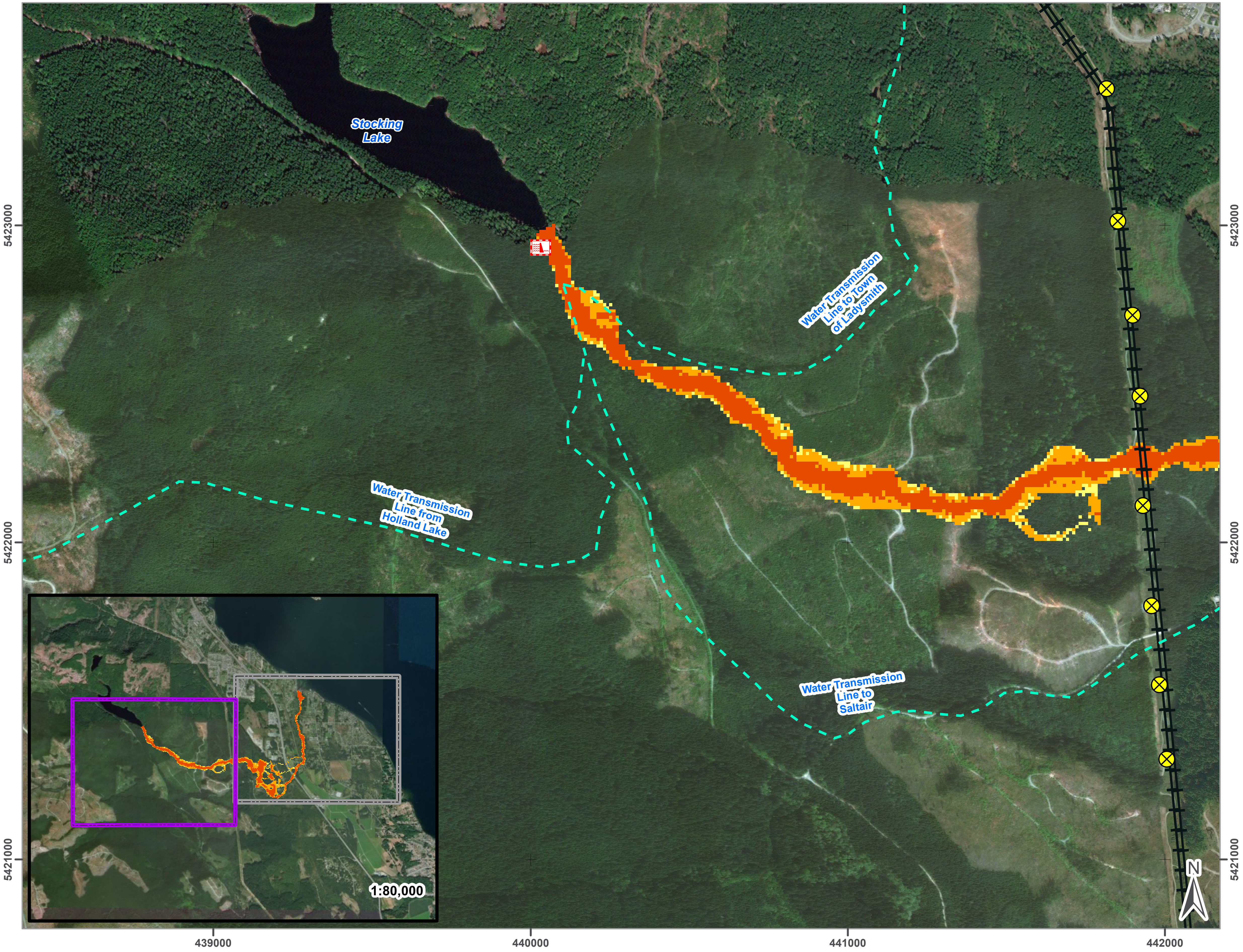
0 100 200 300 400 500 Meters

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/08/17
Drawn: MT Check: BF

Figure 7.0d

FLOOD HAZARD RATING



DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

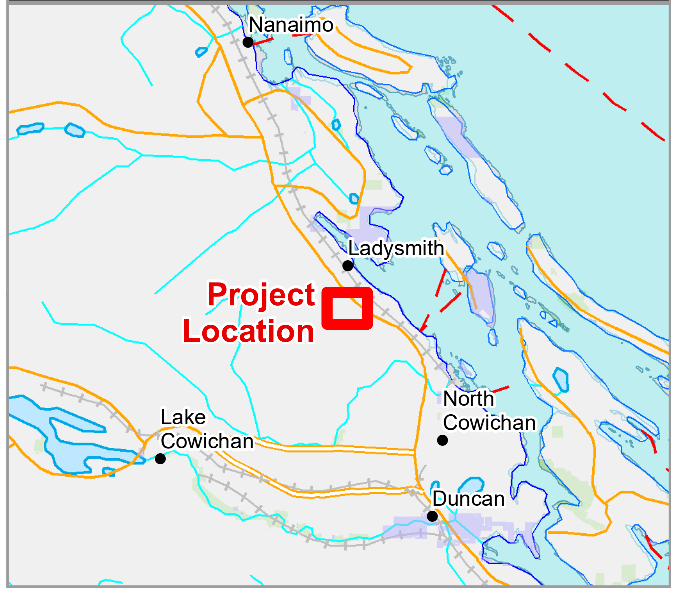
- Stocking Lake Dam Location
- ⊗ Transmission Towers
- Assumed Residence
- Other Building
- Water Transmission Line
- Transmission Lines

Flood Hazard Rating and Total Area of Flooding Downstream of Dam Spillway Point

- 🟨 Low (0.04 km ^{2})
- 🟧 Medium (0.25 km ^{2})
- 🟥 High (0.36 km ^{2})

Hazard Level	Description
High	Persons are in danger both inside and outside of buildings. Structures are at risk of being destroyed.
Medium	Persons are in danger outside of buildings. Structures may suffer damage and possible destruction depending on construction characteristics.
Low	Danger to persons is low or non-existent. Buildings may suffer little structural damage, however may undergo significant non-structural damage to interiors.
Reference: Garcia, et al., 2003, 2005	

LOCATION MAP



1:12,000

0 100 200 300 400 500 Meters

Project No.: GK-18-020-CVD
Date: 2018/08/17
Client: Cowichan Valley Regional District
Drawn: MT Check: BF
NAD 1983 UTM Zone 10N

Figure 7.0e

FLOOD HAZARD RATING

DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

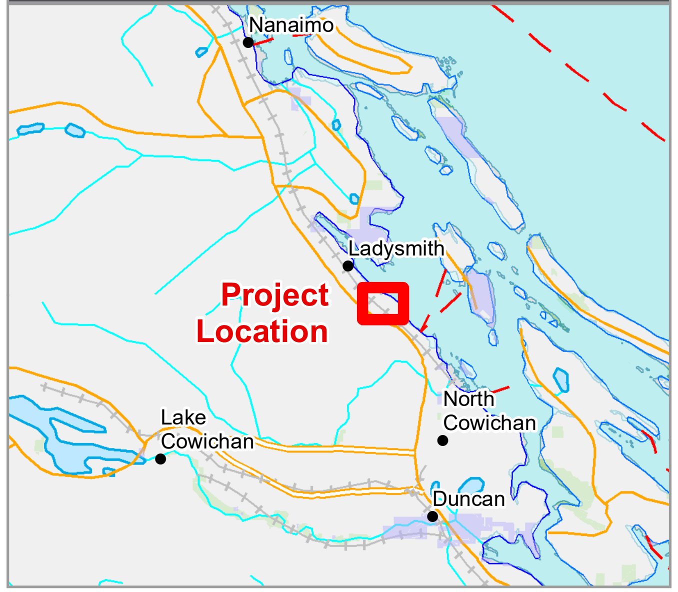
- ⊗ Transmission Towers
- ⌂ Assumed Residence
- ⌂ Other Building
- - - Water Transmission Line
- ┼ Transmission Lines
- ═ Highways

Flood Hazard Rating and Total Area of Flooding Downstream of Dam Spillway Point

- 🟨 Low (0.04 km ^{2})
- 🟧 Medium (0.25 km ^{2})
- 🟥 High (0.36 km ^{2})

Hazard Level	Description
High	Persons are in danger both inside and outside of buildings. Structures are at risk of being destroyed.
Medium	Persons are in danger outside of buildings. Structures may suffer damage and possible destruction depending on construction characteristics.
Low	Danger to persons is low or non-existent. Buildings may suffer little structural damage, however may undergo significant non-structural damage to interiors.
Reference: Garcia, et al., 2003, 2005	

LOCATION MAP



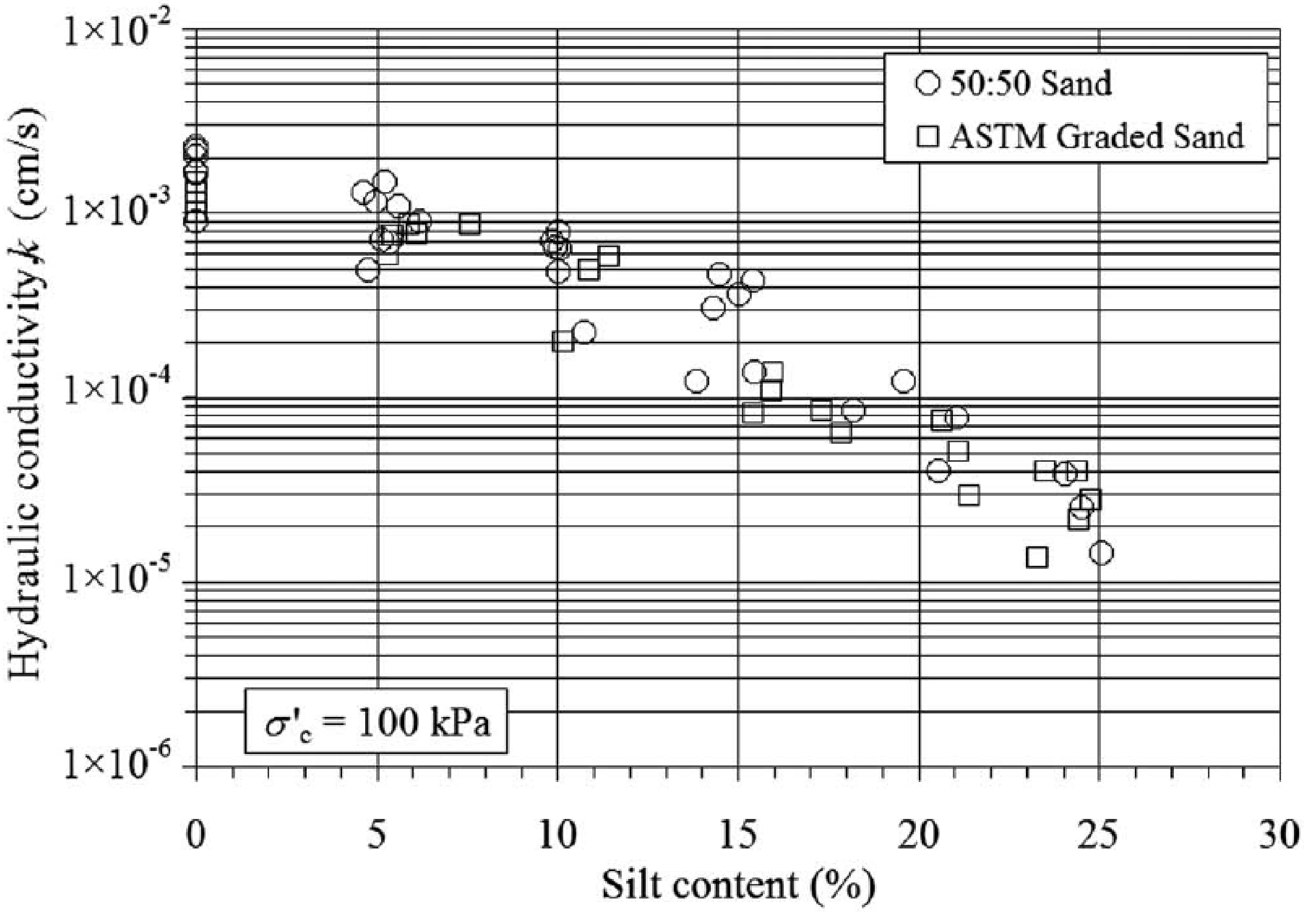
1:12,000

0 100 200 300 400 500 Meters

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/08/17
Drawn: MT Check: BF

Figure 7.0f



Notes:
Adapted from Figure 3 of Bandini et. al. (2009)

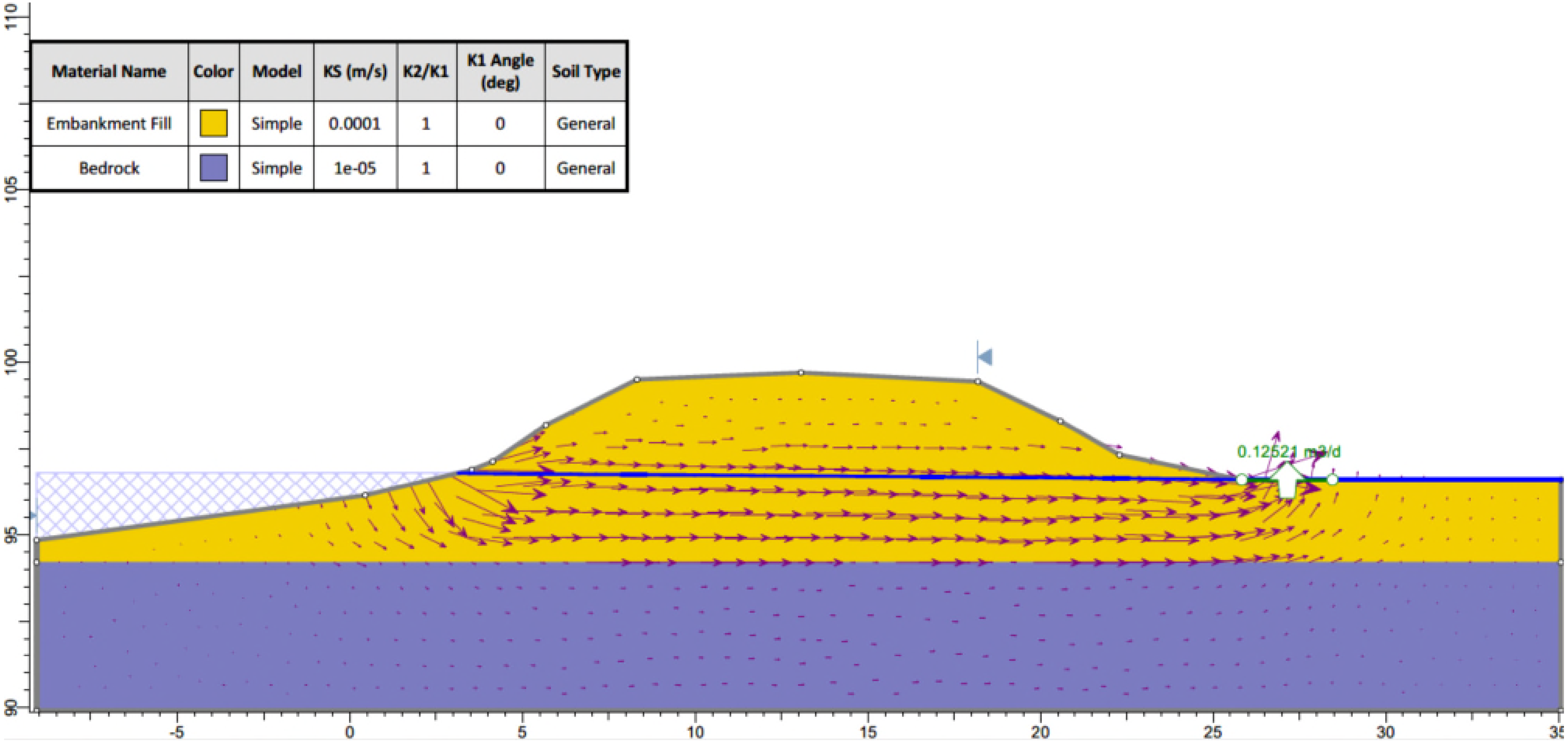
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Saturated Hydraulic Conductivities for Sand-Silt Mixtures with Various Silt Content

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	CE CHK: MJL

ecora

Figure 10.2



Material Name	Color	Model	KS (m/s)	K2/K1	K1 Angle (deg)	Soil Type
Embankment Fill	🟨	Simple	0.0001	1	0	General
Bedrock	🟪	Simple	1e-05	1	0	General

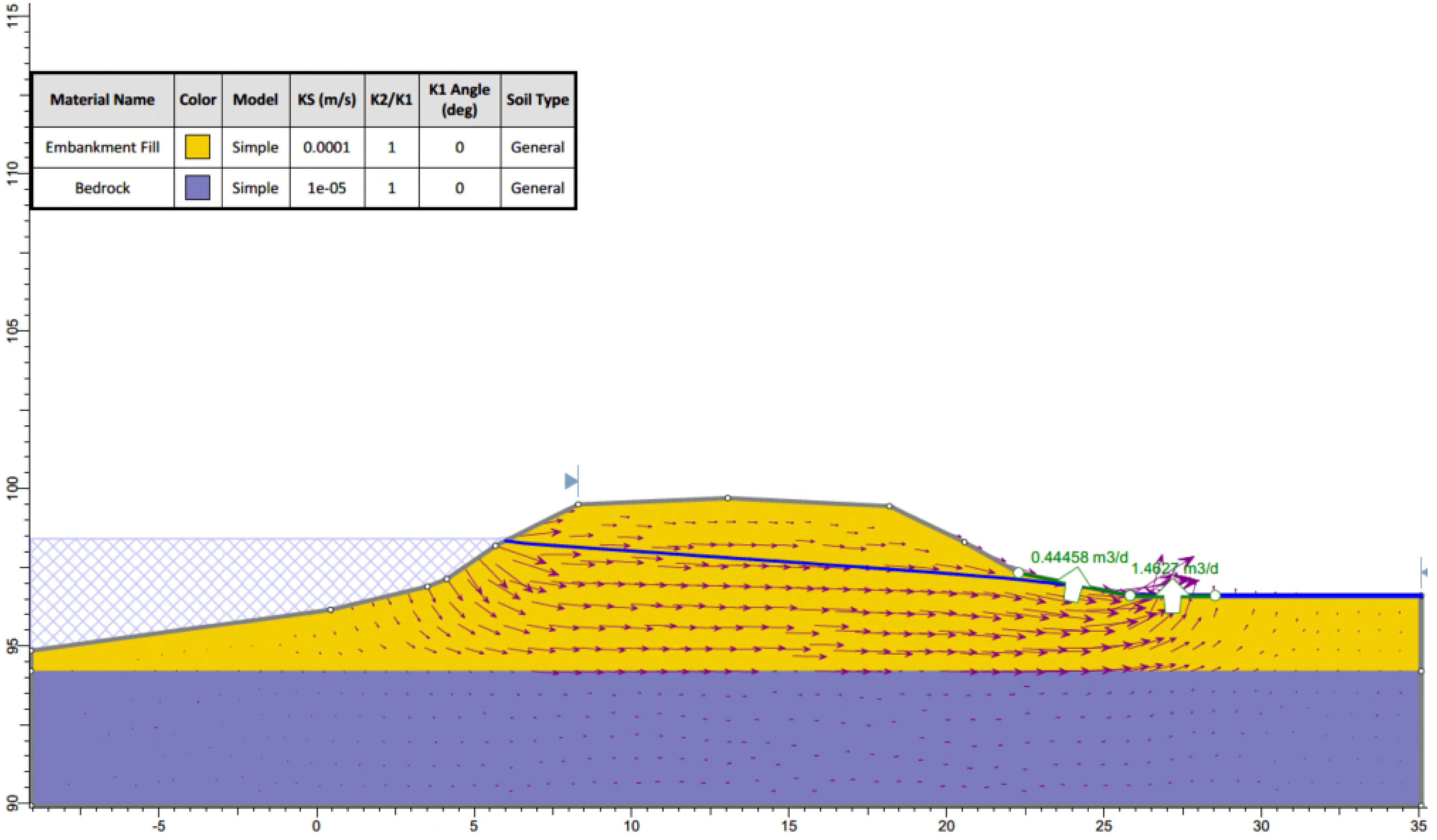
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Steady State Seepage Analysis: Reservoir Level at 359.0 m

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	CE CHK: MJL

Figure 10.3a



Material Name	Color	Model	KS (m/s)	K2/K1	K1 Angle (deg)	Soil Type
Embankment Fill	🟨	Simple	0.0001	1	0	General
Bedrock	🟪	Simple	1e-05	1	0	General

Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Steady State Seepage Analysis: Reservoir Level at 360.8 m

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

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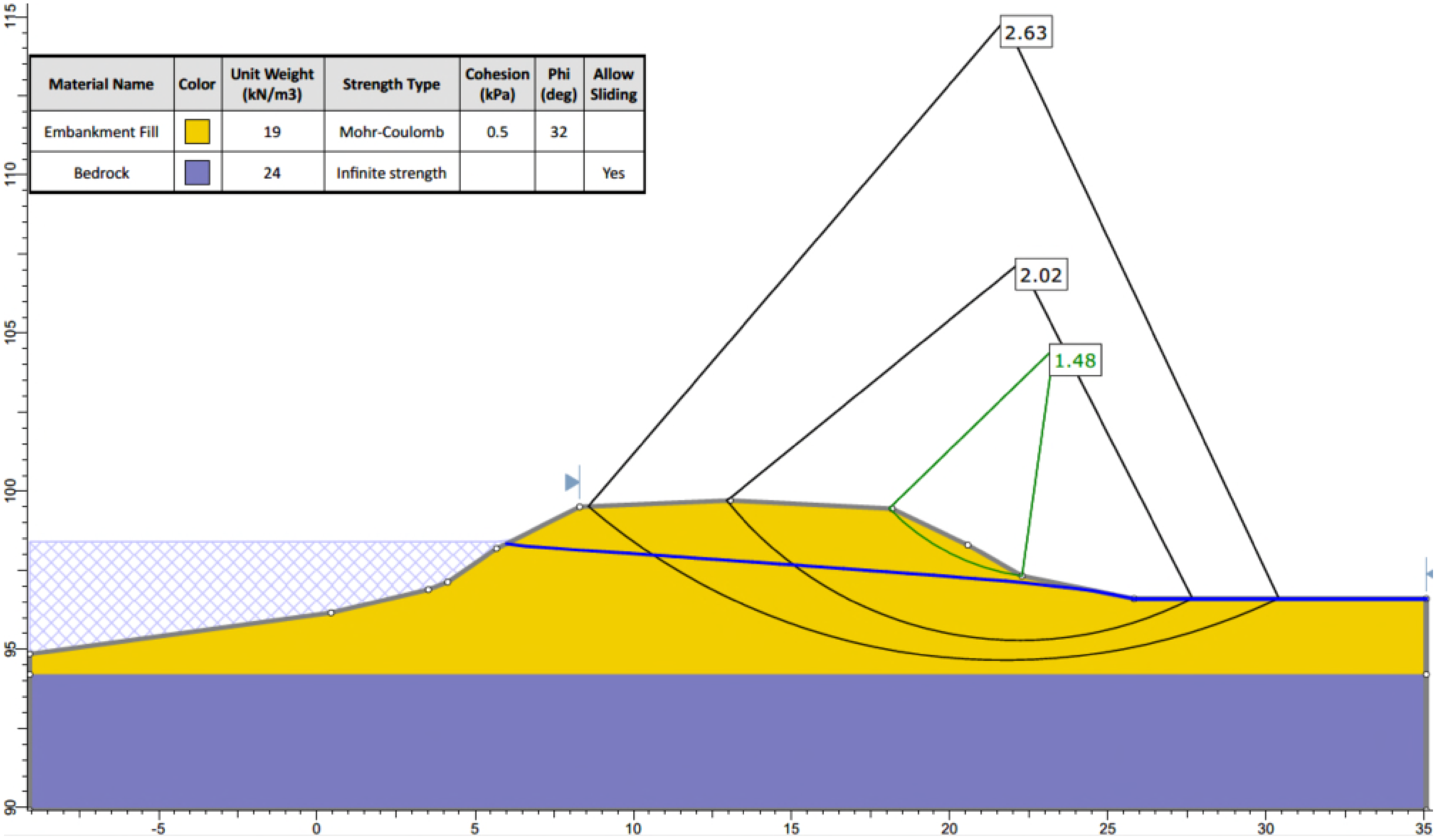
Figure 10.3b



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

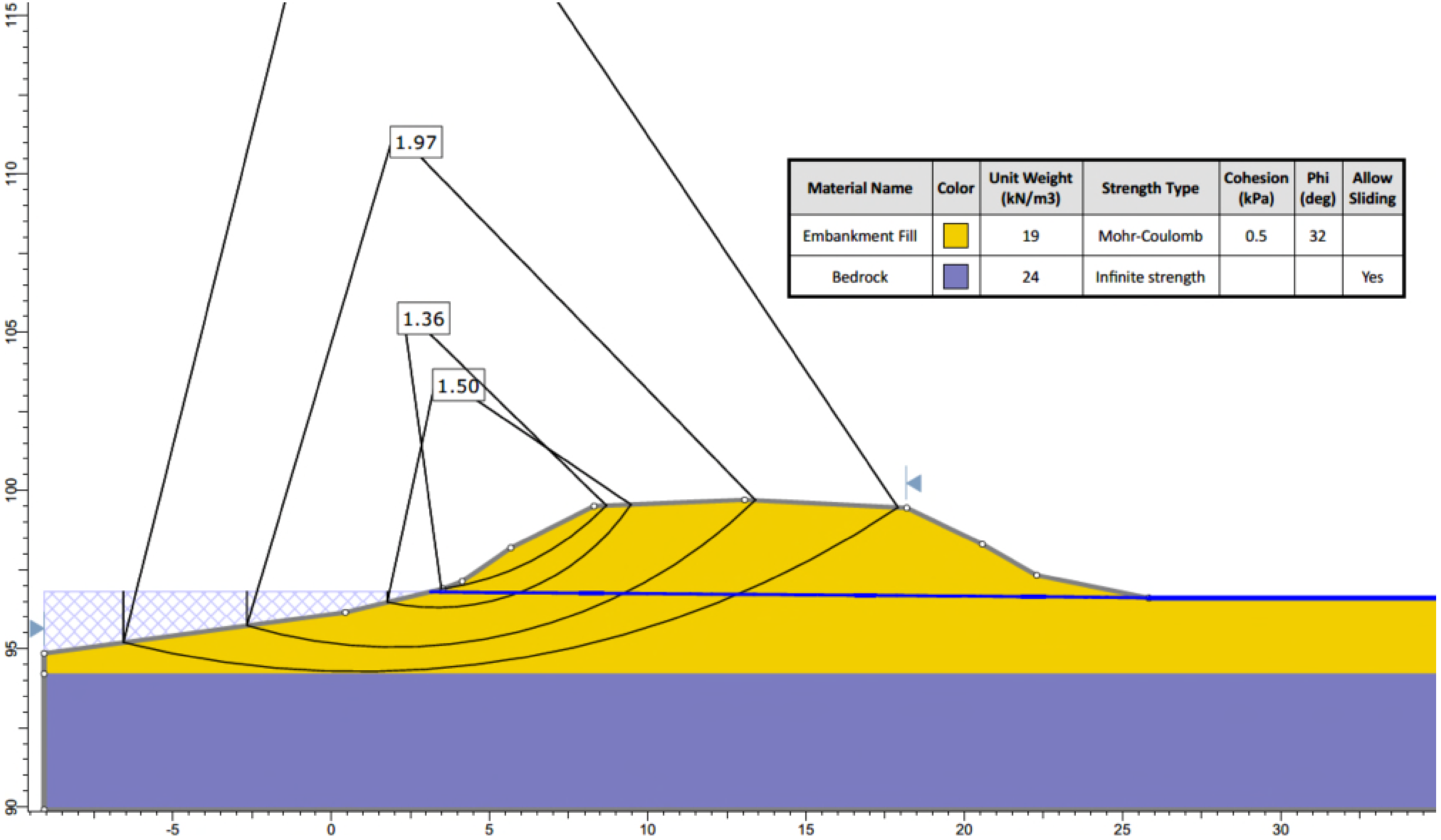
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Static Long Term Stability Analysis: Downstream Slope

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

Figure 10.5a



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

Notes:

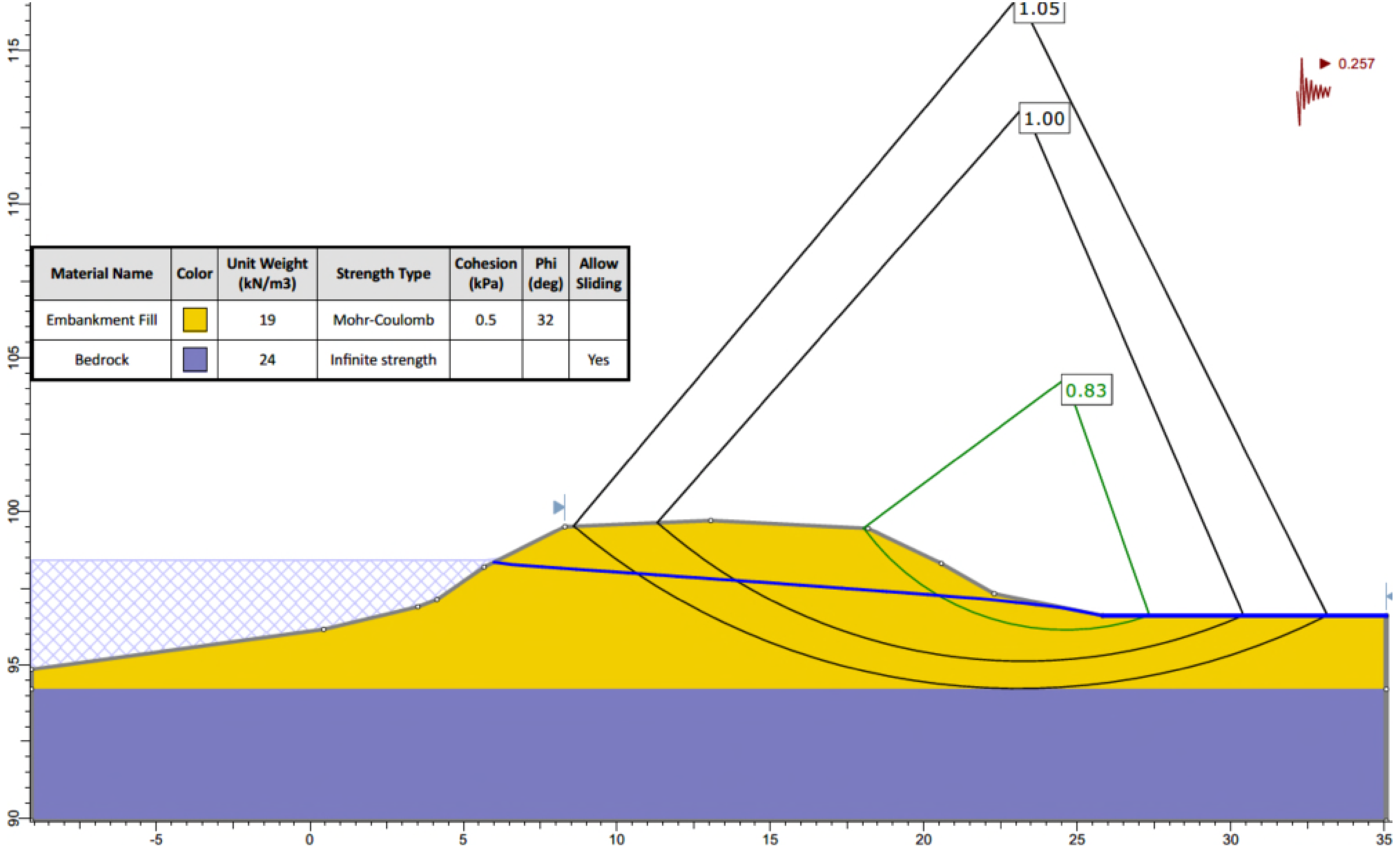
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Static Long Term Stability Analysis: Upstream Slope

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

ecora

Figure 10.5b



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

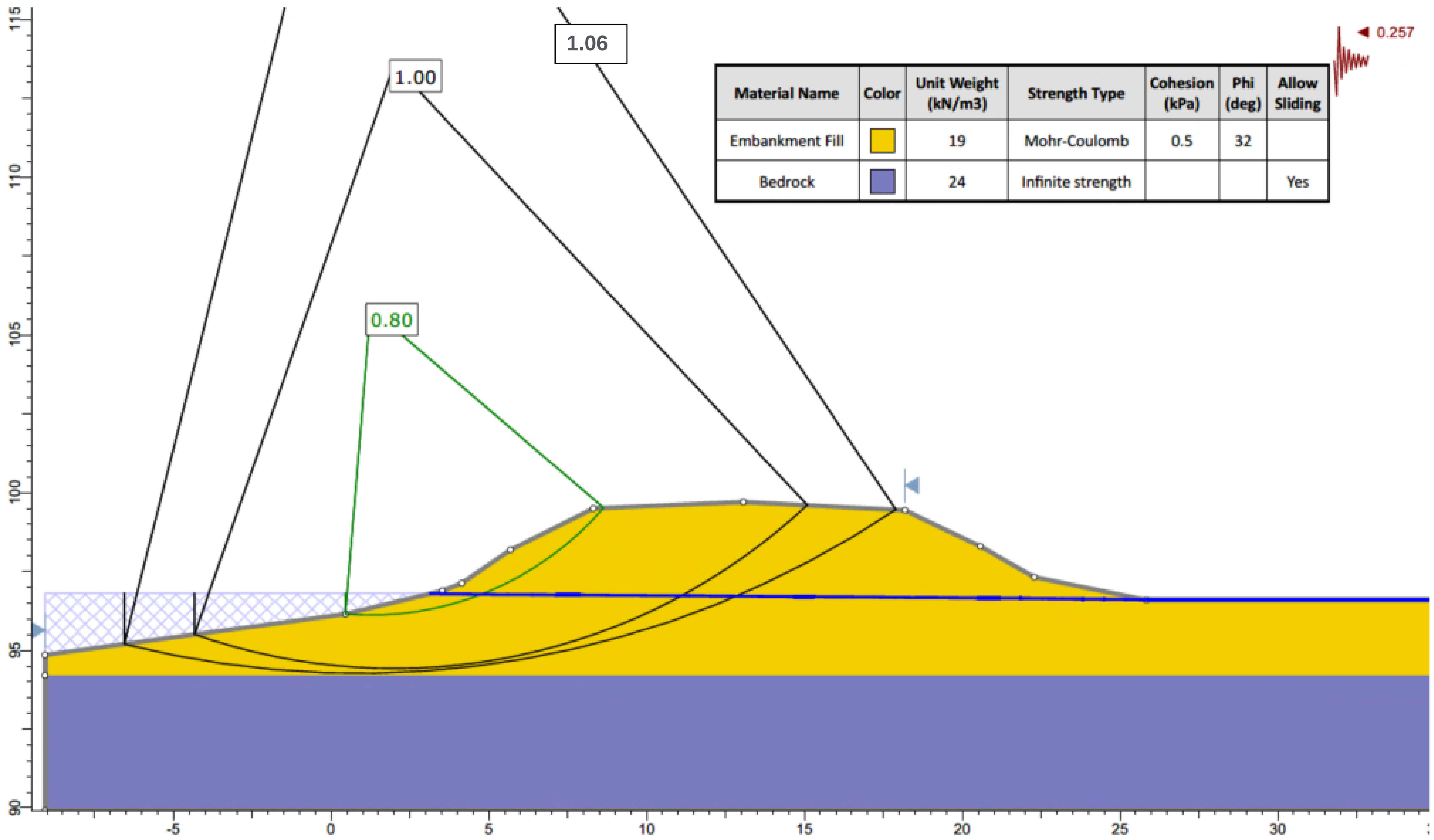
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Pseudo Static Stability Analysis (1/475-year): Downstream Slope

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 24, 2018
DWN: CE CHK: MJL

Figure 10.5c



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

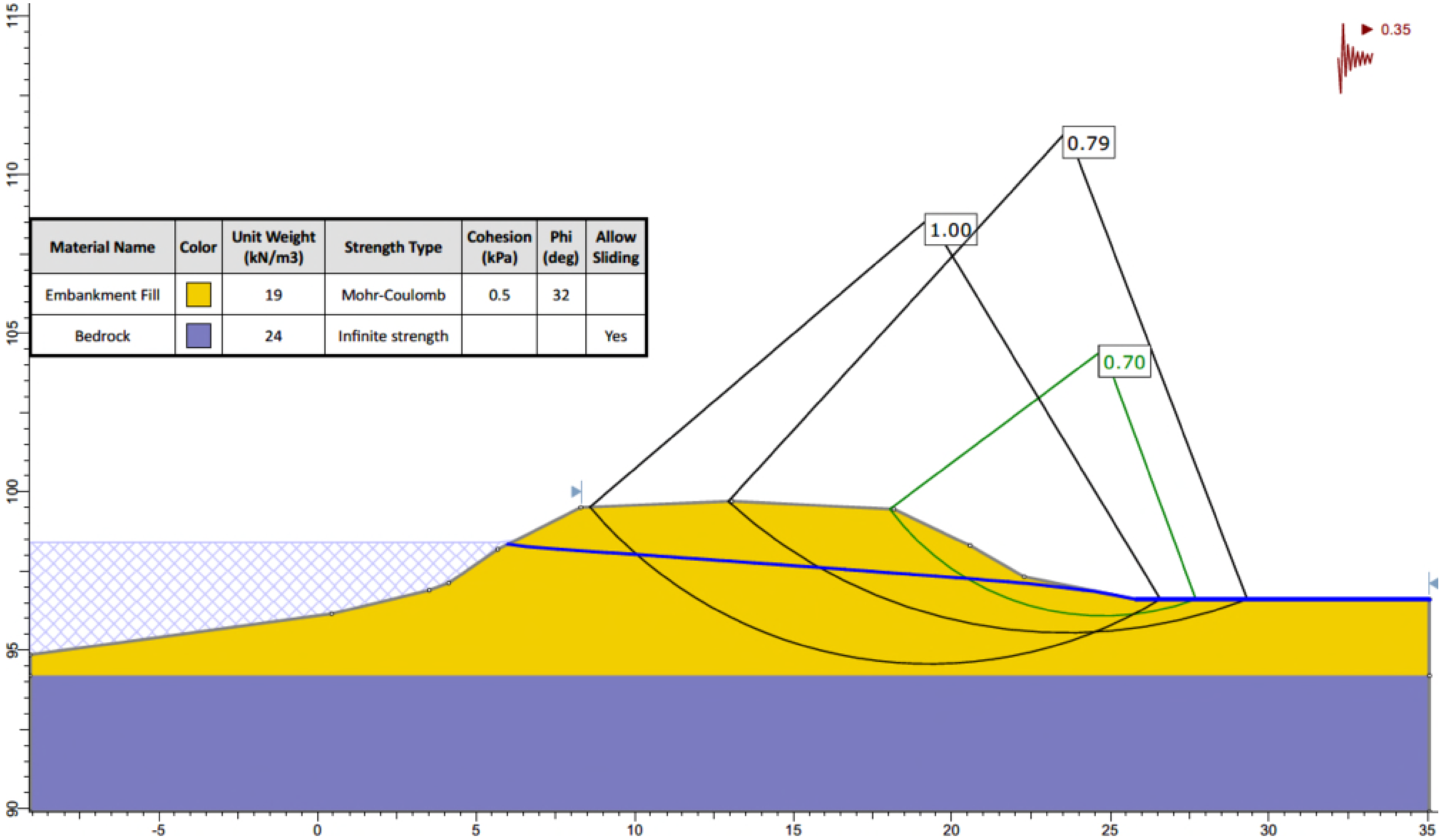
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Pseudo Static Stability Analysis (1/475-year): Upstream Slope

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 24, 2018
DWN: CE CHK: MJL

Figure 10.5d



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Pseudo Static Stability Analysis (1/1,000-year): Downstream Slope

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

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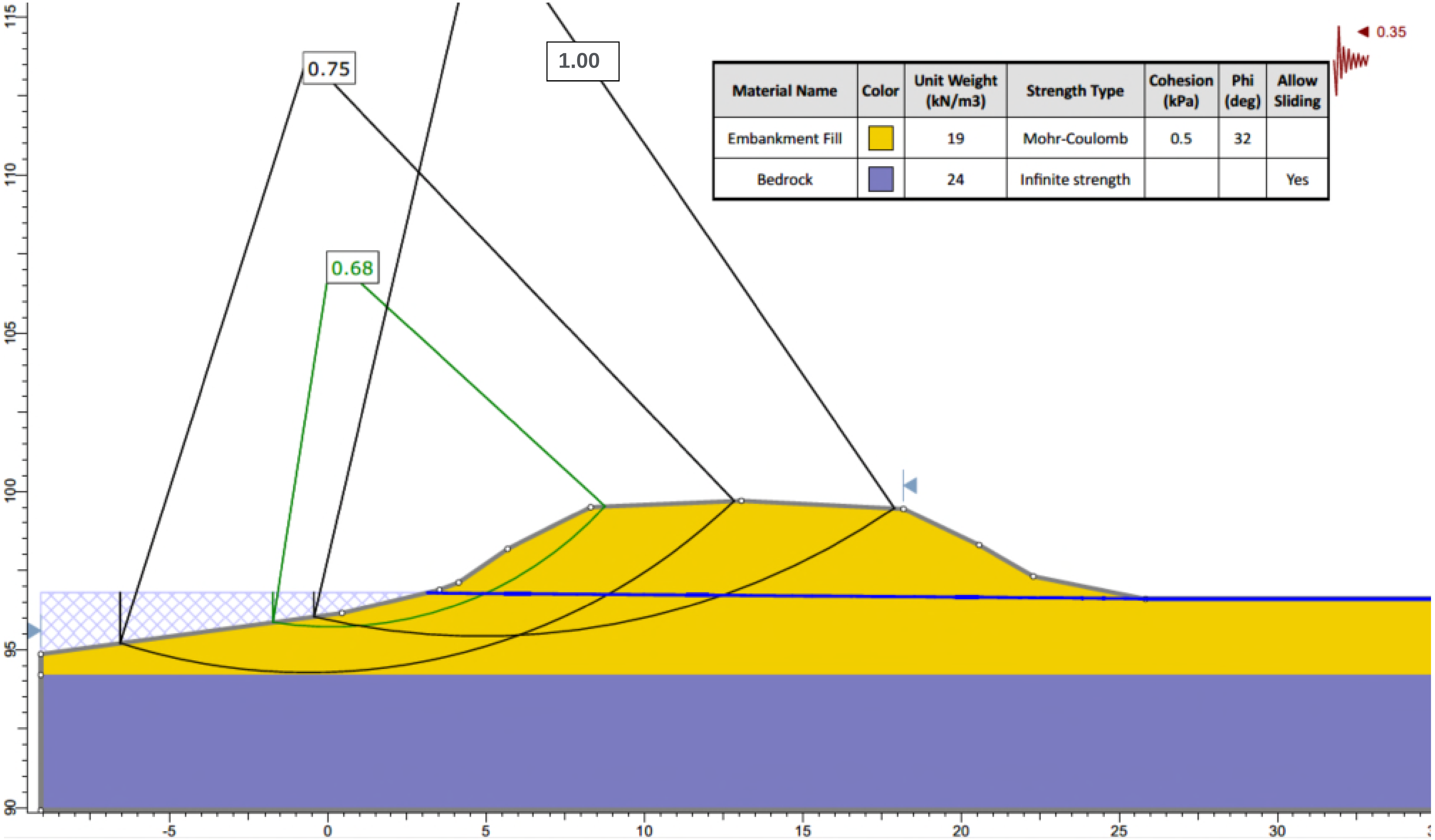
Figure 10.5e



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

Notes:

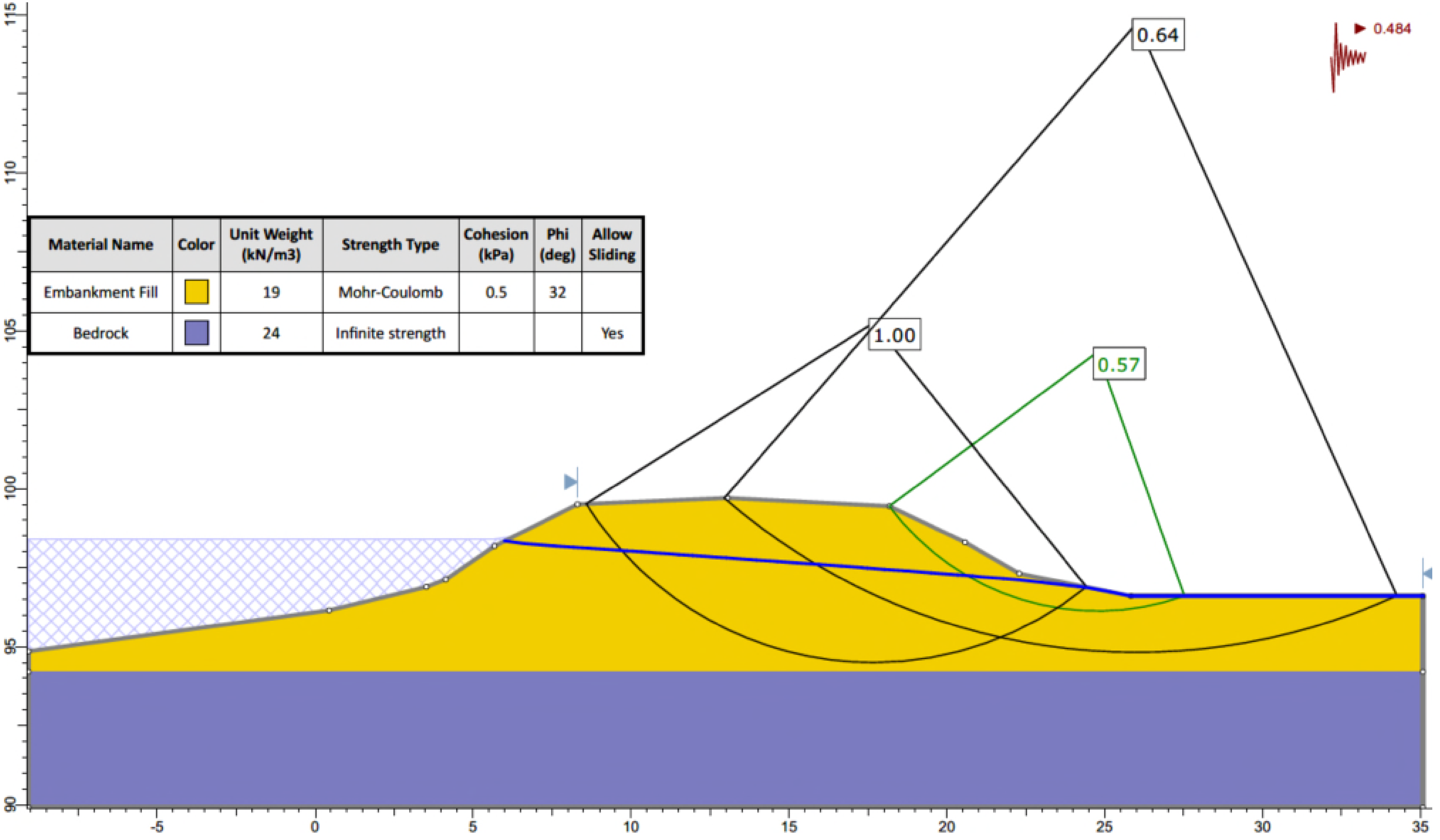
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Pseudo Static Stability Analysis (1/1,000-year): Upstream Slope

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

ecora™

Figure 10.5f



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

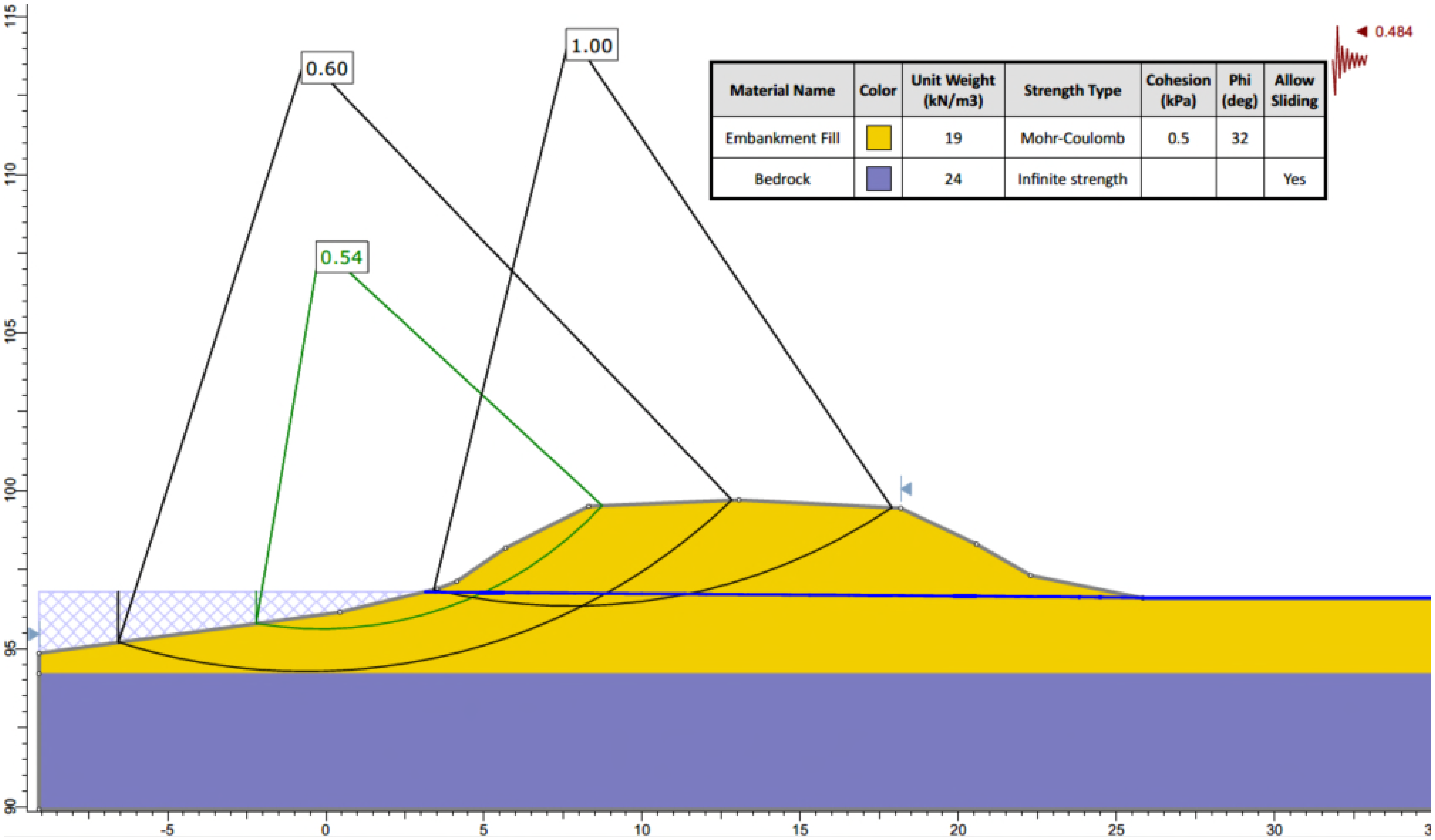
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Pseudo Static Stability Analysis (1/2,475-year): Downstream Slope

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 24, 2018
DWN: CE CHK: MJL

Figure 10.5g



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

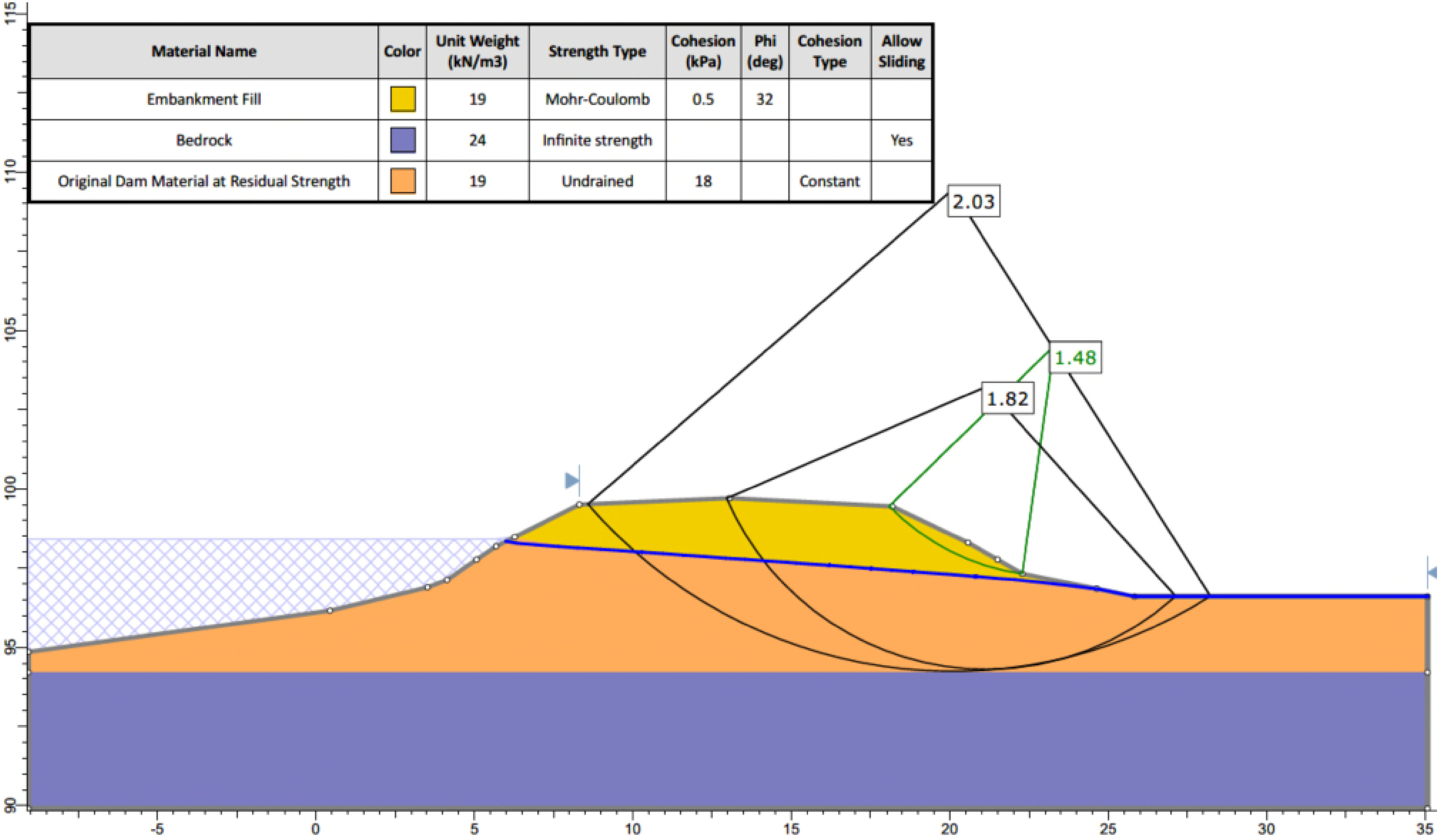
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Pseudo Static Stability Analysis (1/2,475-year): Upstream Slope

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

Figure 10.5h



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Cohesion Type	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32		
Bedrock	🟪	24	Infinite strength				Yes
Original Dam Material at Residual Strength	🟧	19	Undrained	18		Constant	

Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Post-Earthquake Embankment Foundation at Residual Shear Strength: Downstream

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 24, 2018
DWN: CE CHK: MJL

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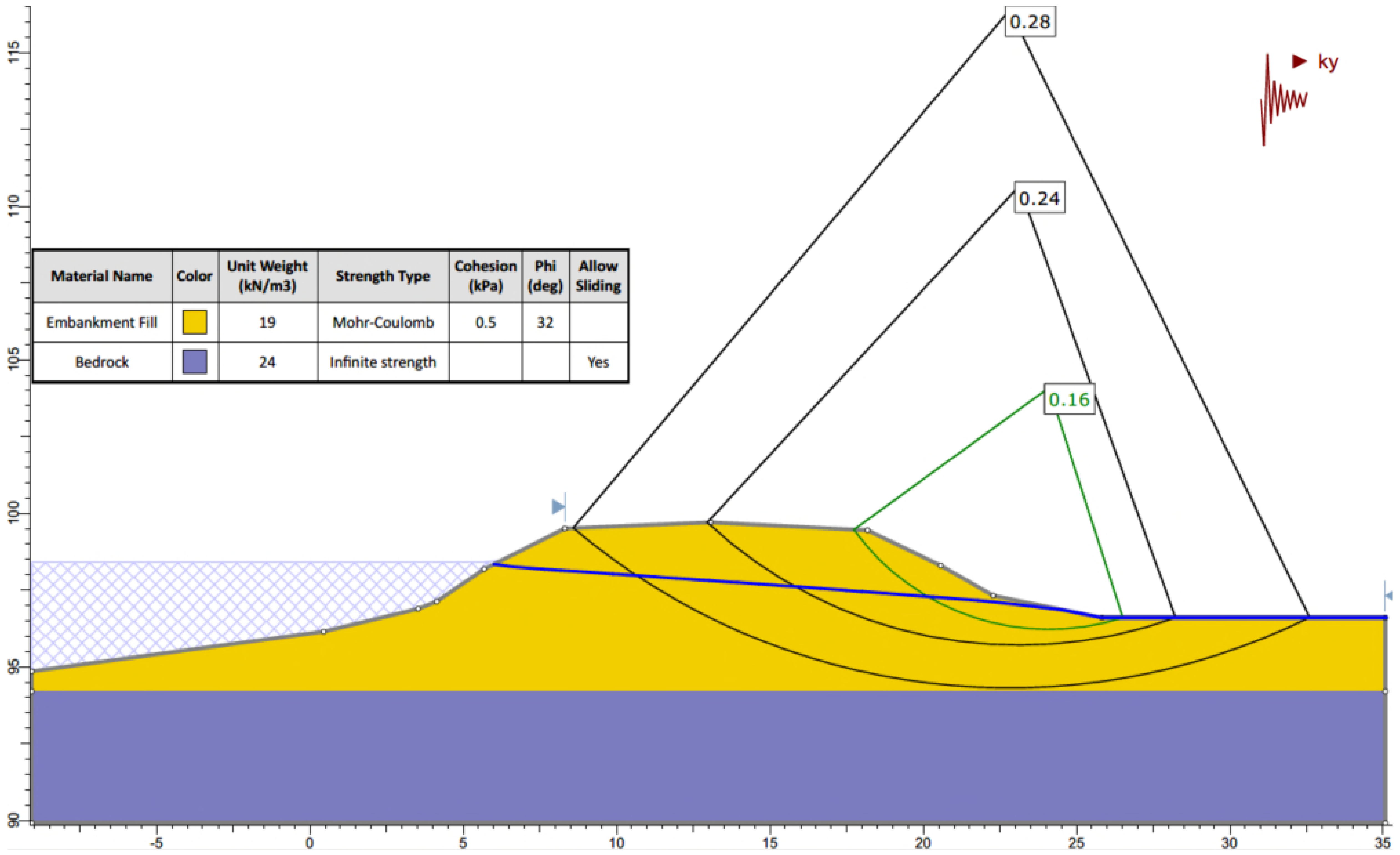
Figure 10.5i



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

Slope	Slide #	Ky	Height of Slide (m)	Ts	Low Sa(1.5Ts)	Significant Sa(1.5Ts)	High Sa(1.5Ts)	D (cm) Low	D (cm) Significant	D (cm) High
Downstream	1	0.16	2.84	0.11	0.591	0.805	1.109	12.5	24.6	47.1
	2	0.24	3.27	0.13	0.591	0.805	1.109	5.5	11.5	23.8
	3	0.28	2.93	0.12	0.591	0.805	1.109	3.9	8.4	17.8

Notes:

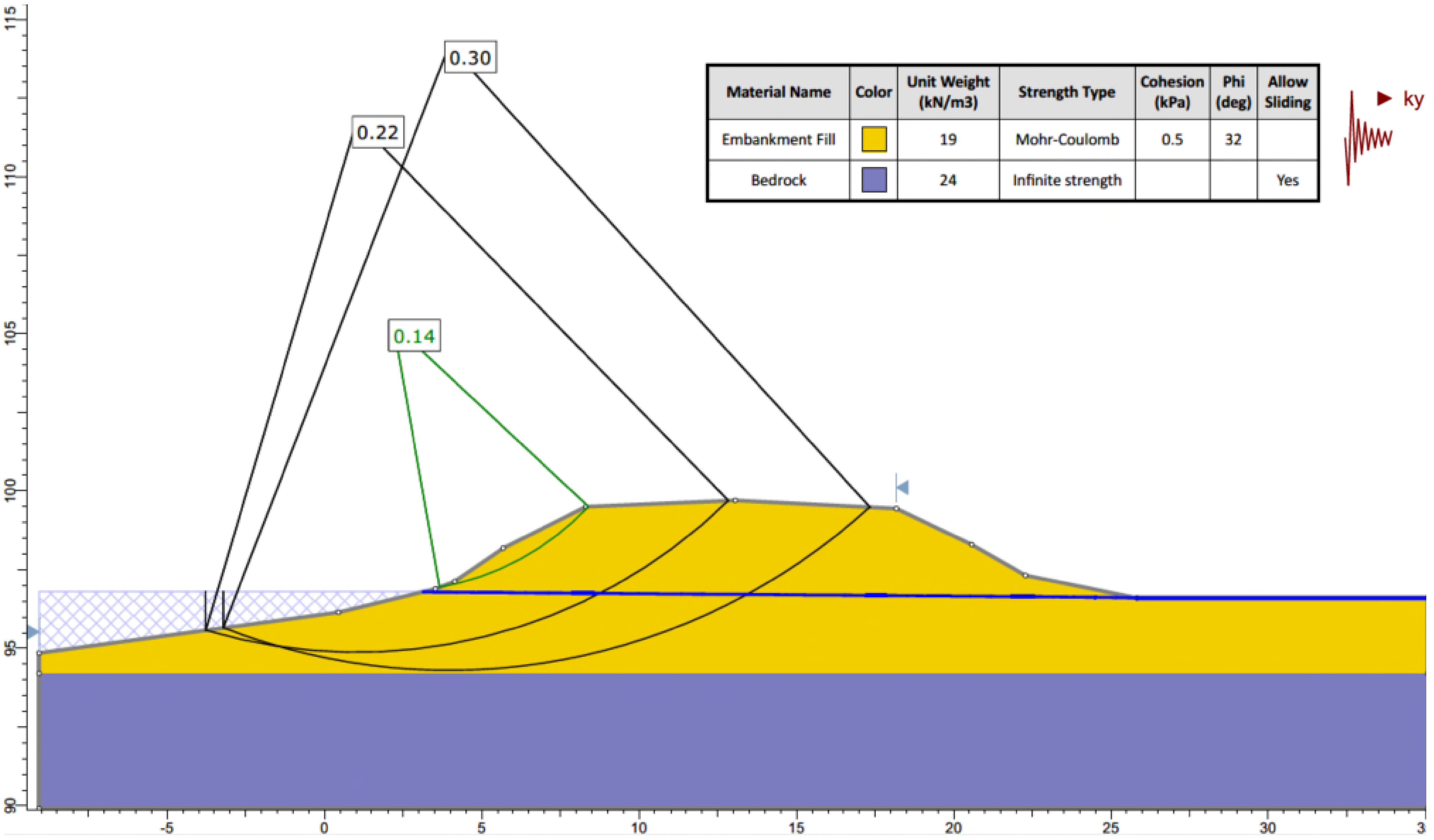
Constants used: V_s = 100 m/s, M_w = 8.83. Displacements calculated using the methods of Bray and Travasarou (2007).

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Yield Acceleration and Displacement Analysis Downstream Earthquake

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 24, 2018
DWN: CE CHK: MJL

Figure 10.6a



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Allow Sliding
Embankment Fill	🟨	19	Mohr-Coulomb	0.5	32	
Bedrock	🟪	24	Infinite strength			Yes

Slope	Slide #	Ky	Height of Slide (m)	Ts	Low Sa(1.5Ts)	Significant Sa(1.5Ts)	High Sa(1.5Ts)	D (cm) Low	D (cm) Significant	D (cm) High
Upstream	1	0.14	2.61	0.1	0.591	0.805	1.109	16.0	30.8	57.6
	2	0.22	4.06	0.16	0.591	0.805	1.109	6.6	13.7	27.8
	3	0.3	3.75	0.15	0.591	0.805	1.109	3.3	7.2	15.6

Notes:

Constants used: Vs = 100 m/s, Mw = 8.83. Displacements calculated using the methods of Bray and Travasarou (2007).

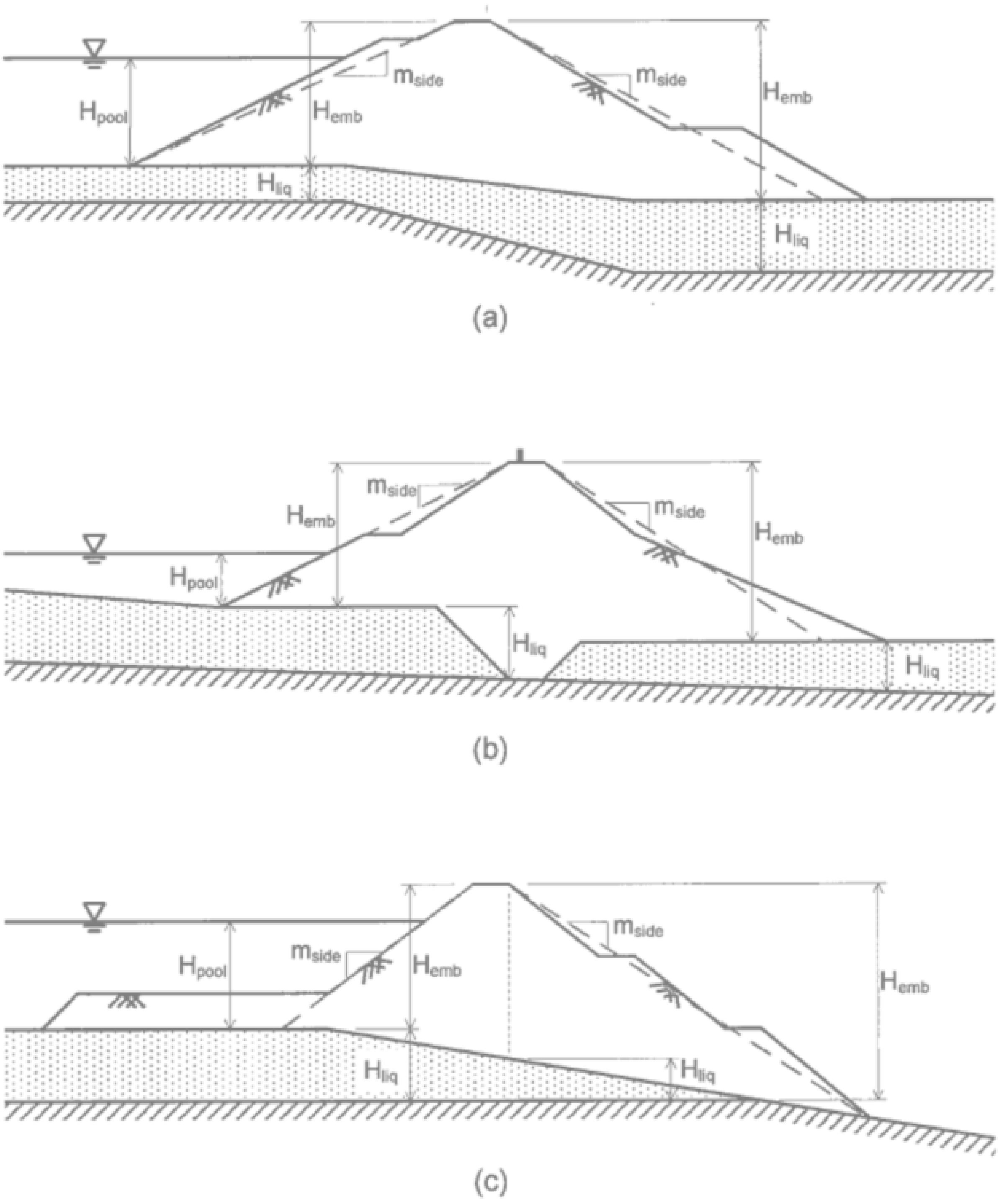
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Yield Acceleration and Displacement Analysis Upstream Earthquake

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 24, 2018
DWN:	CE CHK: MJL

ecora

Figure 10.6b



Notes:
Adapted from Rauch et al. 2007

DAM SAFETY REVIEW OF STOCKING LAKE DAM

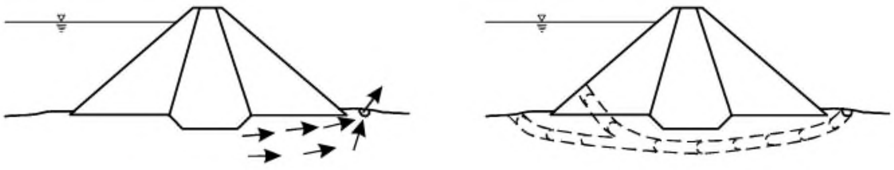
Input Variables for the Liquefaction-Induced Deformation Screening Assessment

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	CE CHK: MJL

Figure 10.6c

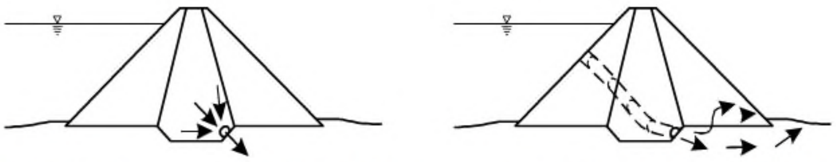
INITIATION	CONTINUATION	PROGRESSION	BREACH
Concentrated leak forms, erosion initiates along walls of crack	Continuation of erosion	Enlargement of concentrated leak	Breach mechanism forms

(A) INTERNAL EROSION IN THE EMBANKMENT INITIATED BY EROSION IN A CONCENTRATED LEAK



INITIATION	→ CONTINUATION	→ PROGRESSION	→ BREACH
Leakage exits from the foundation and backward erosion initiates	Continuation of erosion	Backward erosion in progresses to form a pipe	Breach mechanism forms

(B) INTERNAL EROSION IN THE FOUNDATION INITIATED BY BACKWARD EROSION



INITIATION	CONTINUATION	PROGRESSION	BREACH
Leakage exits the core into the foundation and backward erosion initiates as core erodes into the foundation	Continuation of erosion	Backward erosion progresses to form a pipe. Eroded soil is transported in the foundation	Breach mechanism forms

(C) INTERNAL EROSION FROM THE EMBANKMENT TO FOUNDATION INITIATED BY BACKWARD EROSION

Notes:
Adapted from Figure 2.1 of ICOLD Bulletin 164 Internal Erosion of Existing Dams, Levees and Dikes, and Their Foundations.

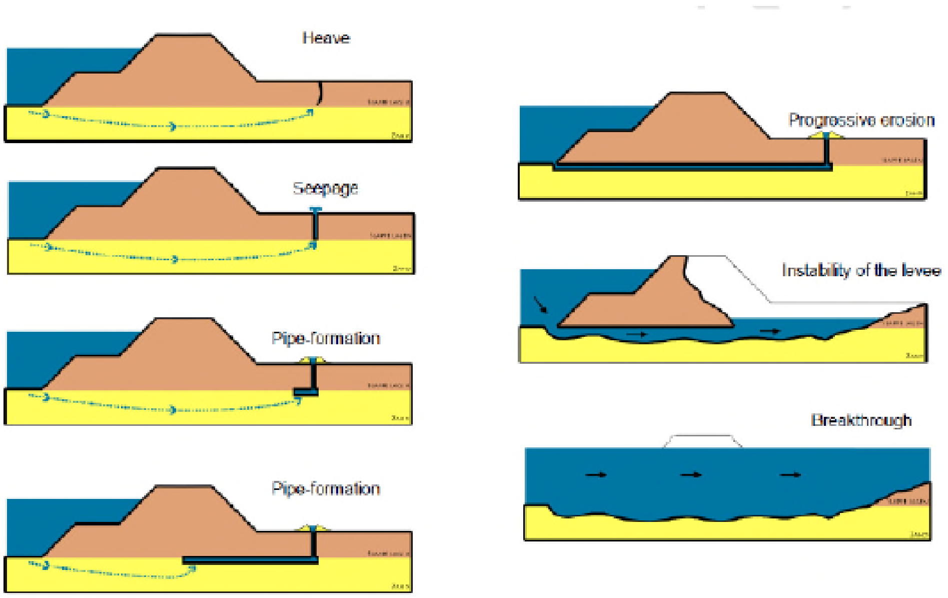
DAM SAFETY REVIEW STOCKING LAKE DAM

Models for the Developments of Embankment Failures Due to Internal Erosion

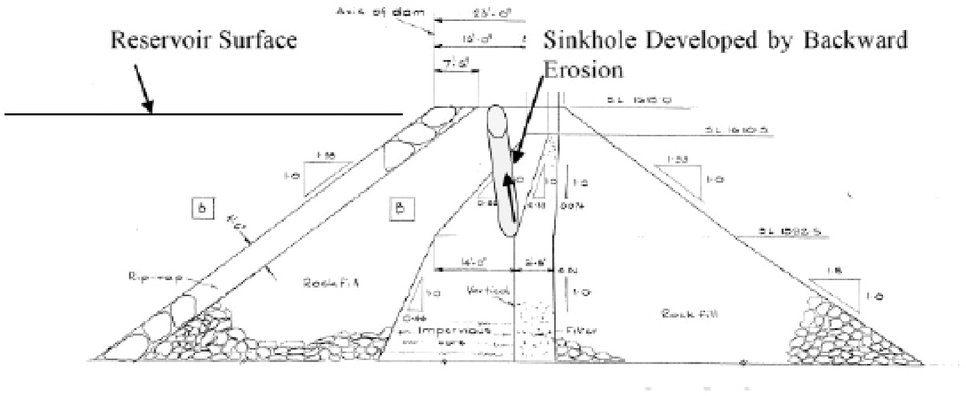
Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	CE CHK: MJL

Figure 10.7a

Example of Backward Erosion Piping



Example of Global Backward Erosion



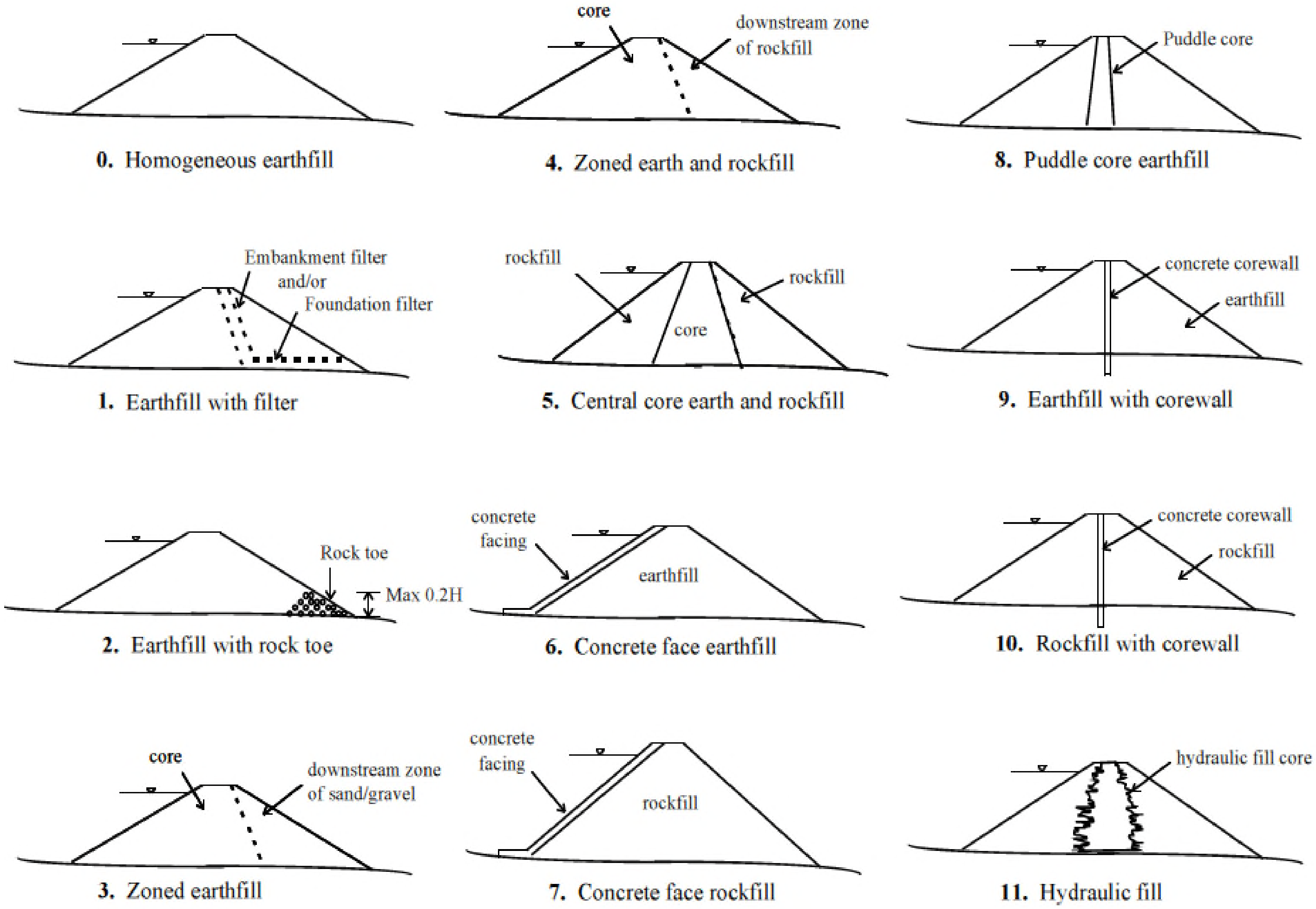
Notes:
Adapted from Figure 2.7 of ICOLD Bulletin 164 Internal Erosion of Existing Dams, Levees and Dikes, and Their Foundations.

DAM SAFETY REVIEW STOCKING LAKE DAM

Examples of Backwards Erosion

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 19, 2018
DWN: CE CHK: MJL

Figure 10.7b



0. Homogeneous earthfill

4. Zoned earth and rockfill

8. Puddle core earthfill

1. Earthfill with filter

5. Central core earth and rockfill

9. Earthfill with corewall

2. Earthfill with rock toe

6. Concrete face earthfill

10. Rockfill with corewall

3. Zoned earthfill

7. Concrete face rockfill

11. Hydraulic fill

Notes:
Adapted from Figure 2.9 of ICOLD Bulletin 164 Internal Erosion of Existing Dams, Levees and Dikes, and Their Foundations.

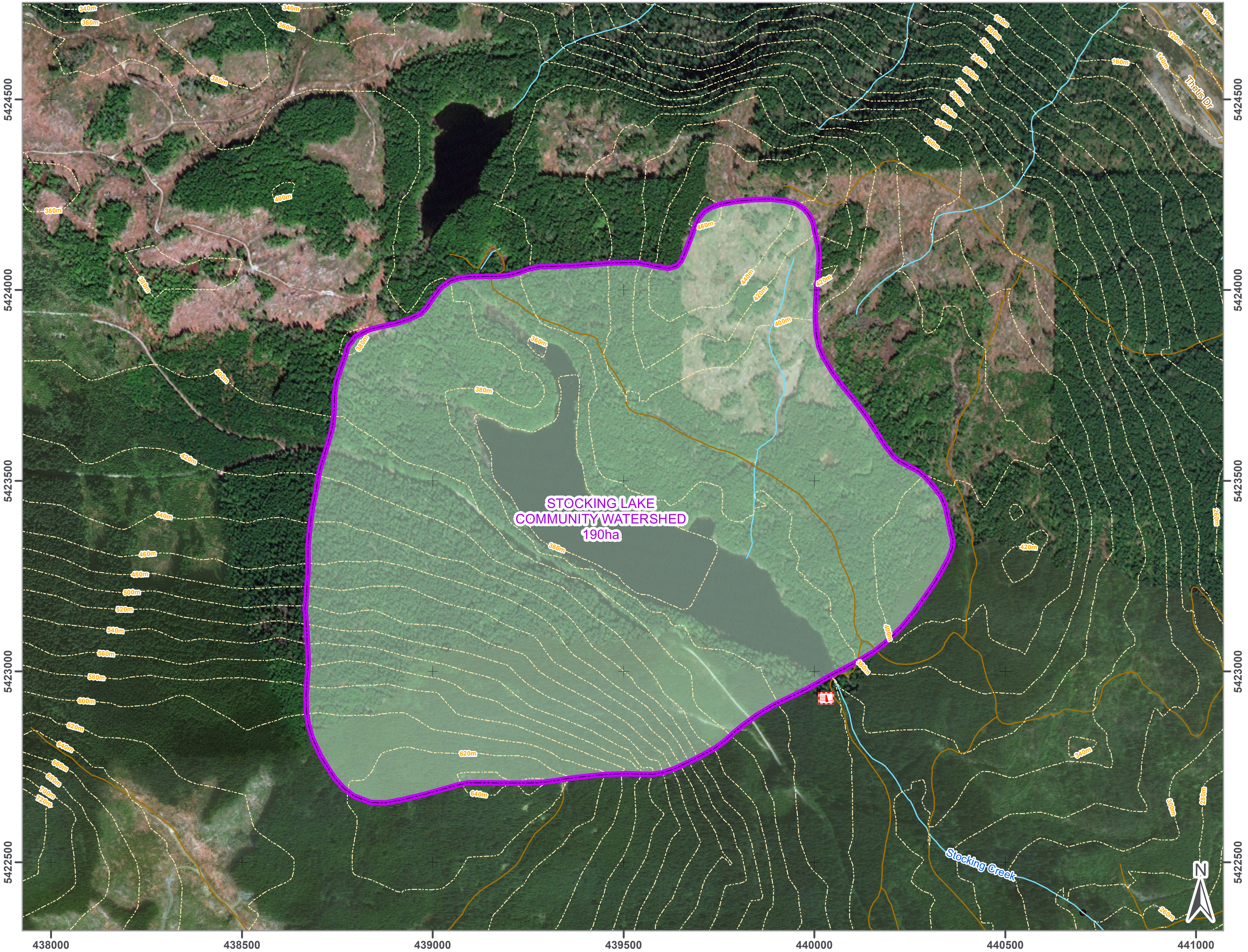
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Dam Zoning Categories

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	CE CHK: MJL

Figure 10.7c

STOCKING LAKE WATERSHED

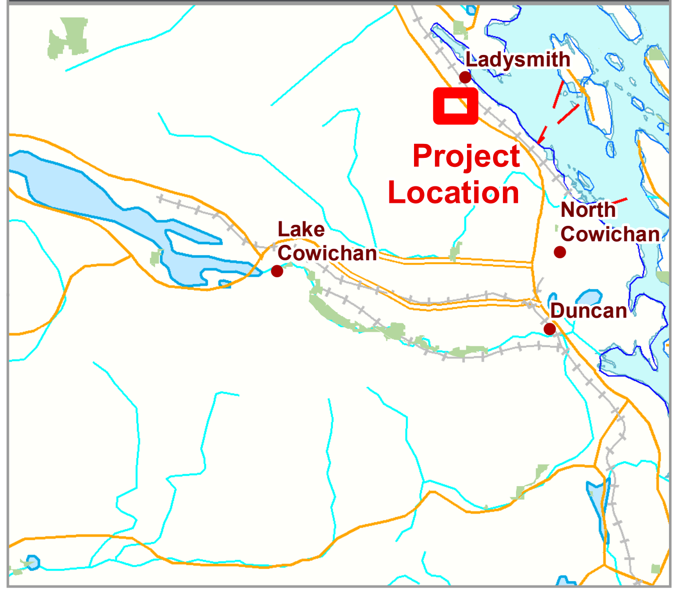


DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

🟥	Stocking Lake Dam
┈	20m TRIM Contours
—	Fresh Water Atlas Streams
—	Digital Atlas Roads
▭	Stocking Lake Community Watershed

LOCATION MAP



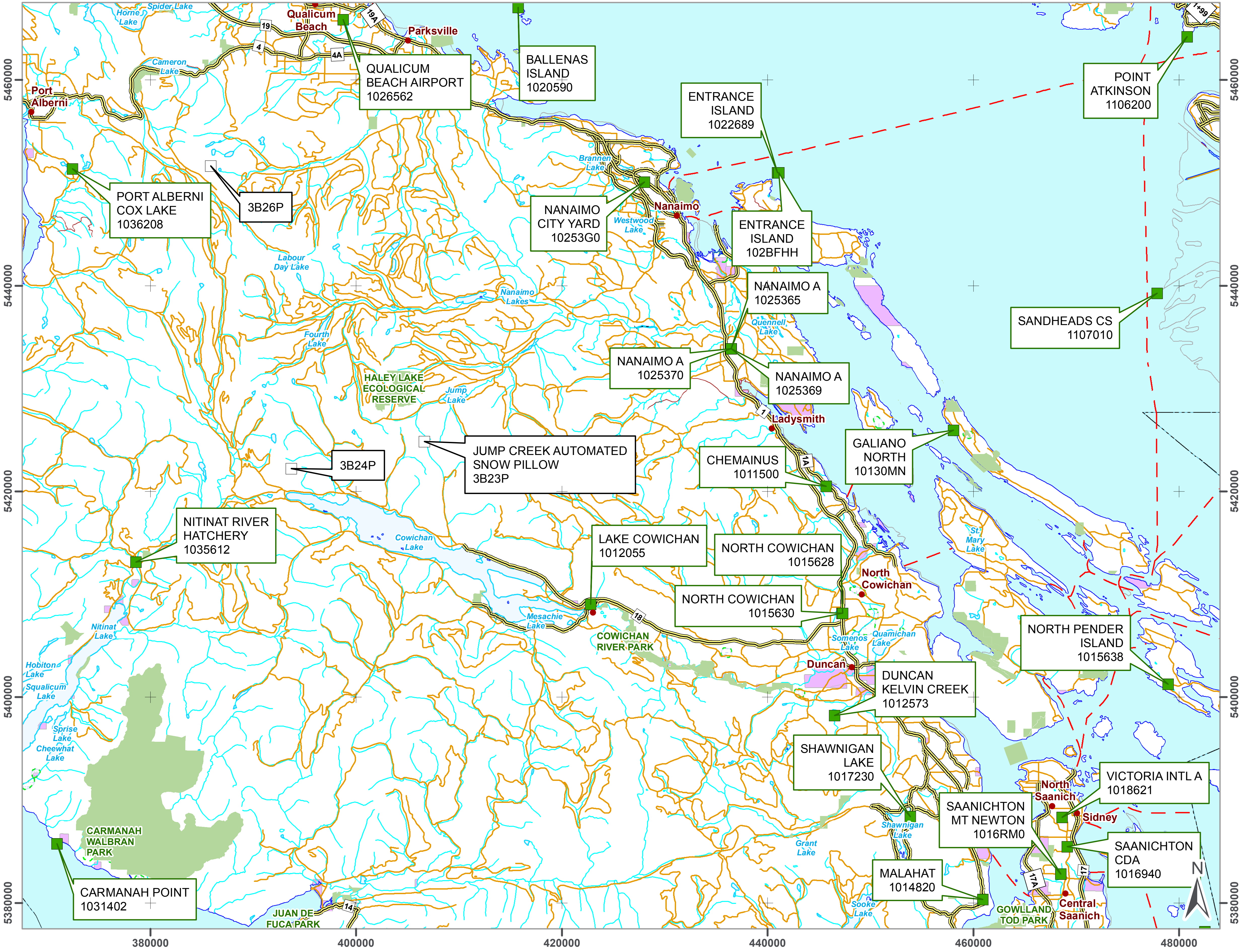
1:10,000

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/07/03
Drawn: MT Check: AG

Figure 11.1

CLIMATE AND AUTOMATED SNOW PILLOW STATIONS

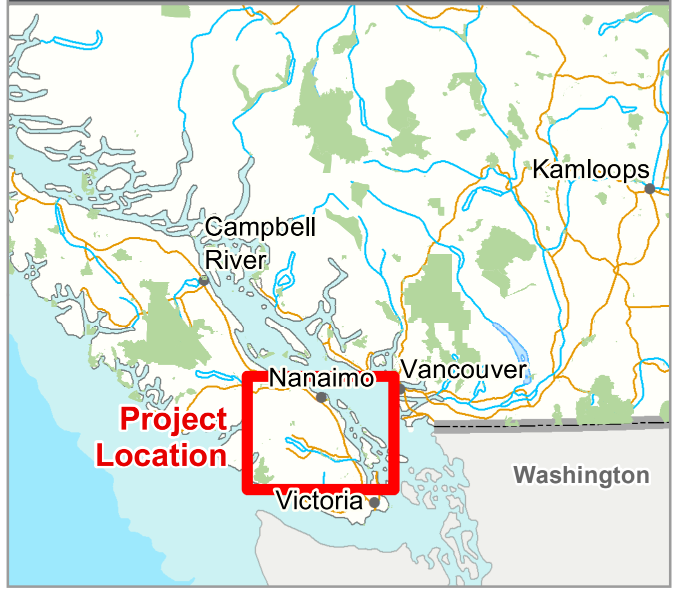


DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

- ● Cities
- 🟩 Climate Station (Environment and Climate Change Canada)
- ☐ Automated Snow Pillow Station (British Columbia)
- Highways
- Streams
- Ferry Route
- Roads
- Trail
- Bridge
- Reserves
- Parks

LOCATION MAP



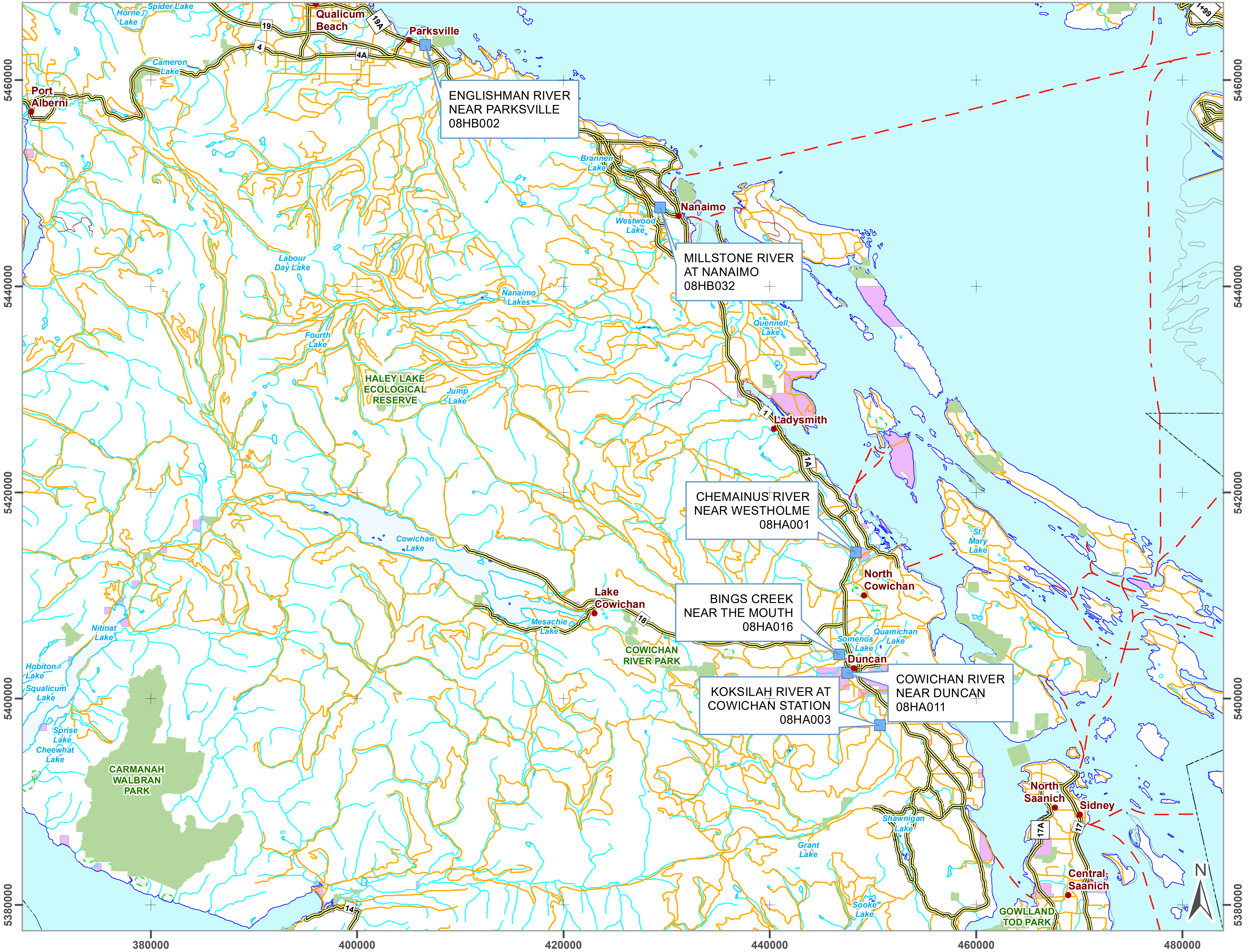
1:370,000

0 5 10 15 20 km

Project No.: GK-18-020-CVD
Date: 2018/03/29
Client: Cowichan Valley Regional District
Drawn: TJM Check: AG
NAD 1983 UTM Zone 10N

Figure 11.2

HYDROMETRIC STATIONS



DAM SAFETY REVIEW OF STOCKING LAKE DAM LADYSMITH, BC

Legend

- ● Cities
- ■ Hydrometric Station (Water Survey of Canada)
- ═ Highways
- — Streams
- – – Ferry Route
- — Roads
- — Trail
- — Bridge
- ▨ Reserves
- 🟩 Parks

LOCATION MAP

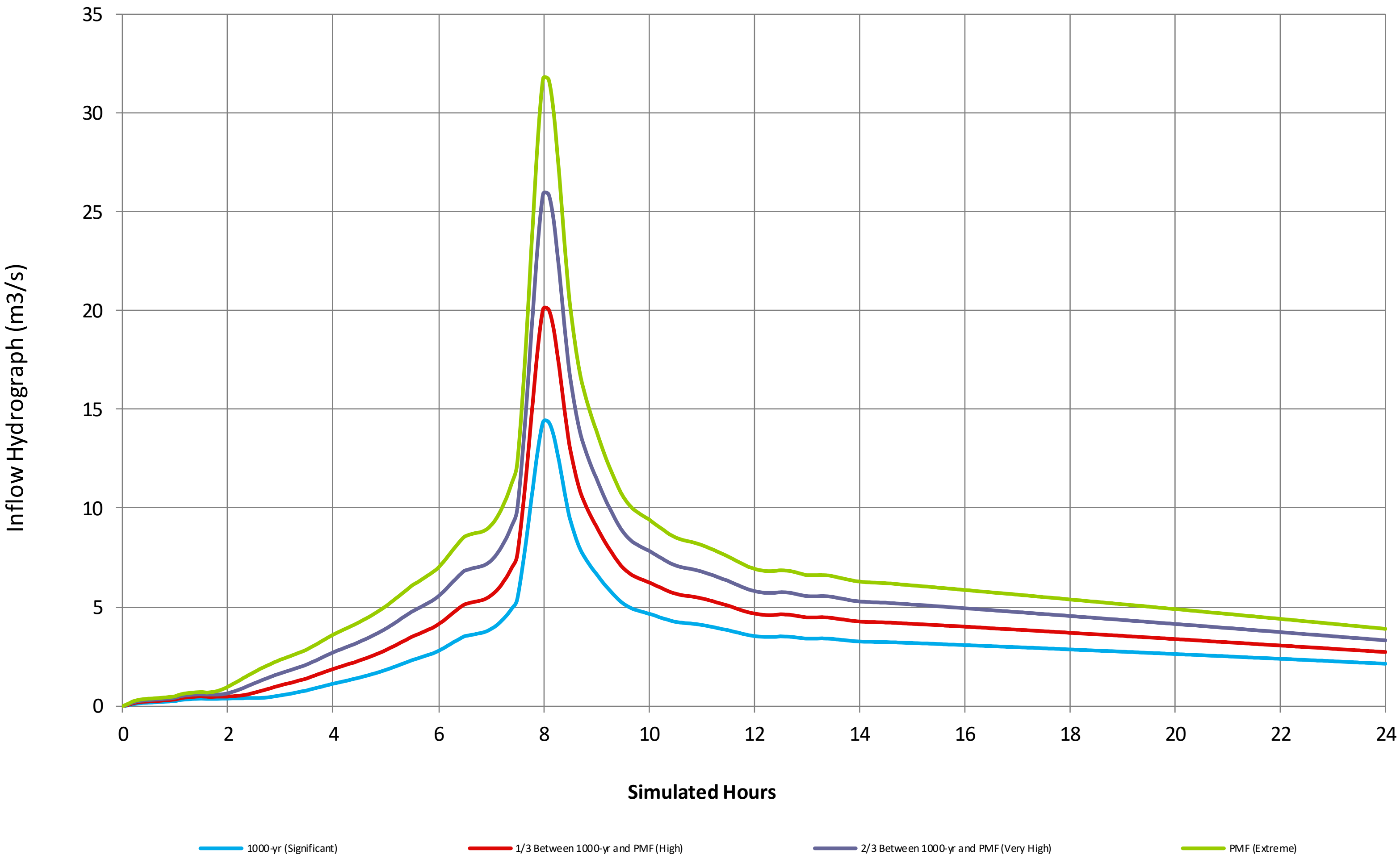
1:370,000

0 5 10 15 20 km

Project No.: GK-18-020-CVD
Client: Cowichan Valley Regional District
NAD 1983 UTM Zone 10N

Date: 2018/09/13
Drawn: MT Check: AG

Figure 11.3



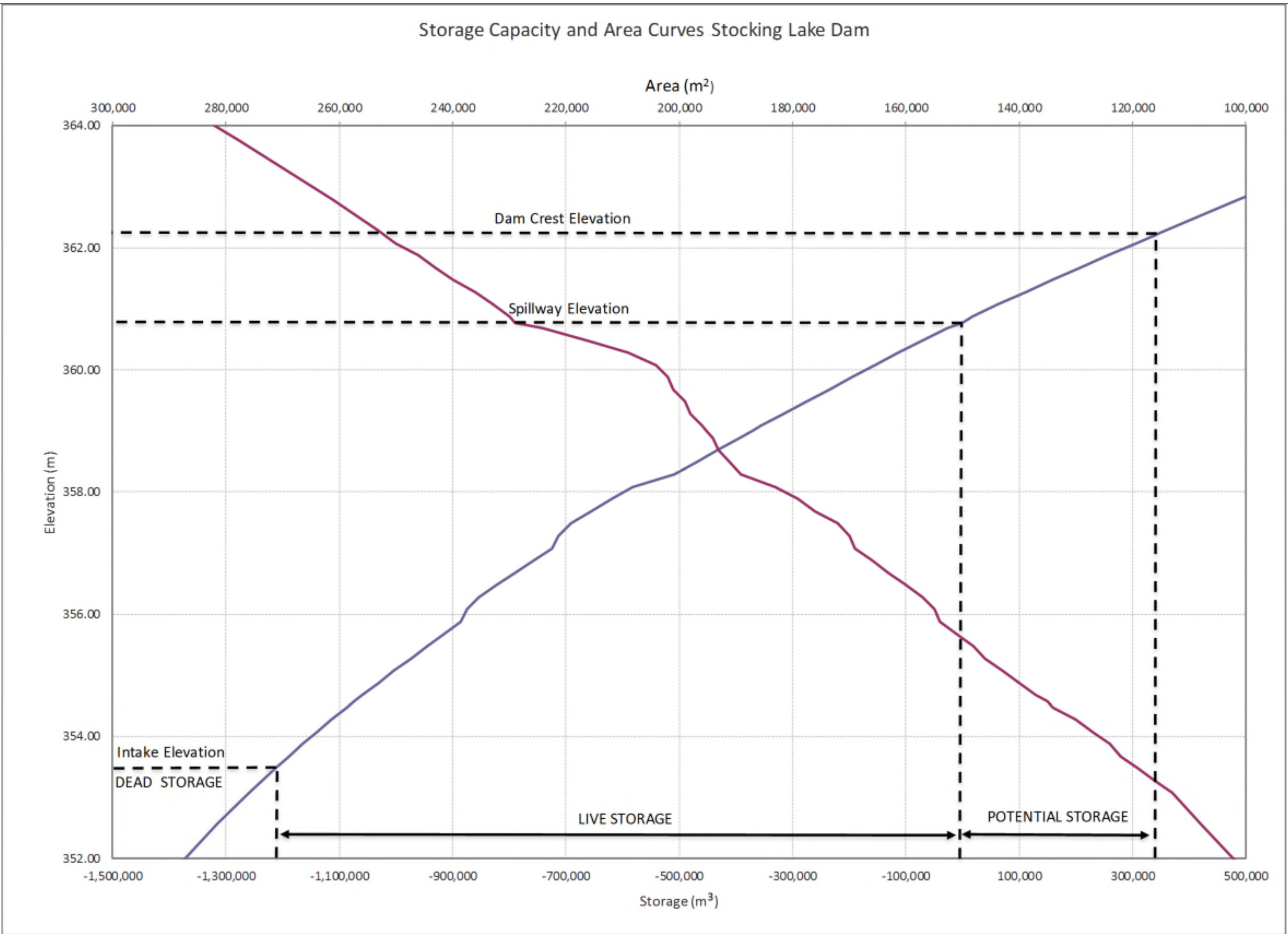
Notes:

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Inflow Design Flood Hydrographs

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	AG CHK: MJL

Figure 11.4



Notes:

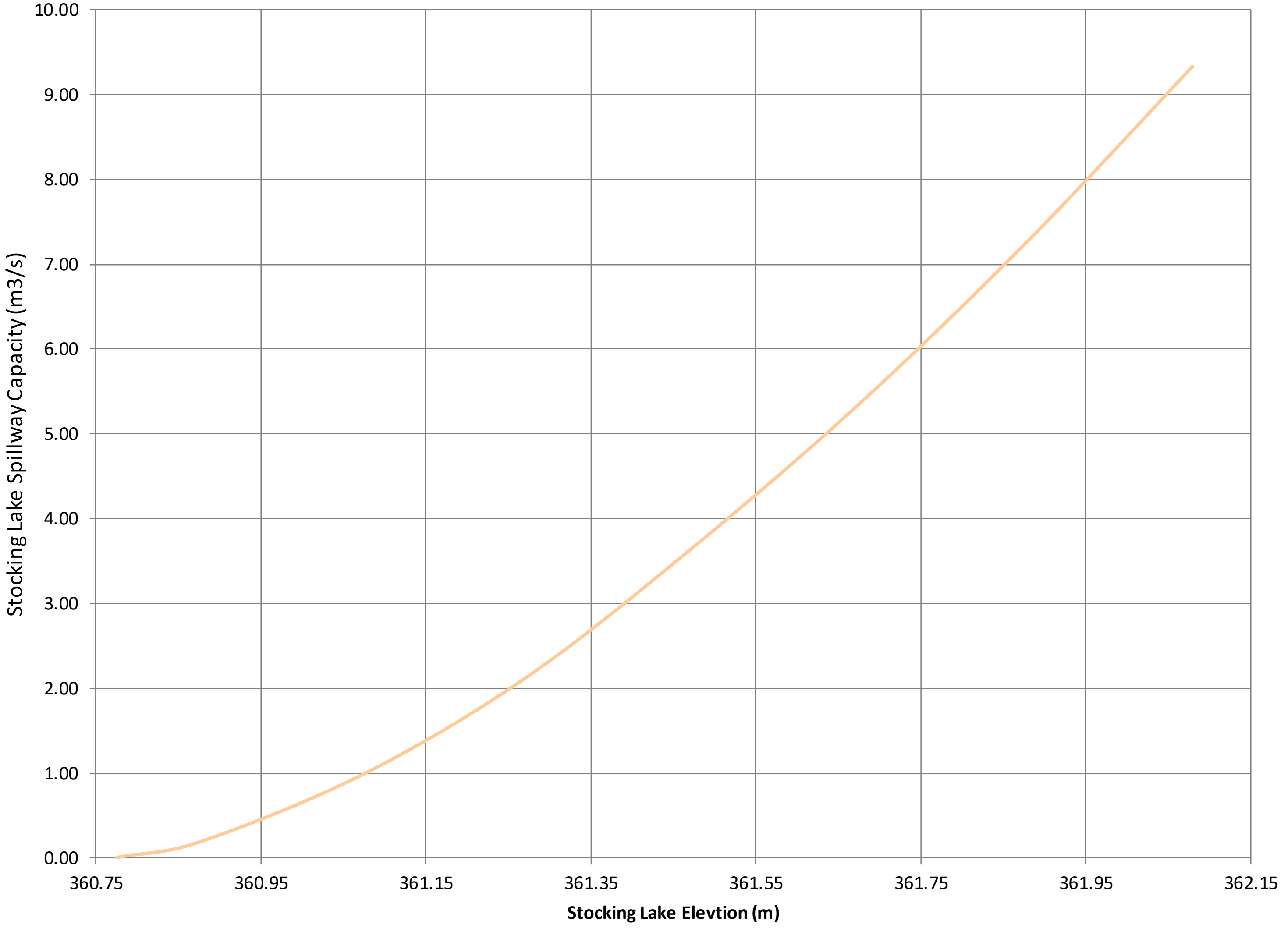
DAM SAFETY REVIEW STOCKING LAKE DAM

Stocking Lake Storage Area Elevation Curves

Project No.	GK-18-020-CVD
Client:	Cowichan Valley Regional District
Office:	Kelowna
Scale:	NTS
Date:	September 19, 2018
DWN:	AG CHK: MJL

ecora

Figure 11.5a



Notes:

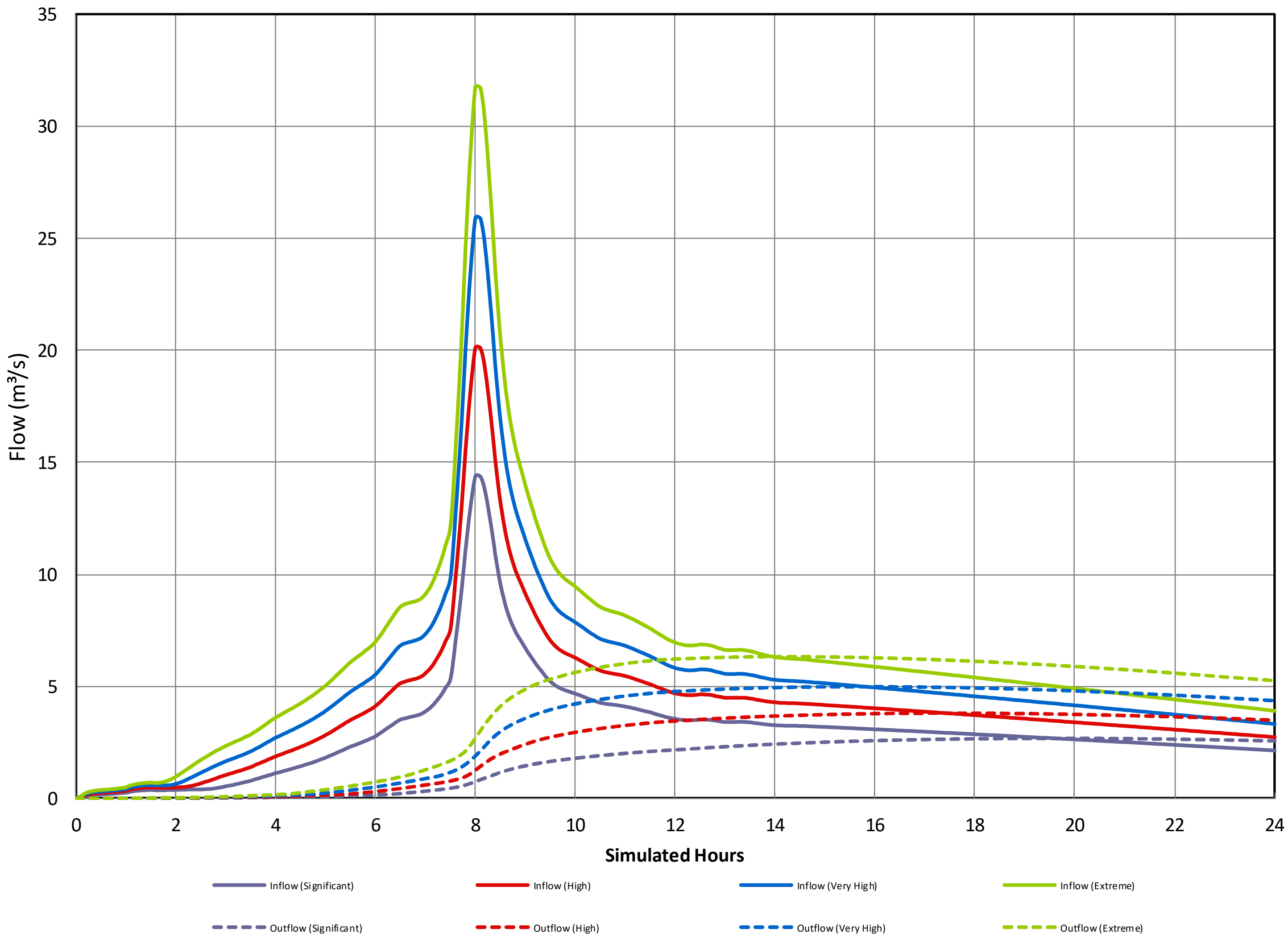
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Spillway Rating Curve

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 19, 2018
DWN: AG CHK: MJL

ecora

Figure 11.5b



Notes:

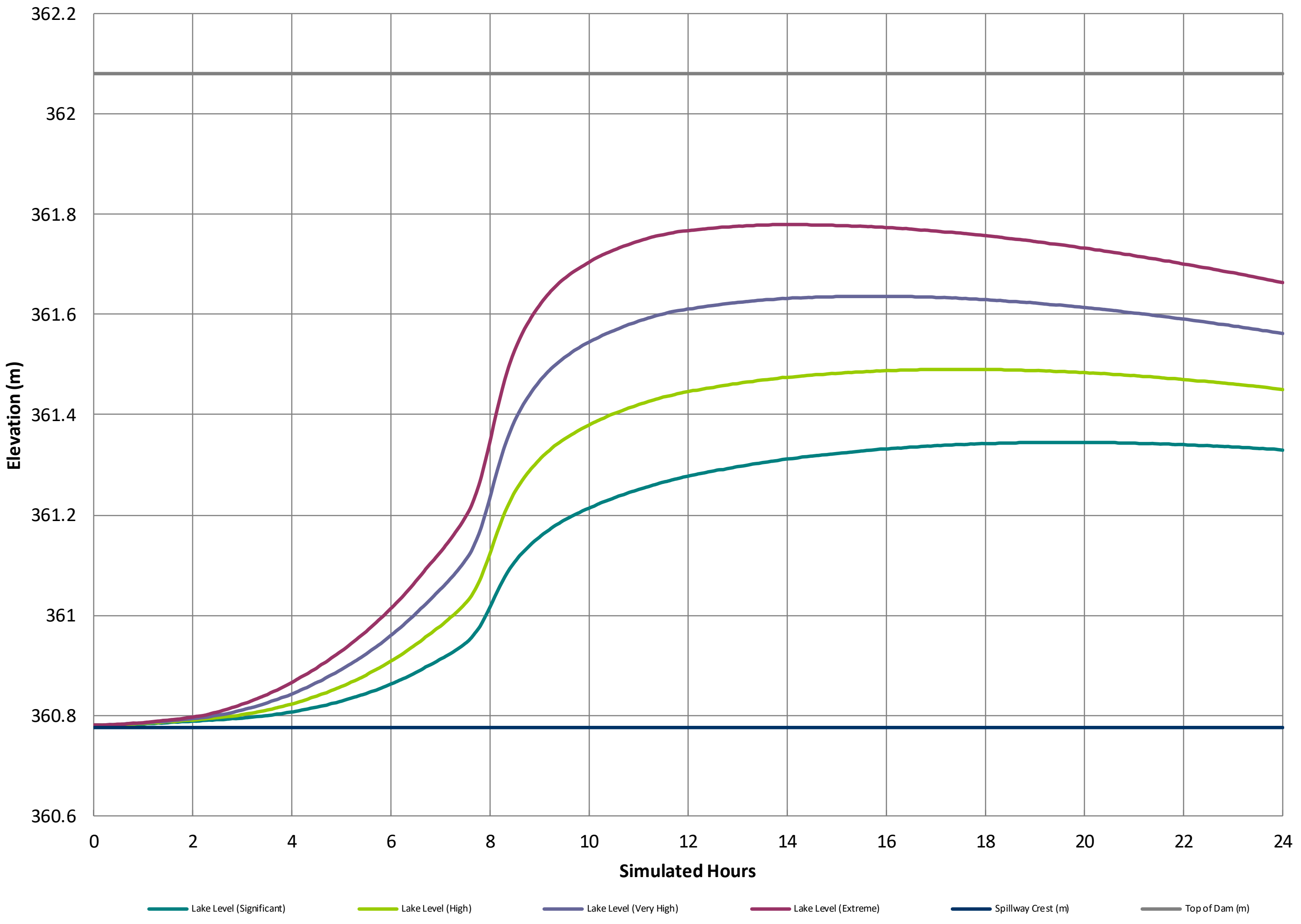
DAM SAFETY REVIEW STOCKING LAKE DAM

Flood Routing Hydrographs

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 19, 2018
DWN: AG CHK: MJL

ecora

Figure 11.5c



Notes:

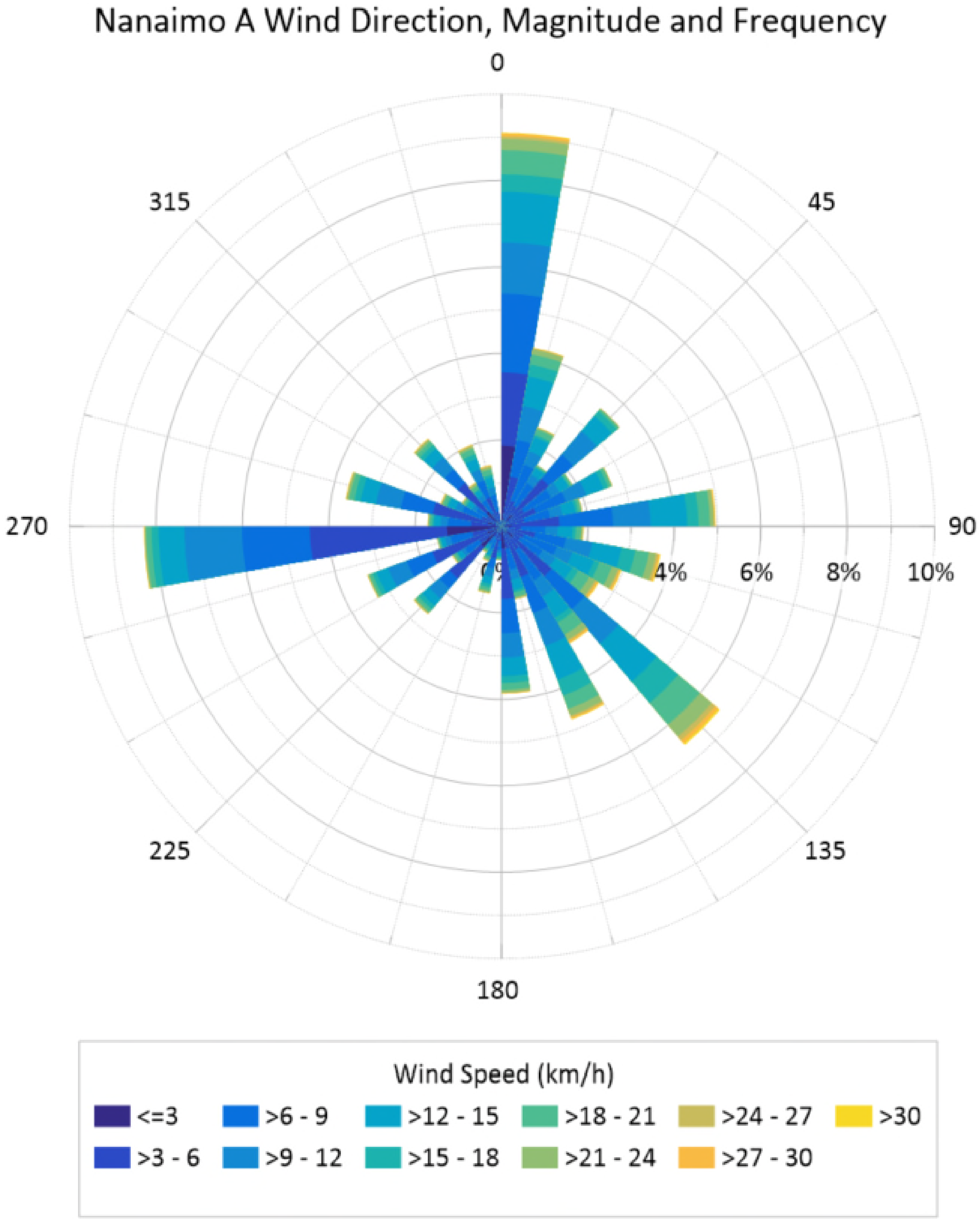
DAM SAFETY REVIEW & RISK ASSESSMENT OF STOCKING LAKE DAM

Flood Lake Levels

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 19, 2018
DWN: AG CHK: MJL

ecora

Figure 11.5d



Notes:
Data retrieved from http://climate.weather.gc.ca

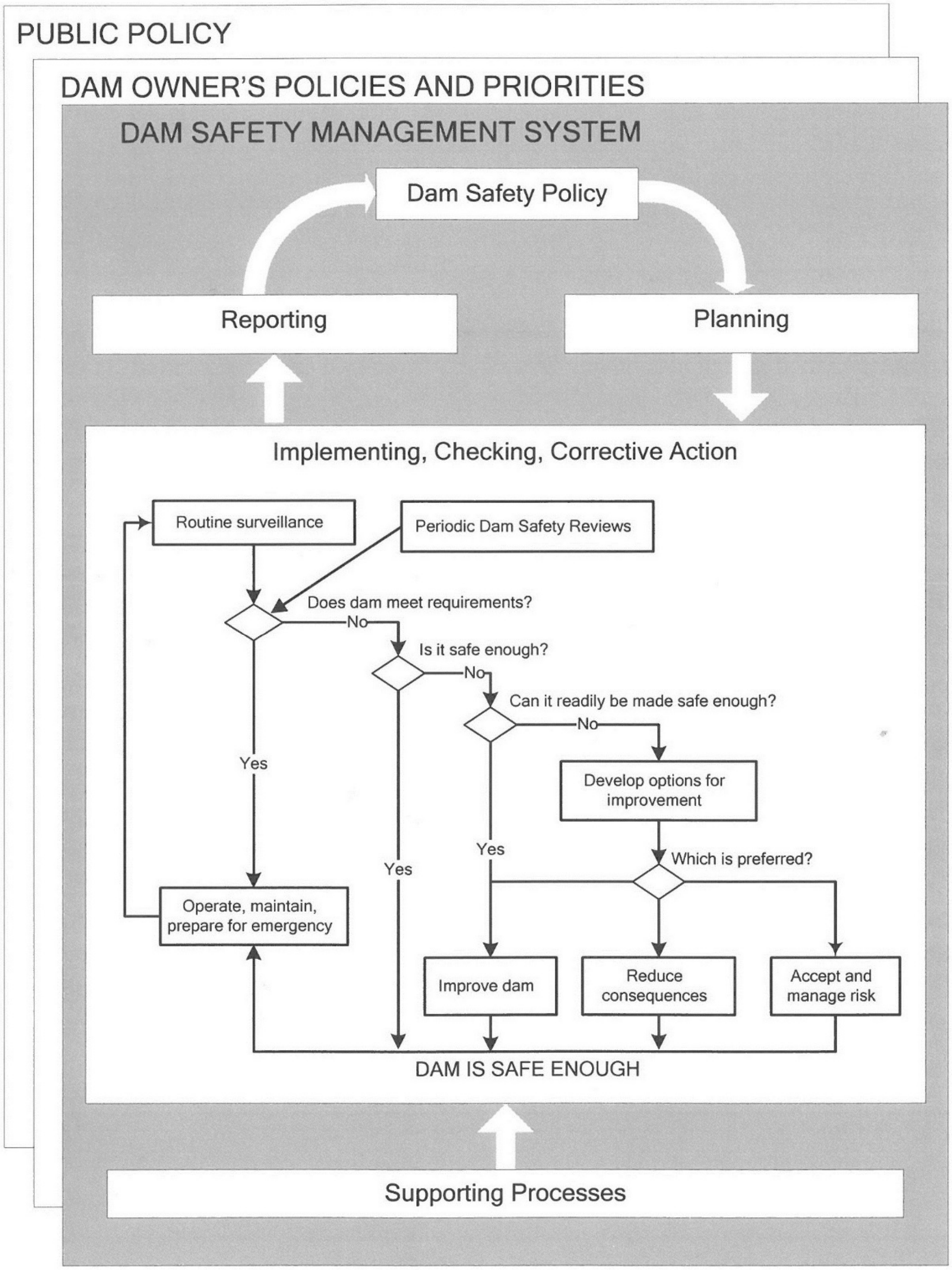
DAM SAFETY REVIEW OF STOCKING LAKE DAM

Nanaimo A Climate Station Wind Rose

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 19, 2018
DWN: AG CHK: MJL

ecora

Figure 11.5e



Notes:
Adapted from Figure 1-1 of Canadian Dam Association Dam Safety Guidelines 2007 (2013 Edition).

DAM SAFETY REVIEW OF STOCKING LAKE DAM

Dam Safety Management System

Project No. GK-18-020-CVD
Client: Cowichan Valley Regional District
Office: Kelowna
Scale: NTS
Date: September 19, 2018
DWN: CE CHK: MJL

Figure 12.1

Photographs

Photo 1	Stocking Lake Dam, January 17, 2018
Photo 2	Stocking Lake Dam, March 29, 2018
Photo 3	Seepage located at the left end of the downstream toe, March 29, 2018
Photo 4	Sinkhole at the location of the right abutment observed in January 2018, not flowing on March 29, 2018
Photo 5	Spillway channel, January 17, 2018
Photo 6	Spillway channel, March 29, 2018
Photo 7	Debris in the spillway outlet channel, March 29, 2018
Photo 8	Sign in place at the base of the dam in the spillway channel
Photo 9	Spillway inlet channel log boom in place
Photo 10	Backfilled water main trench downstream of the dam, note saturated surface.
Photo 11	Dam wider than design drawings
Photo 12	Rip-rap 0.4 m – 0.7 m extending to toe on upstream side of dam

Photo 1 **Stocking Lake Dam, January 17, 2018**

Photo 2 **Stocking Lake Dam, March 29, 2018**

Photo 3 Seepage located at the left abutment toe, March 29, 2018

Photo 4 Sinkhole at the location of the right abutment observed in January 2018, not flowing on March 29, 2018

Photo 5 **Spillway channel, January 17, 2018**

Photo 6 **Spillway channel, March 29, 2018**

Photo 7 **Debris in the spillway outlet channel, March 29, 2018**

Photo 8 **Sign in place at the base of the dam in the spillway channel**

Photo 9 Spillway inlet channel log boom in place

Photo 10 Backfilled water main trench downstream of the dam, note saturated surface

Photo 11 Dam crest wider than design drawings

Photo 12 Rip-rap 0.4 m – 0.7 m extending to toe on upstream side of dam

Appendix A

Background Information Reviewed

Background Review

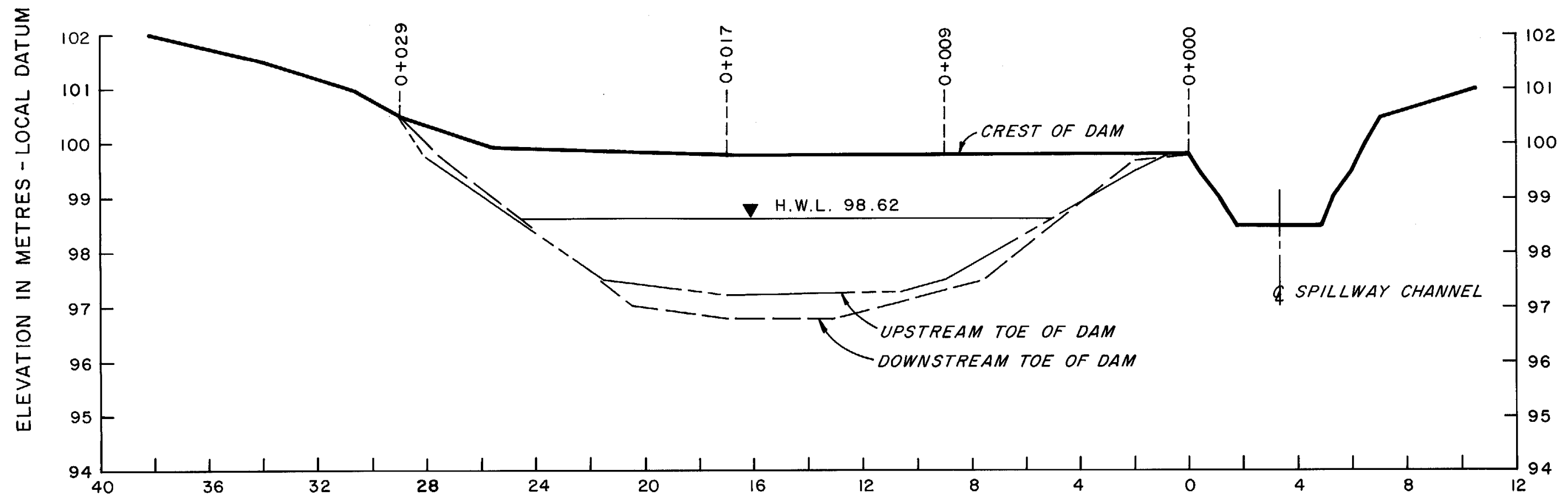
- September 2014 – Holland Lake and Stocking Lake Hydrology Update – Tetra Tech EBA
- November 2016 – Stocking Lake Dam Site Inspection – Tetra Tech EBA
- December 2016 – Stocking Lake Dam November 28, 2016 Site Inspection – Tetra Tech EBA
- May 1986 – Stocking Lake Reservoir General Plan of Dam – Ministry of Environment Water Management Branch
- May 1986 – Stocking Lake Reservoir General Plan of Dam Markup for MASW– Ministry of Environment Water Management Branch
- June 1981 – Stocking Lake Reservoir Plan of Reservoir – Ministry of Environment Water Management Branch
- April 1988 – Fig.25 Stocking Lake Dam – Province of British Columbia Ministry of Environment
- November 2011 – Upgrading of Stocking Lake Dam, Geotechnical Services – Tetra Tech EBA
- July 2016 – Stocking Lake Dam Audit Check Sheet – Water Management Branch – Dam Safety

Appendix B

Historical Dam Drawings

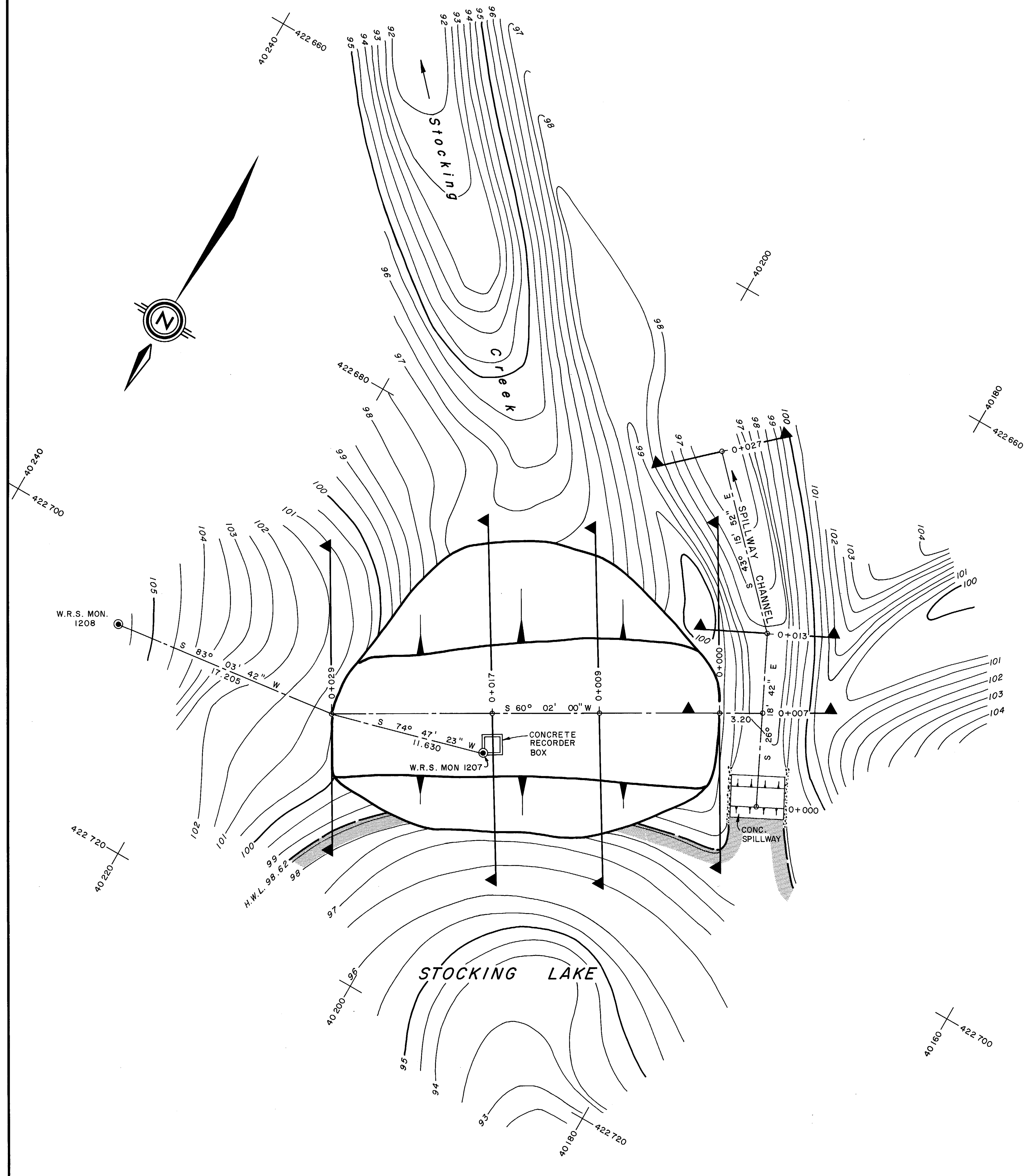
NOTE:

This information is considered historical as it represents channel conditions and configuration at the time of the survey only. User must accept responsibility of ensuring that the accuracy and completeness of provided data is suitable for the needs of the project or study. It is provided "as is" without warranty of any kind, whether express or implied. Under no circumstances will the Government of British Columbia be liable to any person or business entity for any direct, indirect, special, incidental, consequential, or other damages based on any use of this information.



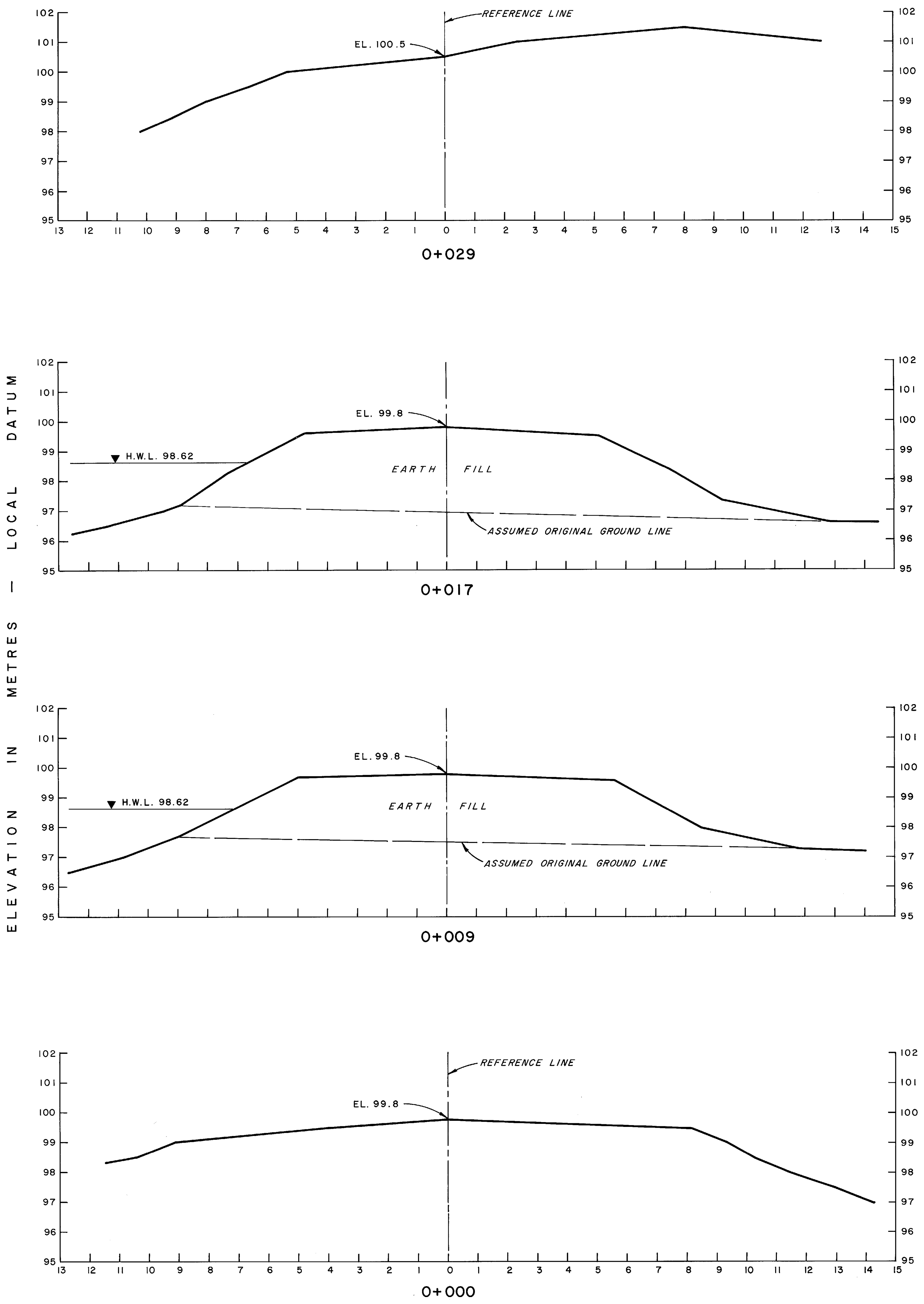
LONGITUDINAL SECTION OF DAM

SCALE HORIZ. 1:200
VERT. 1:100



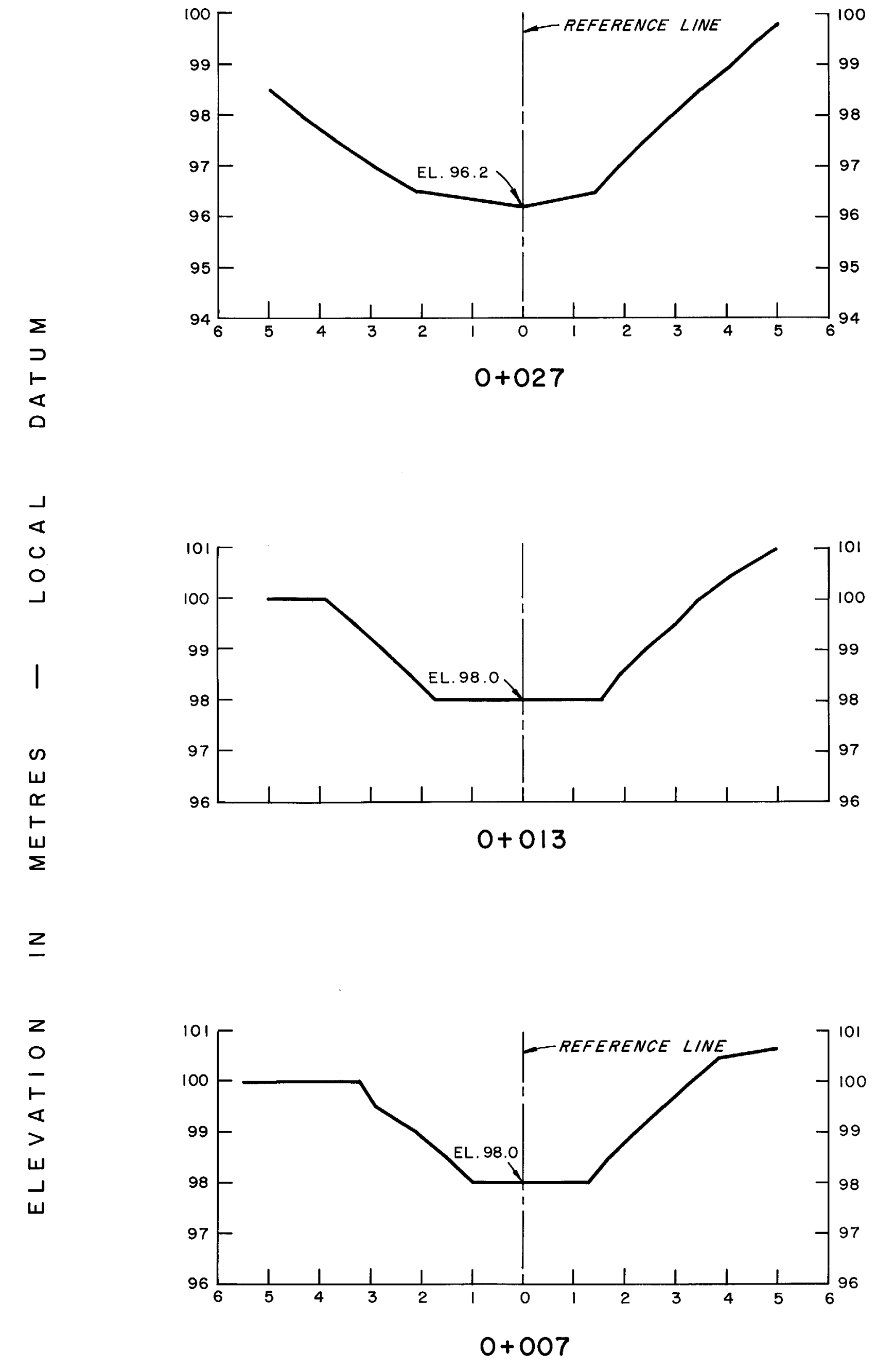
GENERAL PLAN OF DAM

0 5 10 15 20
SCALE 1:200



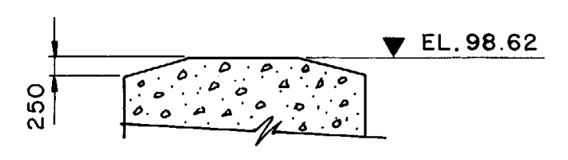
CROSS SECTIONS OF DAM

SCALE HORIZ. 1:100
VERT. 1:100

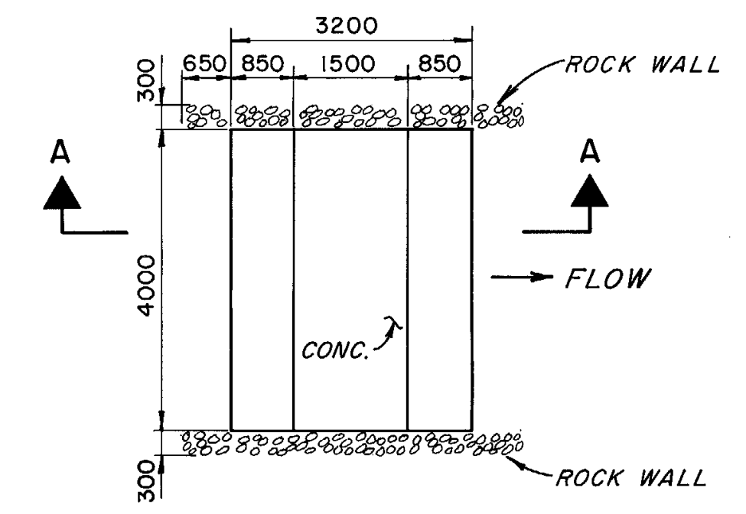


CROSS SECTIONS OF SPILLWAY

SCALE HORIZ. 1:100
VERT. 1:100



SECTION A-A



PLAN OF SPILLWAY

SCALE 1:100

NOTE : DIMENSIONS ARE IN MILLIMETRES

NOTES

FOR NOTES SEE DWG. No. 4984-12

LEGEND

- △ TRIANGULATION STATION
- ◉ REFERENCE MONUMENT (P. CON. or P. ROCK)
- ▣ 50 000 HORIZ. CONTROL POINT
- ● 60 000 VERT. CONTROL POINT
- ←T.B.M. TEMPORARY BENCH MARK
- ⬡ TEMPORARY REFERENCE POINT
- ===== ROAD, ---- TRAIL, ～ CREEK,
- CREEK INTERMITTENT, SWAMP
- BASIN, SUB BASIN
- ○→ AIR PHOTO CENTER

STORAGE LICENCES

LICENCE	PRIORITY DATE	AUTHORIZED ACRE-FEET	DEVELOPED ACRE-FEET

REFERENCES

DWG. No.	DESCRIPTION	DATE
4984-12	PLAN OF RESERVOIR	MAY 1986

REVISIONS

No.	DESCRIPTION	DATE

SURVEYED B. SCHUBERT
DATE JUNE 1981
COMPILED R. EDWARD
CHECKED G. H. Roberts
DATE MAY 1986
DRAWN B. Woods
APPROVED P. Dixon
HEAD, DRAFTING SECTION
DATE MAY 1986
ENGINEER

Province of British Columbia
Ministry of Environment
WATER MANAGEMENT BRANCH

STORAGE INVENTORY PROGRAMME
COWICHAN BASIN — VANCOUVER ISLAND SYSTEM

STOCKING LAKE RESERVOIR

GENERAL PLAN OF DAM

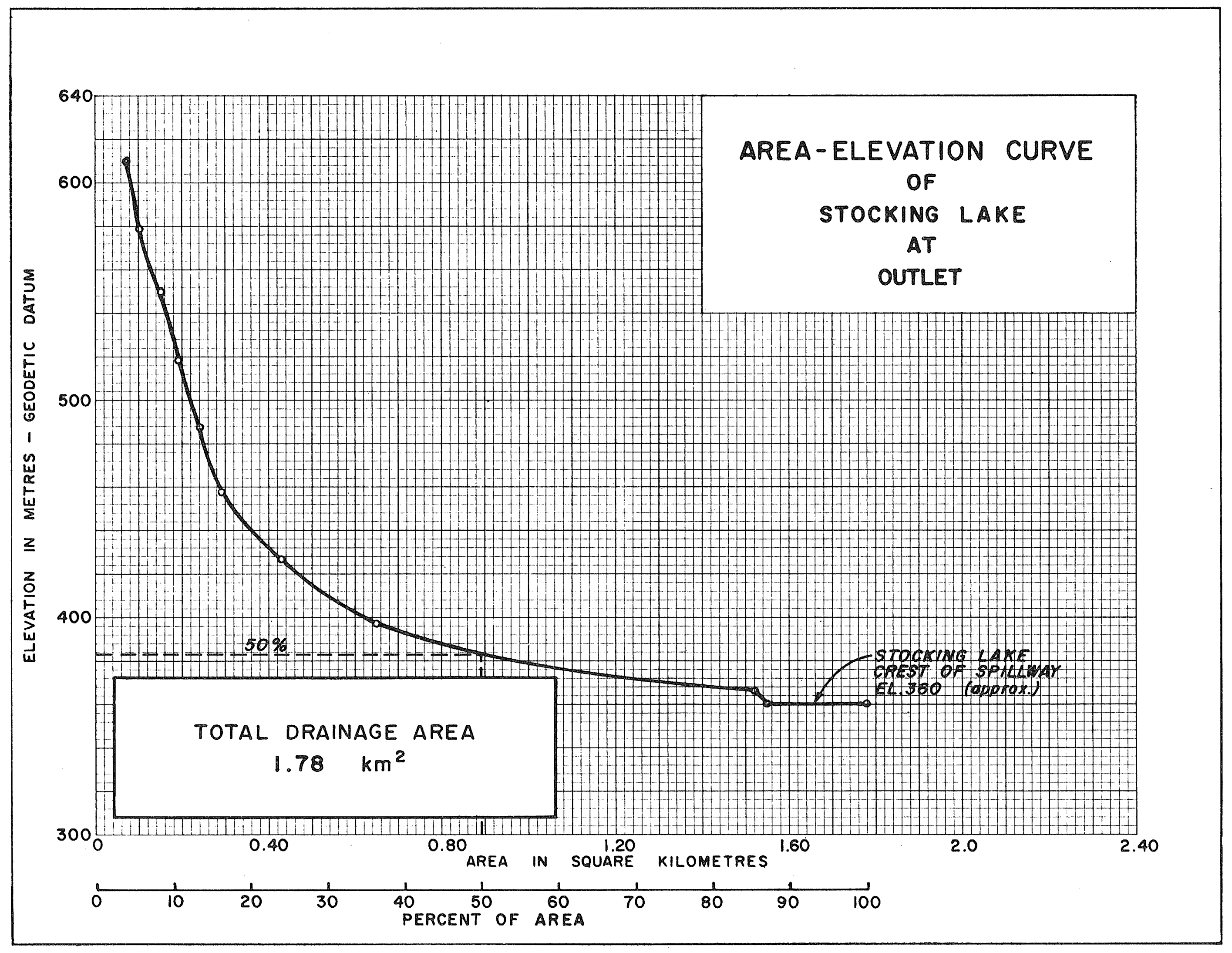
APPROVED Bernd R. Schubert
HEAD, SURVEYS SECTION
DATE MAY 1986

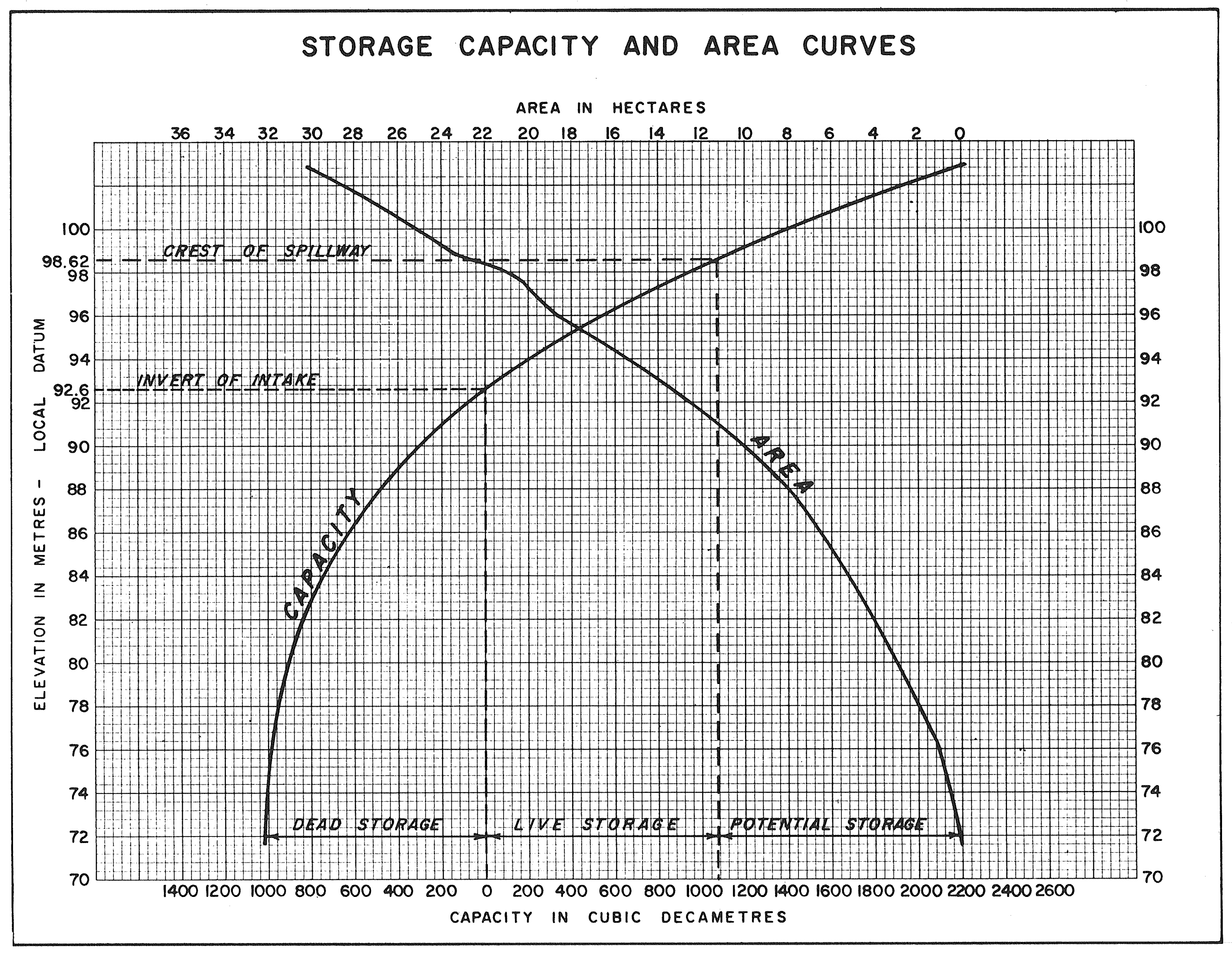
FILE Nos.
Branch
Inventory 0305080-36
SURVEY PROJECT No. 81-SIP-1
NTS Map No. 93B/13W
Map Scale.
SCALE AS SHOWN
NEGATIVE No. 272461
DRAWING No. 4984-12A
SHEET 1 of 1

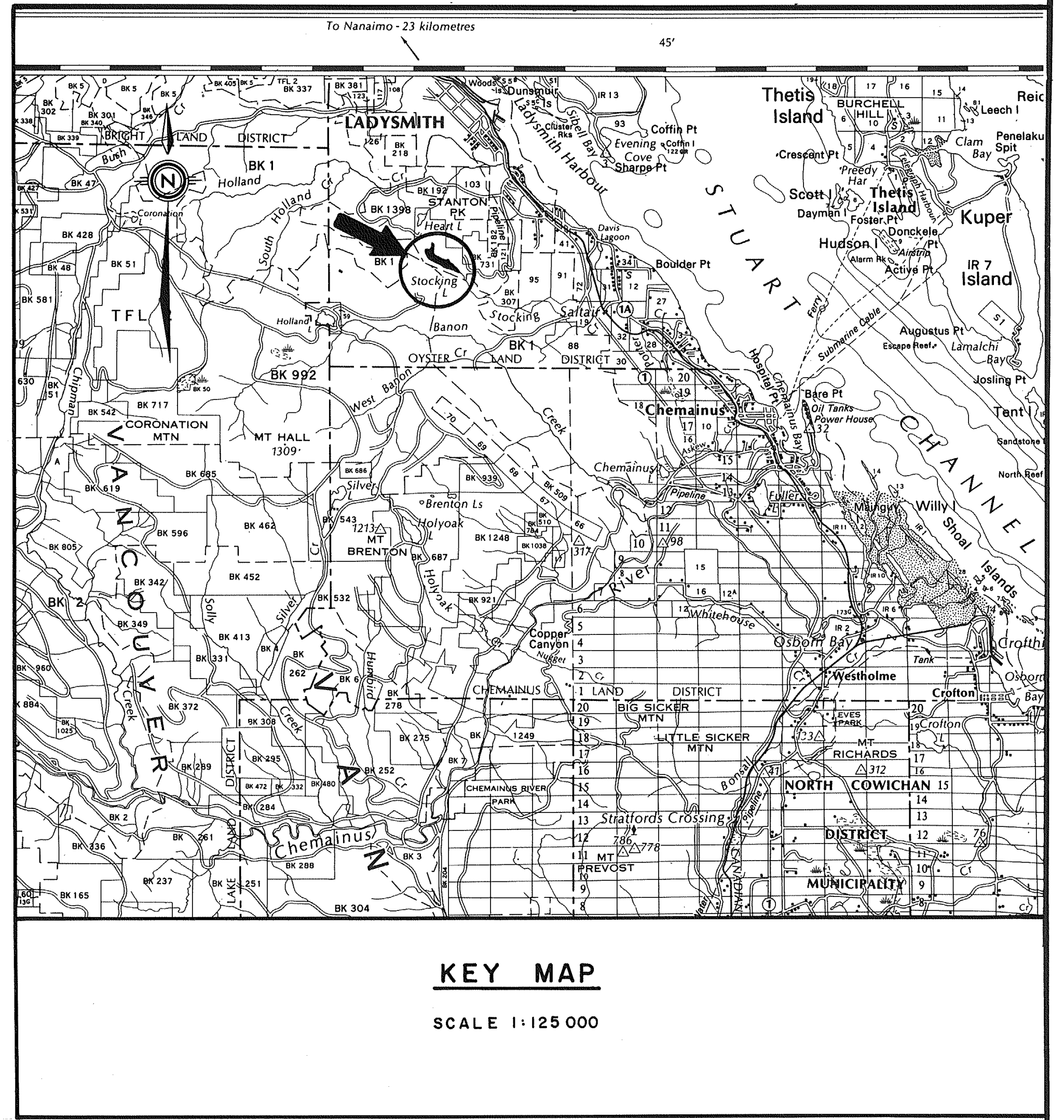
DD

STORAGE CAPACITY TABLE

MAPPING DATA			STORAGE CAPACITY				REMARKS
CONTOUR ELEVATION		Area in Hectares	Depth of Storage in Metres	Area in Hectares	STORAGE IN DECAMETRES^3		
Geodetic Datum	Local Datum				Net	Gross	
	71.7	0	-20.9			1018	Low Point in Lake
					0		
	72.0	0.18					
					2		
			-20.0	0.37		1016	
					2		
	73.0	0.50					
					3		
			-19.0	0.61		1011	
					3		
	74.0	0.68					
					4		
			-18.0	0.79		1004	
					3		
	75.0	0.86					
					6		
			-17.0	0.99		995	
					4		
	76.0	1.08					
					7		
			-16.0	1.33		984	
					6		
	77.0	1.50					
					10		
			-15.0	1.71		968	
					7		
	78.0	1.85					
					12		
			-14.0	2.18		949	
					9		
	79.0	2.40					
					15		
			-13.0	2.77		925	
					12		
	80.0	3.01					
					19		
			-12.0	3.33		894	
					14		
	81.0	3.55					
					22		
			-11.0	3.86		858	
					16		
	82.0	4.07					
					25		
			-10.0	4.42		817	
					18		
	83.0	4.65					
					29		
			-9.0	5.00		770	
					21		
	84.0	5.24					
					33		
			-8.0	5.62		716	DEAD STORAGE
					23		
	85.0	5.87					
					36		
			-7.0	6.25		657	
					26		
	86.0	6.51					
					40		
			-6.0	6.95		591	
					28		
	87.0	7.24					
					45		
			-5.0	7.71		518	
					32		
	88.0	8.02					
					50		
			-4.0	8.73		436	
					36		
	89.0	9.21					
					57		
			-3.0	9.78		343	
					40		
	90.0	10.2					
					62		
			-2.0	10.8		241	
					44		
	91.0	11.2					
					11		
			-1.5	11.3		186	
					23		
			-1.3	11.6		163	
					24		
			-1.1	11.9		139	
					24		
			-0.9	12.2		115	
					24		
			-0.7	12.4		91	
					12		
	92.0	12.6					
					13		
			-0.5	12.7		66	
					26		
			-0.3	13.0		40	
					27		
			-0.1	13.4		13	
					13		
	92.6		0	13.5		0	Invert of Intake
					14		
			0.1	13.7		14	
					28		
			0.3	14.0		42	
					28		
			0.5	14.3		70	
					29		
			0.7	14.6		99	
					29		
			0.9	14.8		128	
					30		
			1.1	15.1		158	
					30		
			1.3	15.4		188	
					15		
	94.0	15.5					
					16		
			1.5	15.7		219	
					32		
			1.7	16.0		251	
					32		
			1.9	16.3		283	
					33		
			2.1	16.6		316	
					33		
			2.3	16.9		349	
					17		
	95.0	17.0					
					17		
			2.5	17.2		383	
					35		
			2.7	17.6		418	
					36		
			2.9	17.9		454	
					36		
			3.1	18.3		490	
					37		
			3.3	18.6		527	
					37		
			3.5	18.9		564	LIVE STORAGE
					38		
			3.7	19.1		602	
					38		
			3.9	19.3		640	
					39		
			4.1	19.4		679	
					39		
			4.3	19.6		718	
					20		
	97.0	19.7					
					20		
			4.5	19.8		758	
					40		
			4.7	19.9		798	
					40		
			4.9	20.1		838	
					40		
			5.1	20.2		878	
					41		
			5.3	20.4		919	
					20		
	98.0	20.5					
					21		
			5.5	20.9		960	
					42		
			5.7	21.6		1002	
					44		
			5.9	22.4		1046	
					27		
	98.62	22.9	6.0			1073	Crest of Spillway
					18		
			6.1	23.0		1091	
					46		
			6.3	23.3		1137	
					24		
	99.0	23.4					
					24		
			6.5	23.6		1185	
					48		
			6.7	24.0		1233	
					48		
			6.9	24.3		1281	
					49		
			7.1	24.6		1330	POTENTIAL STORAGE
					49		
			7.3	25.0		1379	
					25		
	100.0	25.2					
					26		
			7.5	25.3		1430	
					128		
			8.0	26.1		1558	
					106		
	101.0	26.7					
					163		
			9.0	27.8		1827	
					113		
	102.0	28.5					
					174		
			10.0	29.6		2114	
					120		
	103.0	30.3	10.4			2234	

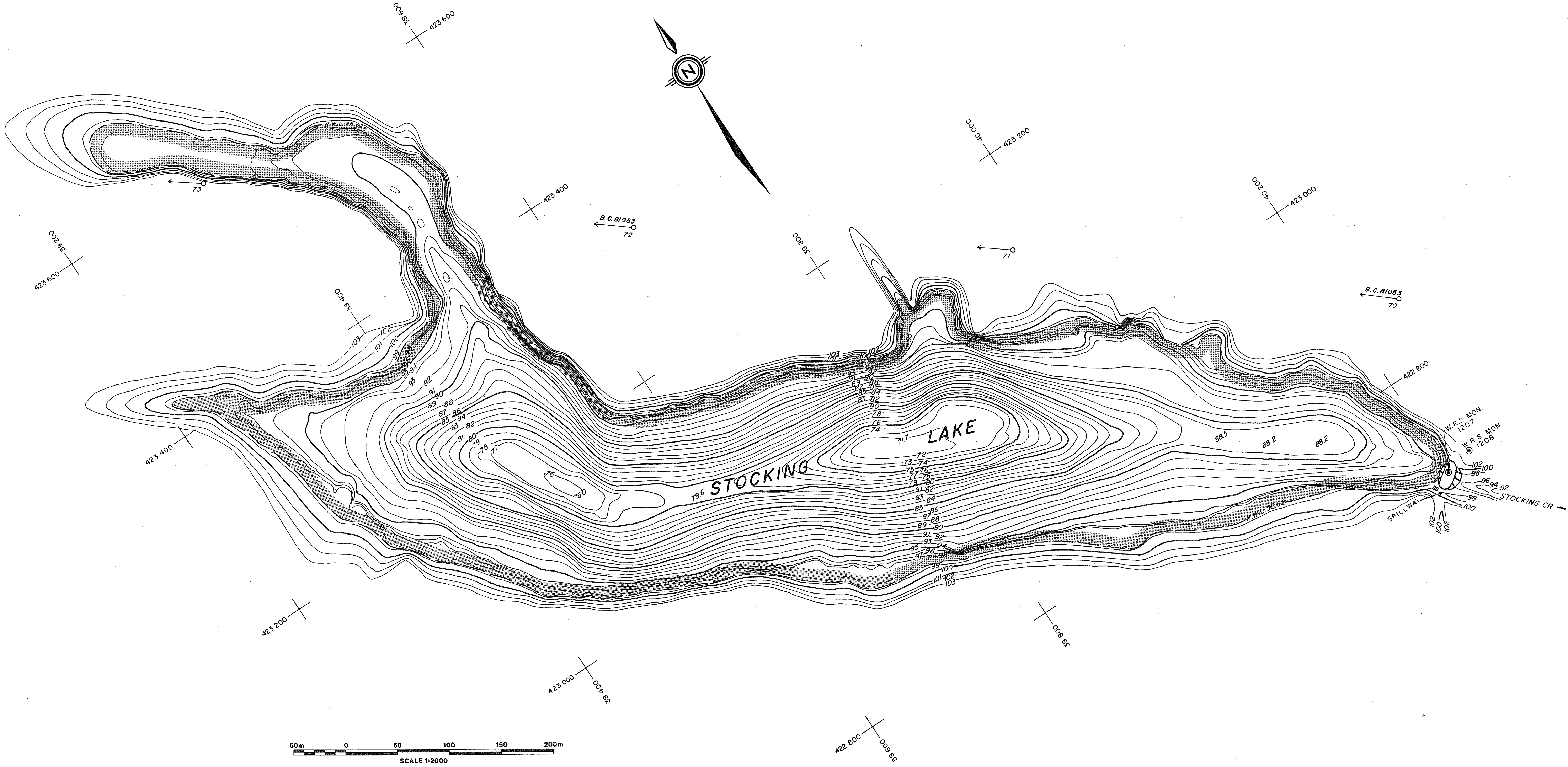






KEY MAP

SCALE 1:125 000



NOTE:

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NOTES

1) This plan has been prepared from field surveys carried out by the Surveys Section, Water Management Branch, Ministry of Environment, Province of British Columbia.

2) SURVEY DATA:
a) Horizontal control was established by traverse using Hewlett-Packard Distance Meter, Model 3800B.
b) Subaqueous contours were drawn from depths established by Raytheon Depth Recorder, Model DE-719B.
c) Position control for the bathymetry was maintained by simultaneous fixes from theodolites which were set over coordinated shore stations.
d) Topography and cross-sections of dam were obtained by stadia.
e) Contours above water level were obtained from shore sections using inclinometer.
f) The survey was carried out in June 1981.

3) DATUM:
a) Coordinates are referred to W.R.S. Monument 1207 which has an assigned value of Northing 422 700 and Easting 40 200.
b) Elevations are in metres and are referred to W.R.S. Monument 1207 which was assigned an arbitrary elevation of 100.
c) Bearings are solar grid referred to Longitude 123°.

4) FIELD BOOKS:
Survey data are recorded in Field Books No. 2293 L-1 and 2293 I-1.

5) AIR PHOTOGRAPHS:
B.C.81053, Frames 69-74.
Date of Photography : July 15, 1981.
Photo Scale: 1:5000 approx.

6) MAPPING:
The lake perimeter was mapped under Project No. 82-037T at a scale of 1:2500 using second order photogrammetric equipment.

7) REFERENCE MONUMENTS:

	Northing	Easting	Elevation
W.R.S. Mon. 1207	422 700.000	40 200.000	100.000
W.R.S. Mon. 1208	422 705.140	40 228.386	105.760
Mon. 1207 → Mon. 1208	N 79-44-07 E	28.847	

8) Intake not located in the field. Invert elevation 92.6 calculated from crest of spillway elevation using live storage height of 6 metres established by Town of Ladysmith in 1968.

LEGEND

- △ TRIANGULATION STATION
- ◎ REFERENCE MONUMENT (P. CON. or P. ROCK)
- ▣ 50 000 HORIZ. CONTROL POINT
- • 60 000 VERT. CONTROL POINT
- ← T.B.M. TEMPORARY BENCH MARK
- ⬡ TEMPORARY REFERENCE POINT
- ===== ROAD, ---- TRAIL, ∼ CREEK,
- CREEK INTERMITTENT, SWAMP
- BASIN, SUB BASIN
- B.C. 81053 / 70 → AIR PHOTO CENTER

STORAGE LICENCES

LICENCE	PRIORITY DATE	AUTHORIZED ACRE-FEET	DEVELOPED
C.L. 23488	July 23,1956	100	1073dam^3

1 cubic decametre = 0.810713 acre foot.

REFERENCES

DWG. No.	DESCRIPTION	DATE
4984-12A	GENERAL PLAN OF DAM	MAY 1986

REVISIONS

No.	DESCRIPTION	DATE

SURVEYED B. SCHUBERT
DATE JUNE 1981
COMPILED J. TAM
CHECKED G. H. Roberts
DATE MARCH 1986
DRAWN B. McInnes
APPROVED P. Dixon
HEAD, DRAFTING SECTION
DATE MARCH 1986
ENGINEER

Province of British Columbia
Ministry of Environment
WATER MANAGEMENT BRANCH

STORAGE INVENTORY PROGRAMME
COWICHAN BASIN - VANCOUVER ISLAND SYSTEM

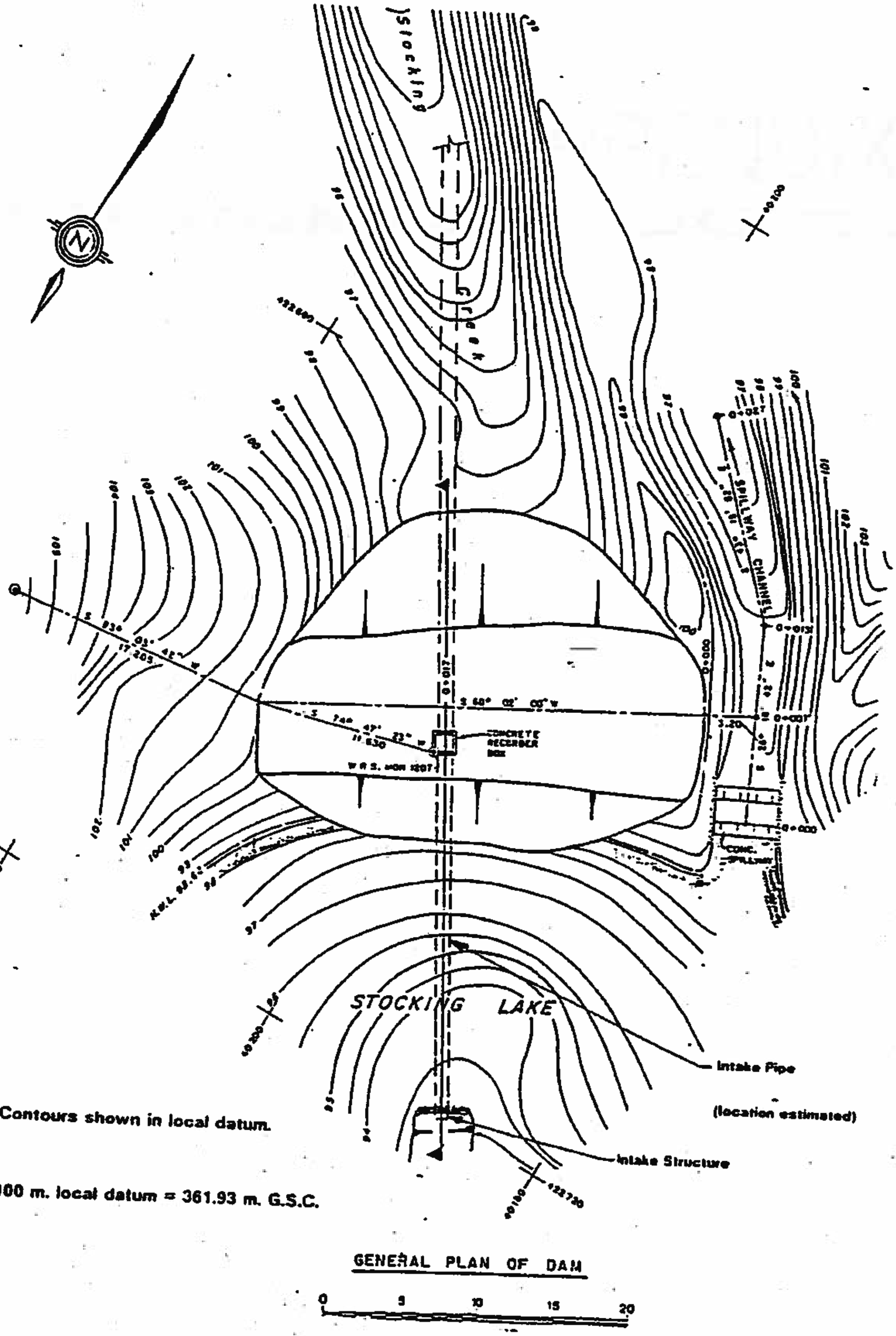
STOCKING LAKE RESERVOIR

PLAN OF RESERVOIR

APPROVED Bernd R. Schubert
HEAD, SURVEYS SECTION
DATE MAY 1986

FILE Nos.
Branch
Inventory 0305080-36
SURVEY PROJECT No. 81-SIP-1
NTS Map No. 92B/13
Map Scale 1:50000
SCALE 1:2000
NEGATIVE No. 272 460
DRAWING No. 4984-12
SHEET of

DD



– Contours shown in local datum.

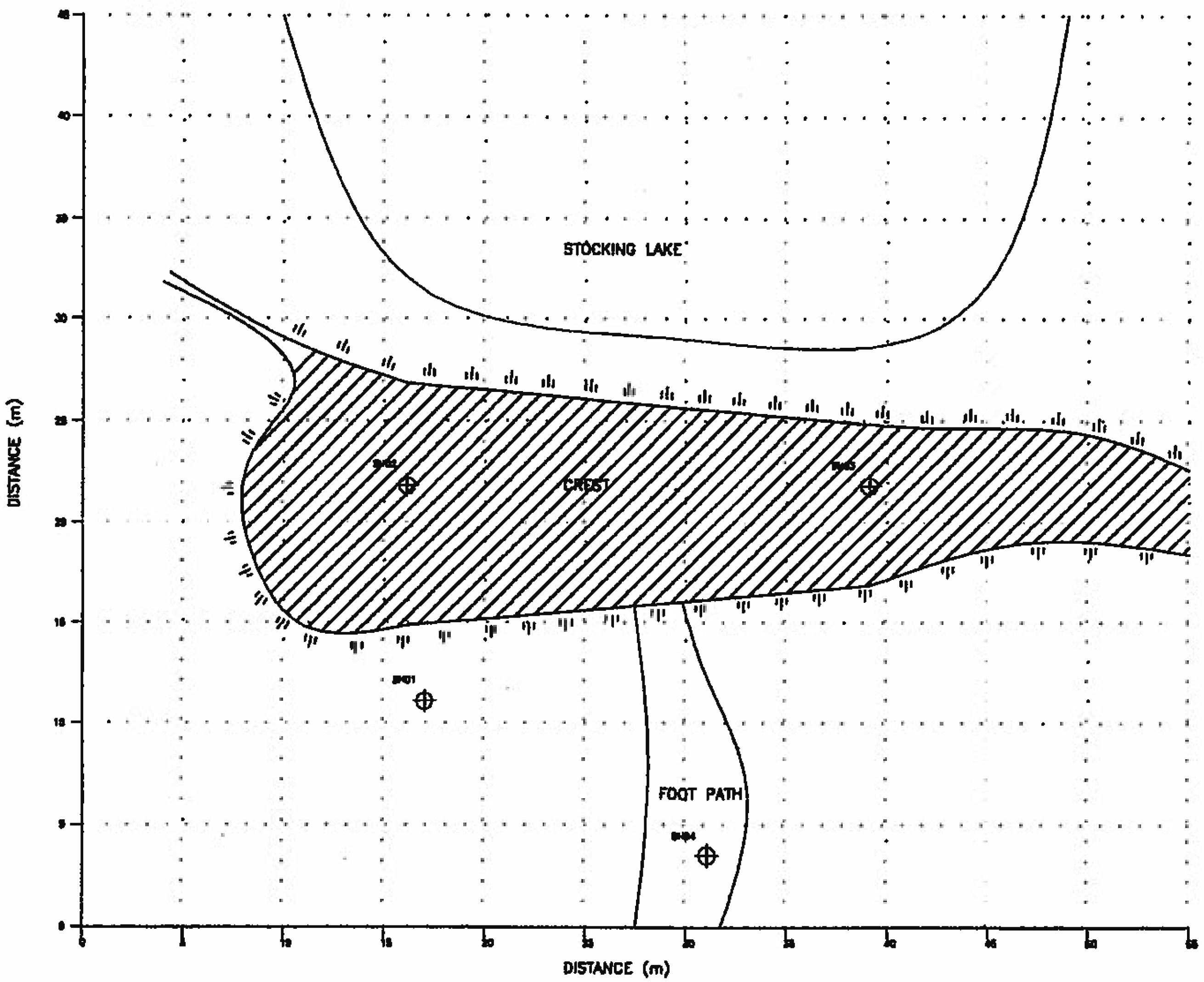
–100 m. local datum = 361.93 m. G.S.C.

GENERAL PLAN OF DAM

Province of British Columbia
Ministry of Environment

FIG. - 25
STOCKING LAKE DAM

SCALE : NTS
DATE : APRIL 1988



LEGEND:

⊕ APPROXIMATE BOREHOLE LOCTION

▨ DAM STRUCTURE

SCALE: NTS

CLIENT

TOWN OF LADYSMITH

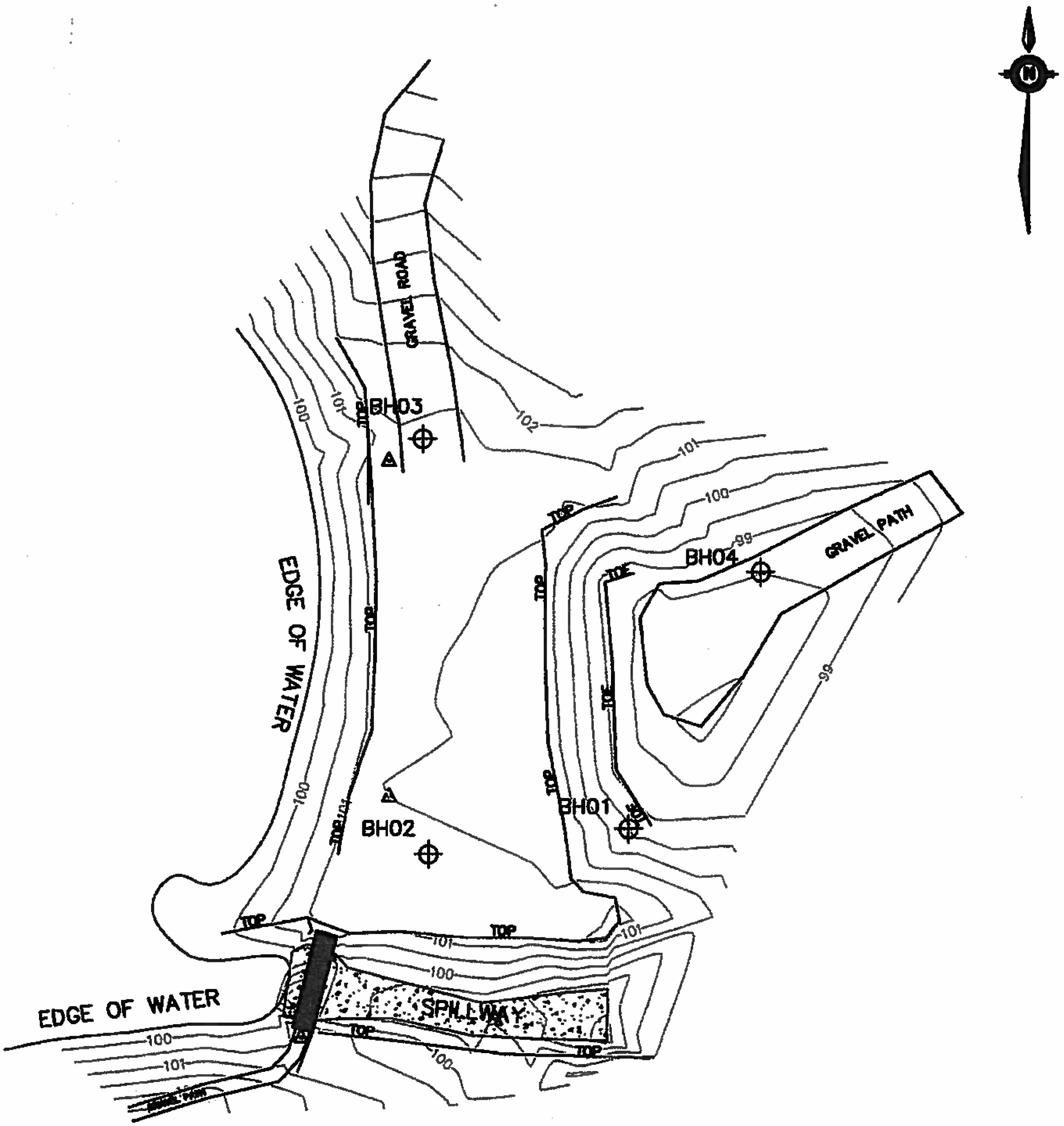
EBA Engineering Consultants Ltd.

STOCKING LAKE DAM
INVESTIGATION

**BOREHOLE LOCATION
PLAN**

PROJECT NO.	DWN	CKD	REV
2800172	RMG	PJM	0
OFFICE	DATE		
EBA-NANAIMO	Sept. 20, 2008		

Figure 1



LEGEND:

⊕ APPROXIMATE BOREHOLE LOCATIONS

SCALE: NTS

CLIENT

TOWN OF LADYSMITH

STOCKING LAKE DAM INVESTIGATION

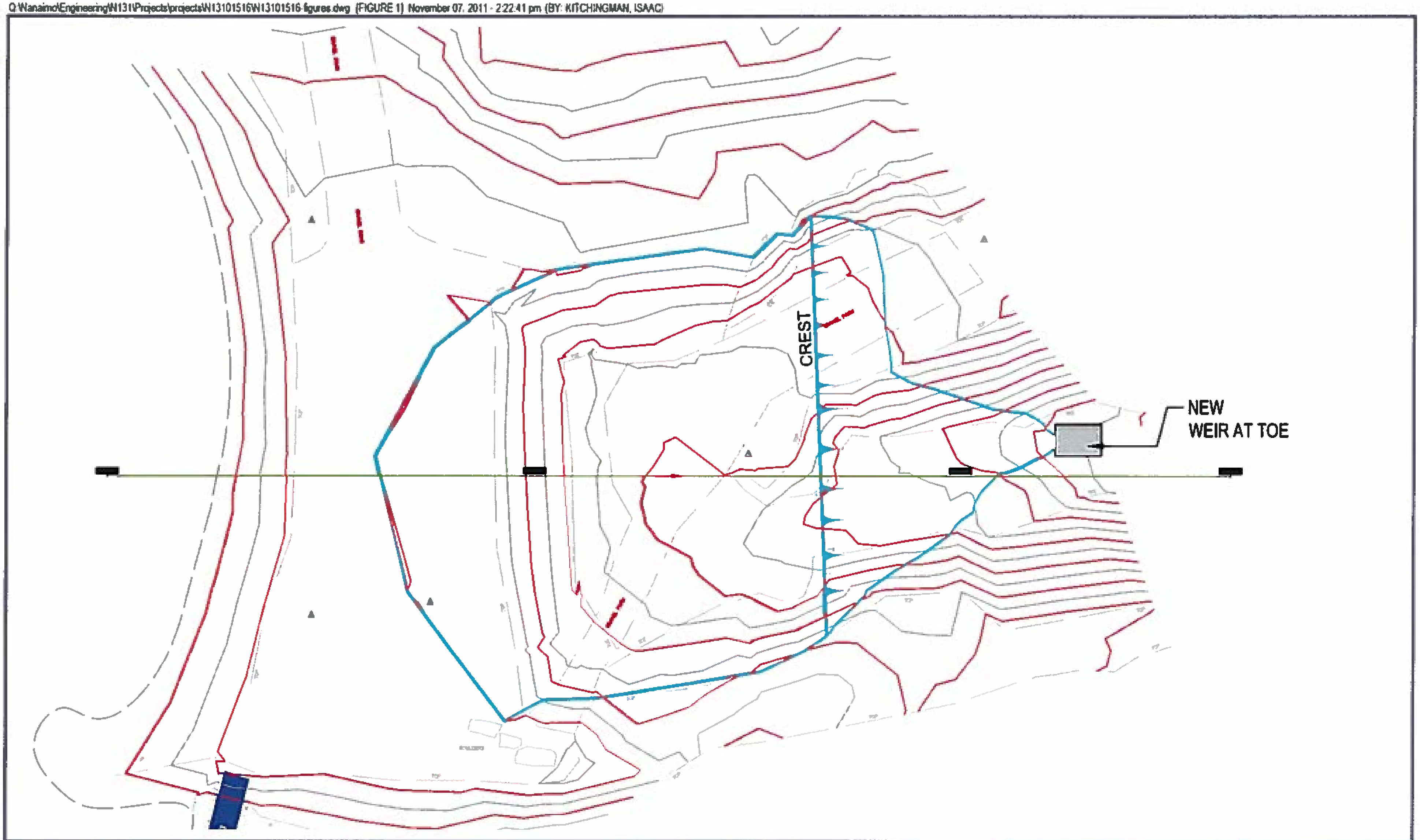
BOREHOLE LOCATION PLAN

EBA Engineering Consultants Ltd.

PROJECT NO.	DWN	CKD	REV
2800172.002	RMG	PJM	0
OFFICE	DATE		
EBA-NANAIMO	Sept. 21, 2006		

Figure 2

Q:\Nanaimo\Engineering\N131\Projects\projects\N13101516\N13101516-figures.dwg (FIGURE 1) November 07, 2011 - 2:22:41 pm (BY: KITCHINGMAN, ISAAC)



LEGEND:

▨ - PROPOSED DAM EXTENTS

0 10m

Scale: 1: 250

ISSUED FOR USE

CLIENT

TOWN OF LADYSMITH

eba A TETRA TECH COMPANY

STOCKING LAKE DAM
STABILIZATION

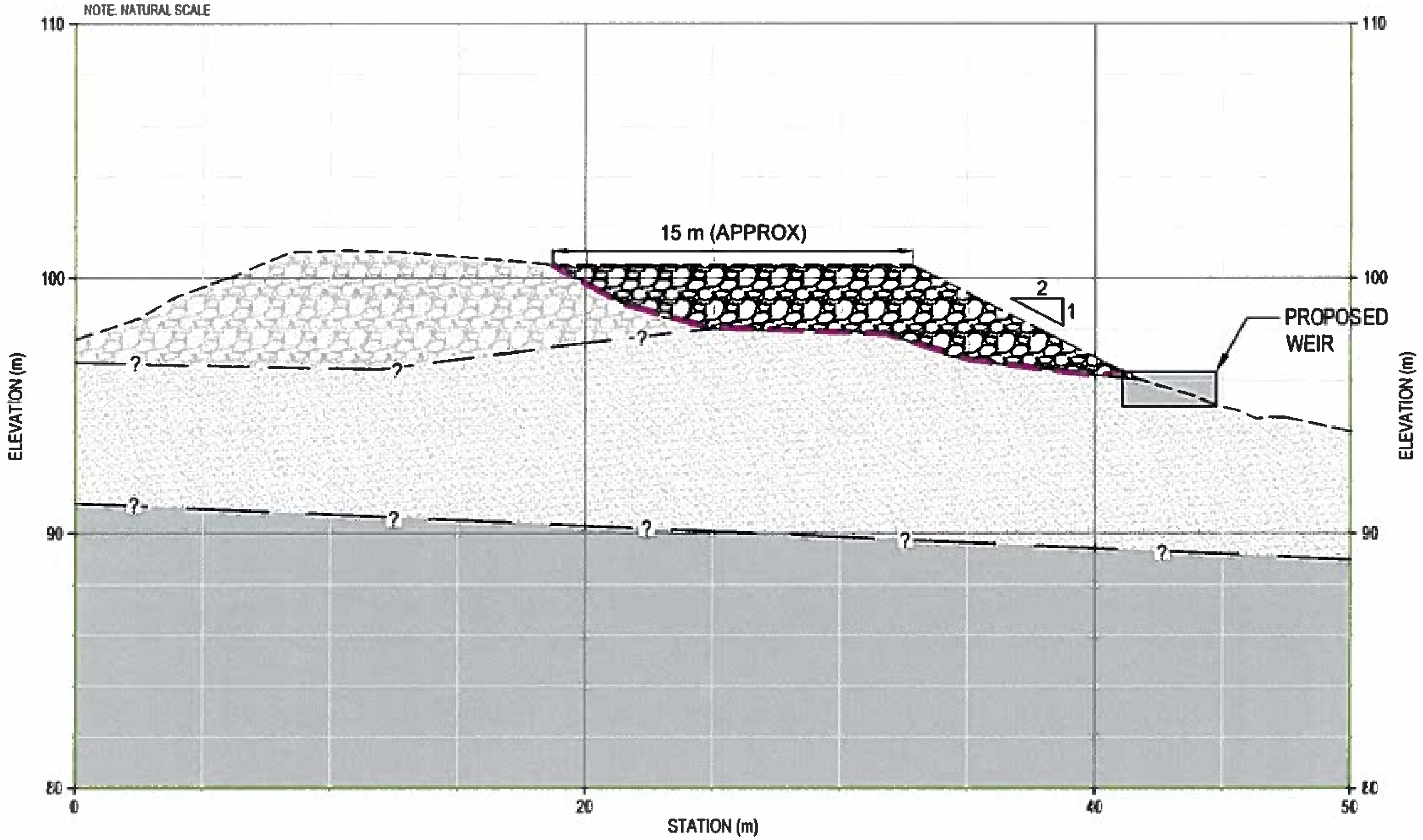
PROPOSED DAM BUTTRESS PLAN

PROJECT NO	DWN	CKD	REV
N13101516	IK	AW	0

OFFICE	DATE
NAN	November 1, 2011

Figure 1

Q:\Nanaimo\Engineering\N131\Projects\projects\N13101516\N13101516-figures.dwg [FIGURE 2] November 07, 2011 - 2:22:59 pm (BY: KITCHINGMAN, ISAAC)



CENTRE SECTION

LEGEND:

- ▨ - NEW DAM BUTTRESS FILL
- ▨ - SAND AND GRAVEL FILL
- ▨ - NATIVE SAND AND GRAVEL
- ▨ - BEDROCK
- ╱ - FILTER OPTION

0 10m

Scale: 1: 250

NOTES

FIGURES 1 & 2 ARE BASED ON DRAWING PROVIDED BY KOERS & ASSOCIATES LTD

SOIL STRATA HAVE BEEN INFERRED AND ARE INTENDED TO BE USED FOR GENERAL SITE ANALYSIS ONLY

ISSUED FOR USE

CLIENT

TOWN OF LADYSMITH

A TETRA TECH COMPANY

STOCKING LAKE DAM STABILIZATION

CENTRE SECTION

PROJECT NO	DWN	CKD	REV
N13101516	IK	AW	0

OFFICE	DATE
NAN	November 1, 2011

Figure 2

Appendix C

Dam Inspection Notes

Table E Site Inspection Observations of the Stocking Lake Dam

General Description of Dam			
Date:	March 29, 2018	Attendees:	Michael J. Laws, P.Eng. (Ecora), Caleb Pomeroy, P.Eng. (Ecora), Dr. Adrian Chantler, P.Eng. (Ecora), Bram Hobuti, P.Eng. (Ecora), David Parker (CVRD)
Weather:	Cloudy	Location:	Cowichan Valley Regional District
Length:	30 m	Outlet type:	450 mm low level outlet
Max. Height:	3.3 m	Sluice gate:	Valve
Crest Elevation:	362.28 m	Spillway:	3.7 m
Crest Width:	8.5 m	Spillway Crest Elevation:	360.78 m
Water Level:	357.294 m	Downstream Slope Angle:	2H:1V
Appurtenances:	Spillway	Upstream Slope Angle:	2.5H:1V
Observations			
Location			
Left Toe	Seepage was noted at the left end of the downstream toe of the dam. The water was noted as appearing clear.		
Right Abutment	Sinkhole noted in previous inspections is no longer flowing.		
Reservoir	A log boom is in place across the spillway channel inlet.		
Dam Crest	No vehicle access is provided, all terrain vehicles are able to travel a pathway down to the dam.		
Dam Crest	Crest is noted to be wider than indicated on design drawings.		
Spillway	Debris was noted to be in the spillway at the time of inspection.		
Spillway	Measured to have 4.7 m top width, approximately 0.8 m deep and have a bottom width of 4 m.		
Downstream Face	Rip-rap, 0.4 – 0.7 m in size, extends to toe of dam.		
Downstream Face	Sign locate at base of dam in the spillway channel.		

Appendix D

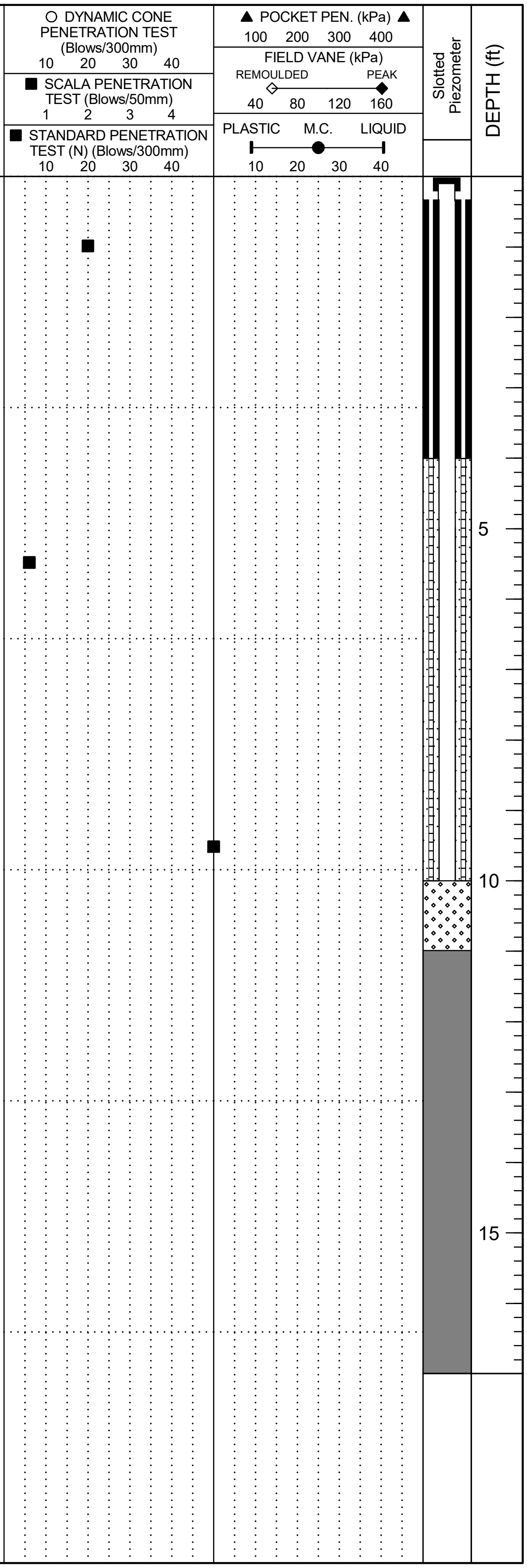
Geotechnical Investigation Data

BOREHOLE: BH18-01

Project: Stocking Lake Dam Drilling Investigation
Location: Stocking Lake
Zone: 10 Northing: 5422984.867 Easting:440055.85

Project No: GK-18-020-CVD
Client: Cowichan Valley Regional District
Elevation: 362.161 m

DEPTH (m)	DRILLING METHOD	DESCRIPTION (For Explanation of Terms, Symbols and Abbreviations See Attached Key Sheet)	SAMPLE TYPE	SAMPLE NUMBER	PARTICLE SIZE DISTRIBUTION GRAVEL (%)	SAND (%)	FINES (%)	STANDARD PENETRATION TEST (N)
0 – 1.22		**SAND (FILL)** (0 m to 1.22 m) Compact, Gravelly SAND, some Silt, Subangular to Subrounded gravel, rootlets up to 1 mm thick in the upper 0.2 m, moist, brown.						5 10 10 N=20
1.22 – 2.44		**SAND (FILL)** (1.22 m to 2.44 m) Loose, SAND, Silty, some Gravel, brown to grey at 2.1 m, wet.	S	1	18.8	60.0	21.2	2 3 3 N=6
2.44 – 2.97		**SAND (TOPSOIL)** (2.44 m to 2.97 m) Loose, Silty SAND, trace Gravel, some black organics and fibrous wood, grey to black, wet.						4 for 10 mm N=50+
2.97 – 3.35		**GRAVEL (TILL)** (2.97 m to 3.35 m) Dense to very dense, Silty Sandy GRAVEL, grey, wet to moist at 3.0 m.						150 24 N=174
3.35 – 5.18		**BEDROCK (BEDROCK)** (3.35 m to 5.18 m) Intrusive Igneous BEDROCK, Medium grained, moderately fractured.						
		End of Borehole at 5.18 m due to Bedrock.						



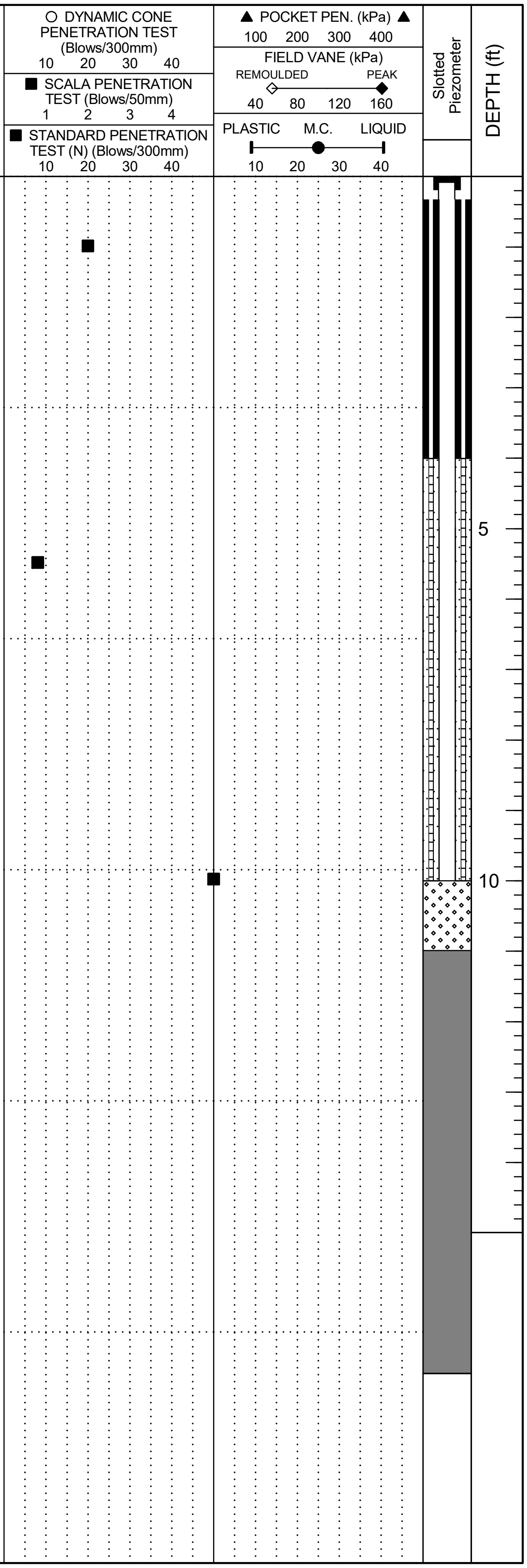
Contractor: Drillwell
Drilling Rig Type: Sonic
Logged By: PW
Reviewed By: MJL
Started: 2018-04-17
Completed: 2018-04-17
Hole Inclination: °
Hole Orientation: °
Completion Depth: 5.18m

BOREHOLE: BH18-02

Project: Stocking Lake Dam Drilling Investigation
Location: Stocking Lake
Zone: 10 Northing: 5422979.28 Easting: 440058.37

Project No: GK-18-020-CVD
Client: Cowichan Valley Regional District
Elevation: 361.96

DEPTH (m)	DESCRIPTION (For Explanation of Terms, Symbols and Abbreviations See Attached Key Sheet)	SAMPLE TYPE	SAMPLE NUMBER	GRAVEL (%)	SAND (%)	FINES (%)	STANDARD PENETRATION TEST (N)
0 – 1.22	**SAND (FILL)** (0 m to 1.22 m) Compact, Silty SAND, some Gravel, Subangular to Subrounded gravel, rootlets up to 1 mm thick in the upper 0.2 m, moist, brown.						2 11 9 N=20
		D	2	18.4	63.3	18.3	
1.22 – 1.52	**WOOD (FILL)** (1.22 m to 1.52 m) Tree truck ~300 mm in diameter.						
1.52 – 1.67	**GRAVEL (FILL)** (1.52 m to 1.67 m) Loose, GRAVEL, some Silt, subrounded to Rounded gravel, wet, rusty.						2 4 4 N=8
1.67 – 3.2	**SAND (ALLUVIAL DEPOSITS)** (1.67 m to 3.2 m) Compact, Silty Gravelly SAND, laminated, wet, brown to Grey	S	3	29.3	41.1	29.6	9 13 45 for 137.5 mm N=50+
3.2 – 3.96	**SAND (TILL)** (3.2 m to 3.96 m) Compact, Silty Gravelly SAND, grey, wet to moist at 3.65 m. Becoming dense at 3.65 m.	D	4	50.4	26.4	23.2	
3.96 – 4.57	**BEDROCK (BEDROCK)** (3.96 m to 4.57 m) Intrusive Igneous BEDROCK, Medium grained, moderately fractured.						
	End of Borehole at 4.57 m due to Bedrock.						

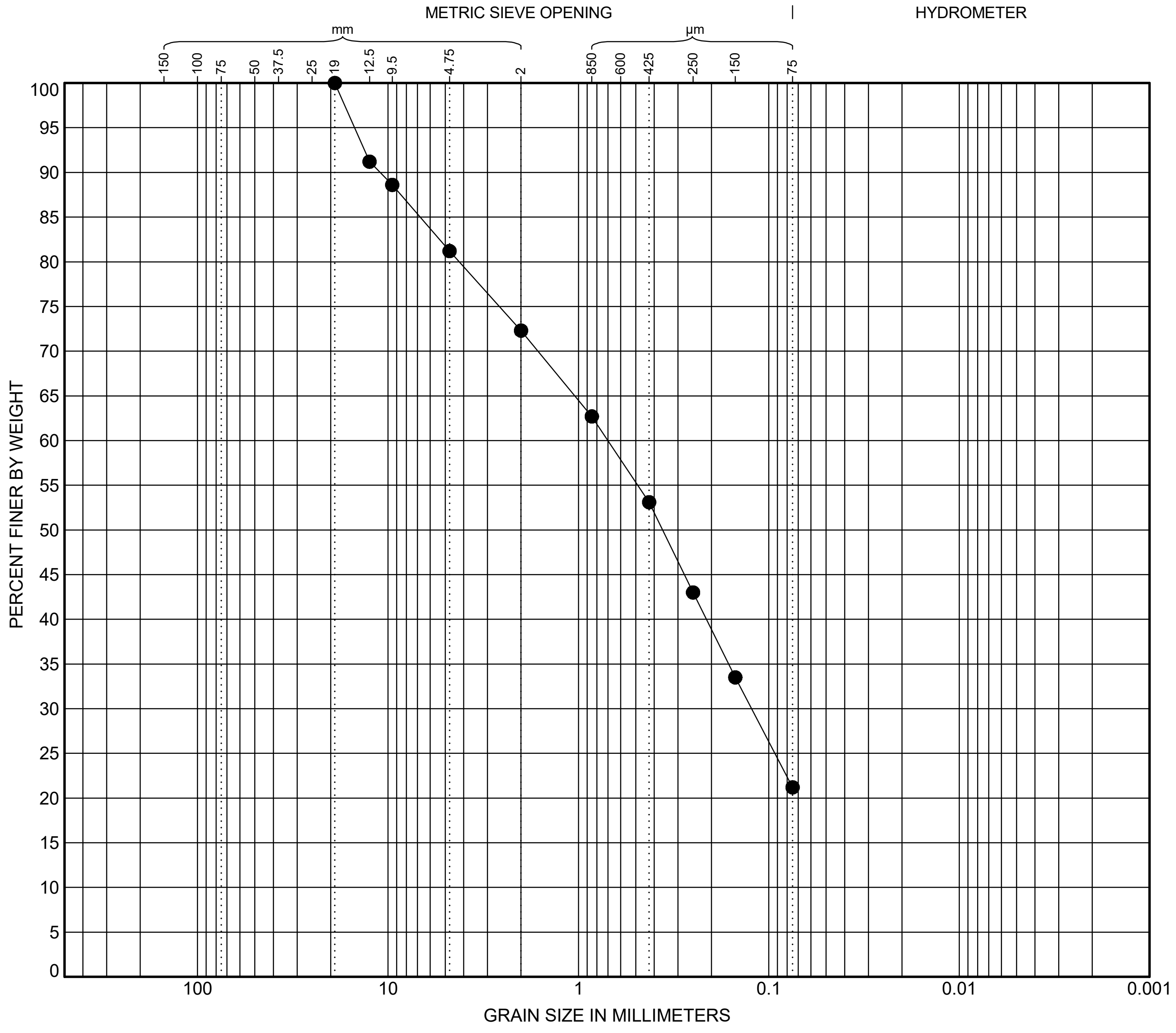


Contractor: Drillwell
Drilling Rig Type: Sonic
Logged By: PW
Reviewed By: MJL
Started: 2018-04-17
Completed: 2018-04-17
Hole Inclination: °
Hole Orientation: °
Completion Depth: 4.57m

GRAIN SIZE DISTRIBUTION ASTM C136

Project: Stocking Lake Dam Drilling Investigation
Location: Stocking Lake, Ladysmith BC
Sample Location/Source: BH18-01

Project No: GK-18-020-CVD
Client: Cowichan Valley Regional District
Depth: 1.37 m to 1.83 m



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

D_{100}	D_{85}	D_{60}	D_{30}	D_{15}	D_{10}	C_c	C_u	%Gravel	%Sand	%Silt	%Clay
19	**6.781**	**0.699**	**0.123**					**19**	**60**	**21**	

Description: Sand, Silty Fines, some Gravel
Natural Moisture Content: 16.7%
Material Specification: N/A
Intended Use: N/A
Comments: N/A

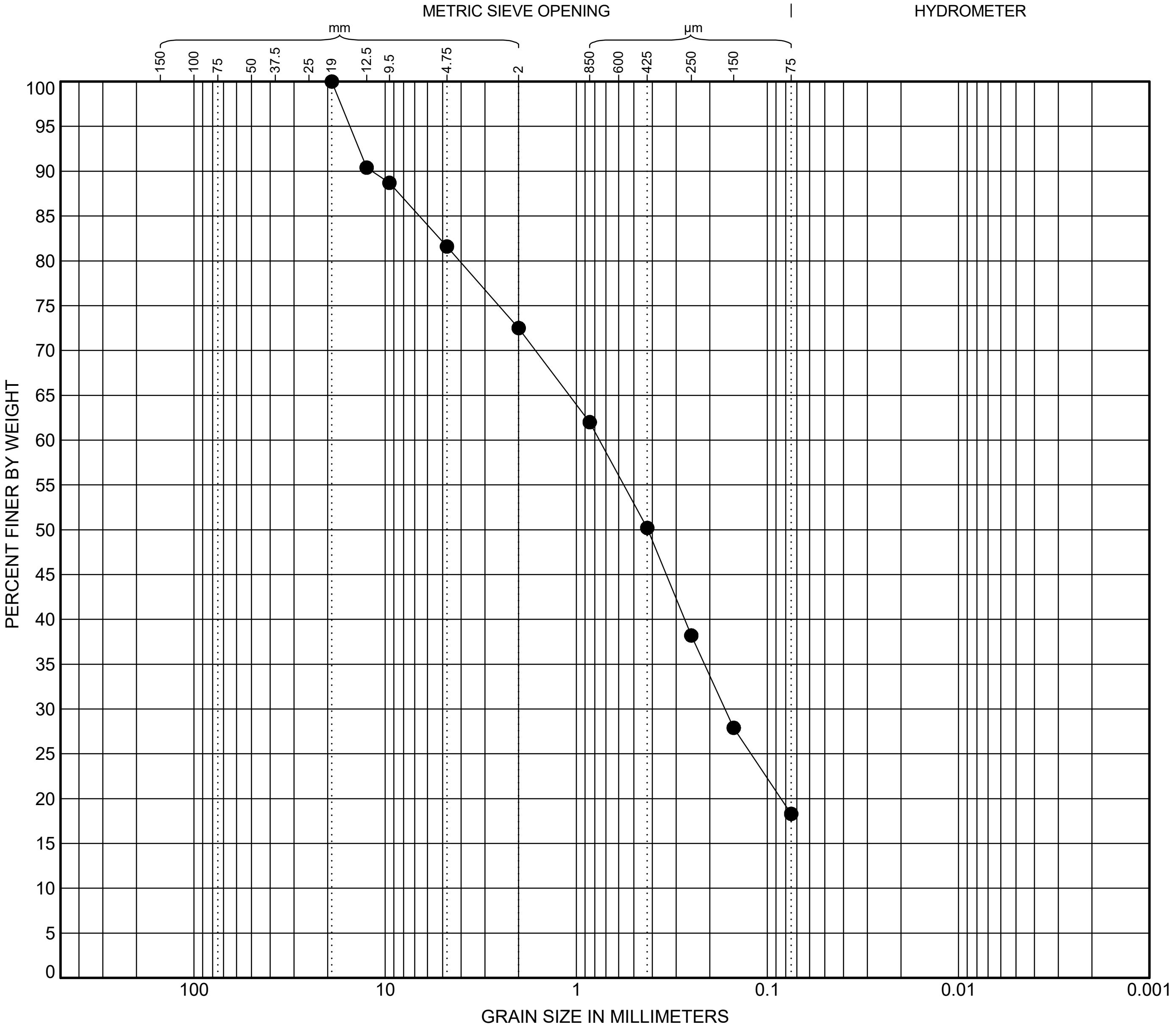
Sample Number: 117-18
Date Tested: 2018-04-25
Tested By: MK

Checked By:

GRAIN SIZE DISTRIBUTION ASTM C136

Project: Stocking Lake Dam Drilling Investigation
Location: Stocking Lake, Ladysmith BC
Sample Location/Source: BH18-02

Project No: GK-18-020-CVD
Client: Cowichan Valley Regional District
Depth: 0.4 m to 0.8 m



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

D_{100}	D_{85}	D_{60}	D_{30}	D_{15}	D_{10}	C_c	C_u	%Gravel	%Sand	%Silt	%Clay
19	**6.62**	**0.756**	**0.166**					**18**	**64**	**18**	

Description: Sand, some Gravel, some Fines
Natural Moisture Content: 12.7%
Material Specification: N/A
Intended Use: N/A
Comments: Sample taken from drill core box, reported moisture content may not be represetative of in-situ condidtions.

Sample Number: 118-18
Date Tested: 2018-04-25
Tested By: MK

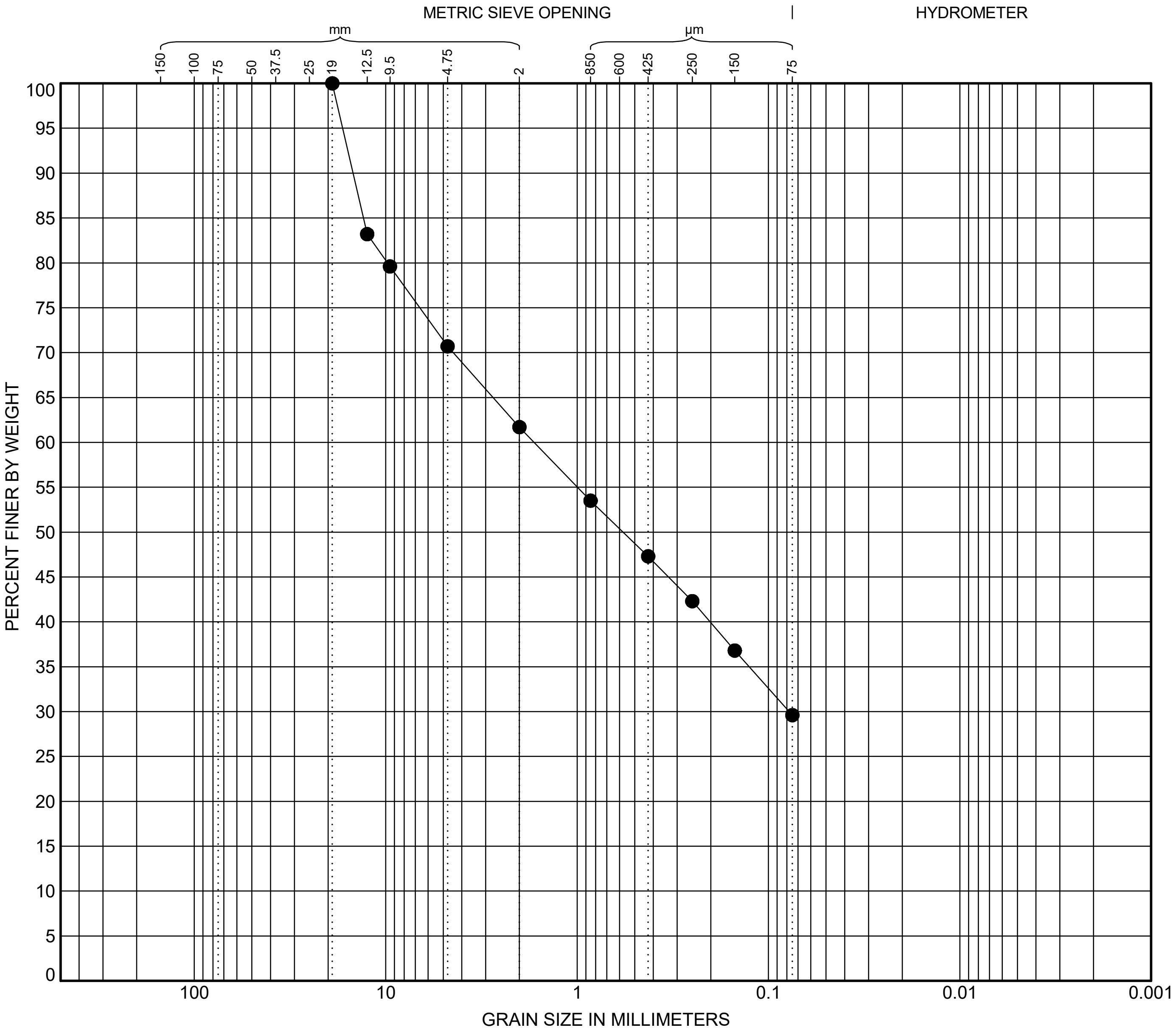
Checked By: ______________________

GRAIN SIZE DISTRIBUTION GK-18-020-CVD.GPJ DATAECORA2015.GDT 18/5/8

GRAIN SIZE DISTRIBUTION ASTM C136

Project: Stocking Lake Dam Drilling Investigation
Location: Stocking Lake, Ladysmith BC
Sample Location/Source: BH18-02

Project No: GK-18-020-CVD
Client: Cowichan Valley Regional District
Depth: 2.74 m to 3.35 m



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

D_{100}	D_{85}	D_{60}	D_{30}	D_{15}	D_{10}	C_c	C_u	%Gravel	%Sand	%Silt	%Clay
19	**13.074**	**1.675**	**0.078**					**29**	**41**	**30**	

Description: Sand, Silty Fines, Gravelly
Natural Moisture Content: 12.5 %
Material Specification: N/A
Intended Use: N/A
Comments: N/A

Sample Number: 119-18
Date Tested: 2018-04-25
Tested By: MK

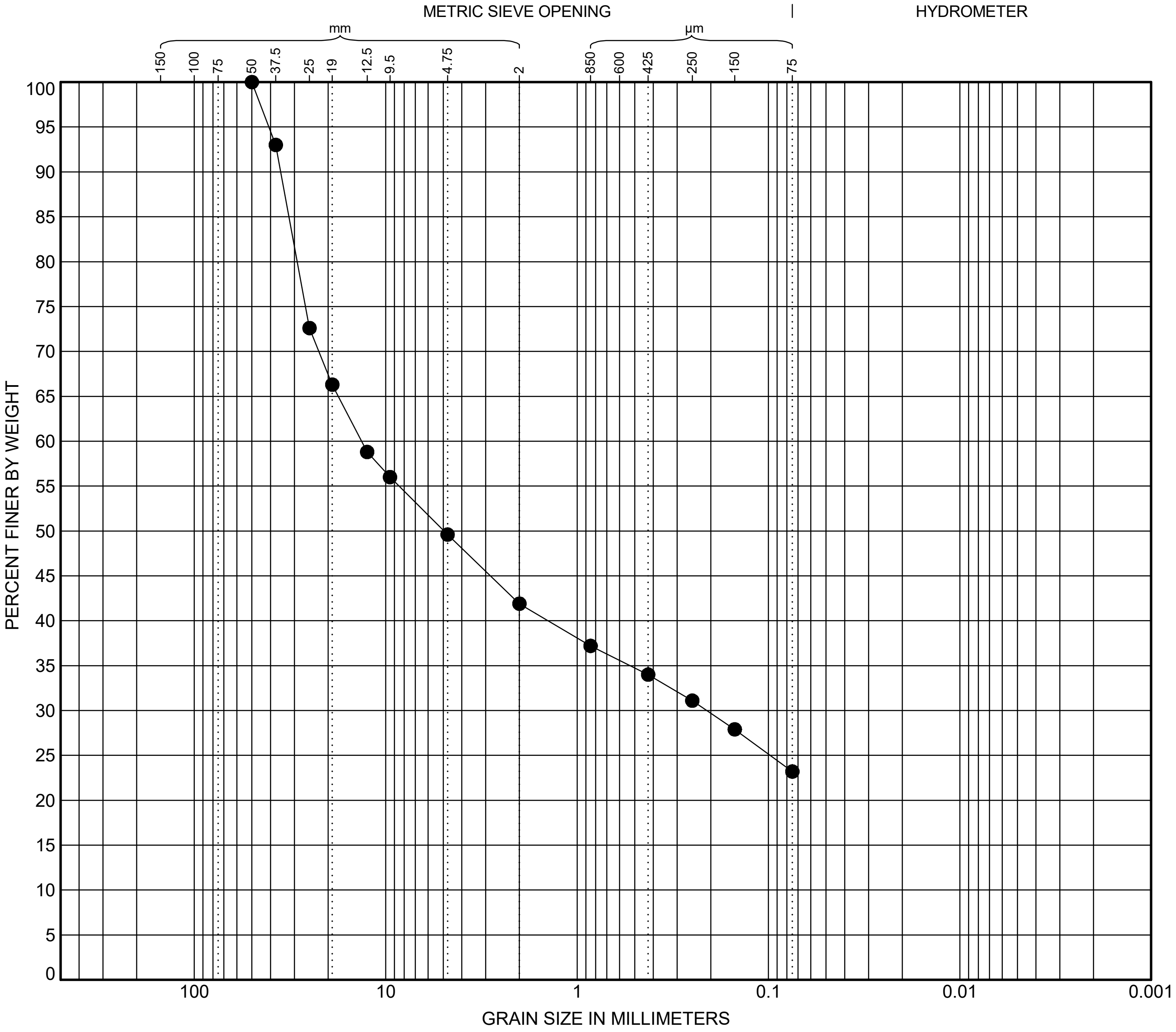
Checked By: ______________

GRAIN SIZE DISTRIBUTION GK-18-020-CVD.GPJ DATAECORA2015.GDT 18/5/8

GRAIN SIZE DISTRIBUTION ASTM C136

Project: Stocking Lake Dam Drilling Investigation
Location: Stocking Lake, Ladysmith BC
Sample Location/Source: BH18-02

Project No: GK-18-020-CVD
Client: Cowichan Valley Regional District
Depth: 3.4 m to 3.7 m



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

D_{100}	D_{85}	D_{60}	D_{30}	D_{15}	D_{10}	C_c	C_u	%Gravel	%Sand	%Silt	%Clay
50	**31.987**	**13.366**	**0.21**					**50**	**26**	**23**	

Description: Gravel, Sandy, Silty Fines
Natural Moisture Content: 1.8%
Material Specification: N/A
Intended Use: N/A
Comments: Sample take from drill core box, reported moisture content may not be represetative of in-situ condidtions.

Sample Number: 120-18
Date Tested: 2018-04-25
Tested By: MK

Checked By:

GRAIN SIZE DISTRIBUTION GK-18-020-CVD.GPJ DATAECORA2015.GDT 18/5/8

PRESENTATION OF SITE INVESTIGATION RESULTS

Stocking Lake Dam, Ladysmith, BC

Prepared for:

Ecora Engineering and Resources Group

ConeTec Job No: 18-02030

Project Start Date: 16-Mar-2018
Project End Date: 16-Mar-2018
Report Date: 21-Mar-2018

Prepared by:

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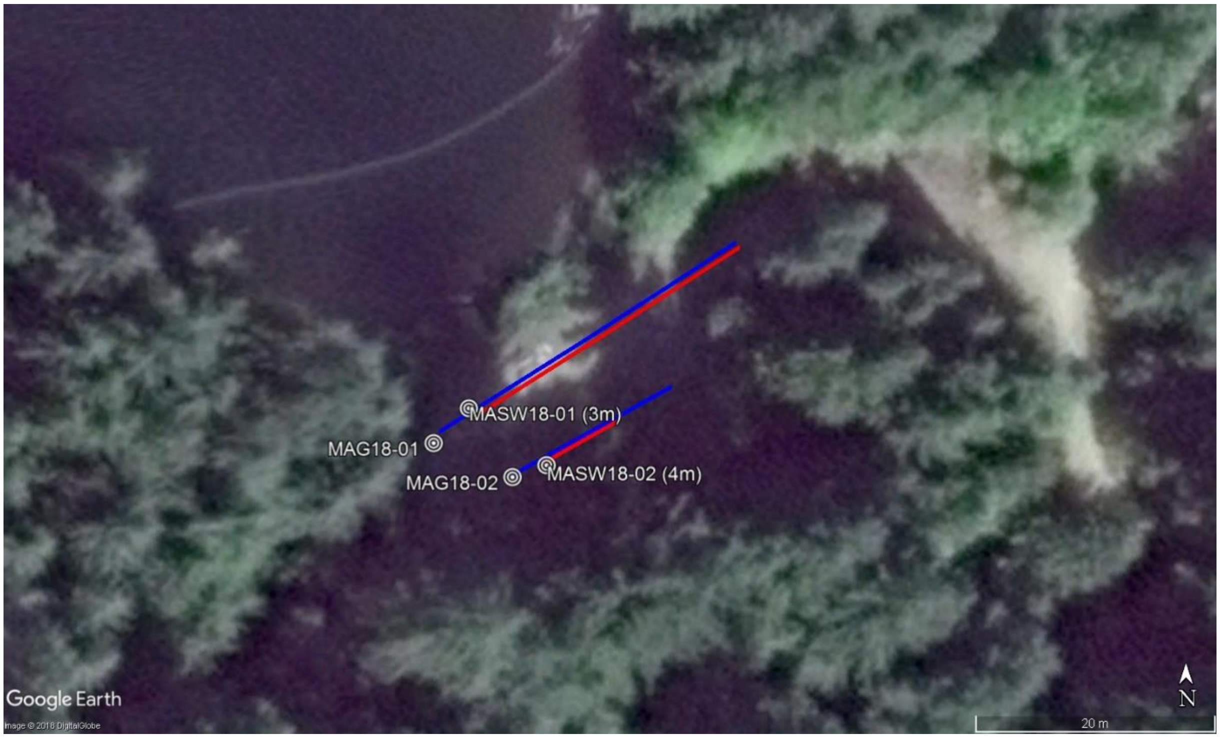
www.conetecdataservices.com

Introduction

The enclosed report presents the results of the geophysical site investigation program conducted by ConeTec Investigations Ltd. for Ecora Engineering and Resources Group at Stocking Lake Dam, Ladysmith, BC. The program consisted of two two-dimensional (2D) Multichannel Analysis of Surface Waves (MASW) tests to provide shear wave velocity profiles and three Magnetometer tests to provide total magnetic field data. Both tests were attempting to discover the location of a buried water pipe.

Project Information

Project	
Client	Ecora Engineering and Resources Group
Project	Stocking Lake Dam, Ladysmith, BC
ConeTec project number	18-02030



Google Earth image indicating the locations of MASW profiles (red) and Magnetometer profiles (blue). The numbers in brackets represent where the first data begin with respect to the start of the cross-section.

Coordinates			
Test Type	Collection Method	EPSG Number	Comments
MASW, Magnetometer	Handheld GPS	32610	Stations that were not surveyed in the field were interpolated from nearby coordinates.

MASW Acquisition Procedures

The MASW data was acquired along two lines on the crest and toe of Stocking Lake Dam. A 48 geophone static array with station spacing of 0.5 meters was used with a roll along method to survey the full line lengths. The source position was moved through the static array with a source spacing of 1 meter. The roll along method was organized such that at least 24 channels were maintained behind the source location with a 1 meter offset from the nearest geophone. The start, midpoint and end coordinates of the array were measured with a consumer grade GPS and checked against landmarks using Google Earth. Line coordinates were adjusted in Google Earth to fit the field measurements. Equipment used is detailed in the table below.

Equipment Used for MASW Testing on this Project				
Seismograph(s)	Geophones	Coupling Mechanism	Trigger Style	Seismic Sources
2 x Geometrics Geode 24	48 x Geospace 4.5 Hz vertical	Steel Pucks or spikes	Geometrics piezoelectric	1 lb ballpeen hammer and steel puck

Magnetometer Acquisition Procedures

The magnetometer data was collected along the same two lines as MASW. Readings were taken every 0.5m at the same positions as the geophones. Along the crest of the dam, magnetic field data was measured at two different vertical sensor locations (2.27m and 0.9m above ground) at every survey position. Coordinates were fitted to the data used the same method as discussed in the MASW section. Equipment used is detailed in the table below.

Equipment Used for Magnetometer Testing on this Project			
Magnetometer	Type	Mount	Datum
GEM GSM-19	Overhauser Magnetometer	Aluminium Staff	57000 nT

Data Analysis and Quality

The MASW data quality was good for this project. The data quality was excellent in areas with at least 3m of soil, but the Northeast end with shallow bedrock produce scattered seismic signals. MASW arrays were digitally cut in areas with shallow bedrock to reduce the detrimental signal scattering effects and improve the data quality. Distance is relative to the measurements taken in the field, so the data can be quickly located using field markings when the client goes back to drill. In general, overtone images show coherent surface wave energy between 20 Hz to 120 Hz. Example time domain traces and overtone images are included in the appendices of this report.

The magnetometer data were excellent for this project and the readings were repeatable to less than 1 nT. All magnetic objects were removed to at least 25m away from the working area to reduce the background noise. Two magnetic anomalies were seen in the profile on the crest of the dam. Magnetic Anomaly 1 was detectable when the sensor was 0.9m from the ground, but not when the sensor was 2.37m from the ground. This anomaly is located at 7m on the profile, is weakly magnetic and near the ground surface. Magnetic Anomaly 2 was detectable by both sensors but was higher amplitude for the sensor nearer to the ground. This Anomaly was located at 28m along the profile and is estimated to be 1.5m below the surface. No magnetic anomalies were discovered along the toe of the dam.

Results

Inverted shear wave velocity (Vs) test results are included in the appendices of this report. The depth of investigation was up to 5 m below the surface. Vs values ranged from approximately 100 m/s increasing with depth to approximately 400 m/s in the soil and 1000m/s in the bedrock. MASW pdf profiles and csv data files are included in the release of this report.

Magnetometer total magnetic field test results are included in the appendices of this report. Two magnetic anomalies were identified along the crest of the dam at 7m and 28m. Magnetic pdf profiles and csv data files are included in the appendices of this report.

Limitations

This report has been prepared for the exclusive use of Ecora Engineering and Resource Group (Client) for the project titled “Stocking Lake Dam, Ladysmith, BC”. The report’s contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety. For further information please refer to the following inserts which describe the test specific details and limitations.

As with all surface based geophysical methods, there is a loss of resolution as well as an increase in uncertainty in the results with increasing depth. Surface geophysical methods provide an educated estimate of the subsurface parameters, however they are no substitute for in-situ measurements. In-situ measurements are always recommended in order to verify the surface geophysical result.

Multichannel analysis of surface waves (MASW) is a non-intrusive in-situ test that uses the principles of elasticity and surface wave dispersion to determine the variation of shear wave velocity with depth at a site. The observation that surface waves (Rayleigh waves) of different wavelengths propagate at different phase velocities in non-ideal media, is called dispersion. This is a direct result of the fact that surface waves of different wavelengths propagate along the surface to varying depths, and hence, if material stiffness changes with depth (as is the case with most non-ideal materials), then an appropriately selected wavelength band will reflect such changes in the velocity of propagation.

The field methods for surface wave testing are very similar to other surface seismic data collection methods. For active source surface wave testing surface geophones are placed in a linear array along a survey line at a known separation (typically 1 m). A series of recordings (shots) are collected with a known in-line source offset from the array. Each shot gather is represented in the time-offset domain and shows the amplitude of wave propagation through the array (see figure MASW-1). For detailed frequency analysis multiple records with different shot offset distances are collected to help better define the broad spectrum frequency-phase velocity response of the medium. Two-dimensional cross sections can be collected by moving the geophone array a small distance (typically 2 m) along the line and repeating the shots at set offsets.

Surface wave data can also be collected using ambient background noise as the seismic source. This type of data collection is referred to as passive MASW as the timing, strength and location of the seismic waves is not controlled. Passive data is collected in a similar manner to active source, however the geophone array does not need to be linear. Circular, 2D or randomly placed geophone arrays can also be used to capture passive data events. The array type will be dependent on the nature of the passive seismic waves and the anticipated arrival directions (if known). Passive data collection also utilizes significantly longer record times to capture as many seismic events as possible. Analysis of the passive data is nearly identical to active source data sets.

Given that surface wave velocity is closely related to the shear wave velocity and the wavelength related to depth, the surface wave results can be used to develop a profile of shear wave velocity versus depth through a process referred to as inversion. The program used to perform the inversion is SurfSeis 4.0, developed by the Kansas Geological Survey. In SurfSeis, the raw time domain traces are transformed to the frequency domain to create what is referred to as an overtone image as shown in figure MASW-2. The overtone image displays the amplitude of the primary surface wave mode and any potential higher modes. A dispersion curve is fitted to the overtone image, and the inversion process is then used to determine the most appropriate shear wave velocity profile. For each test location, the final product is a 1D shear wave velocity profile comprising of a number of velocity layers of variable thickness (see figure MASW-3). For 2D testing a series of 1D tests are combined to produce a shear wave velocity cross section.

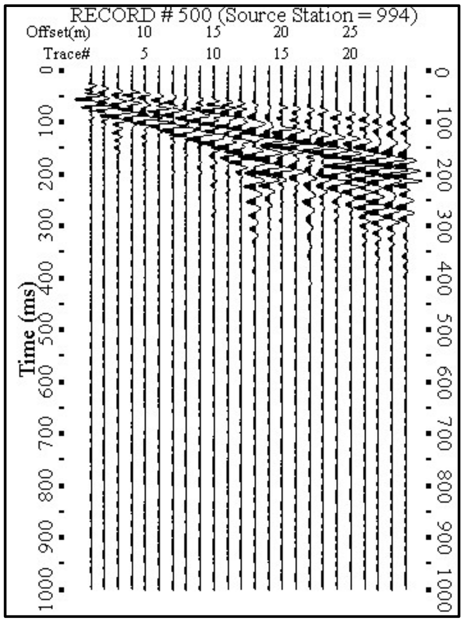


Figure MASW-1. Typical MASW time domain record (shot gather)

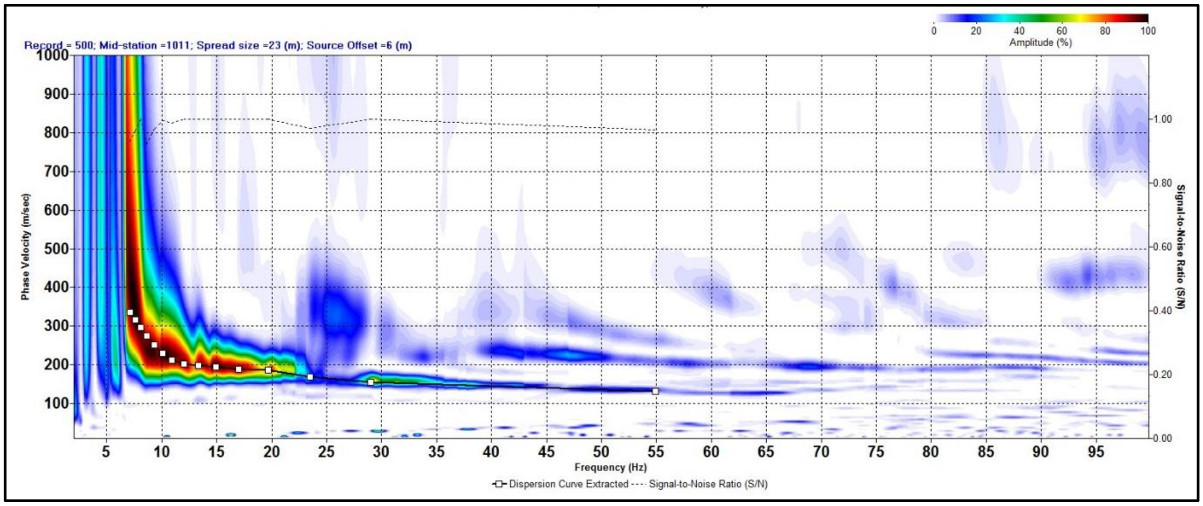


Figure MASW-2. Overtone image and a picked dispersion curve

The depth of investigation is related to the ground conditions and the amount of energy delivered by the surface wave source. The surface wave method uses Rayleigh waves that travel horizontally along the ground surface to a depth of about one wavelength. The actual depth of sampling of the ground is considered to be one-half to one-third of the Rayleigh (surface) wave wavelength. The wavelengths measured by the equipment will be a function of the frequency of the source and the velocity of the surface waves through the ground. As the depth of investigation increases, there will be less certainty in terms of layer boundaries and velocity values.

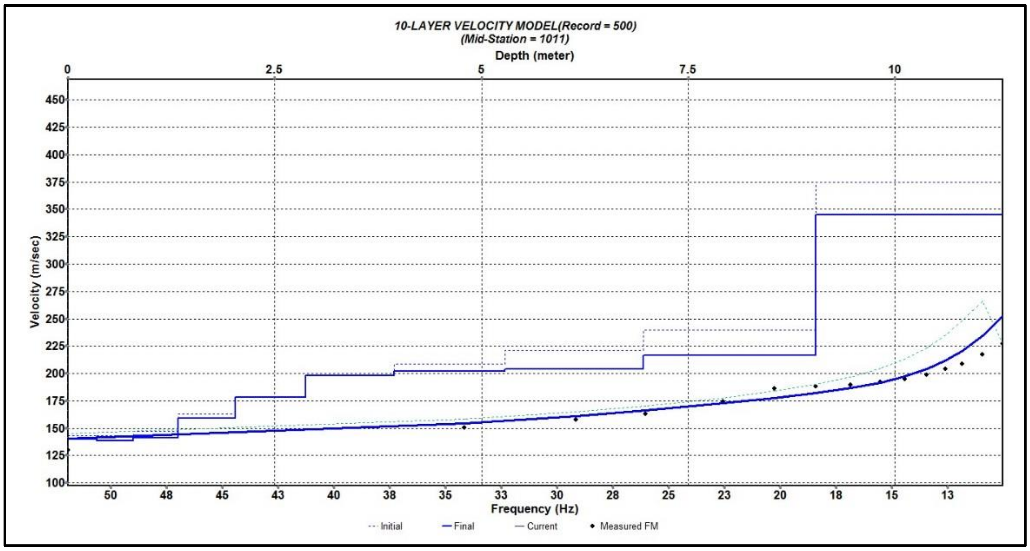


Figure MASW-3. 1D inversion result with fitted dispersion curve

The equipment, field procedures, and analysis software used by ConeTec Investigations Ltd. all conform to the currently accepted best practices for MASW testing. The results of geophysical testing are always interpretative to a certain extent and should be confirmed by drilling or other intrusive testing. While efforts have been made to provide the best possible information ConeTec Investigations Ltd. does not warranty this report to be free from errors or inaccuracies.

References

Miller, R.D., Xia, J., Park, C.B., and Ivanov, J.M., 1999, Multichannel analysis of surface waves to map bedrock, Kansas Geological Survey, The Leading Edge, December, p. 1392-1396.

Park, C.B., Miller, R.D., and Xia, J., 1998b, Ground roll as a tool to image near-surface anomaly: 68th Ann. Internat. Mtg. Soc. Expl.Geophys., Expanded Abstracts, p. 874-877.

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Park, C.B., Miller, R.D., Xia, J., and Ivanov, J., 2007, Multichannel analysis of surface waves (MASW)-active and passive methods: The Leading Edge, January.

Xia, J., R.D. Miller, and C.B. Park, 2000a, Advantages of calculating shear-wave velocity from surface waves with higher modes: [Exp. Abs.]: Soc. Expl. Geophys., p. 1295-1298.

Xia, J., Miller, R.D., Park, C.B., and Ivanov, J., 2000b, Construction of 2-D vertical shear-wave velocity field by the Multichannel Analysis of Surface Wave technique, Proceedings of the Symposium on the Application of Geophysics to Engineering and Environmental Problems (SAGEEP 2000), Washington D.C, February 20-24, p. 1197-1206.

SurfSeis website: http://www.kgs.ku.edu/software/surfseis/index.html

Magnetometry measures variance in the local magnetic field caused by contrasts in magnetic susceptibility. Magnetic susceptibility is the ability of a material to adopt an induced magnetic field caused by it’s immersion in the Earth’s magnetic field. Metallic objects have high magnetic susceptibility and therefore have anomalous induced magnetic fields in contrast to their surroundings.

Data collected in a magnetometry survey can be used to discover magnetic anomalies. Analysis of the shape and size of the anomalies can help determine the shape, size and location of the source. Although the determination of these characteristics can be complex because:

- Magnetic fields are vectors with magnitude and direction. One implication is that a small metallic object near the sensor can have the same magnitude as a larger or more magnetically susceptible object farther away. Also, the magnetic field can be influenced by objects in all directions so objects to the side of the magnetometer can have similar characteristics as ones below.
- The magnitude and direction of Earth’s magnetic field changes depending on the location of the survey.
- Earth’s magnetic field changes slowly throughout the day due to a variety of disturbances.

The Earth’s magnetic field is well understood so many of these complications can be mitigated. Distortions due to latitude and longitude can be corrected and diurnal variation can be accounted for with a base station magnetometer. Also, magnetometry instrumentation is capable of detecting changes of 1nT making it a very sensitive data collection technique.

The following appendices listed below are included in the report:

- MASW and Magnetometer Summaries and Results
- MASW Time Domain Traces and Overtone Images

MASW and Magnetometer Summaries and Results

CONETEC

Job No: 18-02030
Client: Ecora Engineering and Resources Group
Project: Stocking Lake Dam, Ladysmith, BC
Start Date: 16-Mar-2018
End Date: 16-Mar-2018

2D MASW TEST SUMMARY

Section ID	Date	Source Type	Geophone Spacing (m)	Shot Spacing (m)	Section Length (m)	Array Length (m)	Start of Section Northing[1] (m)	Start of Section Easting (m)	End of Section Northing (m)	End of Section Easting (m)	Refer to Notation Number
MASW18-01	16-Mar-2018	Ballpeen Hammer	0.5	1.0	29	35.5	5422970	440044	5422985	440069	1
MASW18-02	16-Mar-2018	Ballpeen Hammer	0.5	1.0	8	16	5422966	440050	5422970	440057	1

1. Coordinates were collected with a consumer grade GPS device in datum WGS84/UTM Zone 10 North.

CONETEC

Job No: 18-02030
Client: Ecora Engineering and Resources Group
Project: Stocking Lake Dam, Ladysmith, BC
Start Date: 16-Mar-2018
End Date: 16-Mar-2018

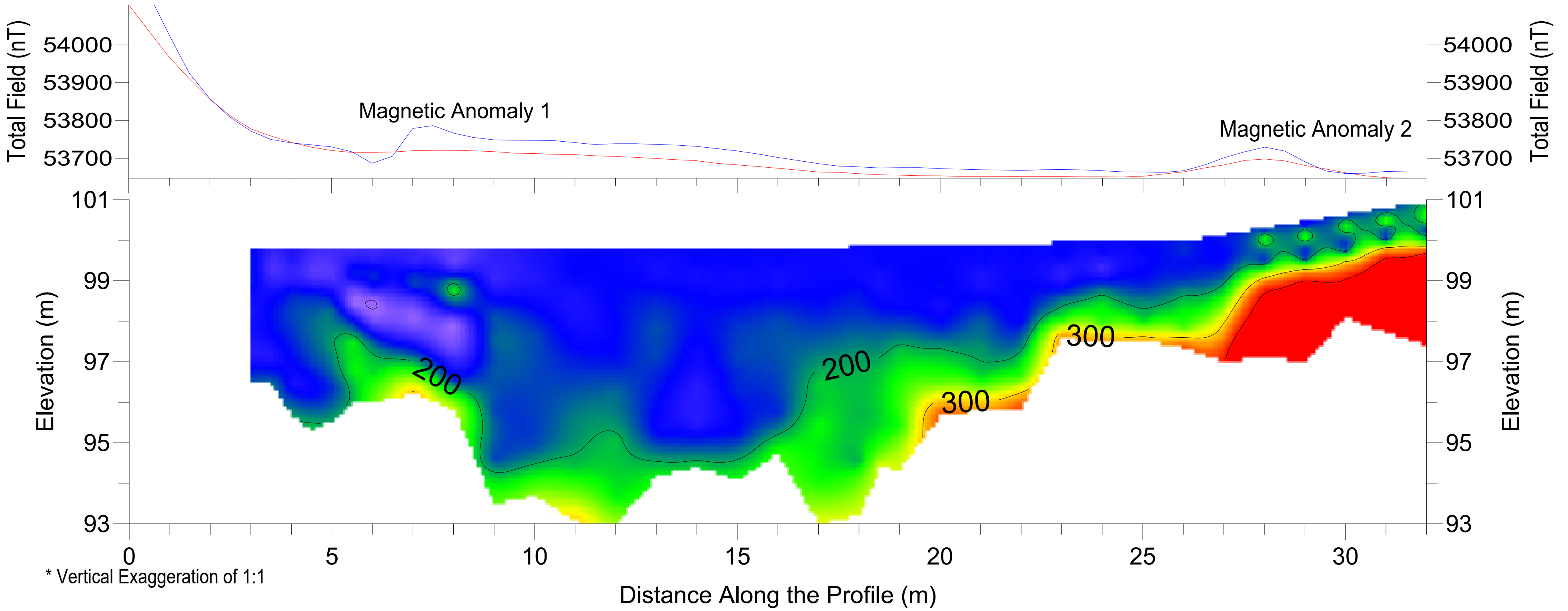
Magnetometer TEST SUMMARY

Section ID	Date	Instrument	Reading Spacing (m)	Section Length (m)	Start of Section Northing[1] (m)	Start of Section Easting (m)	End of Section Northing (m)	End of Section Easting (m)	Refer to Notation Number
MAG18-01 (2.27m)	16-Mar-2018	GEM GSM-19	0.5	32	5422969	440041	5422985	440069	1,2
MAG18-01 (0.9m)	16-Mar-2018	GEM GSM-19	0.5	32	5422969	440041	5422985	440069	1,2
MAG18-02 (2.27m)	16-Mar-2018	GEM GSM-19	0.5	16	5422964	440047	5422972	440061	1

1. Coordinates were collected with a consumer grade GPS device in datum WGS84/UTM Zone 10 North.
2. Readings were taken in the same x positions, but the sensors were located in different vertical positions.

Coordinates at 3 m distance:
Easting (m): 440044
Northing (m): 5422970

Coordinates at 32 m distance:
Easting (m) 440069
Northing (m): 5422985



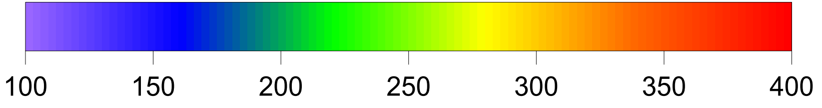
* Vertical Exaggeration of 1:1

Stocking Lake Dam, Ladysmith, BC

FIGURE 1
MASW18-01: 3 to 32 m

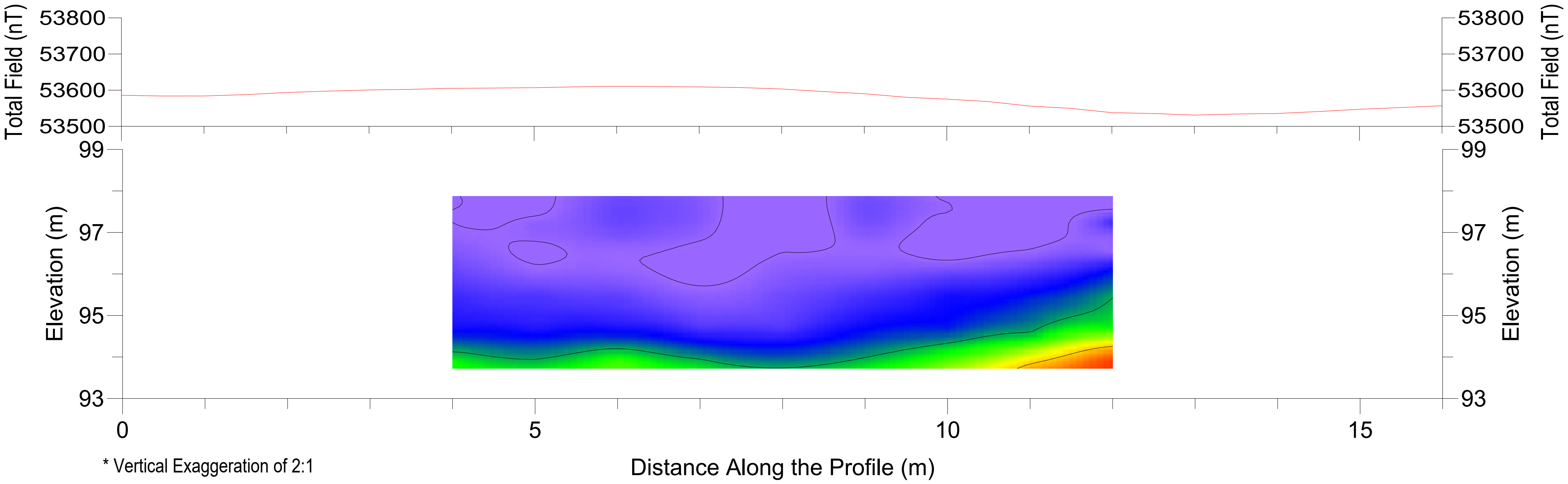
INFORMATION TABLE	
HORIZONTAL DATUM	WGS 84
PROJECTION	UTM Zone 10 North
VERTICAL REFERENCE	Ground Surface
DATE(S) ISSUED / REVISED	20-Mar-2018
SURVEY DATE(S)	16-Mar-2018
CONETEC JOB NUMBER	18-02030

Shear Wave Velocity (m/s)



Coordinates at 4 m distance:
Easting (m): 440050
Northing (m): 5422966

Coordinates at 12 m distance:
Easting (m) 440057
Northing (m): 5422970

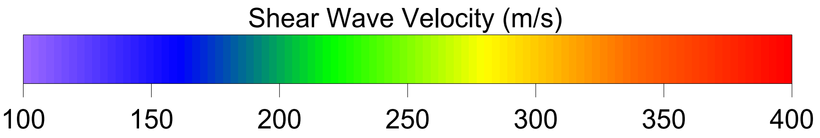


* Vertical Exaggeration of 2:1

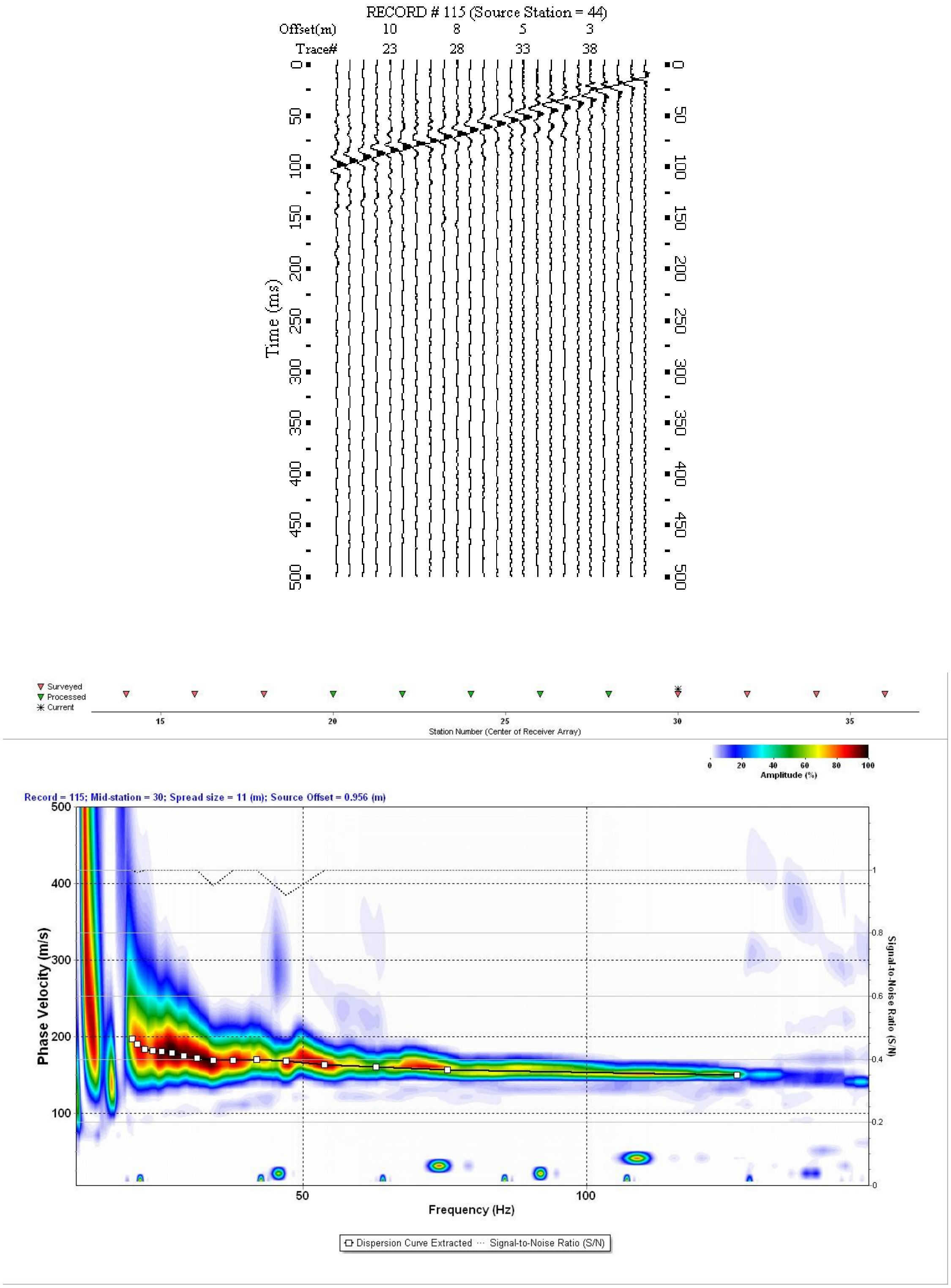
Stocking Lake Dam, Ladysmith, BC

FIGURE 2
MASW18-02: 4 to 12 m

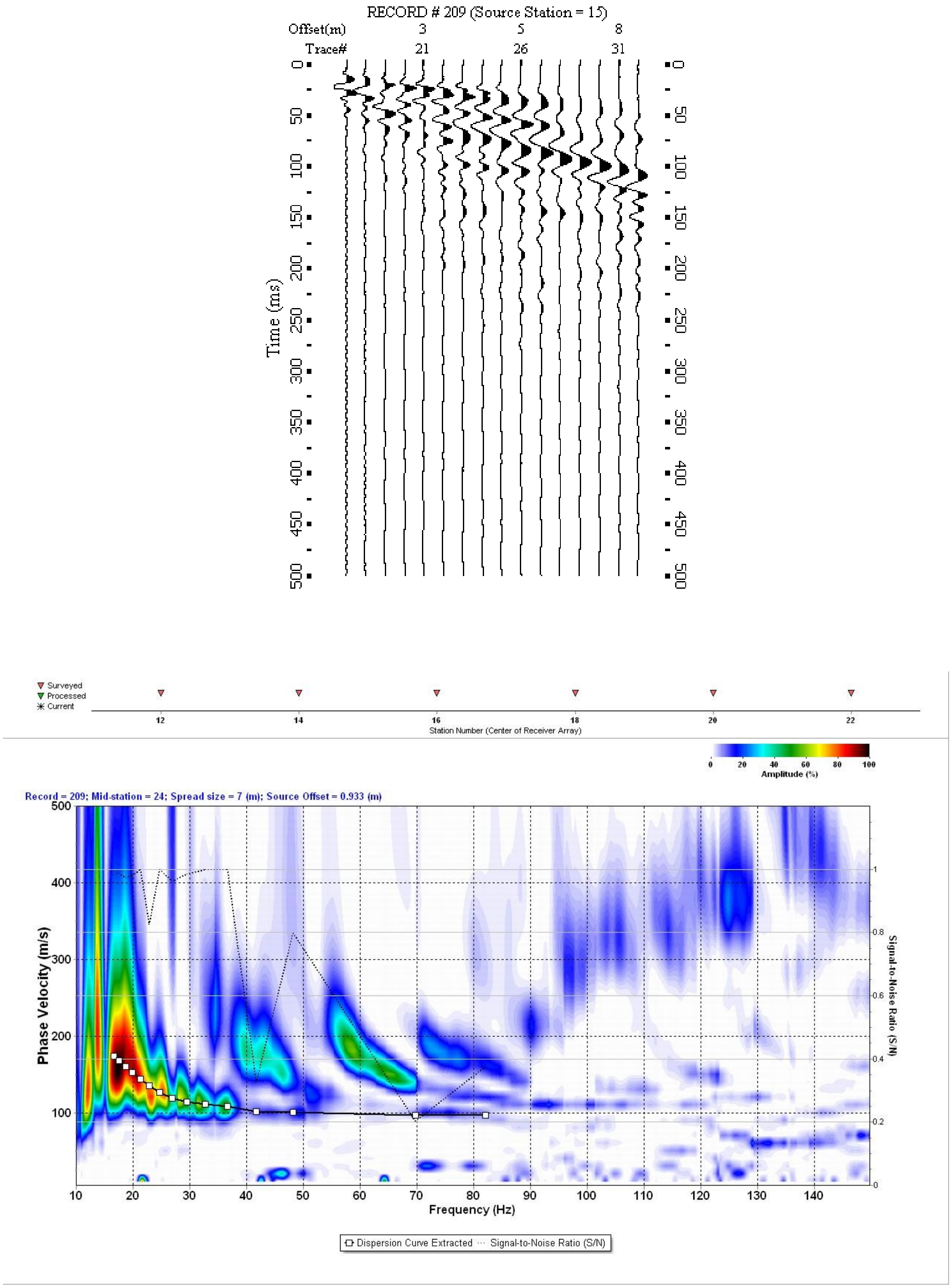
INFORMATION TABLE	
HORIZONTAL DATUM	WGS 84
PROJECTION	UTM Zone 10 North
VERTICAL REFERENCE	Ground Surface
DATE(S) ISSUED / REVISED	20-Mar-2018
SURVEY DATE(S)	16-Mar-2018
CONETEC JOB NUMBER	18-02030



MASW Time Domain Traces and Overtone Images



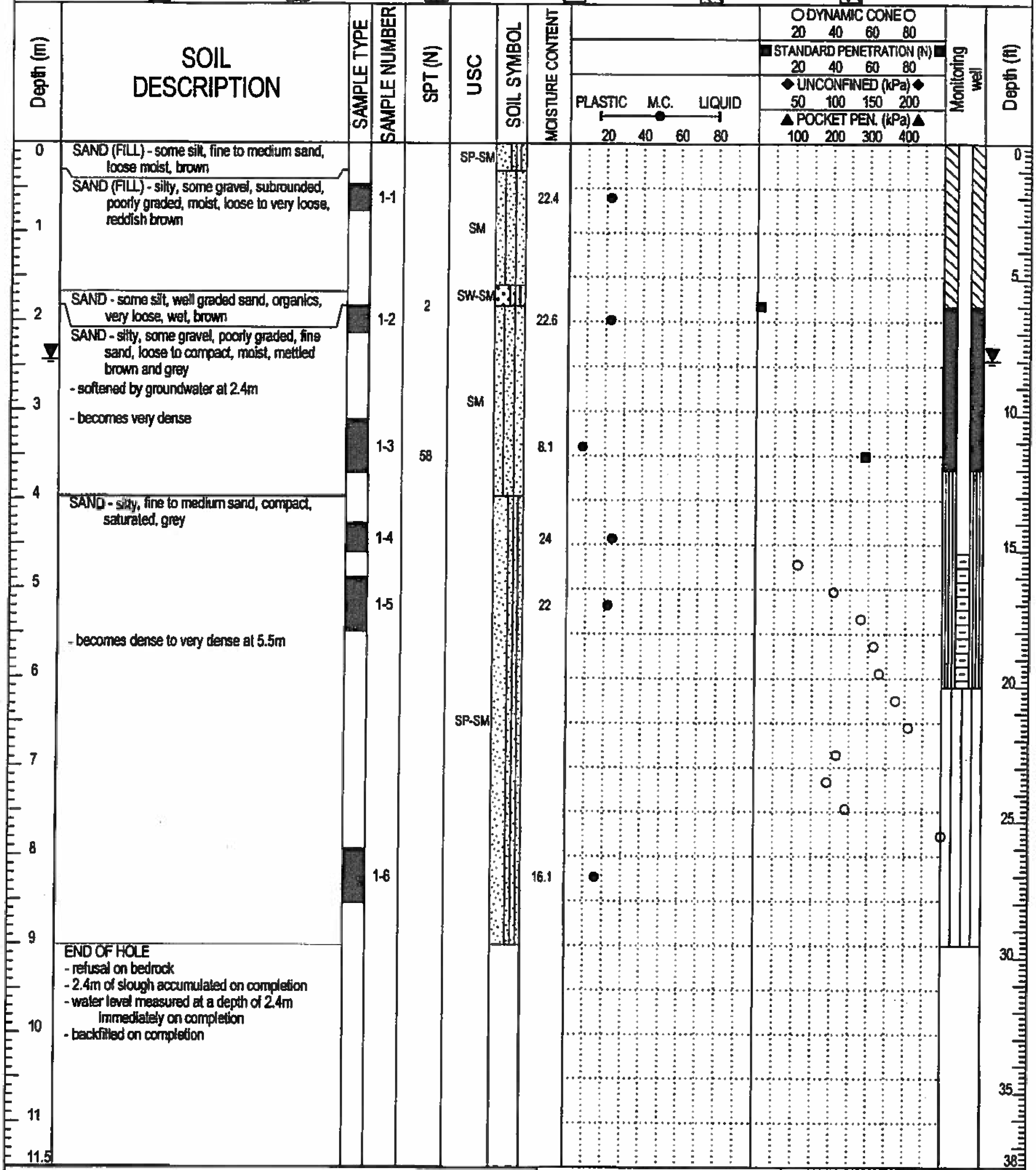
MASW18-01: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).



MASW18-02: Example time domain trace for active source (top) with resulting overtone image with picked dispersion curve (bottom).

Project: Dam Investigation	Client: Town of Ladysmith	BOREHOLE NO: BH 01
Location: Stocking Lake Dam	Drilling Contractor: Drillwell Enterprises Ltd.	PROJECT NO: 2800172.001
	Method: Auger	

SAMPLE TYPE	■ DISTURBED	◿ NO RECOVERY	☒ SPT	▤ A-CASING	▥ SHELBY TUBE	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▧ DRILL CUTTINGS	⁘ SAND



SOIL DESCRIPTION	SAMPLE NUMBER	SPT (N)	USC	MOISTURE CONTENT
SAND (FILL) - some silt, fine to medium sand, loose moist, brown			SP-SM	
SAND (FILL) - silty, some gravel, subrounded, poorly graded, moist, loose to very loose, reddish brown	1-1		SM	22.4
SAND - some silt, well graded sand, organics, very loose, wet, brown	1-2	2	SW-SM	22.6
SAND - silty, some gravel, poorly graded, fine sand, loose to compact, moist, mottled brown and grey - softened by groundwater at 2.4m - becomes very dense	1-3	58	SM	8.1
SAND - silty, fine to medium sand, compact, saturated, grey - becomes dense to very dense at 5.5m	1-4 1-5 1-6		SP-SM	24 22 16.1

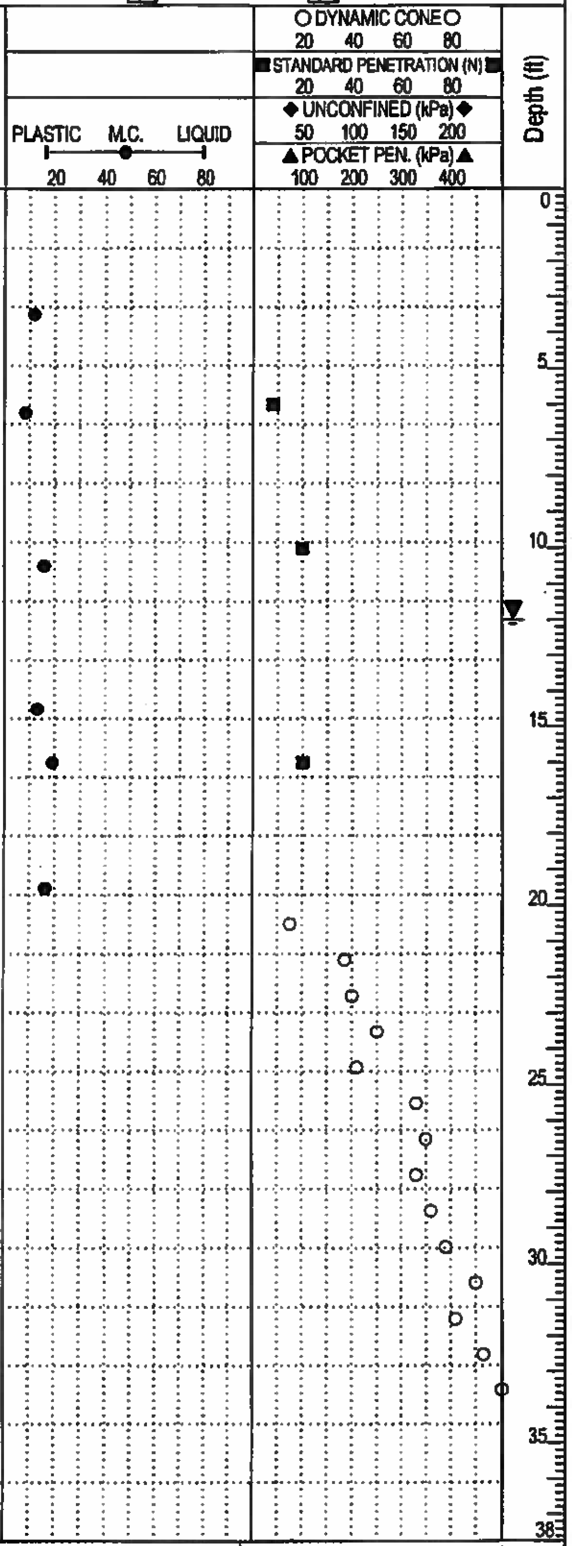
END OF HOLE
- refusal on bedrock
- 2.4m of slough accumulated on completion
- water level measured at a depth of 2.4m immediately on completion
- backfilled on completion

EBA Engineering Consultants Ltd.	LOGGED BY: PM/RG	COMPLETION DEPTH: 9m
	REVIEWED BY: RAP	COMPLETE: 8/23/2006
	DRAWING NO: 1	Page 1 of 1

Project: Dam Investigation	Client: Town of Ladysmith	BOREHOLE NO: BH 02
Location: Stocking Lake Dam	Drilling Contractor: Drillwell Enterprises Ltd.	PROJECT NO: 2800172.001
	Method: Auger	

SAMPLE TYPE	■ DISTURBED	◪ NO RECOVERY	☒ SPT	☰ A-CASING	▥ SHELBY TUBE	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▧ DRILL CUTTINGS	⁘ SAND

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SPT (N)	USC	MOISTURE CONTENT
0	SAND (FILL) - some gravel, some silt, fine to medium sand, subrounded gravel, poorly graded, loose, damp, brown					
1		■	2-1		SWG	11.8
	- minor wood debris (1.4m)					
2	SAND and GRAVEL (FILL) - , trace silt, subrounded gravel, maximum particle size 50mm, well graded, loose, damp, reddish brown, trace organics and ash	■	2-2	8		8.4
3		■	2-3	20		15.9
	- SPT refusal on rock at 3.3m				GWS	
▼	- increased coarse material making drilling difficult at 3.7m					
4						
	GRAVEL and SAND - some silt, well graded, subrounded, wet, compact, brownish grey	■	2-4			13.1
5		■	2-5	20		19.1
	SAND - some silt, trace gravel, poorly graded, coarse sand, wet, brown				SP-SM	
	- becomes fine sand and grey at 5.5m				SP	
6	SAND - medium grained, green	■	2-6		SW	16.4
	DCPT - no samples or soil descriptions possible					
7						
8						
9						
10						
	END OF HOLE -water level measured at 3.7m immediately on completion -6.0m of slough accumulated immediately on completion					
11						
11.5	- backfilled on completion					



EBA Engineering Consultants Ltd.	LOGGED BY: PM/RG	COMPLETION DEPTH: 10.4m
	REVIEWED BY: RAP	COMPLETE: 8/23/2006
	DRAWING NO: 2	Page 1 of 1

Project: Dam Investigation	Client: Town of Ladysmith	BOREHOLE NO: BH 03
Location: Stocking Lake Dam	Drilling Contractor: Drillwell Enterprises Ltd.	PROJECT NO: 2800172.001
	Method: Auger	

SAMPLE TYPE	■ DISTURBED	▨ NO RECOVERY	☒ SPT	☰ A-CASING	▥ SHELBY TUBE	▮ CORE
BACKFILL TYPE	■ BENTONITE	⁘ PEA GRAVEL	▥ SLOUGH	⁘ GROUT	▧ DRILL CUTTINGS	⁘ SAND

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	USC	SOIL SYMBOL	MOISTURE CONTENT	PLASTIC M.C. LIQUID 20 40 60 80	◆ UNCONFINED (kPa) ◆ 50 100 150 200 ▲ POCKET PEN. (kPa) ▲ 100 200 300 400	Depth (ft)
0	SAND (FILL) - some silt, trace gravel, poorly graded, damp, loose, greyish brown , wood debris		SP-SM					0
1.4	SAND - gravelly, some silt, well graded, subrounded, maximum particle size 25mm, compact, moist, light brown		SWG					
2.7	END OF HOLE (2.7m) - refusal on bedrock - no slough accumulated on completion - no groundwater accumulated on completion - backfilled on completion							

EBA Engineering Consultants Ltd.

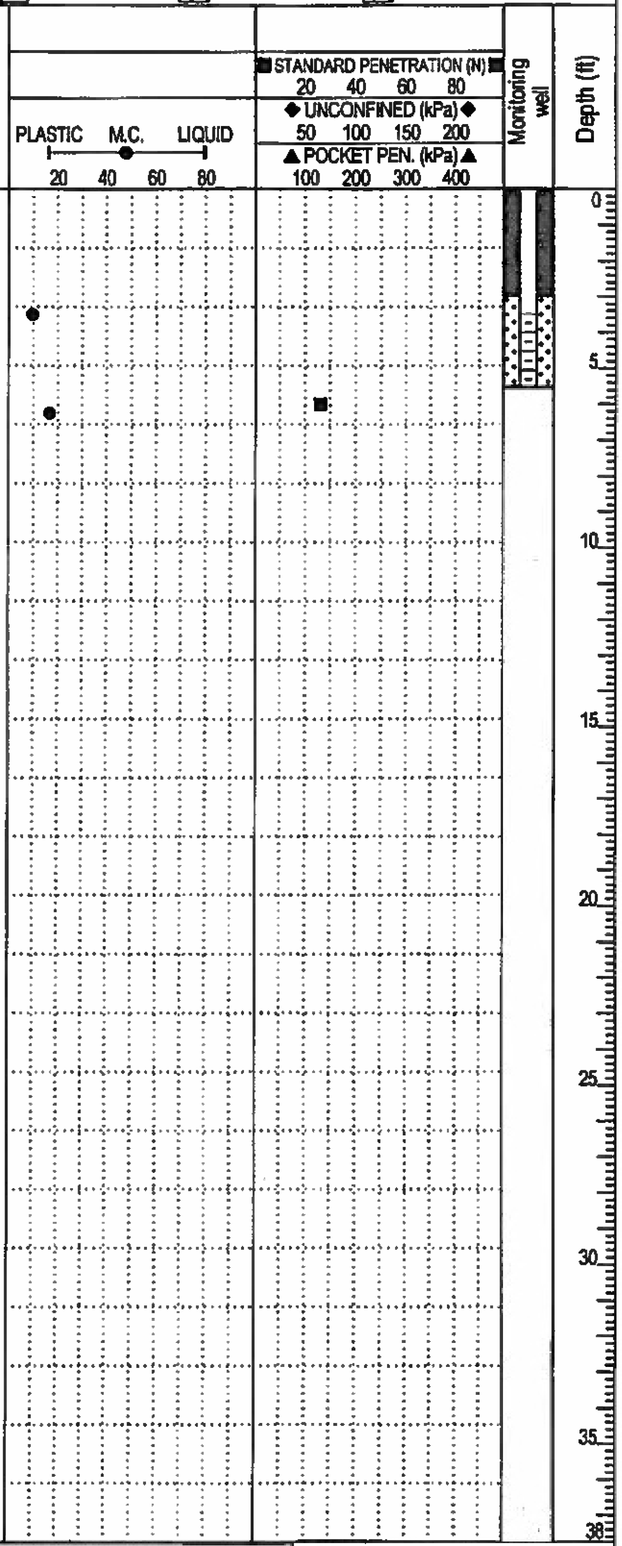
LOGGED BY: PM/RG	COMPLETION DEPTH: 2.7m
REVIEWED BY: RAP	COMPLETE: 8/23/2006
DRAWING NO: 3	Page 1 of 1

GEOTECHNICAL 2800170.001.GPJ EBA.GDT 08/09/21

Project: Dam Investigation	Client: Town of Ladysmith	BOREHOLE NO: BH 04
Location: Stocking Lake Dam	Drilling Contractor: Drillwell Enterprises Ltd.	PROJECT NO: 2800172.001
	Method: Auger	

SAMPLE TYPE	▩ DISTURBED	▨ NO RECOVERY	☒ SPT	▤ A-CASING	▥ SHELBY TUBE	▮ CORE
BACKFILL TYPE	▩ BENTONITE	⛶ PEA GRAVEL	▥ SLOUGH	⛶ GROUT	▧ DRILL CUTTINGS	⛶ SAND

Depth (m)	SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SPT (N)	USC	SOIL SYMBOL	MOISTURE CONTENT
0	SAND (FILL) - gravelly, some silt, poorly graded, subrounded, damp, compact, greyish brown, trace cobbles, trace organics						
1		▩	4-1		SPG		9.6
2	SAND - some silt, fine to medium grained, compact, damp to moist, brownish grey	▩	4-2	26	SP-SM		16.7
	END OF HOLE (2.1m) - refusal on bedrock - no slough accumulated on completion - no groundwater accumulated on completion - backfilled on completion						
3							
4							
5							
6							
7							
8							
9							
10							
11							
11.5							



EBA Engineering Consultants Ltd.	LOGGED BY: PM/RG	COMPLETION DEPTH: 2.1m
	REVIEWED BY: RAP	COMPLETE: 8/23/2006
	DRAWING NO: 4	Page 1 of 1

EBA Engineering

GRAIN SIZE DISTRIBUTION

Project: Stocking Lake Dam Investigation

Project Number: 0802-2800172

Date Tested: 24/8/2006

Borehole Number: BH01 Sa3

Depth: 3.05m to 3.65m

Soil Description: Sand and Gravel, some silt

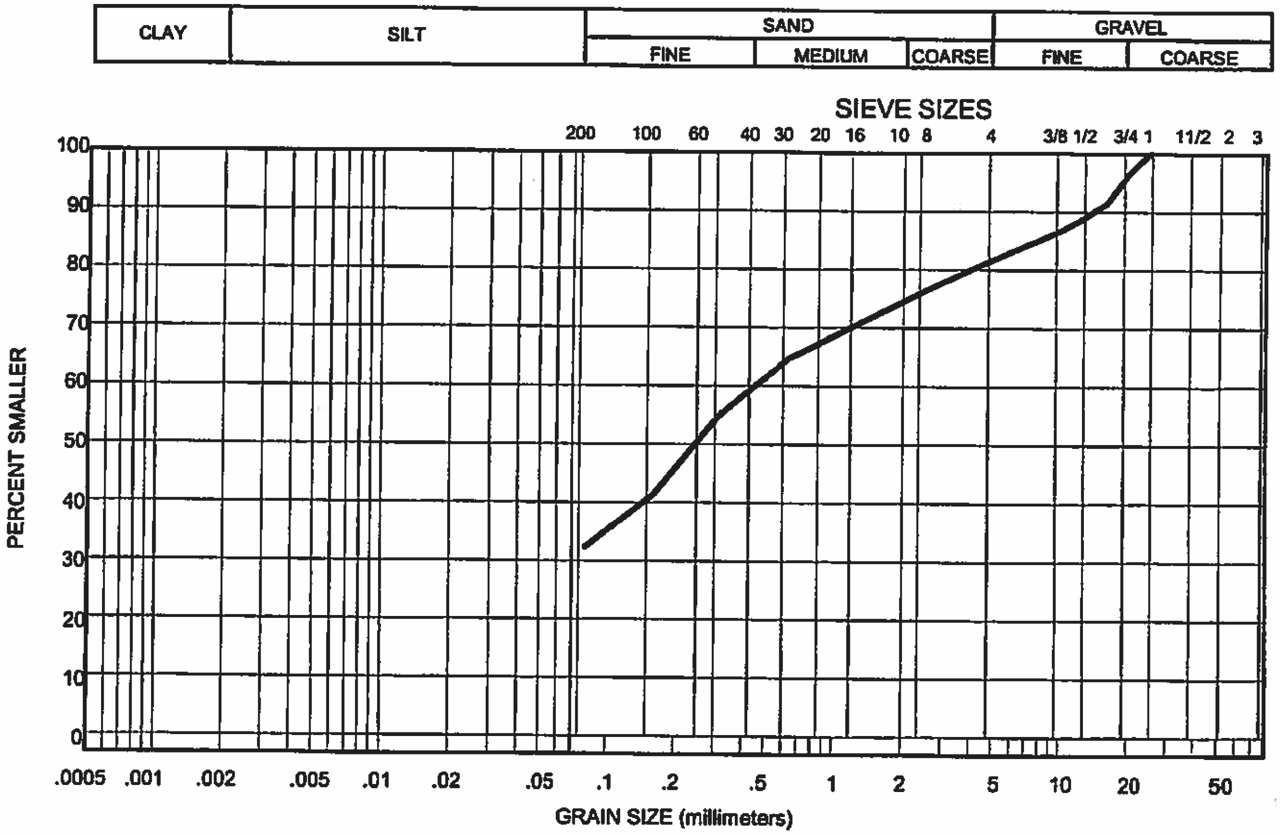
Cu:

Cc:

Natural Moisture Content: 8.1%

Remarks: GRAVEL 18%, SAND 50%, CLAY/SILT 32%

SIEVE	PERCENTAGE PASSING
37.500	#N/A
25.000	100
19.000	97
16.000	91
12.500	89
9.500	87
4.750	82
2.000	76
1.180	71
0.600	65
0.300	55
0.150	41
0.075	32



Reviewed By: P.Eng.

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EBA Engineering

GRAIN SIZE DISTRIBUTION

Project: Stocking Lake Dam Investigation

Project Number: 0802-2800172

Date Tested: 24/8/2006

Borehole Number: BH01 Sa5

Depth: 4.88m to 5.49m

Soil Description: Sand and Gravel, some silt

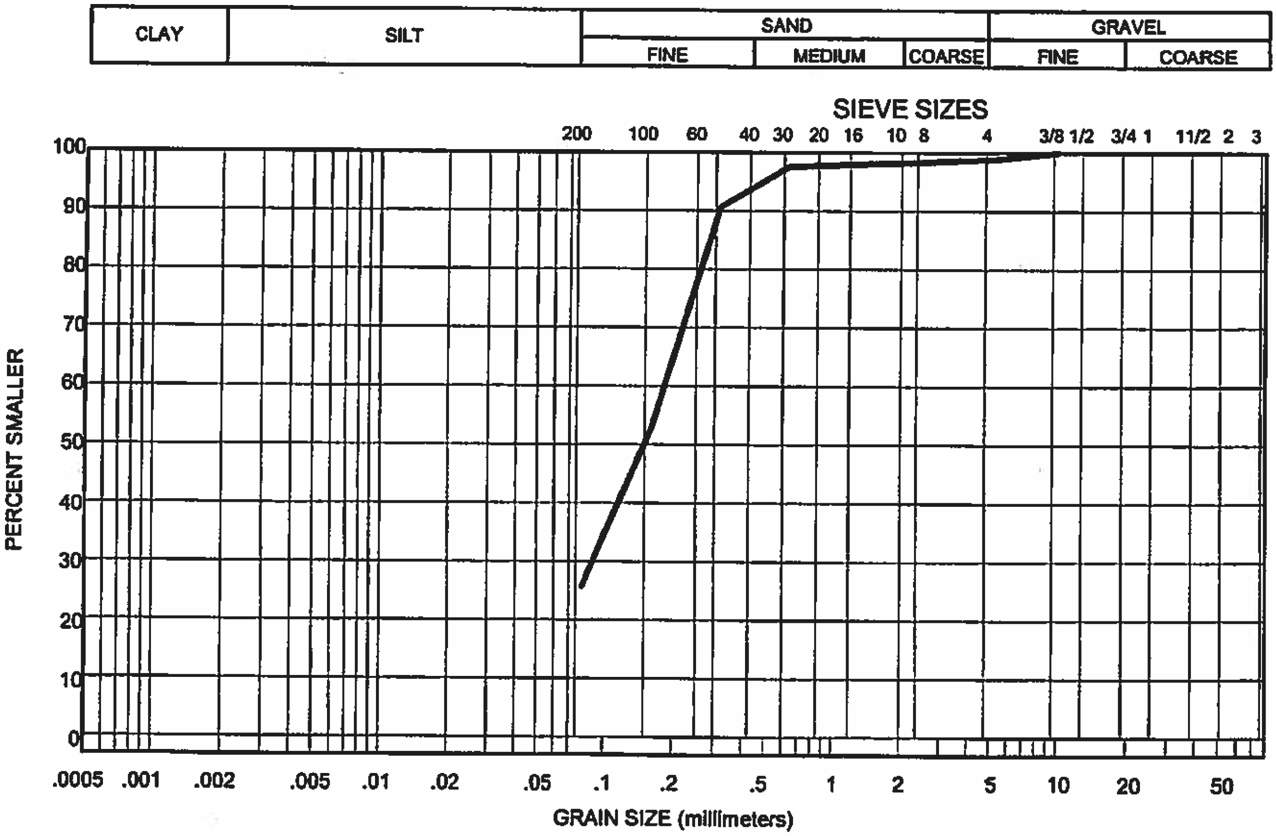
Cu:

Cc:

Natural Moisture Content: 24.0%

Remarks: GRAVEL 1%, SAND 73%, CLAY/SILT 26%

SIEVE	PERCENTAGE PASSING
37.500	#N/A
25.000	#N/A
19.000	#N/A
16.000	#N/A
12.500	#N/A
9.500	100
4.750	99
2.000	98
1.180	98
0.600	98
0.300	91
0.150	53
0.075	26



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EBA Engineering

GRAIN SIZE DISTRIBUTION

Project: Stocking Lake Dam Investigation

Project Number: 0802-2800172

Date Tested: 24/8/2006

Borehole Number: BH01 Sa6

Depth: 7.92m to 8.53m

Soil Description: Sand and Gravel, some silt

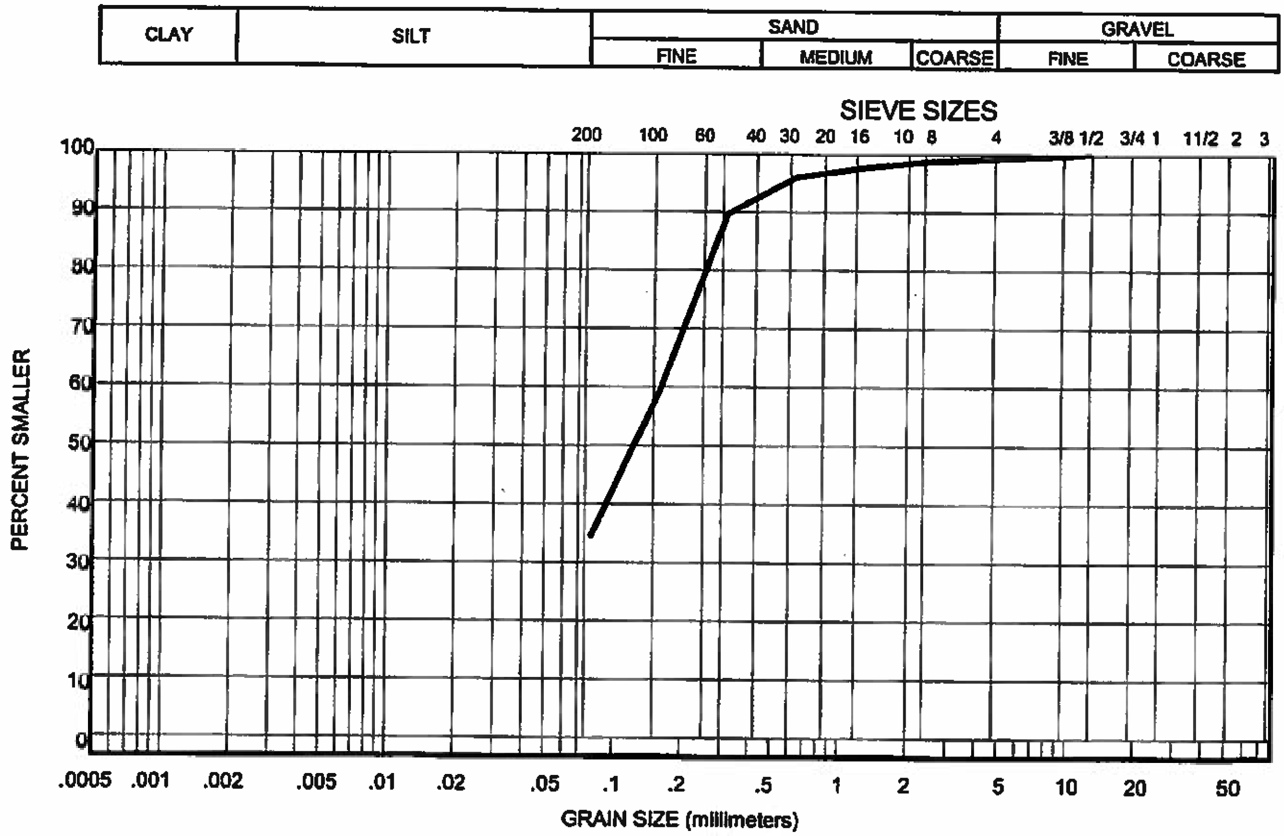
Cu:

Cc:

Natural Moisture Content: 21.9%

Remarks: GRAVEL 1%, SAND 64%, CLAY/SILT 35%

SIEVE	PERCENTAGE PASSING
37.500	#N/A
25.000	#N/A
19.000	#N/A
16.000	#N/A
12.500	100
9.500	100
4.750	99
2.000	99
1.180	98
0.600	96
0.300	90
0.150	60
0.075	35



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Appendix E

Hazard and Failure Modes Analysis

Table F: Hazards and Failure Modes Analysis (HFMM)

Global Failure Modes		Element And/Or Element Function	Most Basic Functional Failure Characteristics	External Hazards				Internal Hazards (Design, Construction, Maintenance, Operation)			
				Meteorological	Seismic	Reservoir Environment	Human and/or Animal Activities	Water barrier	Hydraulic Structure.	Mechanical/Electrical	Infrastructure & Plans
DAM COLLAPSE BY OVERTOPPING (erosion or overturning)	Water elevation too high	Inadequate installed discharge capacity	Meteorological inflow > buffer + outflow capacity	Could a meteorological event cause the inflow to be greater than the outflow capacity and lead to dam overtopping / failure due to insufficient installed discharge capacity?	Could a seismic event cause a meteorological event and cause the dam to be overtopped/fail from a reduced discharge capacity (channels, chutes)?	Could the reservoir environment (landslide? debris?) cause a meteorological event leading to the dam to be overtopped/fail because of insufficient installed discharge capacity?	Could human and/or animal activities cause a meteorological event that leads to the dam being overtopped/fail due to insufficient installed discharge capacity?	Could design or construction of the water barrier cause a meteorological event leading to dam overtopping / failure due to insufficient installed discharge capacity?	Could design or construction of the hydraulic structure cause a meteorological inflow greater than the buffer + outflow capacity and cause the dam to be overtopped/fail?	Could the design or construction of the mechanical/electrical systems cause a meteorological inflow greater than the buffer + outflow capacity and lead to the dam being overtopped/fail due to insufficient installed discharge capacity?	Could inadequate infrastructure and plans cause a meteorological inflow greater than the buffer + outflow capacity and lead to the dam being overtopped/fail due to insufficient installed discharge capacity?
		Inadequate available discharge capacity	Inadequate reservoir operation (rules not followed)	Could the dam be overtopped/fail during a meteorological event if the operating rules are not followed?	Could a seismic event create a condition that prevents the operating rules from being followed, leading to the dam being overtopped/fail?	Could the reservoir environment cause the operating rules to not be followed leading to the dam being overtopped/fail?	Could human and/or animal activities cause the operating rules to not be followed leading to the dam being overtopped/fail?	Could design or construction of the water barrier cause the operating rules to not be followed and cause the dam to be overtopped/fail?	Could the design or construction of the hydraulic structure cause the operating rules to not be followed and lead to dam collapse by overtopping?	Could the design or construction of the mechanical/electrical systems cause the operating rules to not be followed leading to dam overtopping/failure?	Could inadequate infrastructure and plans cause inadequate reservoir operation leading to dam collapse by overtopping?
			Random functional failure on demand	Could the dam be overtopped/fail during a meteorological event if there is a random functional failure of spilling capability?	Could a seismic event cause a random functional failure of spilling capability leading to the dam be overtopped/failed?	Could the reservoir environment cause random functional failure on demand of discharge capability and lead to the dam being overtopped/fail?	Could human and/or animal activities cause random functional failure of spilling capability causing the dam to be overtopped/fail?	Could design or construction of the water barrier cause a random functional failure of spilling capability and cause the dam be overtopped/fail?	Could the design or construction of the hydraulic structure cause random functional failure of spilling capability and lead to the dam being overtopped/fail due to inadequate available discharge capacity?	Could the design or construction of the mechanical/electrical systems cause a random functional failure on demand leading to dam collapse by overtopping?	Could inadequate infrastructure and plans cause random functional failure on demand leading to dam collapse by overtopping?
			Discharge capability not maintained or retained	Could the dam be overtopped/fail during a meteorological event if the discharge capacity is not maintained?	Could a seismic event cause the discharge capacity to be damaged causing the dam to be overtopped/fail?	Could the reservoir environment cause loss of the discharge capability leading to the dam being overtopped/fail?	Could human and/or animal activities cause loss of discharge capability and cause the dam to be overtopped/fail?	Could design or construction of the water barrier cause the discharge capability to be not maintained/retained and cause the dam to be overtopped/fail?	Could the design or construction of the hydraulic structure cause loss of the discharge capability and lead to the dam being overtopped/fail due to inadequate available discharge capacity?	Could the design or construction of the mechanical/electrical systems cause the discharge capability to be not maintained / retained leading to dam collapse by overtopping?	Could inadequate infrastructure and plans cause discharge capacity to not be maintained or retained leading to dam collapse by overtopping?
		Inadequate freeboard	Excessive elevation due to landslide or U/S dam	Could the dam be overtopped/fail during a meteorological event due to a reservoir landslide or upstream dam failure?	Could a seismic event cause the dam to be overtopped/fail by a reservoir landslide or upstream dam failure?	Could the reservoir environment cause excessive elevation of the reservoir leading to the dam being overtopped/fail?	Could human and/or animal activities cause a landslide or upstream dam failure leading to the dam being overtopped/fail?	Could design or construction of the water barrier cause a reservoir landslide or upstream dam failure and cause the dam to be overtopped/fail?	Could the design or construction of the hydraulic structure cause excessive elevation due to a landslide or upstream dam failure leading to the dam being overtopped/fail due to inadequate freeboard?	Could the design or construction of the mechanical/electrical systems cause excessive elevation due to landslide or upstream dam failure leading to dam collapse by overtopping?	Could inadequate infrastructure and/or plans cause the dam to fail due to a reservoir landslide or upstream dam failure?
			Wind-wave dissipation inadequate	Is freeboard and wind wave dissipation adequate to prevent overtopping/failure during a meteorological event?	Could a seismic event cause the dam to be overtopped/fail due to inadequate freeboard and wind wave dissipation?	Is freeboard and wind wave dissipation adequate to prevent overtopping/failure from failure of features in the reservoir environment?	Could human and/or animal activities cause inadequate freeboard and wind wave dissipation leading to dam overtopping/failure?	Could design or construction of the water barrier cause inadequate freeboard and wind wave dissipation and cause overtopping/failure?	Could the design or construction of the hydraulic structure cause inadequate wind-wave dissipation leading to dam collapse by overtopping?	Could the design or construction of the mechanical/electrical systems cause inadequate wind-wave dissipation leading to dam collapse by overtopping?	Could inadequate infrastructure and plans cause inadequate wind-wave dissipation leading to dam collapse by overtopping?
	Management System Failure	Safeguards fail to provide timely detection and correction	Operation, maintenance and surveillance fail to detect/prevent hydraulic adequacy	Could a meteorological event prevent the Dam Safety Engineers activities (based on OMS requirements, see column L) from detecting/prevent hydraulic inadequacy leading to dam overtopping/failure?	Could a seismic event prevent the Dam Safety Engineers activities (based on OMS requirements, see column L) from detecting/preventing hydraulic inadequacy leading to overtopping/failure of the dam?	Could the reservoir environment prevent Dam Safety activities (based on OMS requirements, see column L) from detecting/preventing hydraulic inadequacy leading to dam overtopping/failure?	Could human and/or animal activities cause the OMS activities to not detect/prevent hydraulic inadequacy leading to dam overtopping/failure?	Could inadequate operation, maintenance and surveillance fail to detect / prevent hydraulic adequacy and lead to failure of the water barrier?	Could inadequate operation, maintenance and surveillance fail to detect / prevent hydraulic adequacy and lead to failure of the hydraulic structure?	Could inadequate operation, maintenance and surveillance fail to detect / prevent failure of the mechanical/electrical system leading to dam collapse by overtopping?	Could inadequate operation, maintenance and surveillance of the infrastructure and plans cause the OMS activities to not detect /prevent hydraulic inadequacy before leading to overtopping/failure of dam?
DAM COLLAPSE BY LOSS OF STRENGTH (External orinternal structural failure and weakening)			Operation, maintenance and surveillance fail to detect poor dam performance	Could the meteorological event prevent the OMS rules from being implemented by the DS Engineer leading to dam collapse by loss of strength?	Could a seismic event cause the OMS rules to not be followed leading to collapse by loss of strength during a seismic event?	Could the reservoir environment cause the OMS rules to not be followed leading to dam collapse by loss of strength?	Could human and/or animal activities cause OMS activities to not be followed leading to dam collapse by loss of strength?	Could inadequate operation, maintenance and surveillance fail to prevent poor dam performance and lead to dam collapse by loss of strength?	Could inadequate operation, maintenance and surveillance of the hydraulic structure fail to prevent poor dam performance and lead to dam collapse by loss of strength?	Could inadequate operation, maintenance and surveillance of the mechanical/electrical systems fail to prevent poor dam performance and lead to dam collapse by loss of strength?	Could inadequate surveillance and management of the infrastructure and plans cause the OMS activities to not detect /prevent dam collapse by loss of strength?
	Crest elevation too low	Stability under applied loads	Mass movement (external stability:- displacement, tilting, seismic resistance)	Could loss of strength and static instability occur during a meteorological event and cause dam collapse?	Could a seismic event cause mass external instability and cause dam collapse?	Could the reservoir environment cause external instability of the dam leading to dam collapse?	Could human and/or animal activities cause external instability of the dam and cause dam collapse?	Could design or construction of the water barrier cause external instability and lead to dam collapse?	Could the design or construction of the hydraulic structure cause external instability leading to dam collapse by loss of strength?	Could the design or construction of the mechanical/electrical systems cause external instability leading dam collapse by loss of strength?	Could inadequate infrastructure and plans cause external instability leading to dam collapse by loss of strength?
			Loss of support (foundation or abutment failure)	Could reduction/lack of support in foundation or abutments during a meteorological event cause dam collapse?	Could a seismic event cause reduction/lack of support in foundation or abutments leading to dam collapse?	Could the reservoir environment (debris, ice, landslides) cause foundation or abutment failure leading to dam collapse?	Could human and/or animal activities cause reduction/lack of support in foundation or abutments and cause dam collapse?	Could design or construction of the water barrier cause reduction/lack of support in foundation or abutments and cause dam collapse?	Could the design or construction of the hydraulic structure cause reduction/lack of support in foundation or abutments and lead to dam collapse by loss of strength?	Could the design or construction of the mechanical/electrical systems cause a reduction/lack of support in foundation or abutments leading to dam collapse by loss of strength?	Could inadequate infrastructure and plans cause reduction/lack of support in foundation or abutments leading to dam collapse by loss of strength?
		Watertightness	Seepage around interfaces (abutments, foundation, water stops)	Could seepage around interfaces/abutments/foundation during meteorological event reduce watertightness sufficient to cause dam collapse?	Could a seismic event cause seepage around interfaces / abutments / foundation reduce watertightness sufficient to cause dam collapse?	Could the reservoir environment (debris, ice, landslides) cause seepage around interfaces/abutments/foundation and reduce watertightness sufficient to cause dam collapse?	Could human and/or animal activities seepage around interfaces / abutments / foundation and reduce watertightness sufficient to cause dam collapse?	Could design or construction of the water barrier cause seepage around interfaces / abutments / foundation and reduce watertightness sufficient to cause dam collapse?	Could the design or construction of the hydraulic structure cause seepage around interfaces/ abutments/ foundation leading to dam collapse by loss of strength?	Could the design or construction of the mechanical/electrical systems cause seepage around interfaces/ abutments/ foundation leading to dam collapse by loss of strength?	Could inadequate infrastructure and plans cause seepage around interfaces/ abutments/ foundation and reduce watertightness sufficient to cause dam collapse by loss of strength?
			Through dam seepage control failure (filters, drains, pumps)	Could through -dam seepage (filters/drains/pumps, internal instability) during a meteorological event reduce watertightness and cause dam collapse?	Could a seismic event cause through dam seepage (filters/drains/pumps) to fail and reduce watertightness and cause dam collapse?	Could the reservoir environment (landslides, ice, debris) cause through dam seepage control be lost (filters/drains/pumps) and reduce watertightness and cause dam collapse?	Could human and/or animal activities cause failure of through dam seepage (filters / drains / pumps) control and reduce watertightness and cause dam collapse?	Could design or construction of the water barrier cause through dam seepage (filters / drains / pumps) and reduce watertightness and cause dam collapse?	Could the design or construction of the hydraulic structure cause through dam seepage control failure (filters/ drains/ pumps) and lead to dam collapse by loss of strength?	Could the design or construction of the mechanical/electrical systems cause through dam seepage (filters/ drains/ pumps) and reduce watertightness and cause dam collapse?	Could inadequate infrastructure and plans cause through dam seepage (filters/ drains/ pumps) and cause dam collapse by loss of strength?
		Durability/cracking	Structural weakening (internal erosion, AAR, crushing, gradual strength loss)	Could structural weakening (internal erosion, crushing, cracking, strength loss) caused by a meteorological event cause dam collapse?	Could a seismic event cause internal structural weakening (internal erosion, crushing, cracking, strength loss) and cause dam collapse?	Could the reservoir environment (landslides, ice, debris) cause internal structural weakening (internal erosion, crushing, cracking, strength loss) and lead to dam collapse?	Could human and/or animal activities cause internal structural weakening (internal erosion, crushing, cracking, strength loss) and cause dam collapse?	Could design or construction of the water barrier cause internal structural weakening (internal erosion, crushing, cracking, strength loss) and cause dam collapse?	Could the design or construction of the hydraulic structure cause internal structural weakening (internal erosion, crushing, cracking, strength loss) leading to dam collapse?	Could the design or construction of the mechanical/electrical systems cause internal structural weakening (internal erosion, crushing, cracking, strength loss) leading to dam collapse by loss of strength?	Could inadequate infrastructure and plans cause internal structural weakening (internal erosion, crushing, cracking, strength loss) and cause dam collapse by loss of strength?
			Instantaneous change of state (static liquefaction, hydraulic fracture, seismic cracking)	Could instantaneous change of state occur (Liquefaction, hydraulic fracture) caused by a meteorological event cause dam collapse?	Could a seismic event cause instantaneous change of state to occur (Liquefaction, hydraulic fracture) leading to dam collapse?	Could the reservoir environment (landslides, ice, debris) cause instantaneous change of state to occur (liquefaction, hydraulic fracture) and cause dam collapse?	Could human and/or animal activities cause instantaneous change of state to occur (Liquefaction, hydraulic fracture) and cause dam collapse?	Could design or construction of the water barrier cause instantaneous change of state occur (Liquefaction, hydraulic fracture) and cause dam collapse?	Could the design or construction of the hydraulic structure cause instantaneous change of state to occur (Liquefaction, hydraulic fracture) leading to dam collapse?	Could the design or construction of the mechanical/electrical systems cause instantaneous change of state to occur (Liquefaction, hydraulic fracture) leading to dam collapse by loss of strength?	Could inadequate infrastructure and plans cause instantaneous change of state occur (Liquefaction, hydraulic fracture) and cause dam collapse by loss of strength?

Appendix F

Liquefaction Triggering Assessment

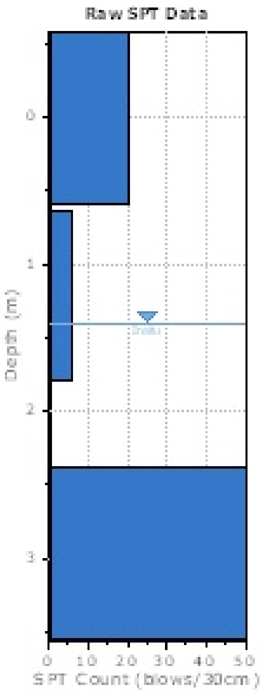
SPT BASED LIQUEFACTION ANALYSIS REPORT

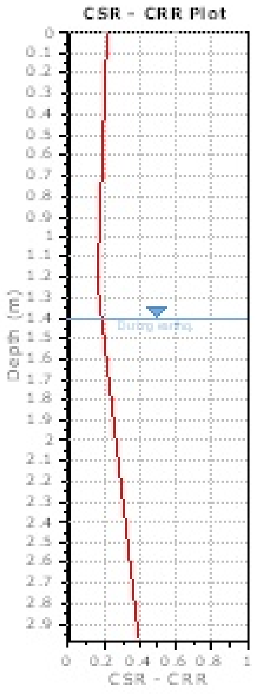
Project title : Stocking Lake Dam **SPT Name: BH18-01**

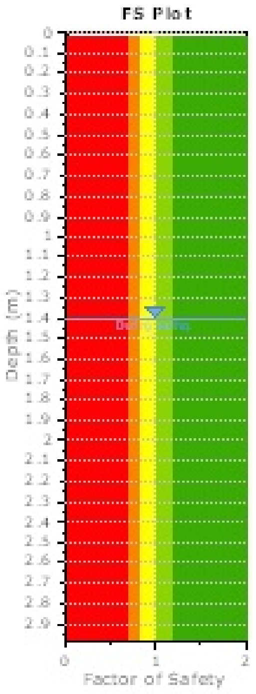
Location : Stocking Lake Dam

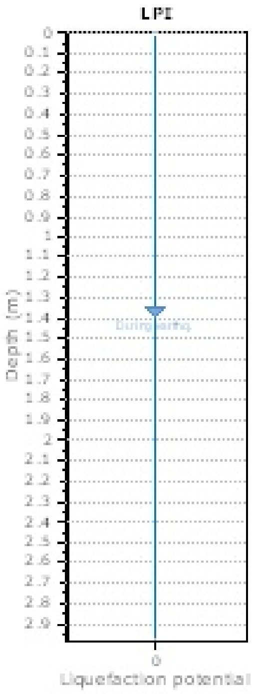
:: Input parameters and analysis properties ::

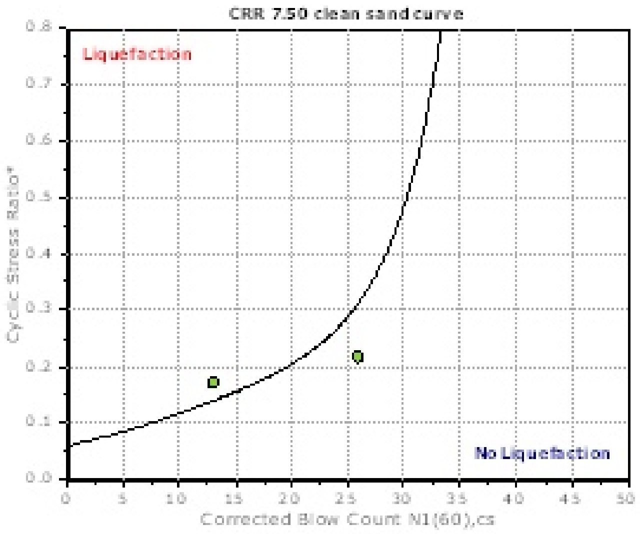
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.40 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.40 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

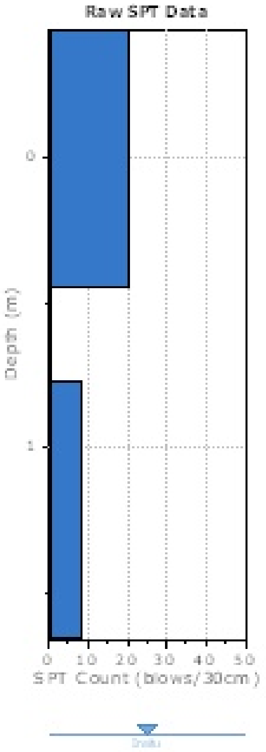
SPT BASED LIQUEFACTION ANALYSIS REPORT

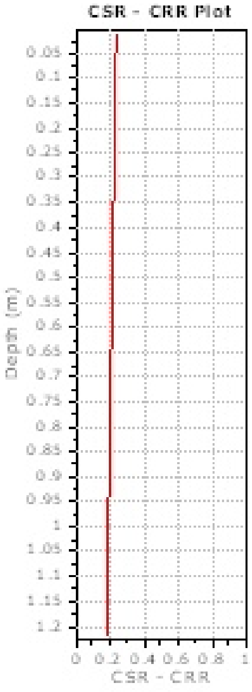
Project title : Stocking Lake Dam **SPT Name: BH18-02**

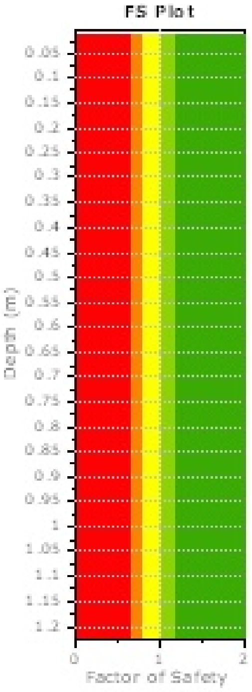
Location : Stocking Lake Dam

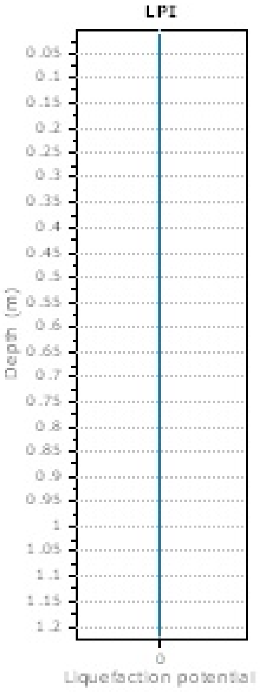
:: Input parameters and analysis properties ::

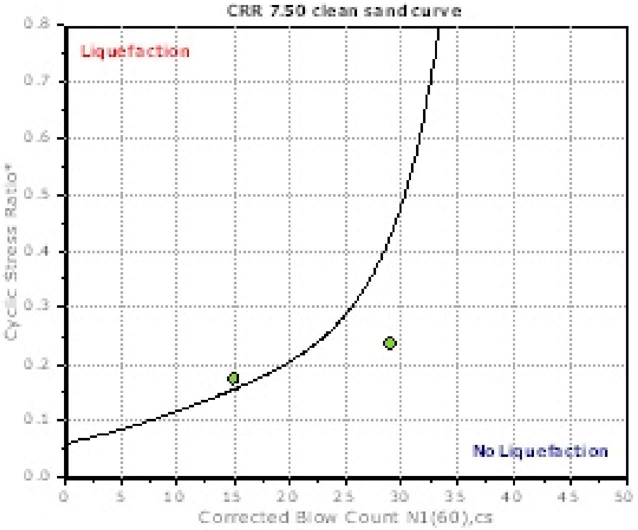
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

- 🟥 Very high risk
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- 🟨 Low risk

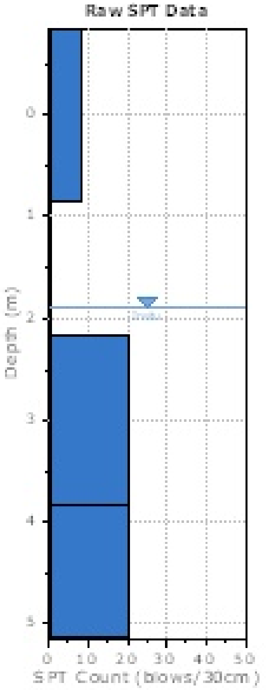
SPT BASED LIQUEFACTION ANALYSIS REPORT

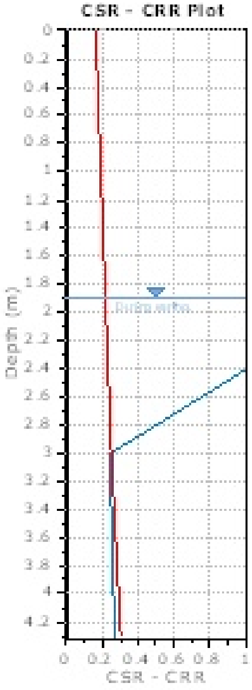
Project title : Stocking Lake Dam **SPT Name: EBA-BH02**

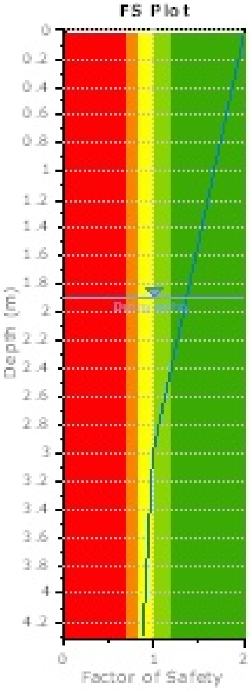
Location : Stocking Lake Dam

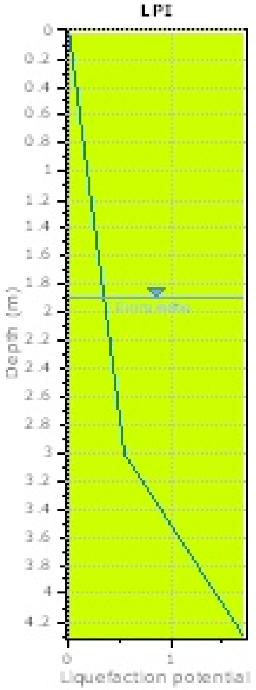
:: Input parameters and analysis properties ::

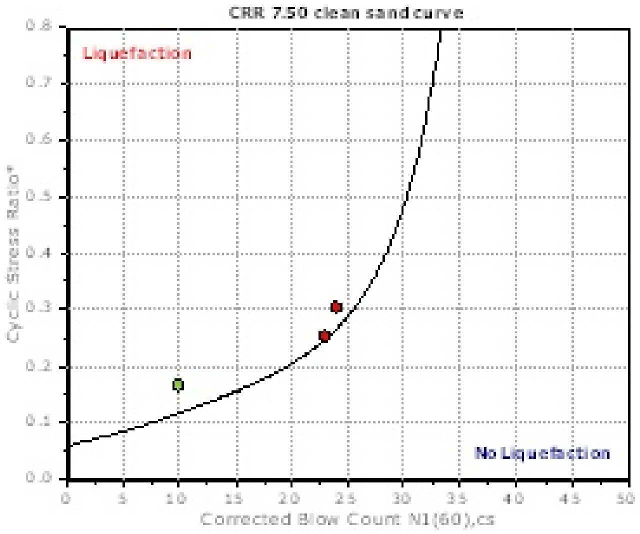
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.90 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.90 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

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- 🟨 Low risk

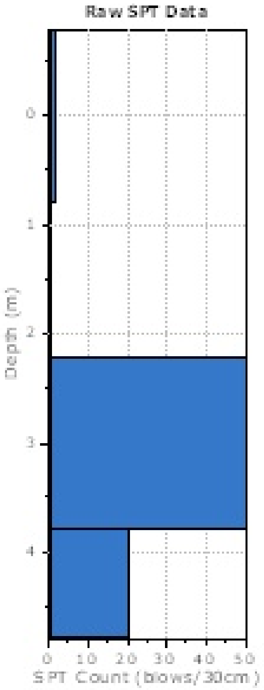
SPT BASED LIQUEFACTION ANALYSIS REPORT

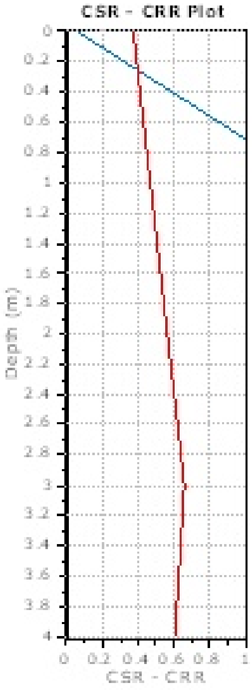
Project title : Stocking Lake Dam **SPT Name: EBA-BH01**

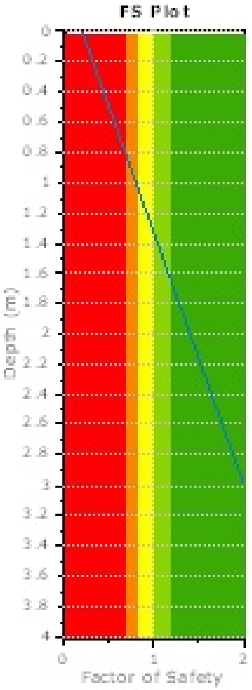
Location : Stocking Lake Dam

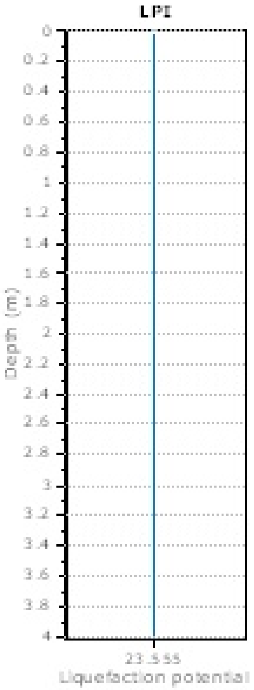
:: Input parameters and analysis properties ::

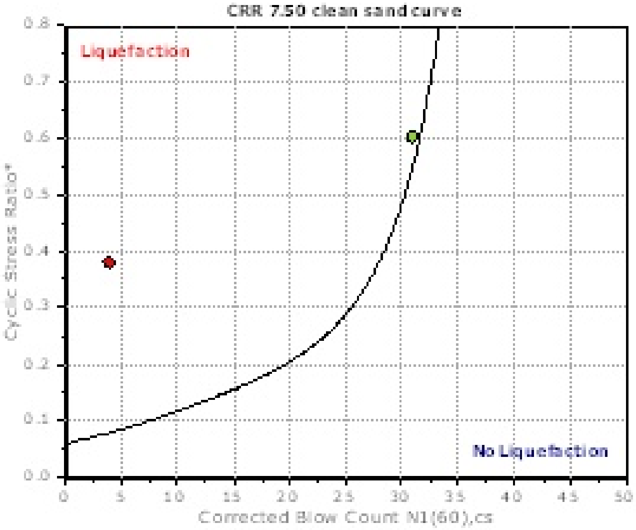
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

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- 🟨 Low risk

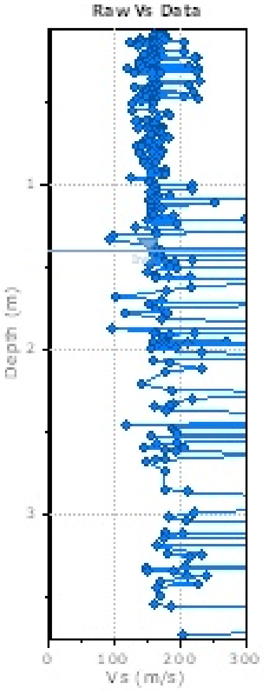
V_s BASED LIQUEFACTION ANALYSIS REPORT

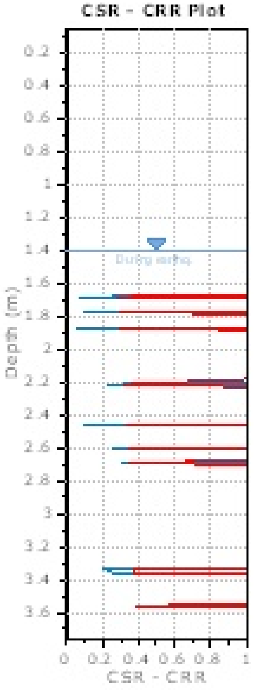
Project title : Stocking Lake Dam **V_s Name: MASW18-01**

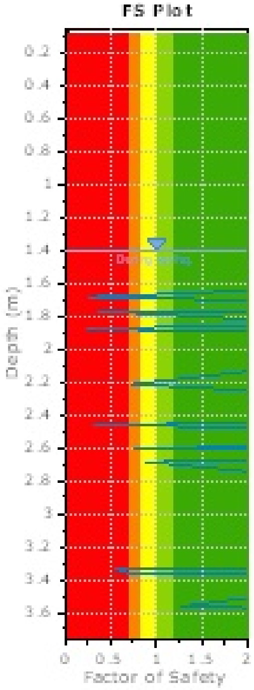
Location : Stocking Lake Dam

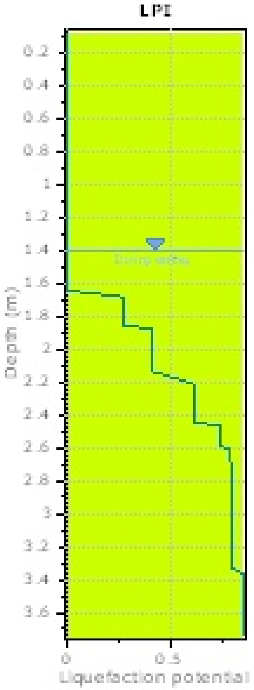
:: Input parameters and analysis properties ::

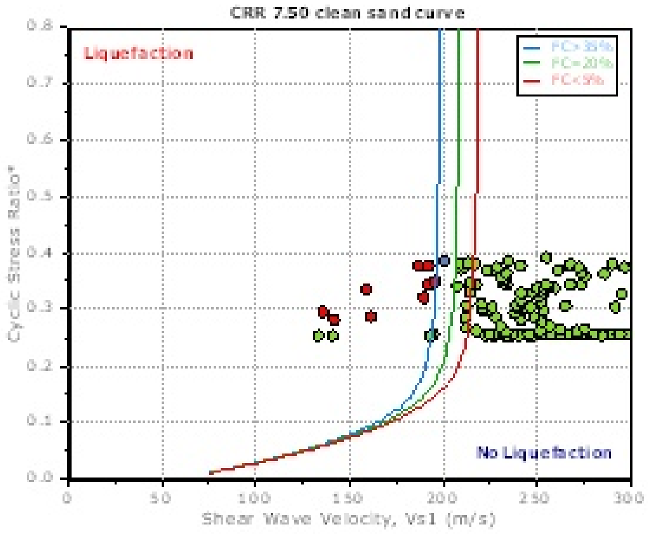
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	1.40 m
G.W.T. (earthq.):	1.40 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.26 g
Eq. external load:	0.00 kPa











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LPI color scheme

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- 🟨 Low risk

V_s BASED LIQUEFACTION ANALYSIS REPORT

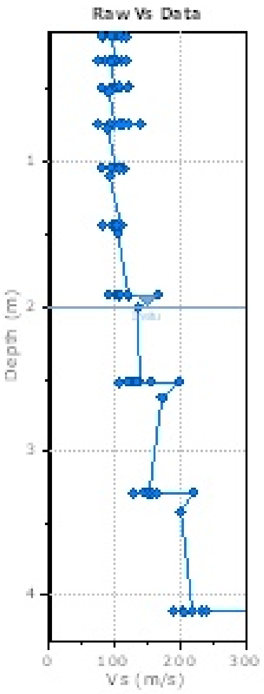
Project title : Stocking Lake Dam

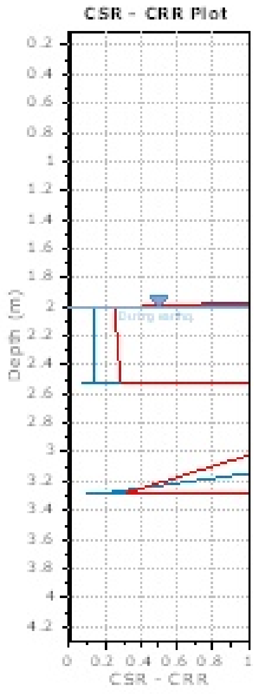
V_s Name: MASW18-02

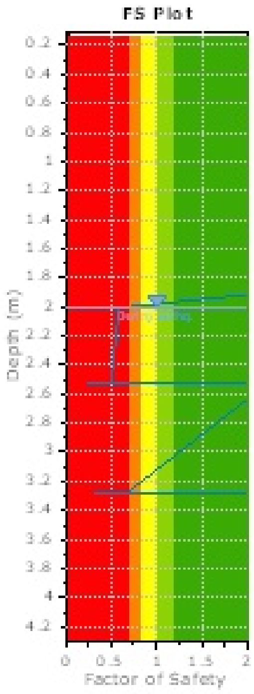
Location : Stocking Lake Dam

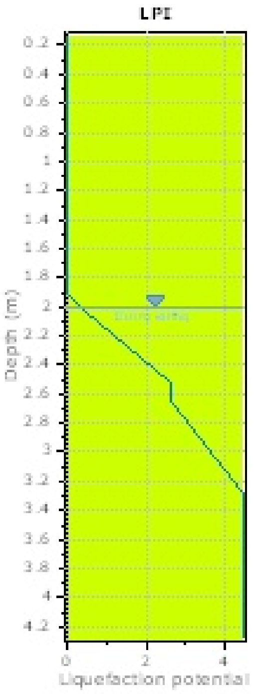
:: Input parameters and analysis properties ::

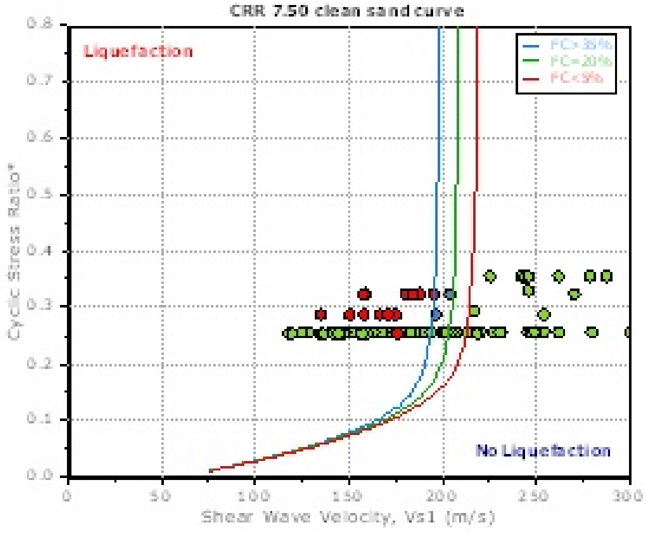
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.00 m
G.W.T. (earthq.):	2.00 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.26 g
Eq. external load:	0.00 kPa











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LPI color scheme

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Ecora

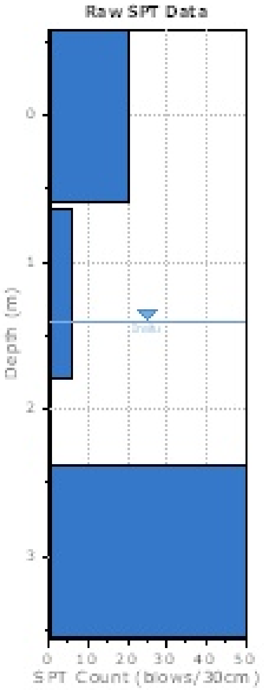
SPT BASED LIQUEFACTION ANALYSIS REPORT

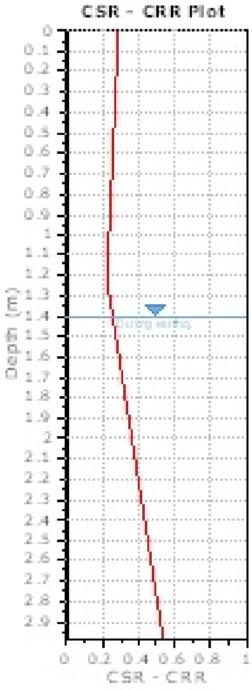
Project title : Stocking Lake Dam **SPT Name: BH18-01**

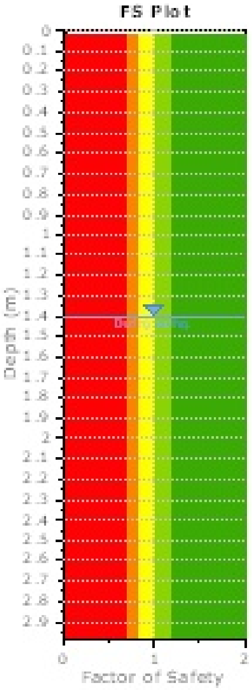
Location : Stocking Lake Dam

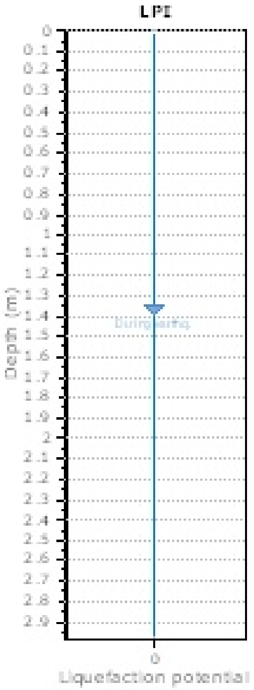
:: Input parameters and analysis properties ::

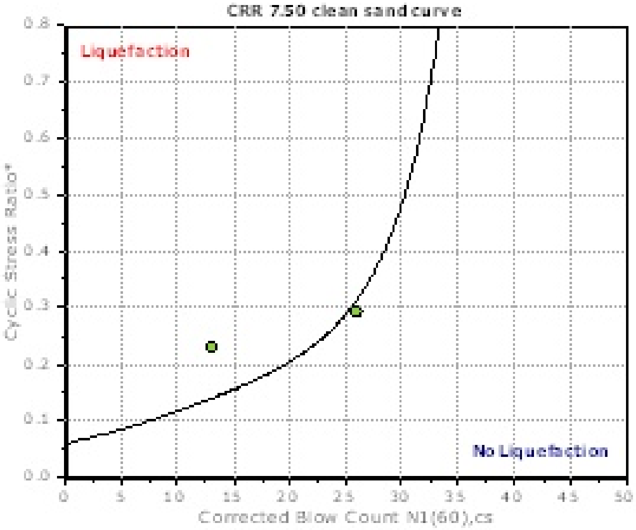
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.40 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.40 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

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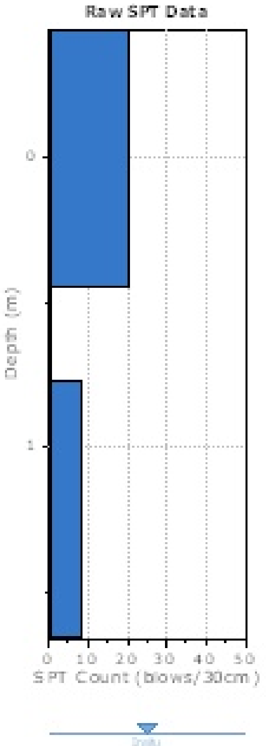
SPT BASED LIQUEFACTION ANALYSIS REPORT

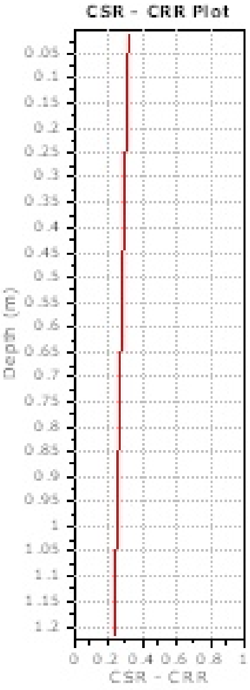
Project title : Stocking Lake Dam **SPT Name: BH18-02**

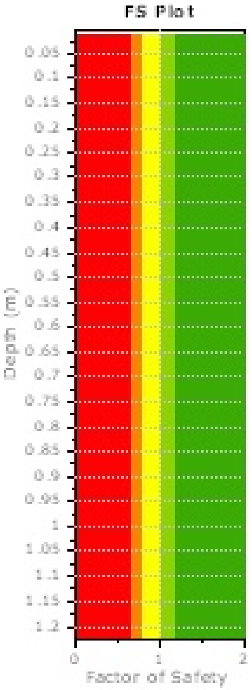
Location : Stocking Lake Dam

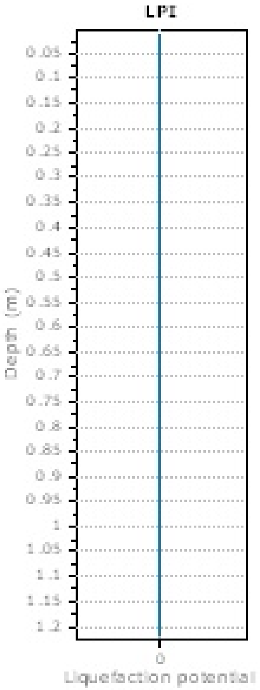
:: Input parameters and analysis properties ::

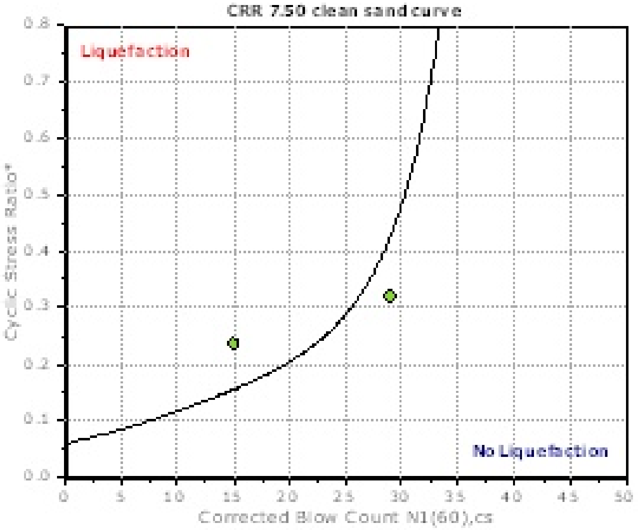
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

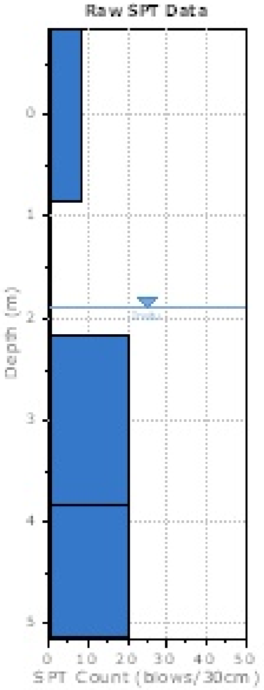
SPT BASED LIQUEFACTION ANALYSIS REPORT

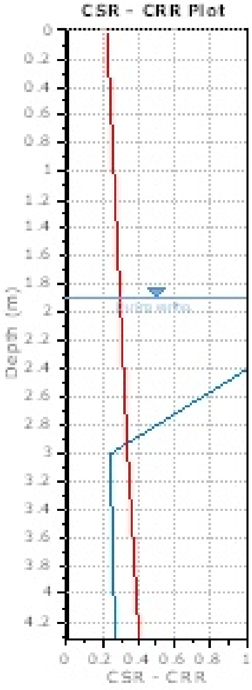
Project title : Stocking Lake Dam **SPT Name: EBA-BH02**

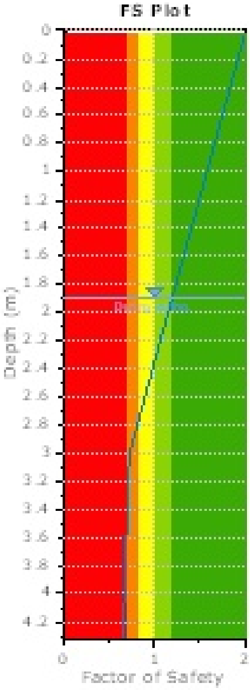
Location : Stocking Lake Dam

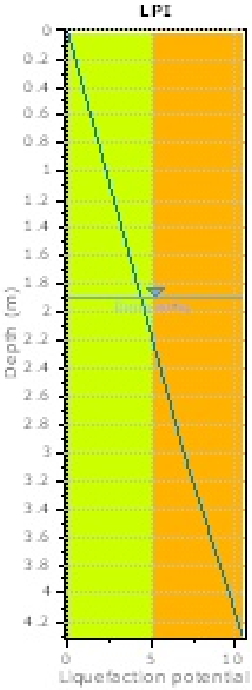
:: Input parameters and analysis properties ::

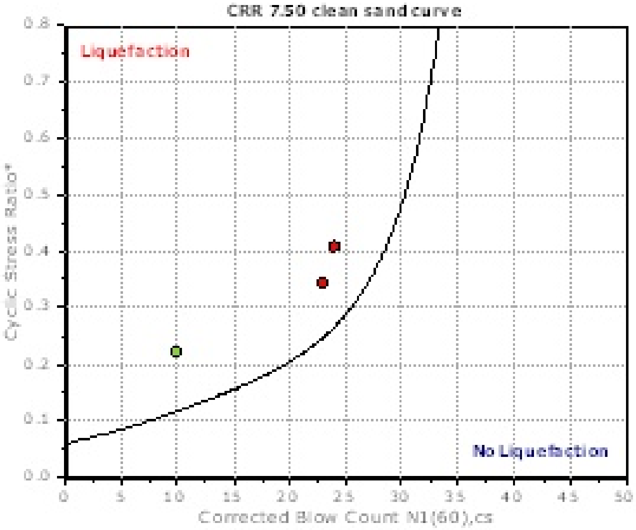
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.90 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.90 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

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- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

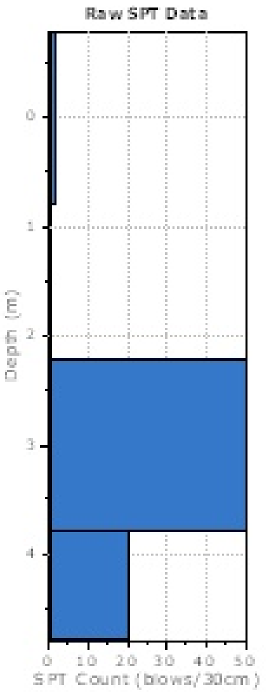
SPT BASED LIQUEFACTION ANALYSIS REPORT

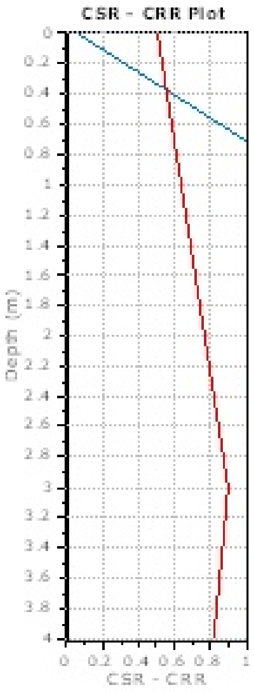
Project title : Stocking Lake Dam **SPT Name: EBA-BH01**

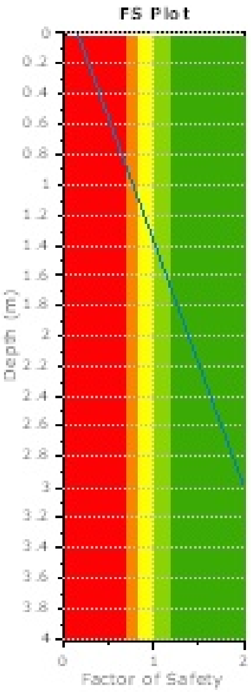
Location : Stocking Lake Dam

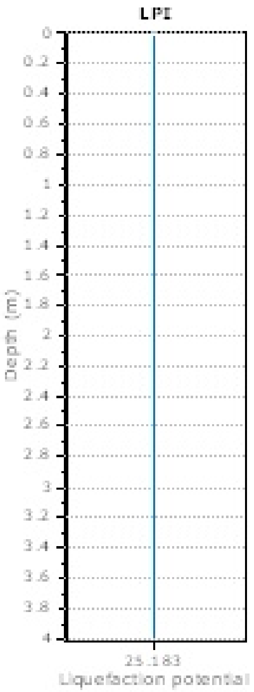
:: Input parameters and analysis properties ::

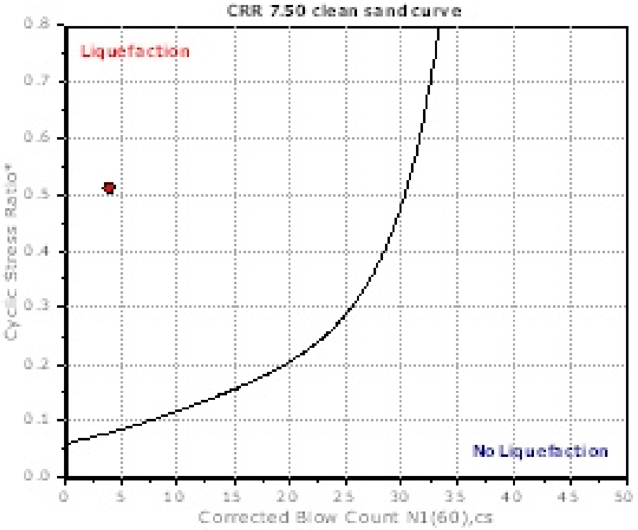
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

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- 🟩 Unlike to liquefy
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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

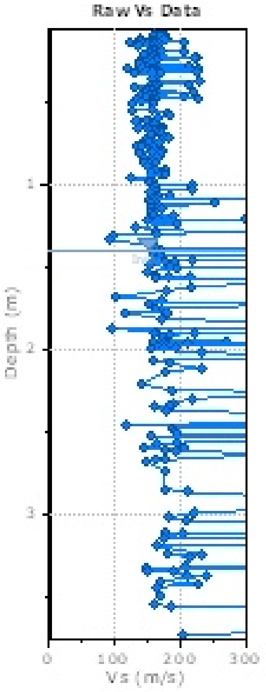
V_s BASED LIQUEFACTION ANALYSIS REPORT

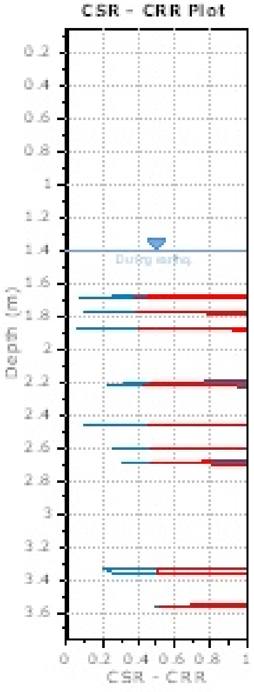
Project title : Stocking Lake Dam **V_s Name: MASW18-01**

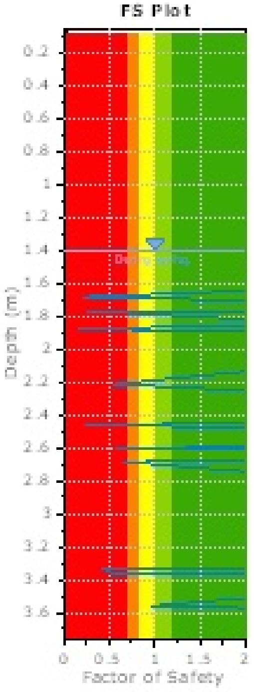
Location : Stocking Lake Dam

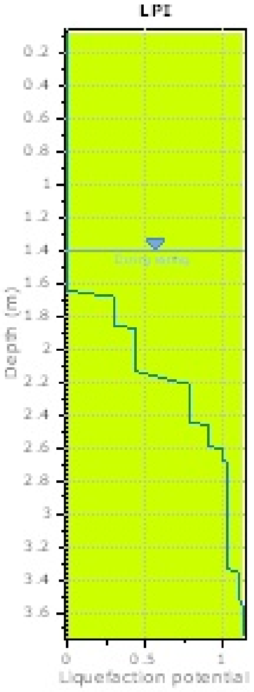
:: Input parameters and analysis properties ::

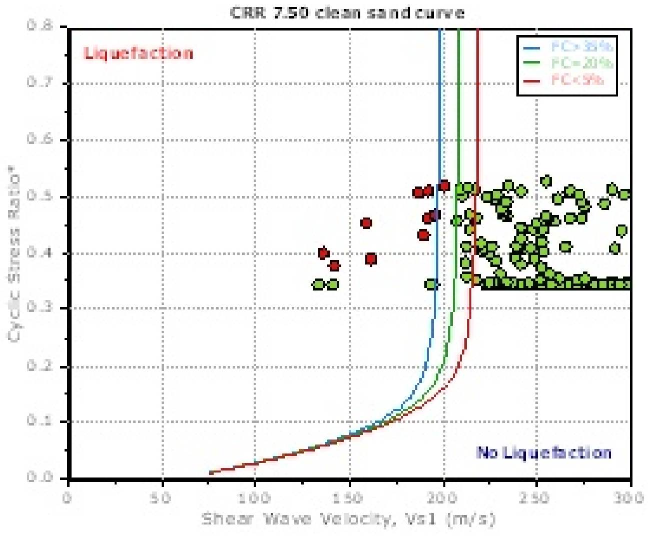
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	1.40 m
G.W.T. (earthq.):	1.40 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.35 g
Eq. external load:	0.00 kPa











F.S. color scheme

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- 🟩 Unlike to liquefy
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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

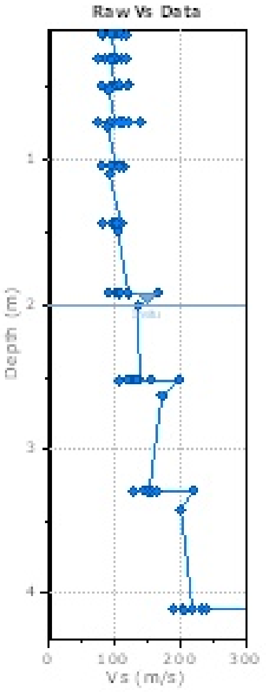
V$_s$ BASED LIQUEFACTION ANALYSIS REPORT

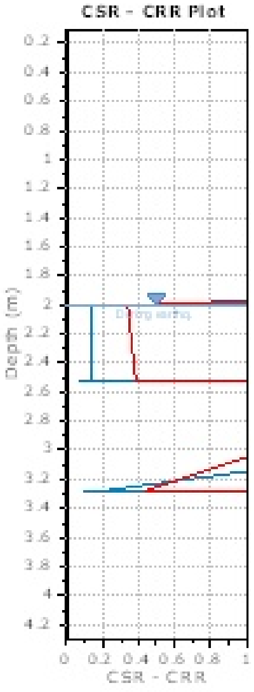
Project title : Stocking Lake Dam **V$_s$ Name: MASW18-02**

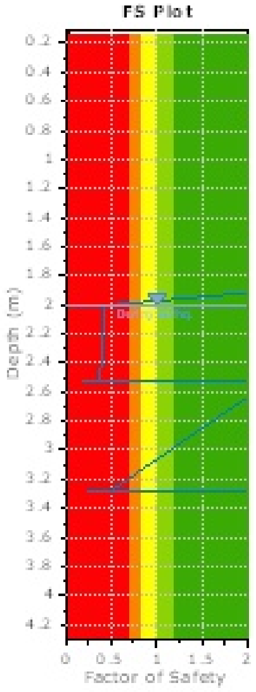
Location : Stocking Lake Dam

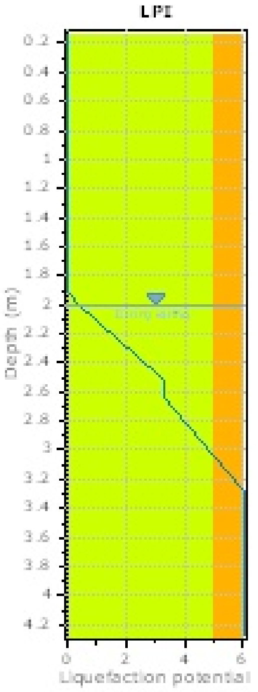
:: Input parameters and analysis properties ::

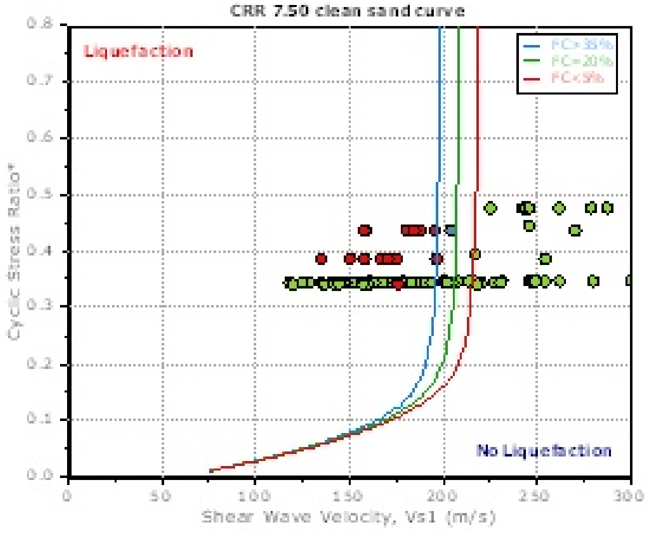
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.00 m
G.W.T. (earthq.):	2.00 m
Earthquake magnitude M$_w$:	8.83
Peak ground acceleration:	0.35 g
Eq. external load:	0.00 kPa











F.S. color scheme

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- 🟩 Unlike to liquefy
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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

SPT BASED LIQUEFACTION ANALYSIS REPORT

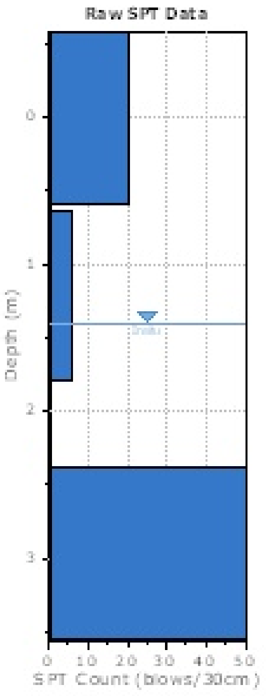
Project title : Stocking Lake Dam **SPT Name: BH18-01**

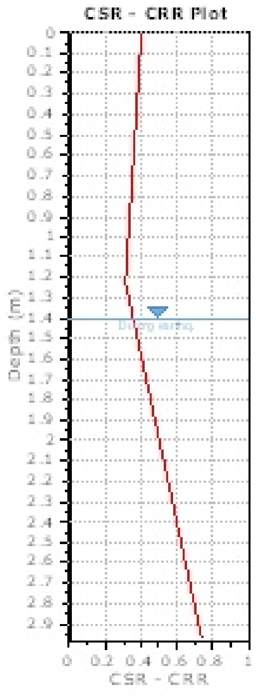
Location : Stocking Lake Dam

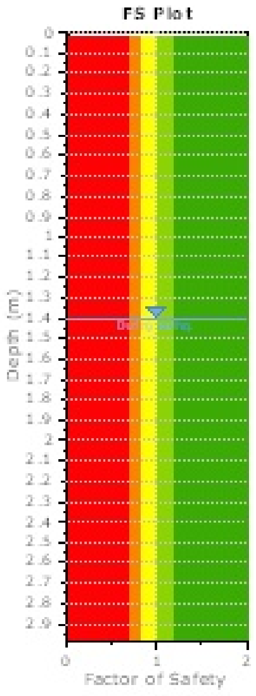
:: Input parameters and analysis properties ::

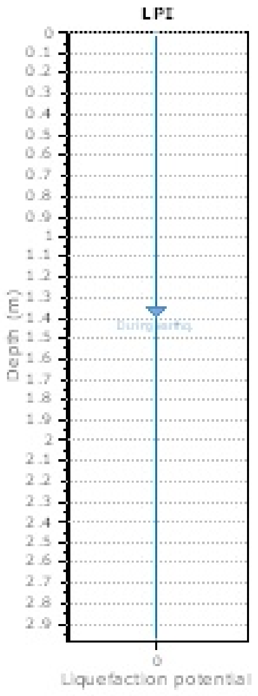
Analysis method: Boulanger & Idriss, 2014
Fines correction method: Boulanger & Idriss, 2014
Sampling method: Standard Sampler
Borehole diameter: 65mm to 115mm
Rod length: 1.00 m
Hammer energy ratio: 1.00

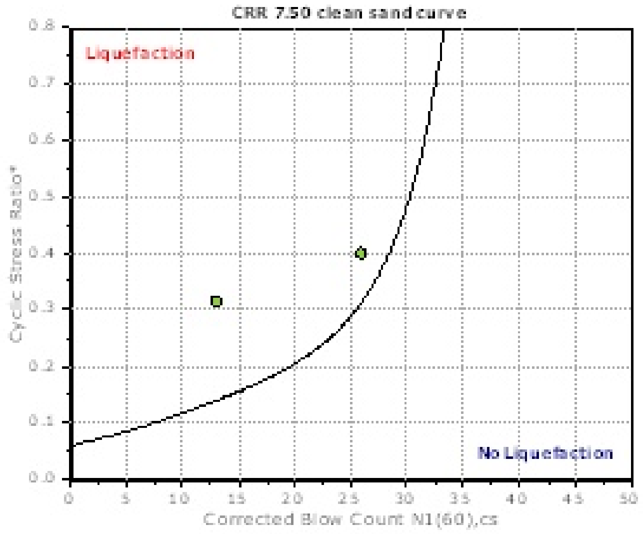
G.W.T. (in-situ): 1.40 m
G.W.T. (earthq.): 1.40 m
Earthquake magnitude M_w: 8.83
Peak ground acceleration: 0.48 g
Eq. external load: 0.00 kPa











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

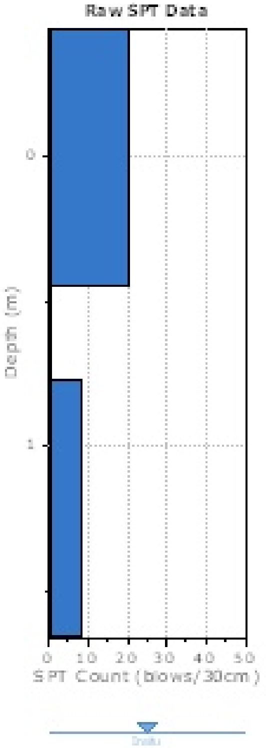
SPT BASED LIQUEFACTION ANALYSIS REPORT

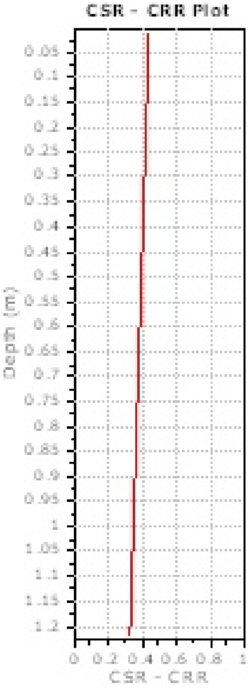
Project title : Stocking Lake Dam **SPT Name: BH18-02**

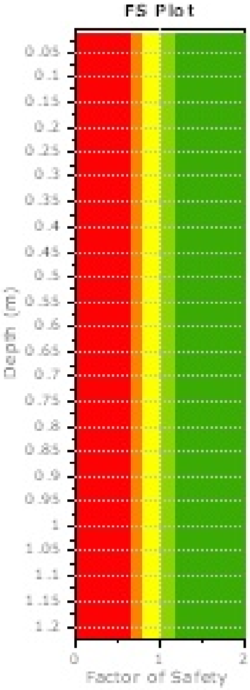
Location : Stocking Lake Dam

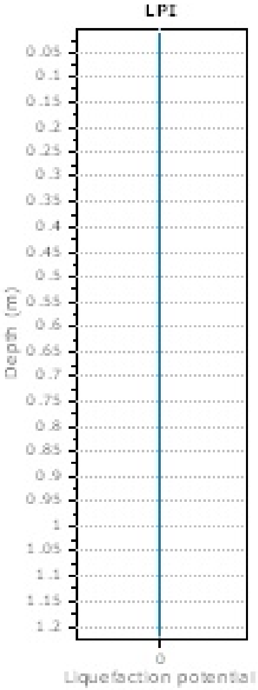
:: Input parameters and analysis properties ::

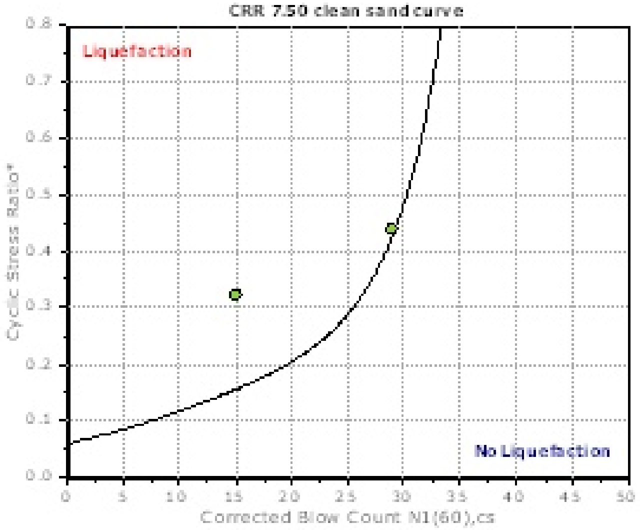
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

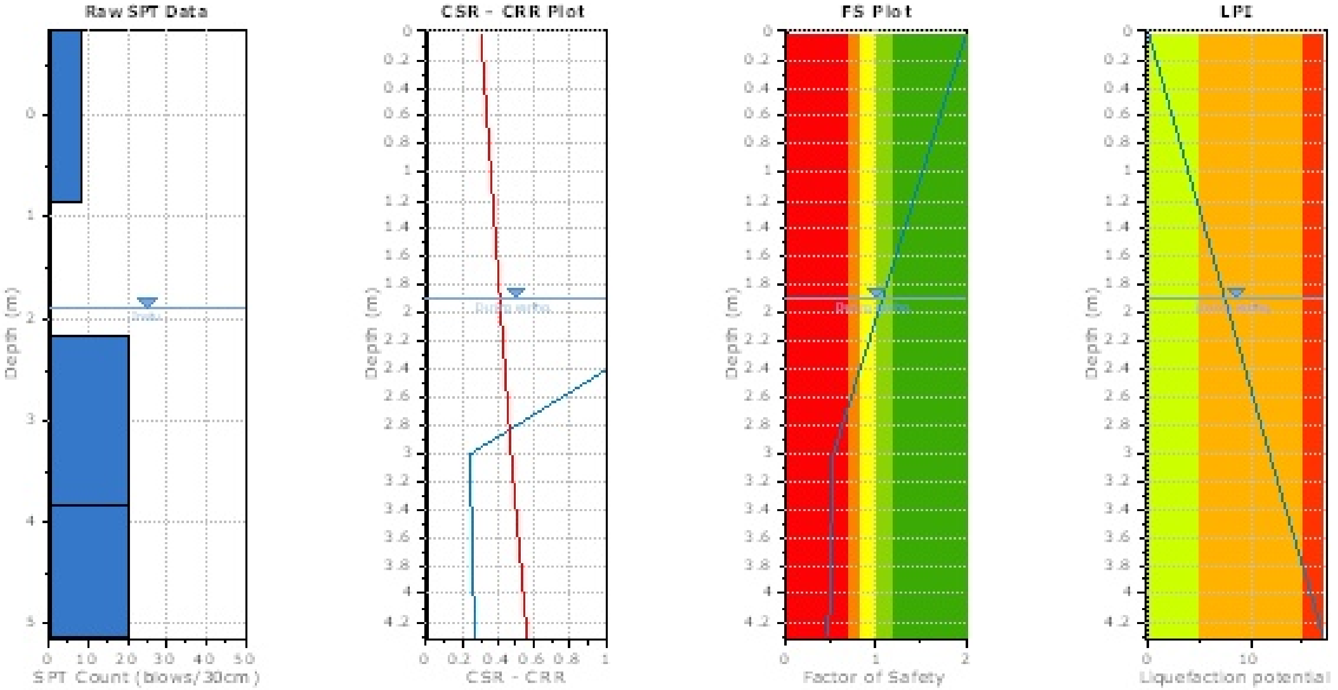
SPT BASED LIQUEFACTION ANALYSIS REPORT

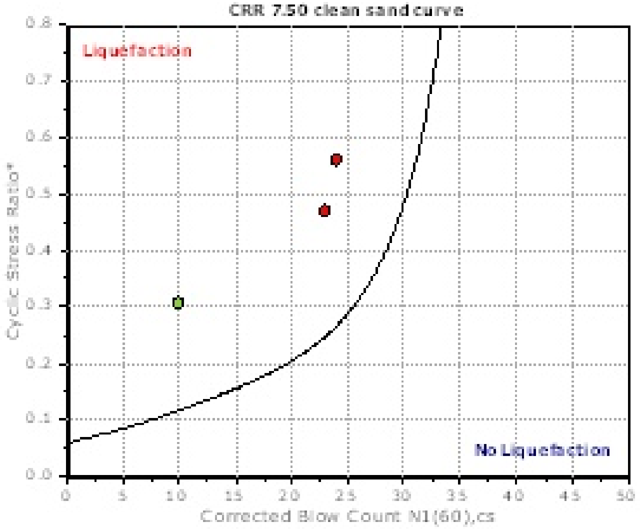
Project title : Stocking Lake Dam **SPT Name: EBA-BH02**

Location : Stocking Lake Dam

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	1.90 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	1.90 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

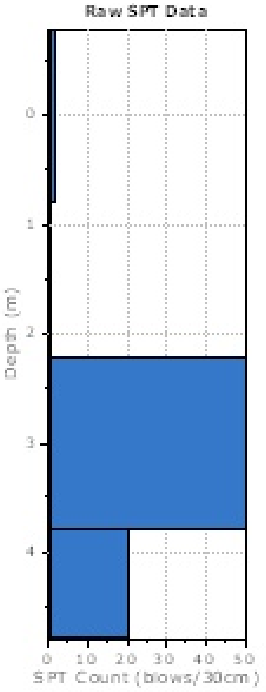
SPT BASED LIQUEFACTION ANALYSIS REPORT

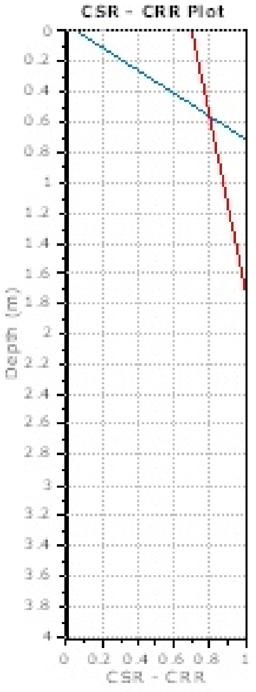
Project title : Stocking Lake Dam **SPT Name: EBA-BH01**

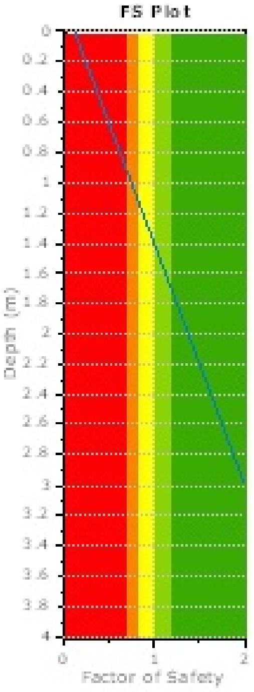
Location : Stocking Lake Dam

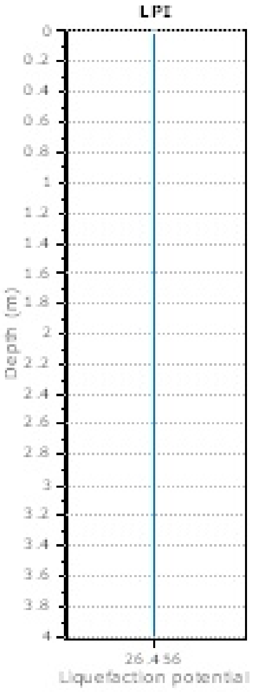
:: Input parameters and analysis properties ::

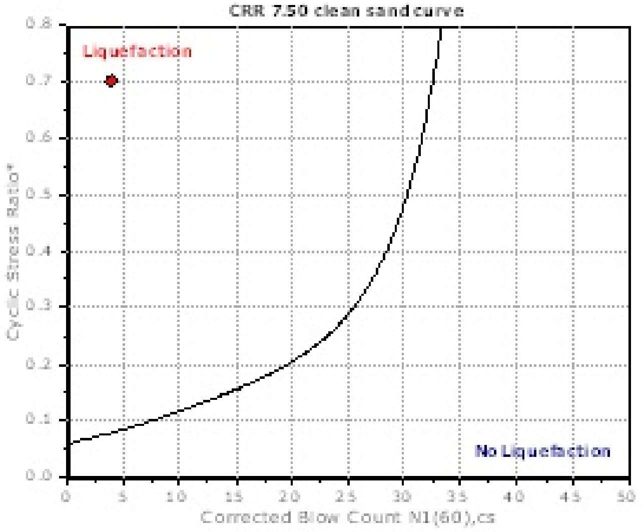
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

V_s BASED LIQUEFACTION ANALYSIS REPORT

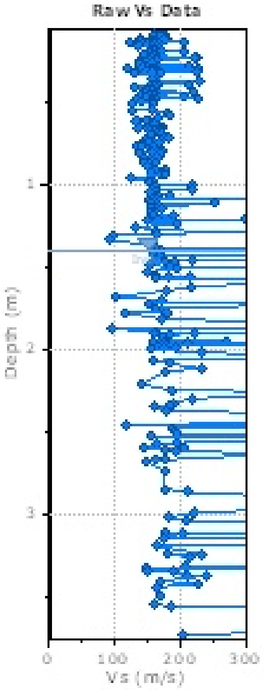
Project title : Stocking Lake Dam

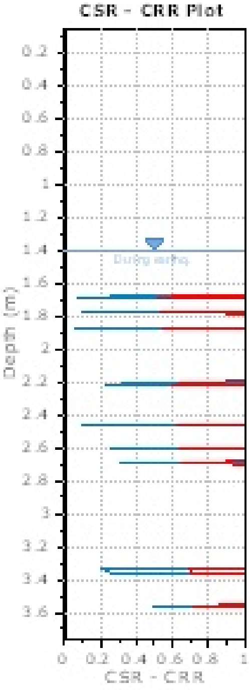
V_s Name: MASW18-01

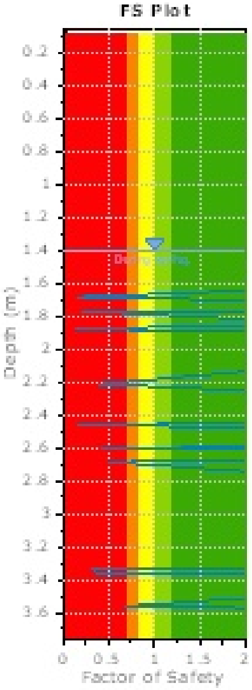
Location : Stocking Lake Dam

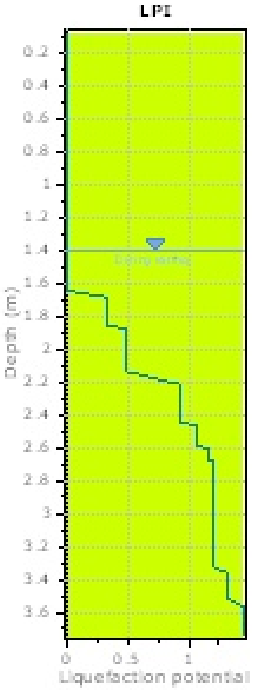
:: Input parameters and analysis properties ::

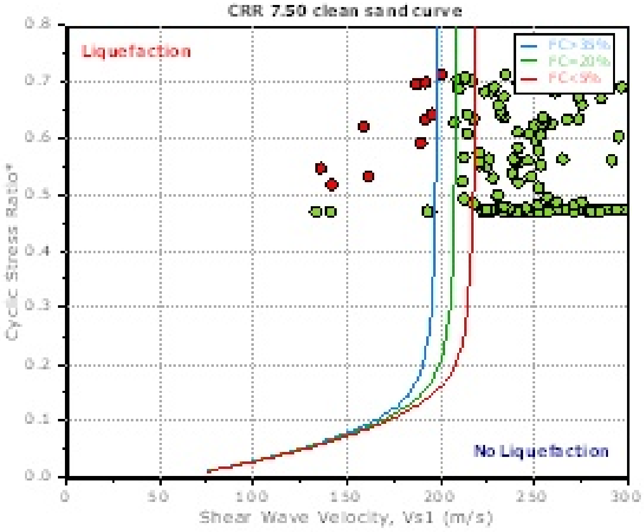
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	1.40 m
G.W.T. (earthq.):	1.40 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.48 g
Eq. external load:	0.00 kPa











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

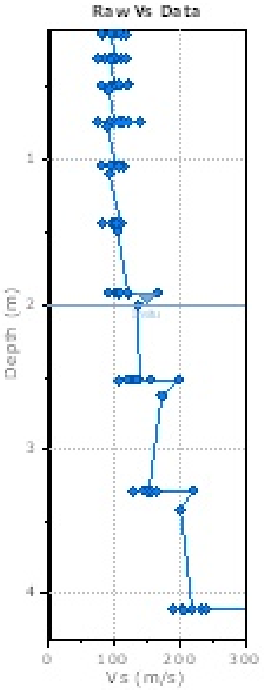
V_s BASED LIQUEFACTION ANALYSIS REPORT

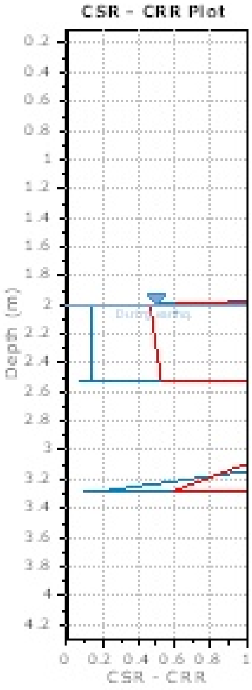
Project title : Stocking Lake Dam **V_s Name: MASW18-02**

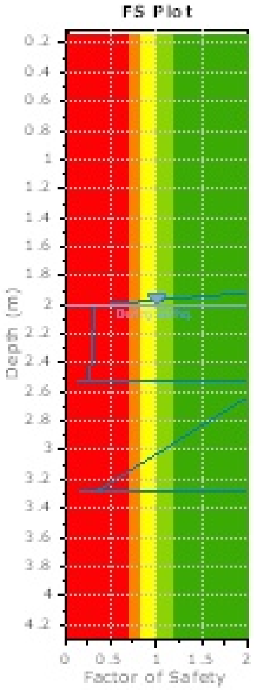
Location : Stocking Lake Dam

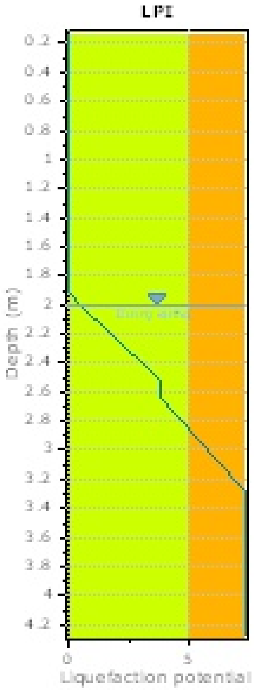
:: Input parameters and analysis properties ::

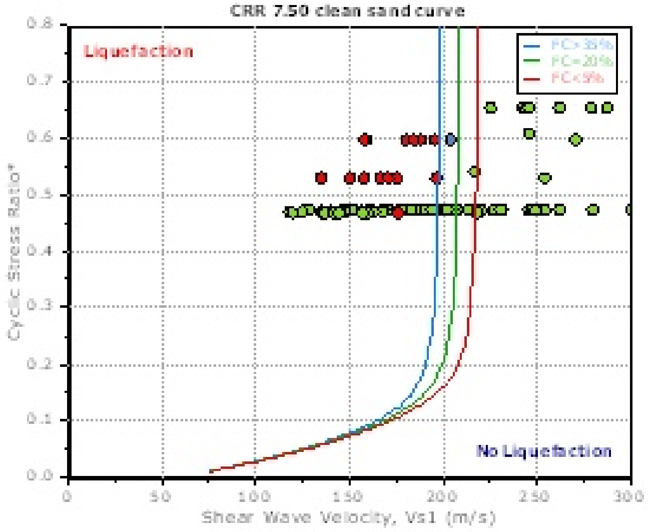
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.00 m
G.W.T. (earthq.):	2.00 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.48 g
Eq. external load:	0.00 kPa











F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

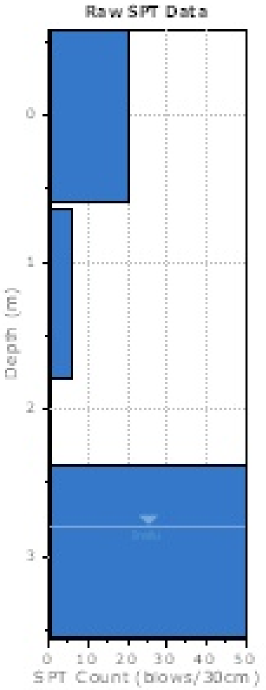
SPT BASED LIQUEFACTION ANALYSIS REPORT

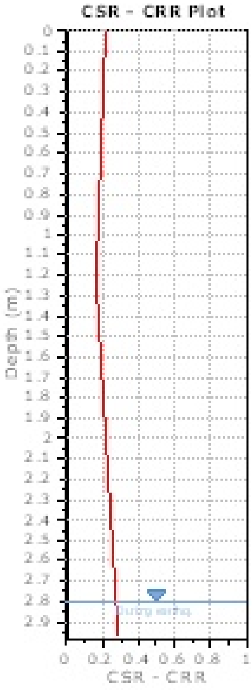
Project title : Stocking Lake Dam **SPT Name: BH18-01**

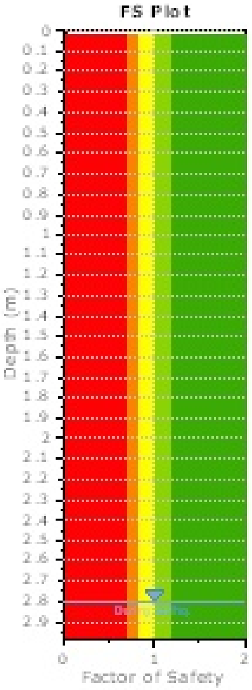
Location : Stocking Lake Dam

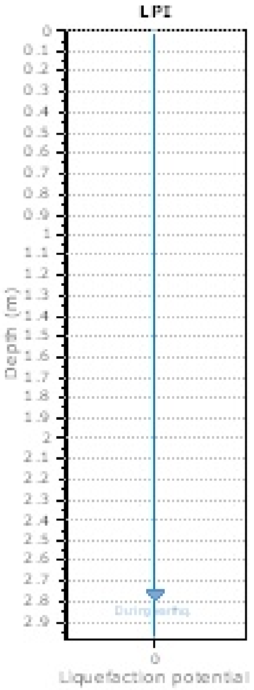
:: Input parameters and analysis properties ::

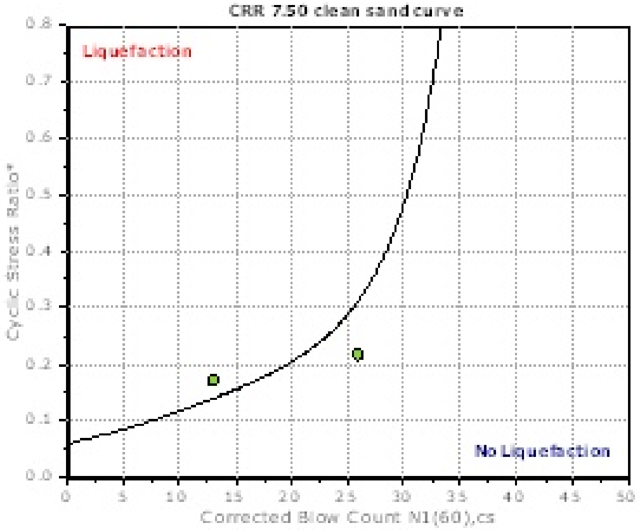
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

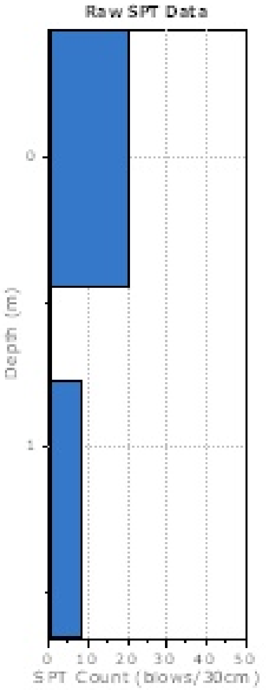
SPT BASED LIQUEFACTION ANALYSIS REPORT

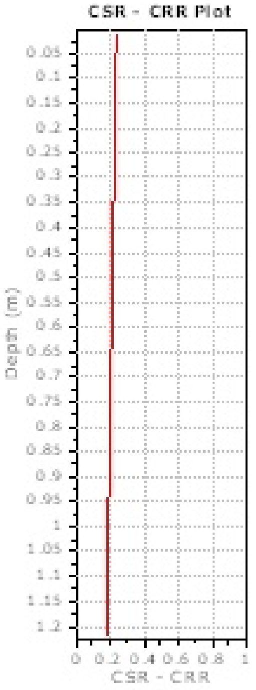
Project title : Stocking Lake Dam **SPT Name: BH18-02**

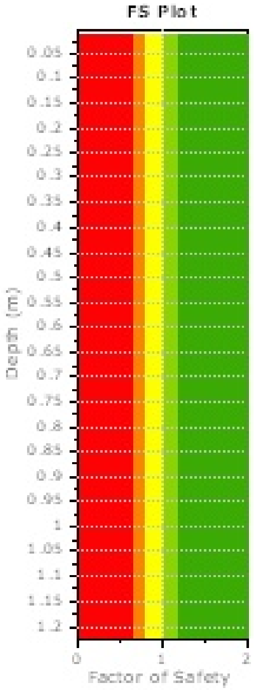
Location : Stocking Lake Dam

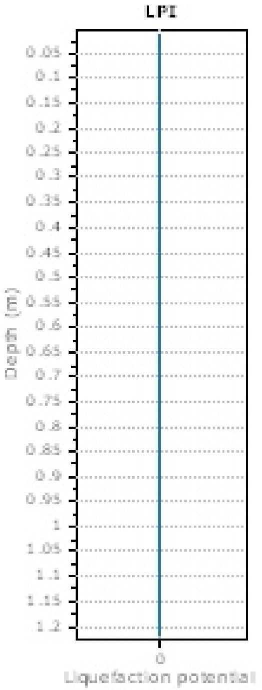
:: Input parameters and analysis properties ::

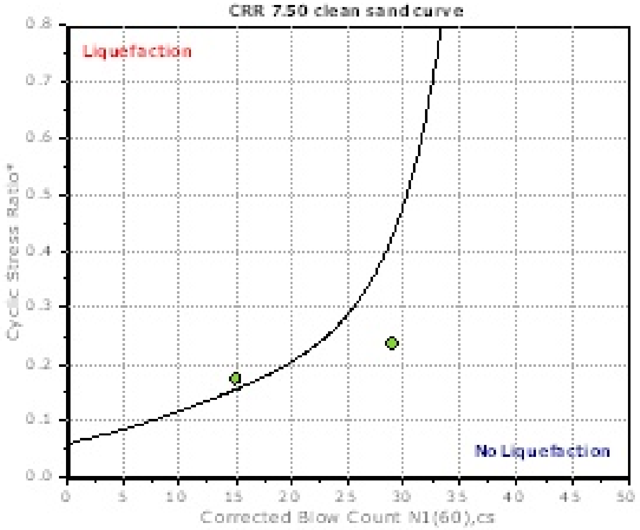
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

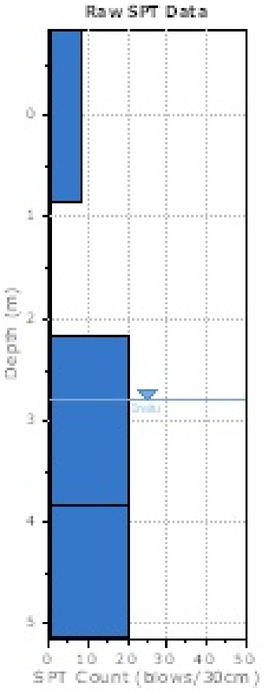
SPT BASED LIQUEFACTION ANALYSIS REPORT

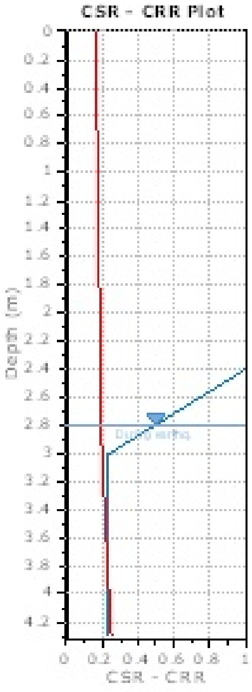
Project title : Stocking Lake Dam **SPT Name: EBA-BH02**

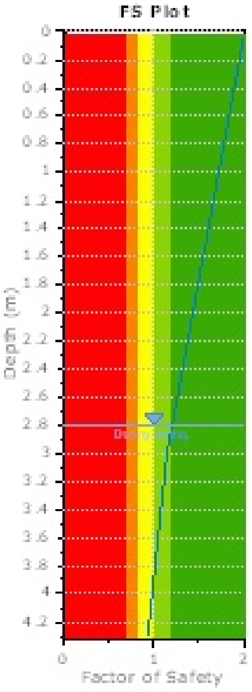
Location : Stocking Lake Dam

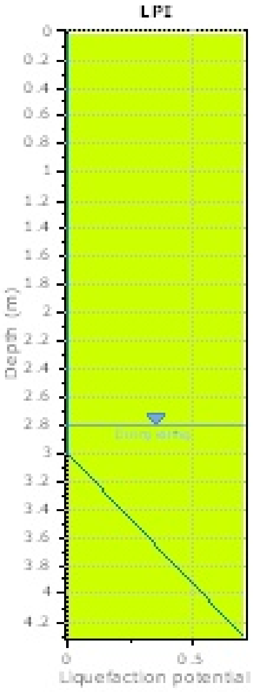
:: Input parameters and analysis properties ::

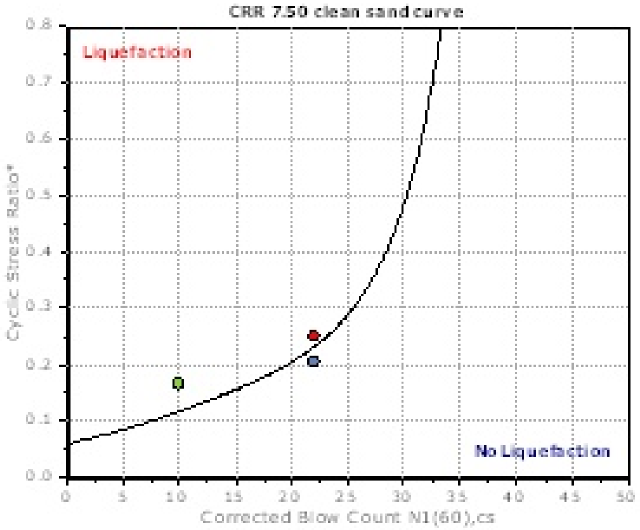
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

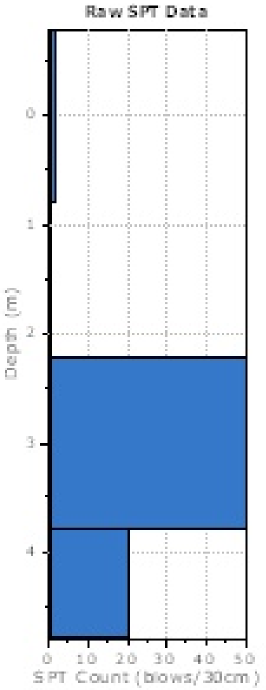
SPT BASED LIQUEFACTION ANALYSIS REPORT

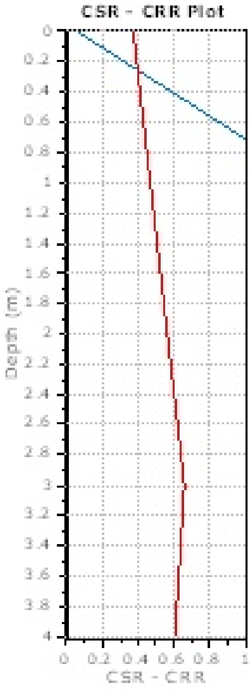
Project title : Stocking Lake Dam **SPT Name: EBA-BH01**

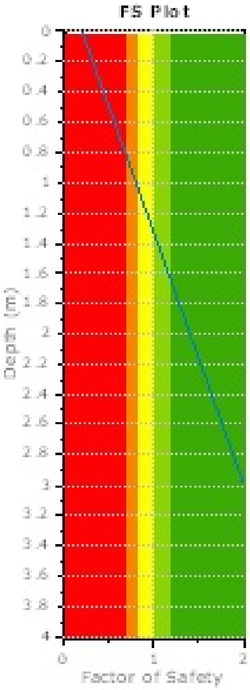
Location : Stocking Lake Dam

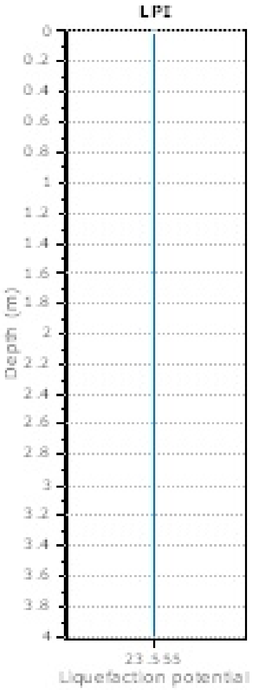
:: Input parameters and analysis properties ::

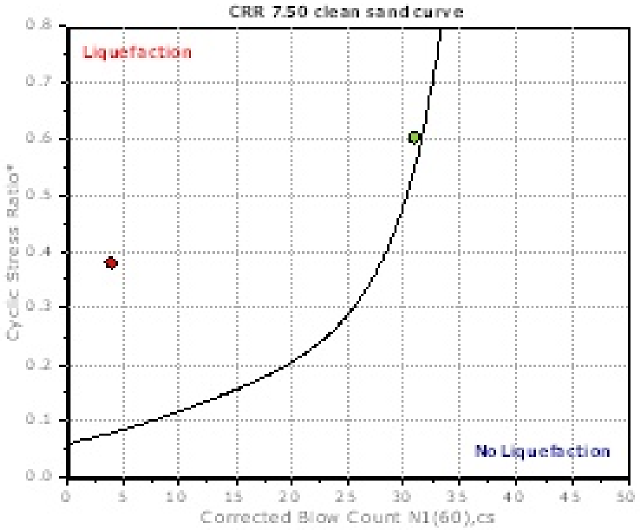
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.26 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

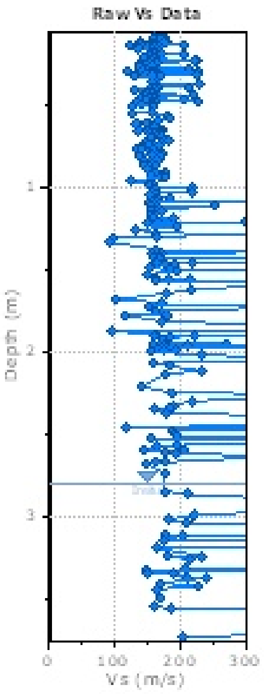
V_s BASED LIQUEFACTION ANALYSIS REPORT

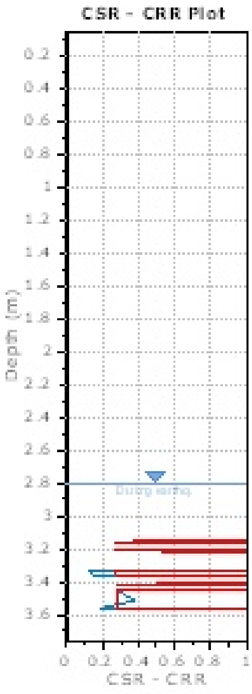
Project title : Stocking Lake Dam **V_s Name: MASW18-01**

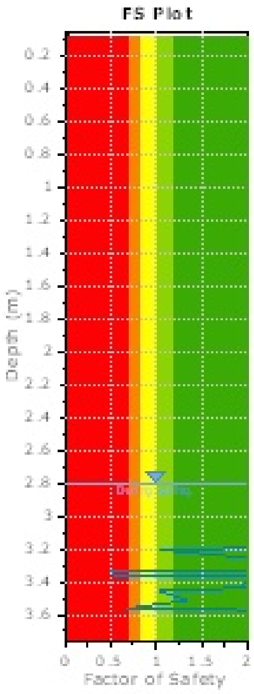
Location : Stocking Lake Dam

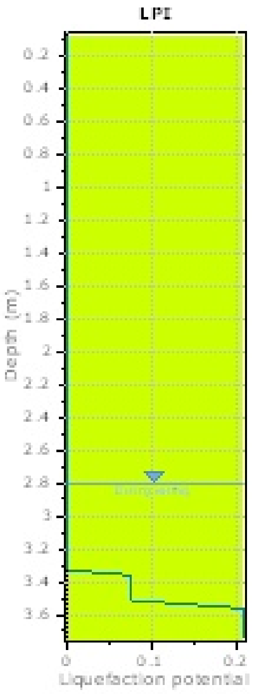
:: Input parameters and analysis properties ::

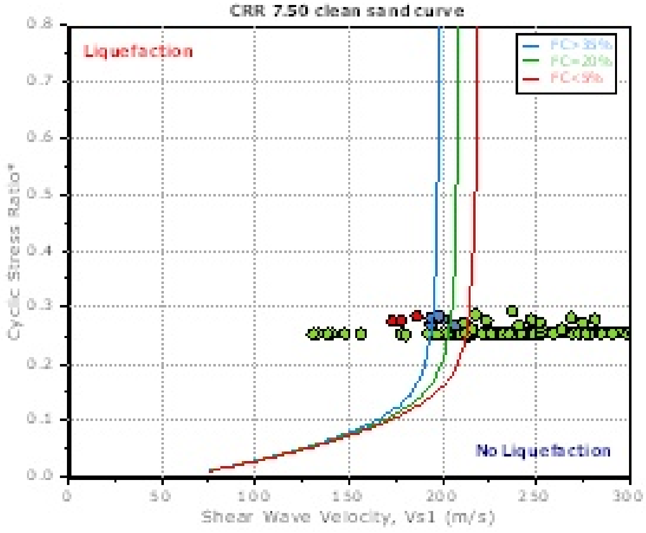
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.80 m
G.W.T. (earthq.):	2.80 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.26 g
Eq. external load:	0.00 kPa











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

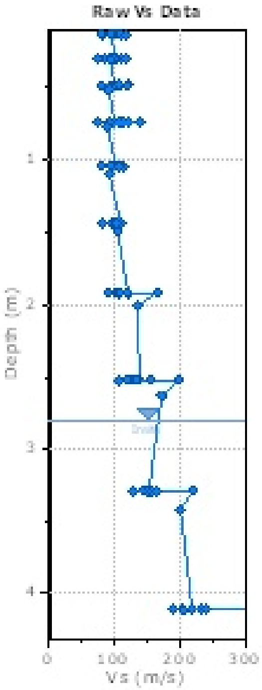
V_s BASED LIQUEFACTION ANALYSIS REPORT

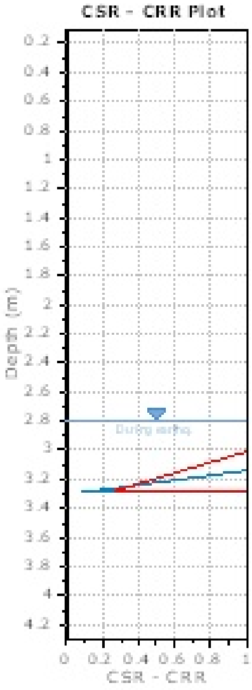
Project title : Stocking Lake Dam **V_s Name: MASW18-02**

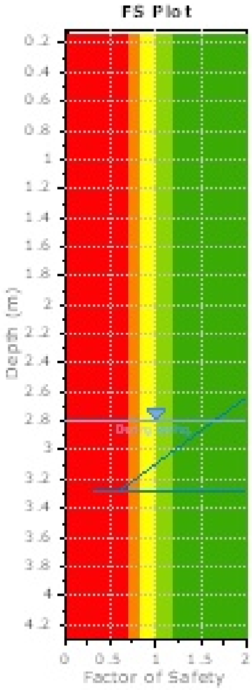
Location : Stocking Lake Dam

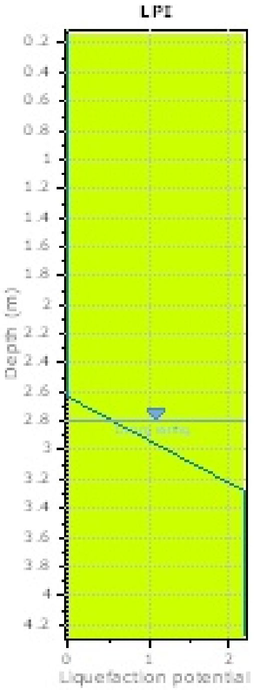
:: Input parameters and analysis properties ::

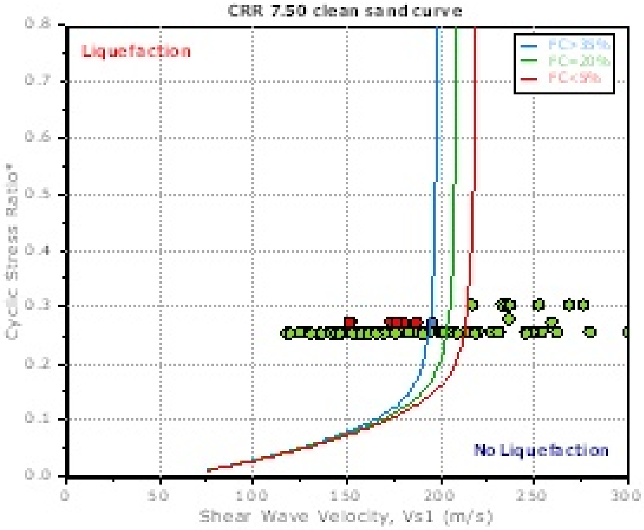
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.80 m
G.W.T. (earthq.):	2.80 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.26 g
Eq. external load:	0.00 kPa











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LPI color scheme

- 🟥 Very high risk
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- 🟨 Low risk

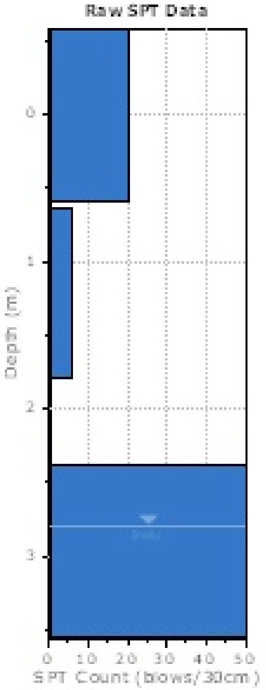
SPT BASED LIQUEFACTION ANALYSIS REPORT

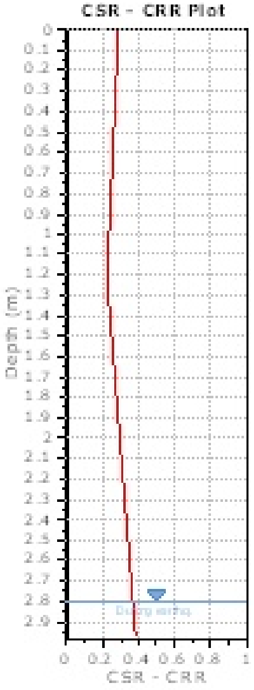
Project title : Stocking Lake Dam **SPT Name: BH18-01**

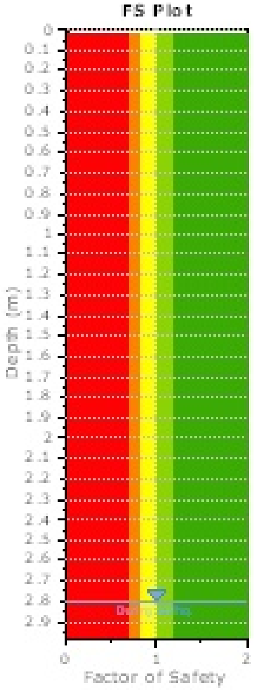
Location : Stocking Lake Dam

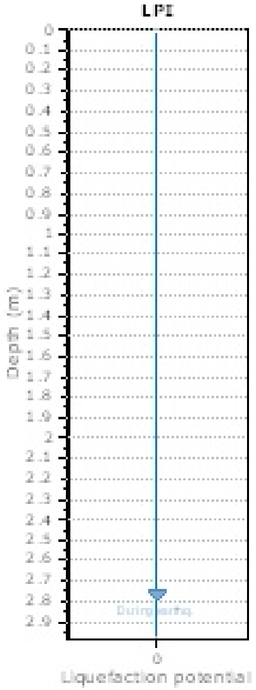
:: Input parameters and analysis properties ::

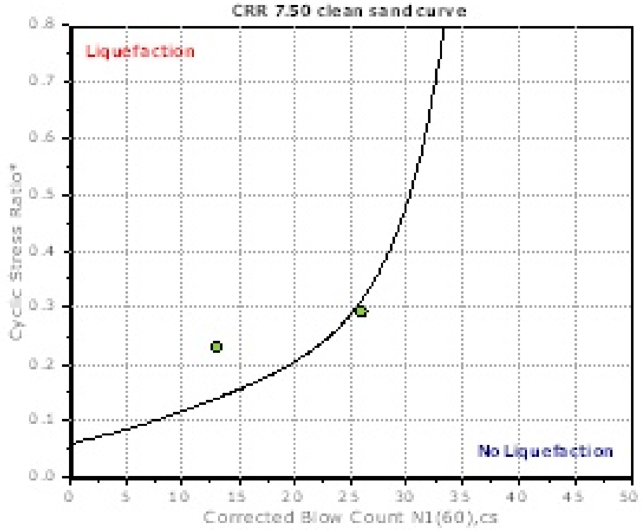
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











F.S. color scheme

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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

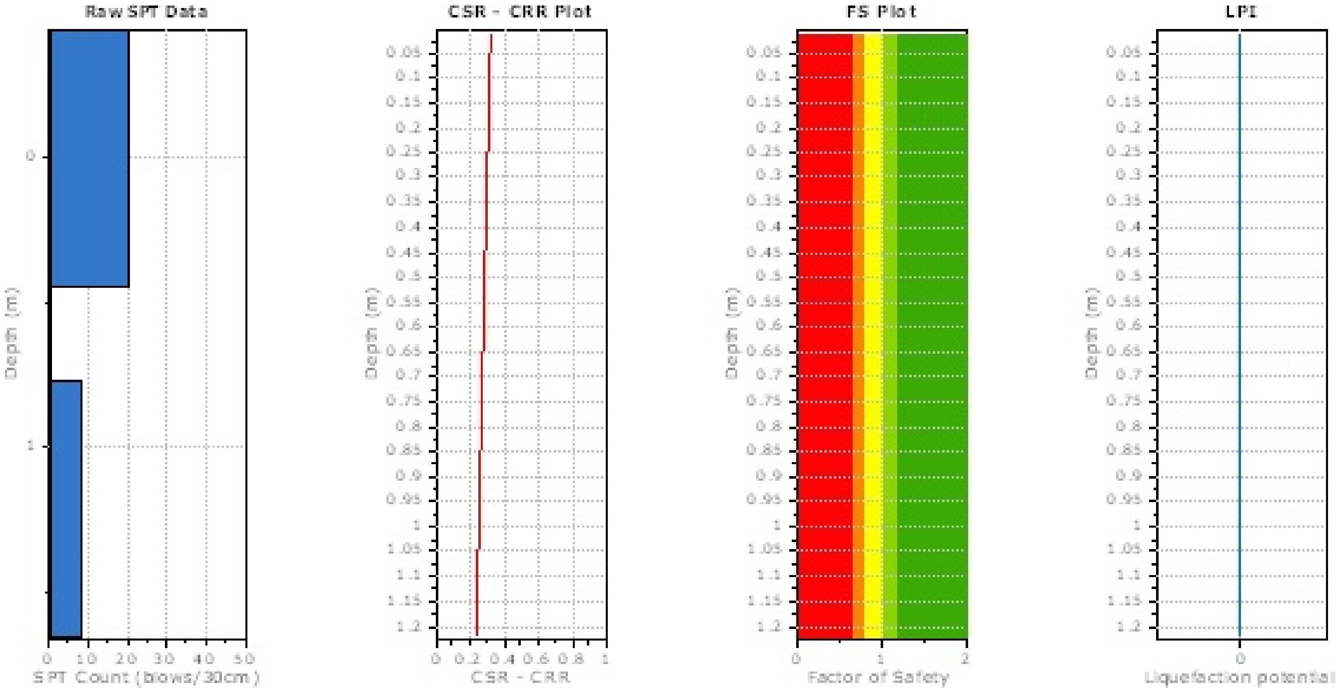
SPT BASED LIQUEFACTION ANALYSIS REPORT

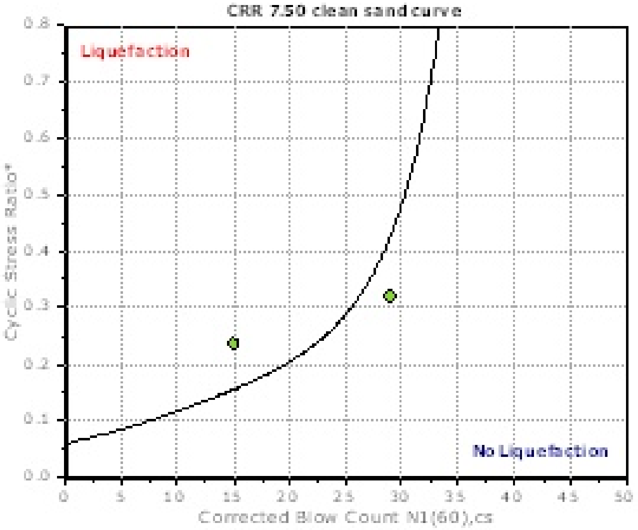
Project title : Stocking Lake Dam **SPT Name: BH18-02**

Location : Stocking Lake Dam

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		





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LPI color scheme

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- 🟨 Low risk

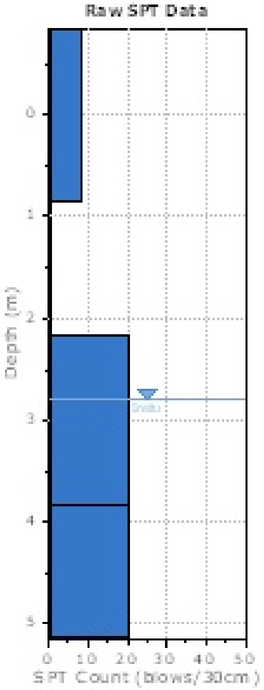
SPT BASED LIQUEFACTION ANALYSIS REPORT

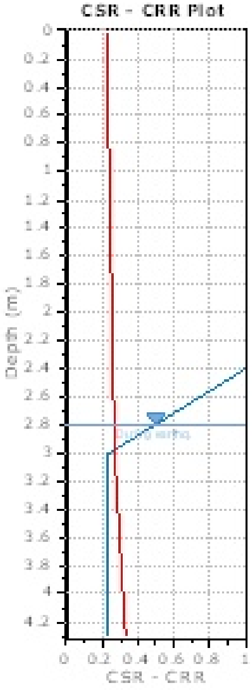
Project title : Stocking Lake Dam **SPT Name: EBA-BH02**

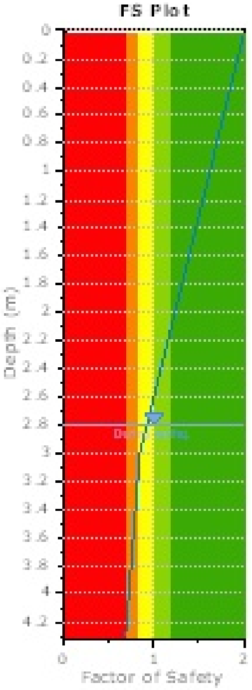
Location : Stocking Lake Dam

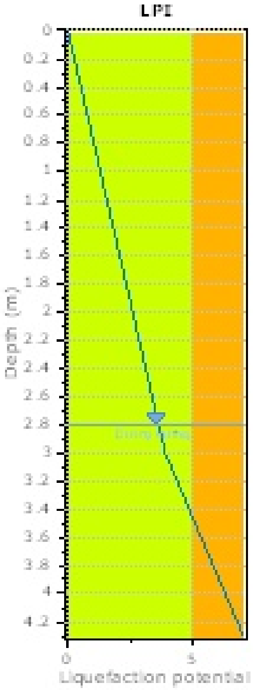
:: Input parameters and analysis properties ::

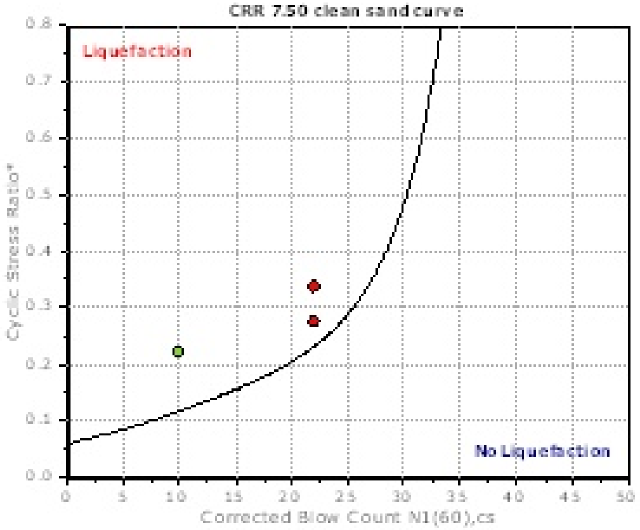
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

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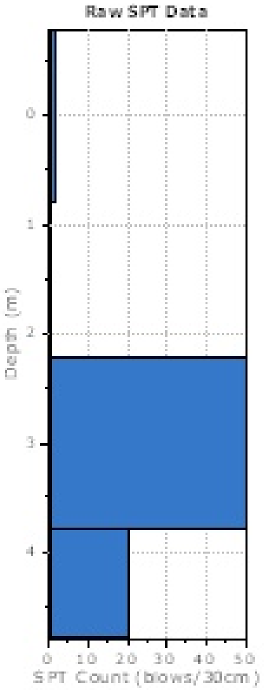
SPT BASED LIQUEFACTION ANALYSIS REPORT

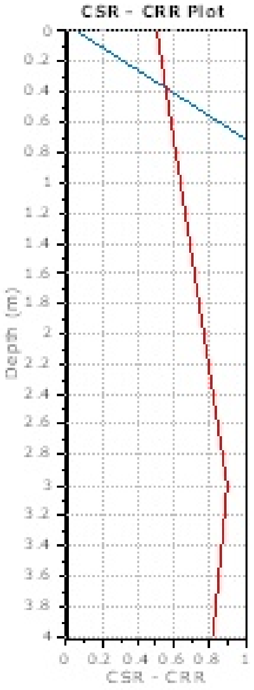
Project title : Stocking Lake Dam **SPT Name: EBA-BH01**

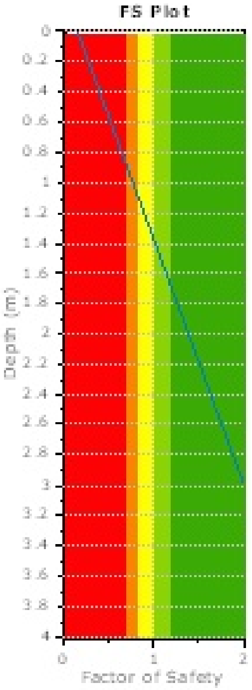
Location : Stocking Lake Dam

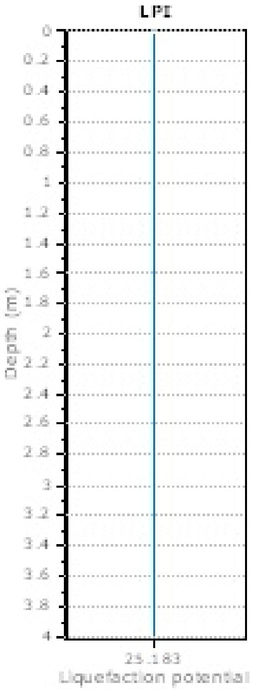
:: Input parameters and analysis properties ::

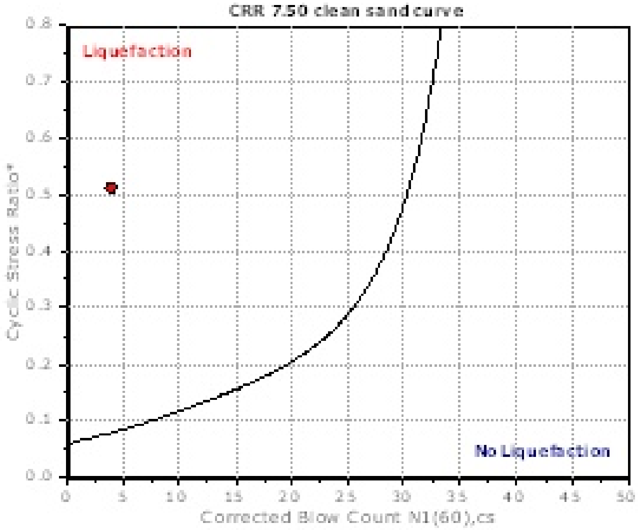
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.35 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

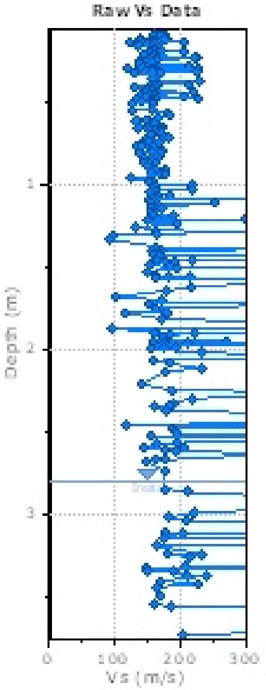
V_s BASED LIQUEFACTION ANALYSIS REPORT

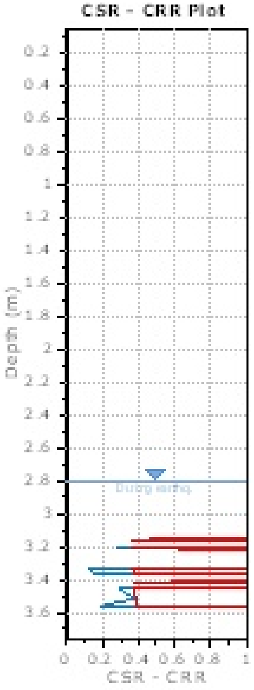
Project title : Stocking Lake Dam **V_s Name: MASW18-01**

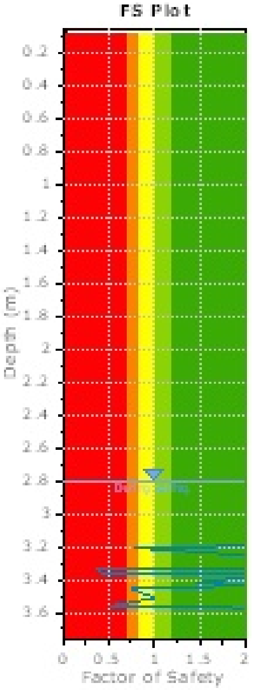
Location : Stocking Lake Dam

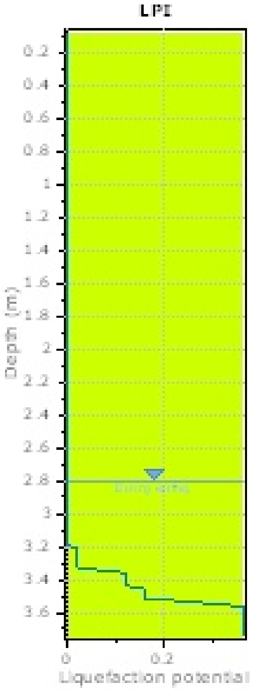
:: Input parameters and analysis properties ::

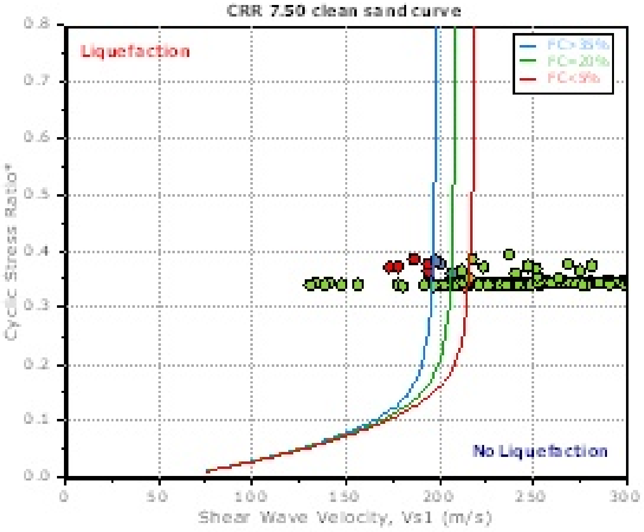
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.80 m
G.W.T. (earthq.):	2.80 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.35 g
Eq. external load:	0.00 kPa











F.S. color scheme

- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
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LPI color scheme

- 🟥 Very high risk
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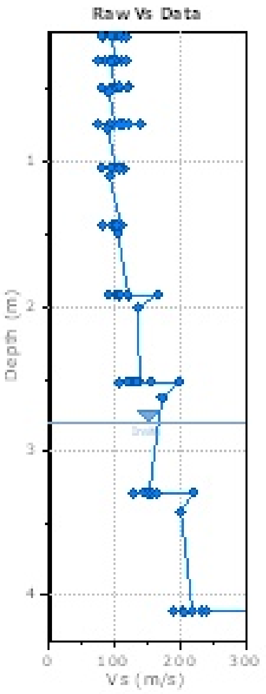
V_s BASED LIQUEFACTION ANALYSIS REPORT

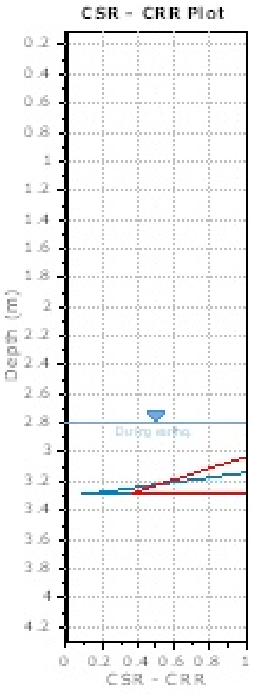
Project title : Stocking Lake Dam **V_s Name: MASW18-02**

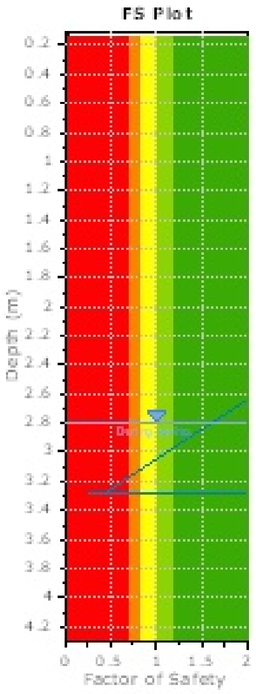
Location : Stocking Lake Dam

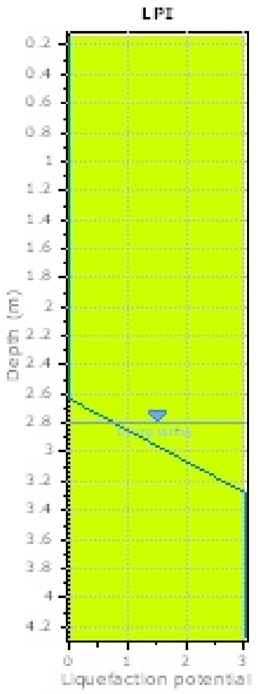
:: Input parameters and analysis properties ::

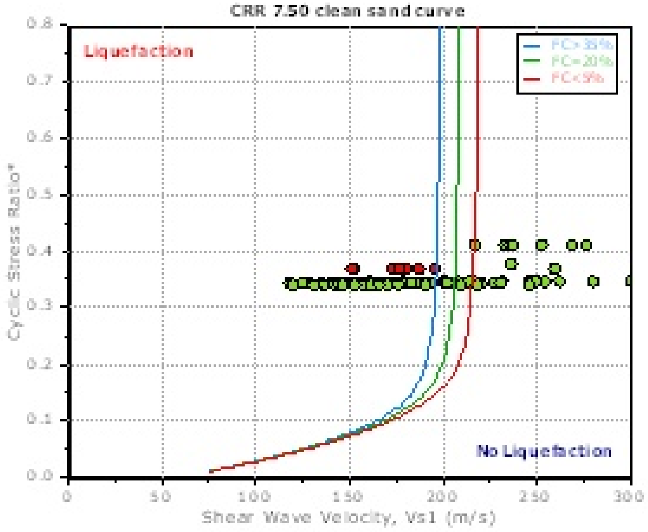
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.80 m
G.W.T. (earthq.):	2.80 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.35 g
Eq. external load:	0.00 kPa











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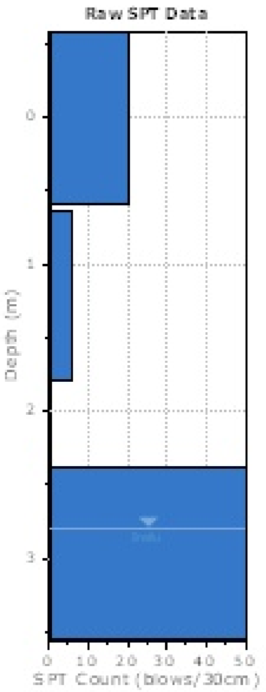
SPT BASED LIQUEFACTION ANALYSIS REPORT

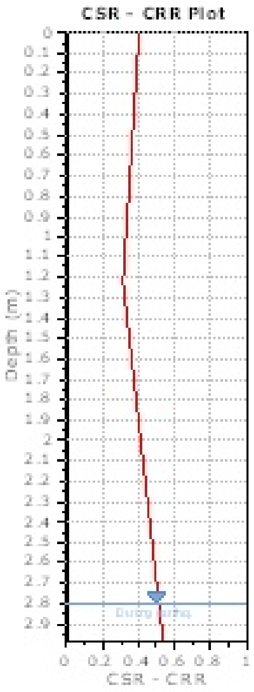
Project title : Stocking Lake Dam **SPT Name: BH18-01**

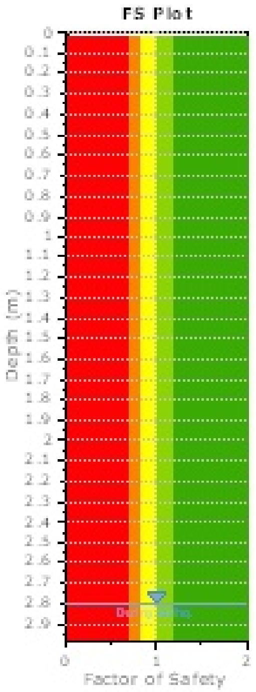
Location : Stocking Lake Dam

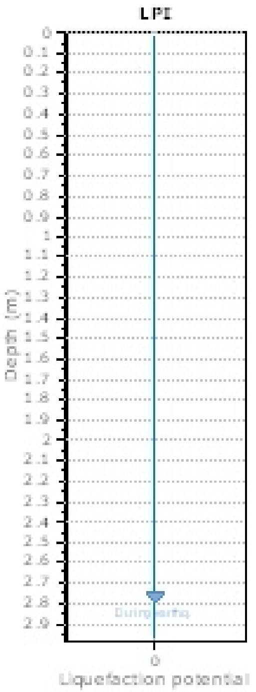
:: Input parameters and analysis properties ::

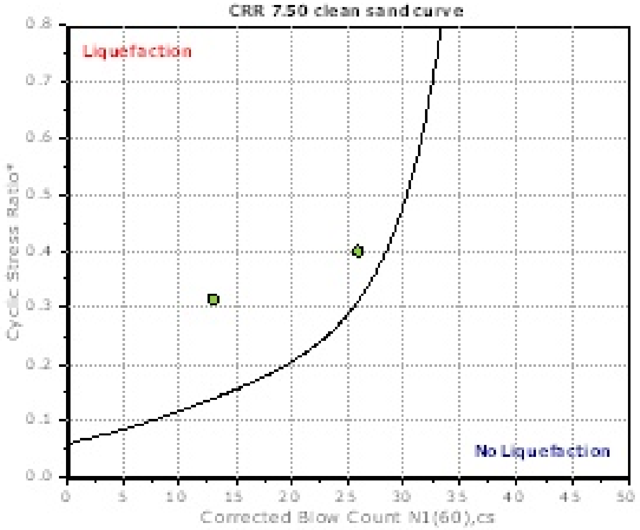
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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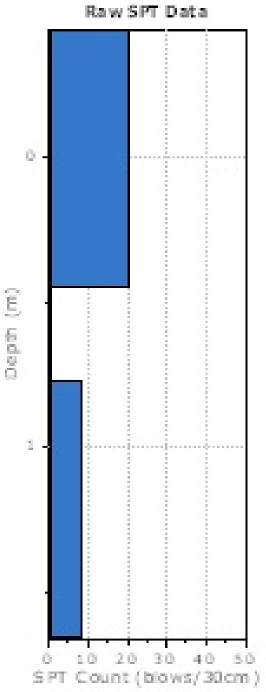
SPT BASED LIQUEFACTION ANALYSIS REPORT

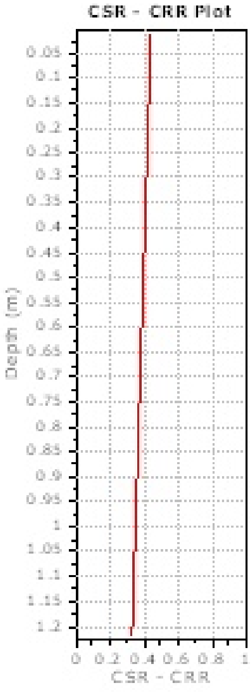
Project title : Stocking Lake Dam **SPT Name: BH18-02**

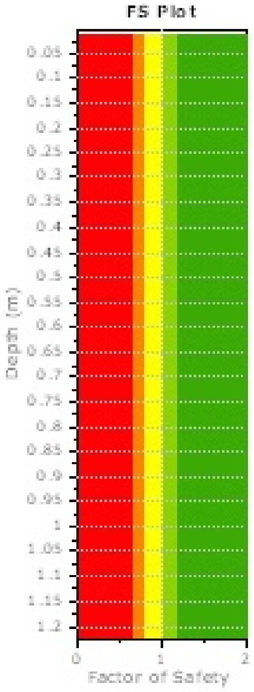
Location : Stocking Lake Dam

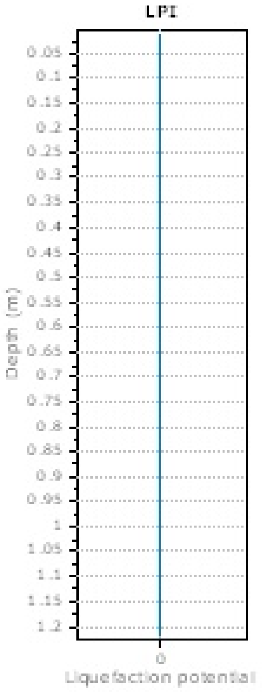
:: Input parameters and analysis properties ::

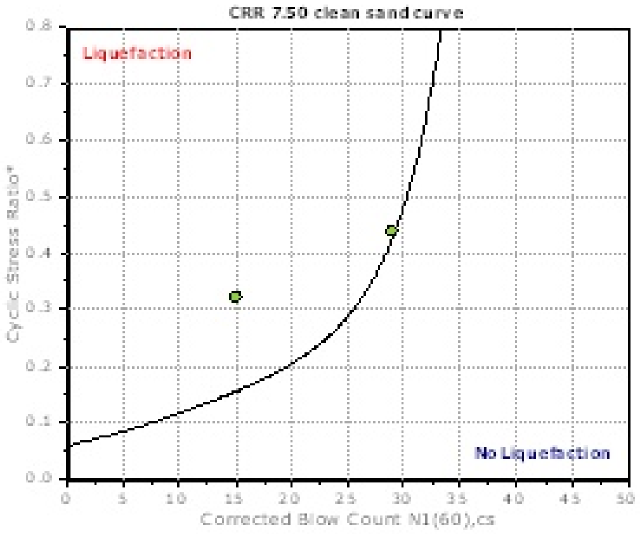
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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LPI color scheme

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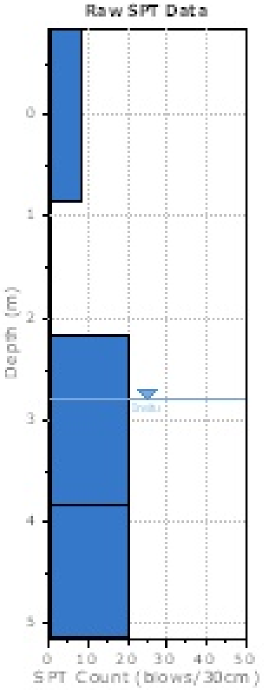
SPT BASED LIQUEFACTION ANALYSIS REPORT

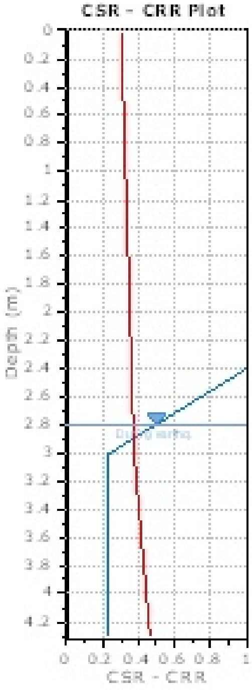
Project title : Stocking Lake Dam **SPT Name: EBA-BH02**

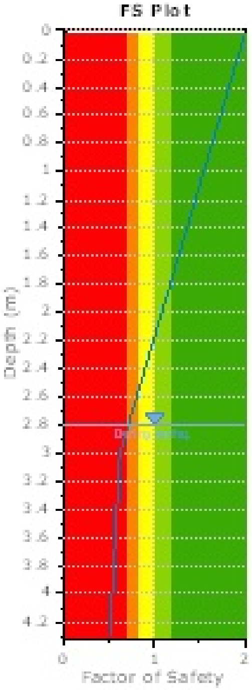
Location : Stocking Lake Dam

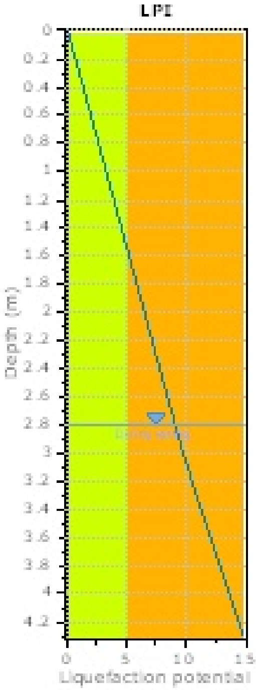
:: Input parameters and analysis properties ::

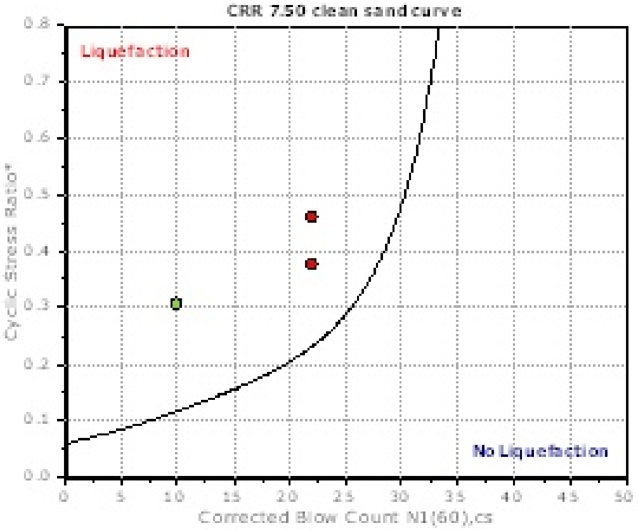
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	2.80 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	2.80 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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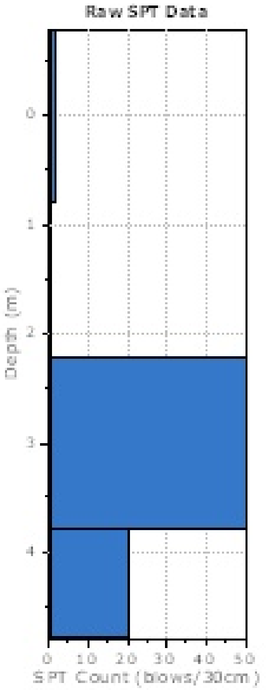
SPT BASED LIQUEFACTION ANALYSIS REPORT

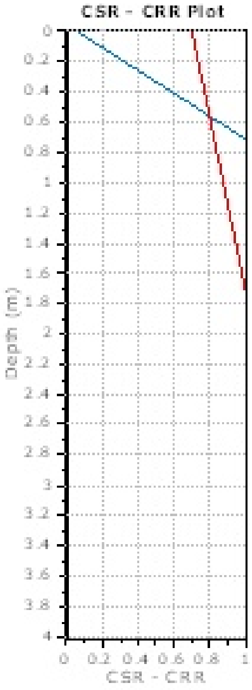
Project title : Stocking Lake Dam **SPT Name: EBA-BH01**

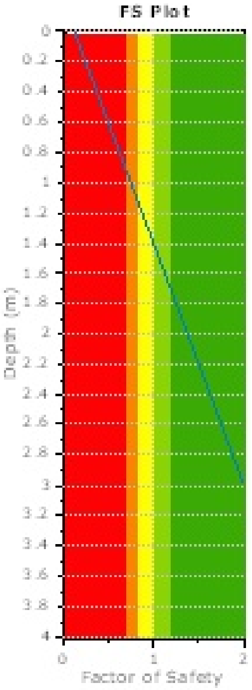
Location : Stocking Lake Dam

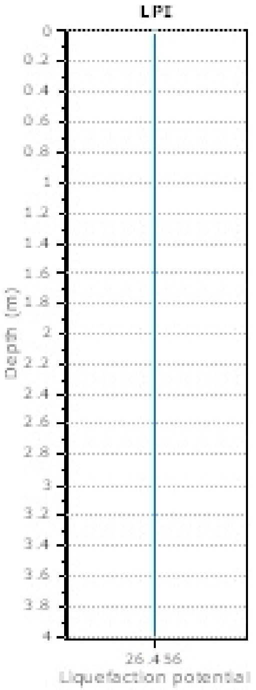
:: Input parameters and analysis properties ::

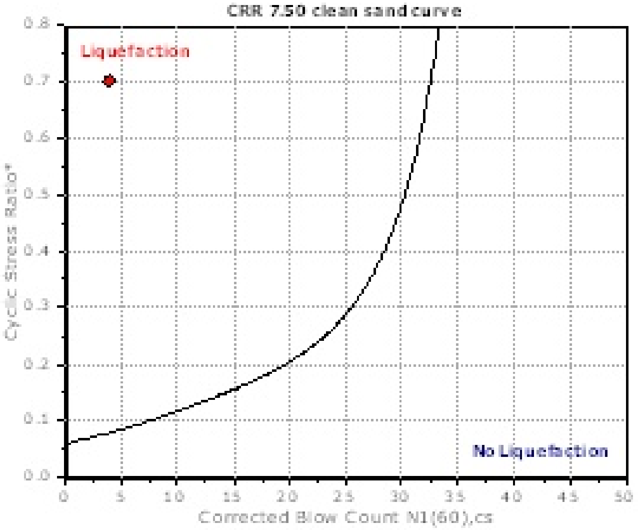
Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	0.00 m
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	0.00 m
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	8.83
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.48 g
Rod length:	1.00 m	Eq. external load:	0.00 kPa
Hammer energy ratio:	1.00		











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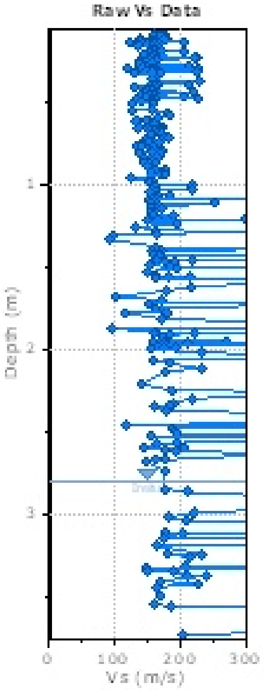
V_s BASED LIQUEFACTION ANALYSIS REPORT

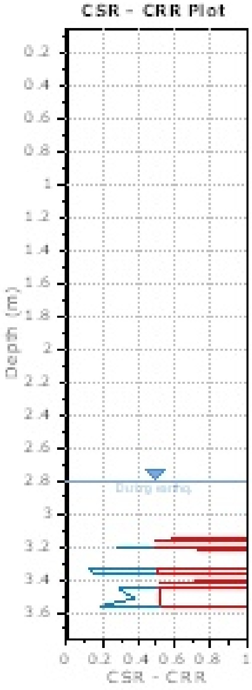
Project title : Stocking Lake Dam **V_s Name: MASW18-01**

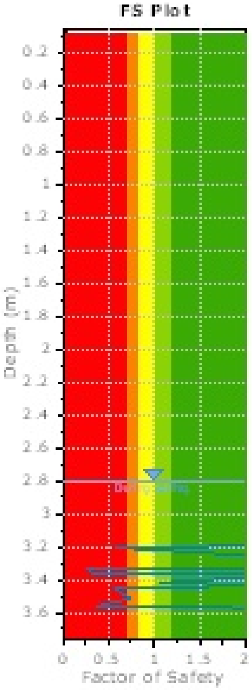
Location : Stocking Lake Dam

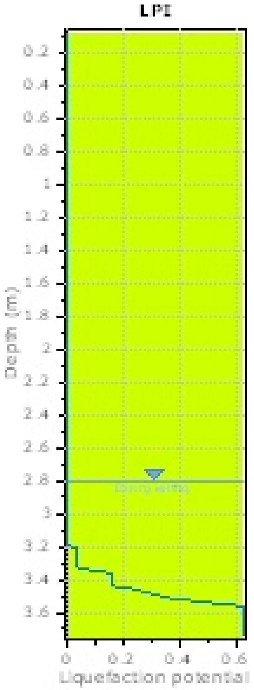
:: Input parameters and analysis properties ::

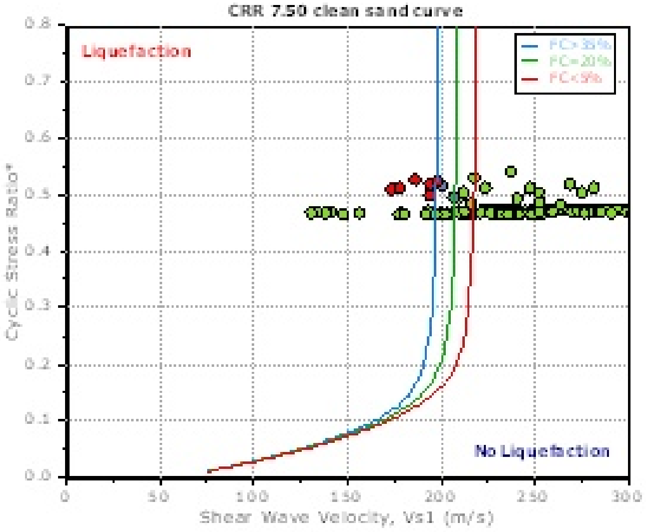
Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.80 m
G.W.T. (earthq.):	2.80 m
Earthquake magnitude M_w:	8.83
Peak ground acceleration:	0.48 g
Eq. external load:	0.00 kPa











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Ecora

V_s BASED LIQUEFACTION ANALYSIS REPORT

Project title : Stocking Lake Dam **V_s Name: MASW18-02**

Location : Stocking Lake Dam

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998 (Youd et al. 2001)
G.W.T. (in-situ):	2.80 m
G.W.T. (earthq.):	2.80 m
Earthquake magnitude M_w:	8.83
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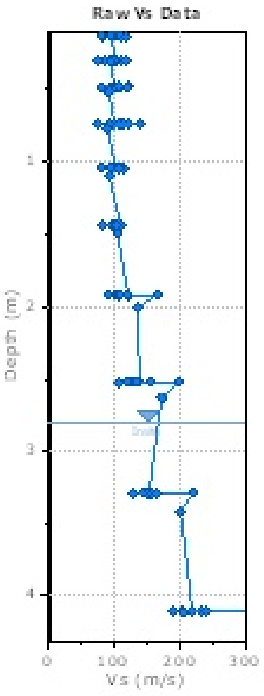

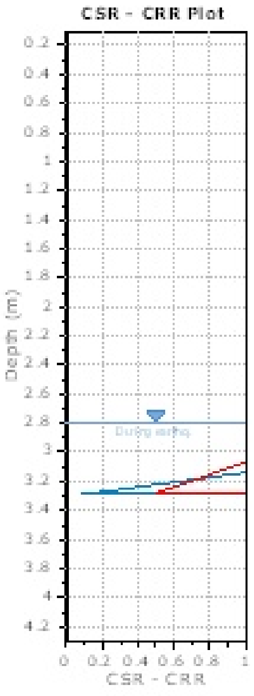

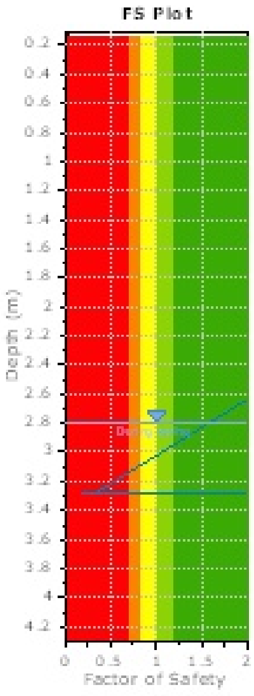

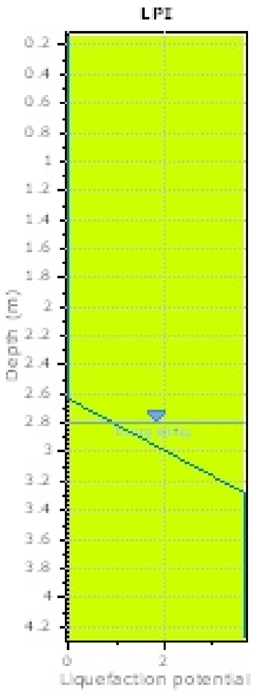

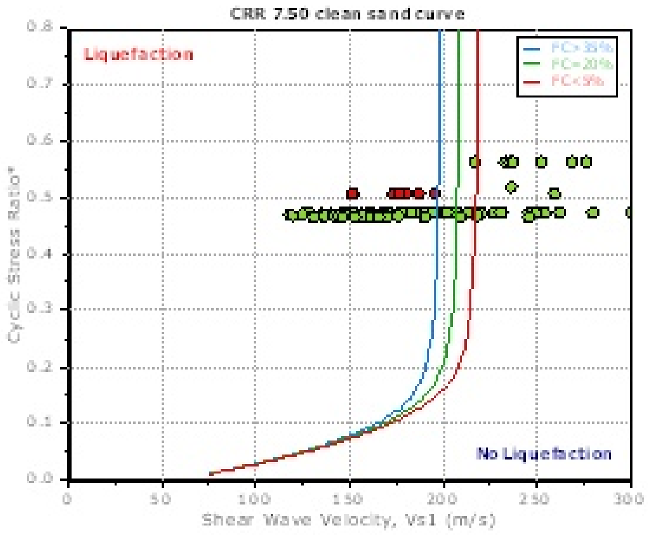

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Appendix G

UNSW Piping Failure Risk Assessment

1. UNSW Piping Failure Risk Assessment

The UNSW method of assessing the probability of piping failure for dams involves the following steps:

- Assess the average annual frequencies of failure for embankment piping (P_e) foundation piping (P_f) and piping of the embankment into foundation (P_{ef}). This includes consideration of whether the dam is greater than or less than 5 years in age as 2/3 of piping failures have been found to occur in the first five years following first filling.
- Calculate weighting factors for each of the aforementioned piping failure modes (w_E, $w_{F,}$ and w_{EF}) which take into account dam characteristics such as core properties, compaction, and foundation geology and past performance of the dam. The weighting factors are the product of a series of weighting factors for each particular characteristic of the dam or foundation.
- Calculate the annual likelihood of failure by piping (P_p) using the following formula:

$$\mathbf{P_p = P_e \times w_E + P_f \times w_F + P_{ef} \times w_{EF}}$$

A drawback of the UNSW method is that is based on a retrospective study, which tends to lump together the factors, that influence the initiation and progression of piping, and breach formation; for historic failures and dam safety incidents (an event where the integrity of the dam has been compromised but failure has not occurred) documented in the ICOLD database of dam failures. As such, it is not possible to specifically isolate the influence of each factor. Another key consideration is the inherent assumption that the Stocking Lake Dam will have enough similar characteristics to the population of dams within the database and that the findings of the database review are statistically relevant for the purposes of this assessment.

Based on the design information available, Ecora has assumed the following zoning categories as defined in Table 1 from the Foster et al. (2000b) study:

- Homogenous earthfill

The database figures for after the first 5 years of operation were selected due to the age of the dam.

The average annual probability of failure presented for Stocking Lake Dam was selected from the Foster et al. (2000b) study and the weighting factors were calculated using the descriptors presented in the same paper. The tabulated weighting factors are in the Tables presented below.

Table 1 Calculation of Annual Likelihood of Piping Failure

Piping Failure Mode	Zoning Category	Average Annual Probability of Failure			Overall Weighting Facture		Weighted Likelihood of Piping Failure		
Piping through embankment (P_e)	Homogenous earthfill	P_e =	1.9	x 10^{-4}	w_E =	72.00	P_e x w_E =	136.80	x 10^{-4}
Piping through the foundation (P_f)		P_f =	0.19	x 10^{-4}	w_F =	14.40	P_f x w_F =	2.74	x 10^{-4}
Piping from embankment into foundation (P_{ef})		P_{ef} =	0.04	x 10^{-4}	w_{EF} =	1.27	$_{Pef}$ x w_{EF} =	0.05	x 10^{-4}
Annual Likelihood of Piping Failure (P_p)							**P_p=**	**139.59**	**x 10^{-4}**

Table 1.1 Weighing Factors for Piping through the Embankment Mode of Failure - Calculation of w_E

Factor	Stocking Lake Dam	Weighting	Comment
Embankment Filters	No filter	2.0	Historical drawings indicate that there is no filter in the dam
Core geological origin	Glacial	0.5	Foundation soils silty till deposits
Core soil type	Silty sands	1.2	Embankment soils silty sands
Compaction	No formal compaction	5.0	Historical drawings do not indicate compaction was undertaken
Conduits	Conduit through embankment, many poor details	5.0	Outlet pipe installed in trench beneath embankment, unknown size, material or condition
Foundation treatment	Irregularities in foundation or abutment, steep abutments	1.2	Steep abutments, trench beneath embankment
Observations of seepage	Leakage gradually increasing at toe of downstream, clear	2.0	Seepage located at the left abutment toe, clear
Monitoring and surveillance	Inspections weekly	1.0	Dam is inspected weekly by ToL staff, weather permitting
w_E, product of individual weighting factors		**72.00**	

Table 1.2 Weighting Factors for Piping through the Foundation Mode of Failure - Calculation of w_F

Factor	Stocking Lake Dam	Weighting	Comment
Filters	No foundation filter present when required	1.2	Historical drawings indicate that there is no filter in the dam
Foundation below cut off	Soil foundation	5.0	Foundation soils silty till deposits
Cutoff (soil foundation)	Shallow or no cut-off trench	1.2	Historical drawings do not indicate a cut-off trench
Soil geology, below cutoff	Glacial	0.5	Foundation soils silty till deposits
Observations of seepage	Leakage gradually increasing at toe of downstream, clear	2.0	Seepage located at the left abutment toe, clear
Observations of pore pressures	Gradually increasing pressures in foundation	2.0	Seepage located at the left abutment toe increasing
Monitoring and surveillance	Inspections weekly	1.0	Dam is inspected weekly by ToL staff, weather permitting
w_F, product of individual weighting factors		**14.40**	

Table 1.3 Weighting Factors for Piping from the Embankment into the Foundation Mode of Failure – Calculations of w_{EF}

Factor	Stocking Lake Dam	Weighting	Comment
Filters	Mode of failure independent of filters	1.0	
Foundation cut off trench	Shallow or none	0.8	Historical drawings do not indicate a cut-off trench
Foundation	Founding on or partly on soil foundations.	0.5	Foundation soils silty till deposits
Erosion control measures of foundation	No erosion-control, average foundation conditions	1.2	None provided for in design, foundation soils silty till deposits with trench for the outlet pipe through dam foundation
Grouting	Soil foundation only, not applicable	1.0	
Soil geology types	Glacial	2.0	Foundation soils silty till deposits
Core geological origin	Glacial	0.5	Embankment fill comprises reworked natural till deposits
Core soil type	Silty sands	1.2	Embankment soils silty sands
Core compaction	Mode of failure independent of compaction	1.0	
Foundation treatment	Irregularities in foundation or abutment, steep abutments	1.1	Steep abutments, trench beneath embankment
Observations of seepage	Leakage gradually increasing at toe of downstream, clear	2.0	Seepage located at the left abutment toe, clear
Monitoring and surveillance	Inspections weekly	1.0	Dam is inspected weekly by ToL staff, weather permitting
w_{EF}, product of individual weighting factors		**1.27**	

A method for assessing the relative likelihood of failure of embankment dams by piping

Mark Foster, Robin Fell, and Matt Spannagle

Abstract: A method for estimating the relative likelihood of failure of embankment dams by piping, the University of New South Wales (UNSW) method, is based on an analysis of historic failures and accidents in embankment dams. The likelihood of failure of a dam by piping is estimated by adjusting the historical frequency of piping failure by weighting factors which take into account the dam zoning, filters, age of the dam, core soil types, compaction, foundation geology, dam performance, and monitoring and surveillance. The method is intended only for preliminary assessments, as a ranking method for portfolio risk assessments, to identify dams to prioritise for more detailed studies, and as a check on event-tree methods. Information about the time interval in which piping failure developed and the warning signs which were observed suggest that the piping process often develops rapidly, giving little time for remedial action. In the piping accidents, the piping process reached some limiting condition allowing sufficient time to draw down the reservoir or carry out remedial works to prevent breaching.

Key words: dams, failures, risk, probability, piping.

Résumé : Une méthode pour évaluer la probabilité relative de rupture de barrages en terre par formation de renard, la méthode UNSW, est basée sur une analyse de l’histoire des ruptures et des accidents dans les barrages en terre. La probabilité de rupture d’un barrage par formation de renard est estimée en ajustant la fréquence historique de rupture par renard au moyen de facteurs de pondération qui prennent en compte le zonage du barrage, les filtres, l’âge du barrage, les types de sol dans le noyau, le compactage, la géologie de la fondation, la performance du barrage, et les mesures et la surveillance. La méthode est destinée à réaliser seulement des évaluations préliminaires, comme une méthode de classement pour un portfolio de classement d’évaluations de risques, pour identifier les barrages auxquels une priorité doit être accordée pour des études détaillées, et comme une vérification pour les méthode de représentation en arbre des événements. L’information sur l’intervalle de temps durant lequel la rupture par renard s’est développée et les signes d’alerte ont été observés suggère que le processus de renard se développe souvent rapidement, laissant peu de temps pour les interventions de confortement. Dans les accidents de renards, le processus de renard atteint une certaine condition limite laissant suffisamment de temps pour la vidange du réservoir ou pour réaliser les travaux de confortement afin d’éviter la formation d’une brèche.

Mots clés : barrages, ruptures, risque, probabilité, renard.

[Traduit par la Rédaction]

Introduction

Internal erosion and piping are a significant cause of failure and accidents affecting embankment dams. For large dams, up to 1986, the failure statistics are as follows (Foster et al. 1998, 2000; Foster 1999):

Mode of failure	% of total failures
Piping through embankment	31
Piping through foundation	15
Piping from embankment to foundation	2
Slope instability	4
Overtopping	46
Earthquake	2

Hence, about half of all failures are due to piping. About 42% of these failures occur on first filling, and 66% on first filling and within the first 5 years of operation, but there is an ongoing piping hazard. This has been recognised by many dam authorities when assessing the safety of their existing dams.

Traditionally, the assessment of safety against piping has been based on the zoning of the dam, the nature of filters (if present), the quality of construction of the dam, the foundation conditions, and the performance of the dam (e.g., seepage flow rates, evidence of piping). This requires a degree of judgement, and is sometimes difficult. As a result in many cases, engineers carrying out dam safety assessments have concentrated more on those aspects which they can more readily quantify, e.g., risk of flooding, slope failure, and

Received February 5, 1999. Accepted February 10, 2000. Published on the NRC Research Press website on October 10, 2000.

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Table 1. Average historic frequency of failure of embankment dams by mode of failure and dam zoning.

	Embankment			Foundation			Embankment into foundation		
		Average annual P_e ($\times10^{-6}$)			Average annual P_f ($\times10^{-6}$)			Average annual P_{ef} ($\times10^{-6}$)	
Zoning category	Average P_{Te} ($\times10^{-3}$)	First 5 years operation	After 5 years operation	Average P_{Tf} ($\times10^{-3}$)	First 5 years operation	After 5 years operation	Average P_{Tef} ($\times10^{-3}$)	First 5 years operation	After 5 years operation
Homogeneous earthfill	16	2080	190	1.7	255	19	0.18	19	4
Earthfill with filter	1.5	190	37	1.7	255	19	0.18	19	4
Earthfill with rock toe	8.9	1160	160	1.7	255	19	0.18	19	4
Zoned earthfill	1.2	160	25	1.7	255	19	0.18	19	4
Zoned earth and rockfill	1.2	150	24	1.7	255	19	0.18	19	4
Central core earth and rockfill	(<1)	(<140)	(<34)	1.7	255	19	0.18	19	4
Concrete face earthfill	5.3	690	75	1.7	255	19	0.18	19	4
Concrete face rockfill	(<1)	(<130)	(<17)	1.7	255	19	0.18	19	4
Puddle core earthfill	9.3	1200	38	1.7	255	19	0.18	19	4
Earthfill with core wall	(<1)	(<130)	(<8)	1.7	255	19	0.18	19	4
Rockfill with core wall	(<1)	(<130)	(<13)	1.7	255	19	0.18	19	4
Hydraulic fill	(<1)	(<130)	(<5)	1.7	255	19	0.18	19	4
All dams	3.5	450	56	1.7	255	19	0.18	19	4

Note: P_{Te}, P_{Tf}, and P_{Tef} are the average frequencies of failure over the life of the dam; P_e, P_f, and P_{ef} are the average annual frequencies of failure. Values in parentheses are based on an assumption of <1 failure.

earthquake. In recent years, some organisations have been using quantitative risk assessment (QRA) techniques to assist in dam safety management, including BC Hydro, Canada; U.S. Bureau of Reclamation (USBR), United States; Norwegian Geotechnical Institute, Norway; and several Australian dam authorities. In some cases, the probability of failure due to piping has been included in the assessment. Some examples are described in Johansen et al. (1997) and Landon-Jones et al. (1996). These use event-tree methods, which require assessments of the probability of initiation, progression to form a pipe, and development of a breach. Unless the dam is one of a population of similar dams (such as the earthfill and rockfill dams in Johansen et al. 1997), where there is a good history of performance, including some accidents, it is very difficult to assign probabilities. Usually an “expert panel” approach is used, but the experts have little to base their judgements on. Others, such as the USBR and some of the assessments of groups (portfolios) of dams in Australia, have used the historic average failure frequencies for piping obtained from ICOLD (1983) and adjusted to take account of the characteristics and performance of the dam. These have lumped the three piping modes together, and the factors used to assess whether a dam was more or less likely to fail were listed, but no guidance was given on relative or absolute weightings.

As part of a research project which is developing methods to assess the probability of failure of dams for use in QRA, we have carried out a detailed statistical analysis of failures and accidents affecting embankment dams and the influencing factors (Foster et al. 1998, 2000). This paper takes the results of that analysis, broadly quantifies the influence of each factor affecting the likelihood of piping, and presents a method of estimating the relative likelihood of failure of all types of embankment dams by piping. The results are expressed in terms of likelihood, meaning a qualitative measure of probability. We do not represent that the results are absolute estimates of probabilities.

The paper also includes information about the time interval in which piping failures have developed and the warning signs which were evident before failures. This information can be used to aid in estimating the likely warning time, which might allow intervention to prevent failure or allow evacuation of persons downstream before the failure. This paper should be read with Foster et al. (2000) so the basis for the method can be understood.

Overview of the method

The method, referred to here as the University of New South Wales (UNSW) method, is based on the assumption that it is reasonable to make estimates of the relative likelihood of failure of embankment dams by piping from the historic frequency of failures. This is done using the dam zoning as the primary means of differentiating between dams and the frequencies of failures calculated by Foster et al. (1998, 2000). The historic frequencies of failure by the three modes of piping are adjusted to take account of the characteristics of the dam, such as core properties, compaction, and foundation geology, and to take account of the past performance of the dam. These adjustments are made with the use of weighting factors which are multiplied by the average historical frequencies of failure.

To assess the annual likelihood of failure of an embankment dam by piping, we first determine the average annual frequencies of failure from Table 1 for each of the three modes of piping failure, namely piping through the embankment, piping through the foundation, and piping from the embankment into the foundation. We consider whether the dam is less than or greater than 5 years old (because two

Table 2. Summary of the weighting factors (values in parentheses) for piping through the embankment mode of failure.

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Embankment filters $w_{\mathrm{E(filt)}}$		No embankment filter (for dams that usually have filters; refer to text) (2)	Other dam types (1)	Embankment filter present, poor quality (0.2)	Embankment filter present, well designed, and well constructed (0.02)
Core geological origin $w_{\mathrm{E(cgo)}}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil $w_{\mathrm{E(cst)}}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (0.8)	High-plasticity clays (CH) (0.3)
Compaction $w_{\mathrm{E(cc)}}$	No formal compaction (5)	Rolled, modest control (1.2)	Puddle, hydraulic fill (1.0)		Rolled, good control (0.5)
Conduits $w_{\mathrm{E(con)}}$	Conduit through the embankment, many poor details (5)	Conduit through the embankment, some poor details (2)	Conduit through embankment, typical USBR practice (1.0)	Conduit through embankment, including downstream filters (0.8)	No conduit through the embankment (0.5)
Foundation treatment $w_{\mathrm{E(ft)}}$	Untreated vertical faces or overhangs in core foundation (2)	Irregularities in foundation or abutment, steep abutments (1.2)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{\mathrm{E(obs)}}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, seepage emerging on downstream slope (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Monitoring and surveillance $w_{\mathrm{E(mon)}}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequencies of failure by piping through the embankment depending on zoning type.

thirds of piping failures occur on first filling or in the first 5 years of operation).

We then calculate the weighting factors w_{E}, w_{F}, and w_{EF} from Tables 2, 3, and 4, respectively, to take account of the characteristics of the dam, such as core properties, compaction, and foundation geology, and to take account of the past performance of the dam. The weighting factors are obtained by multiplying the individual weighting factors from the relevant table. So, for example, $w_{\mathrm{E}} = w_{\mathrm{E(filt)}} \times w_{\mathrm{E(cgo)}} \times w_{\mathrm{E(cst)}} \times w_{\mathrm{E(cc)}} \times w_{\mathrm{E(con)}} \times w_{\mathrm{E(ft)}} \times w_{\mathrm{E(obs)}} \times w_{\mathrm{E(mon)}}$ (weighting factors as defined in Table 2).

We obtain the annual likelihood of failure by piping, P_{p}, by summing the weighted likelihoods of each of the modes:

$$P_{\mathrm{p}} = w_{\mathrm{E}}P_{\mathrm{e}} + w_{\mathrm{F}}P_{\mathrm{f}} + w_{\mathrm{EF}}P_{\mathrm{ef}}$$

If a factor has two or more possible weighting factors that can be selected for a particular dam characteristic, such as different zoning types or different foundation geology types, then the weighting factor with the greater value should be used. This is consistent with the method of analysis that was used to determine the weighting factors, as only the characteristics relevant to the piping incident were included in the analysis.

The UNSW method is intended only for preliminary assessments, as a ranking method for portfolio risk assessments to prioritise dams for more detailed studies, and as a check on event-tree methods. Since the UNSW method is based on a dam-performance database, it tends to lump together the factors which influence the initiation and progression of piping and formation of a breach and it is not possible to assess what influence each of the factors has. We recommend that event-tree methods be used for detailed studies to gain a greater understanding of how each of the factors influences either the initiation or progression of piping or the formation of a breach.

The user of the UNSW method is cautioned against varying the weighting factors significantly, as they have been calibrated to the population of dams so that the net effect when applied to the population is neutral.

The length of the dam is not included in the assessment of the probability of failure using the UNSW method.

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Table 3. Summary of weighting factors (values in parentheses) for piping through the foundation mode of failure.

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{\mathrm{F(filt)}}$		No foundation filter present when required (1.2)	No foundation filter (1.0)	Foundation filter(s) present (0.8)	
Foundation (below cutoff) $w_{\mathrm{F(fnd)}}$	Soil foundation (5)		Rock, clay-infilled or open fractures and (or) erodible rock substance (1.0)	Better rock quality ⟶	Rock, closed fractures and non-erodible substance (0.05)
Cutoff (soil foundation) $w_{\mathrm{F(cts)}}$		Shallow or no cutoff trench (1.2)	Partially penetrating sheetpile wall or poorly constructed slurry trench wall (1.0)	Upstream blanket, partially penetrating, well-constructed slurry trench wall (0.8)	Partially penetrating deep cutoff trench (0.7)
Cutoff (rock foundation) $w_{\mathrm{F(ctr)}}$	Sheetpile wall, poorly constructed diaphragm wall (3)	Well-constructed diaphragm wall (1.5)	Average cutoff trench (1.0)	Well-constructed cutoff trench (0.9)	
Soil geology (below cutoff) $w_{\mathrm{F(sg)}}$	Dispersive soils (5); volcanic ash (5)	Residual (1.2)	Aeolian, colluvial, lacustrine, marine (1.0)	Alluvial (0.9)	Glacial (0.5)
Rock geology (below cutoff) $w_{\mathrm{F(rg)}}$	Limestone (5); dolomite (3); saline (gypsum) (5); basalt (3)	Tuff (1.5); rhyolite (2); marble (2); quartzite (2)		Sandstone, shale, siltstone, claystone, mudstone, hornfels (0.7); agglomerate, volcanic breccia (0.8)	Conglomerate (0.5); andesite, gabbro (0.5); granite, gneiss (0.2); schist, phyllite, slate (0.5)
Observations of seepage $w_{\mathrm{F(obs)}}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, sand boils (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Observations of pore pressures $w_{\mathrm{F(obp)}}$	Sudden increases in pressures (up to 10)	Gradually increasing pressures in foundation (2)	High pressures measured in foundation (1.0)		Low pore pressures in foundation (0.8)
Monitoring and surveillance $w_{\mathrm{F(mon)}}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping through the foundation depending on zoning type.

Vanmarke (1977) demonstrated that the length of the dam might influence the probability of failure by sliding, as long dams are more likely to have some defect in the dam or foundation that could cause failure. However, for piping this may not be a significant factor, as the piping failures often occurred at conduits passing through the dam or steep abutments which are independent of the length of the dam.

Details of the application of the UNSW method

The weighting factors are represents by w, and the subscripts identify the mode of piping: $w_{\mathrm{E}(x)}$ is piping through the embankment, $w_{\mathrm{F}(x)}$ is piping through the foundation, and $w_{\mathrm{EF}(x)}$ is piping from the embankment into the foundation. The letters in parentheses (i.e., x) are abbreviations identifying the purpose of the weighting factors.

The following sections give details relating to the application of the weighting factors listed in Tables 1–4. More information is given in Foster et al. (1998) and Foster (1999).

Piping through the embankment (Table 2)

Embankment filters $w_{E(filt)}$

The weighting factors for embankment filters, $w_{\mathrm{E(filt)}}$, are only applied to the dams with zoning categories that usually have embankment filters present. These are earthfill with filter, zoned earthfill, zoned earth and rockfill, and central core earth and rockfill dams. If an embankment filter is present, an assessment of the quality of the filter is required and this should include an assessment of the filter retention criteria, e.g., comparison with the criteria given by Sherard and

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Table 4. Summary of weighting factors (values in parentheses) for accidents and failures as a result of piping from the embankment into the foundation.

	General factors influencing likelihood of initiation of piping				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{\mathrm{EF(filt)}}$	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)
Foundation cutoff trench $w_{\mathrm{EF(cot)}}$	Deep and narrow cutoff trench (1.5)		Average cutoff trench width and depth (1.0)	Shallow or no cutoff trench (0.8)	
Foundation $w_{\mathrm{EF(fnd)}}$		Founding on or partly on rock foundations (1.5)			Founding on or partly on soil foundations (0.5)
Erosion-control measures of core foundation $w_{\mathrm{EF(ecm)}}$	No erosion-control measures, open-jointed bedrock, or open-work gravels (up to 5)	No erosion-control measures, average foundation conditions (1.2)	No erosion-control measures, good foundation conditions (1.0)	Erosion-control measures present, poor foundations (0.5)	Good to very good erosion-control measures present and good foundation (0.3–0.1)
Grouting of foundations $w_{\mathrm{EF(gr)}}$		No grouting on rock foundations (1.3)	Soil foundation only, not applicable (1.0)	Rock foundations grouted (0.8)	
Soil geology types $w_{\mathrm{EF(sg)}}$	Colluvial (5)	Glacial (2)		Residual (0.8)	Alluvial, aeolian, lacustrine, marine, volcanic (0.5)
Rock geology types $w_{\mathrm{EF(rg)}}$	Sandstone interbedded with shale or limestone (3); limestone, gypsum (2.5)	Dolomite, tuff, quartzite (1.5); rhyolite, basalt, marble (1.2)	Agglomerate, volcanic breccia (1.0); granite, andesite, gabbro, gneiss (1.0)	Sandstone, conglomerate (0.8); schist, phyllite, slate, hornfels (0.6)	Shale, siltstone, mudstone, claystone, (0.2)
Core geological origin $w_{\mathrm{EF(cgo)}}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil type $w_{\mathrm{EF(cst)}}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (CL) (0.8)	High-plasticity clays (CH) (0.3)
Core compaction $w_{\mathrm{EF(cc)}}$	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)
Foundation treatment $w_{\mathrm{EF(ft)}}$	Untreated vertical faces or overhangs in core foundation (1.5)	Irregularities in foundation or abutment, steep abutments (1.1)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{\mathrm{EF(obs)}}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes (2)	Leakage steady, clear, or not monitored (1.0)	Minor leakage (0.7)	No or very small leakage measured (0.5)
Monitoring and surveillance $w_{\mathrm{EF(mon)}}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping from the embankment into the foundation depending on zoning type.

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Dunnigan (1989). The likelihood of segregation of the filter materials should also be assessed by considering the construction methods used and the grading curves of the filter materials.

Compaction $w_{E(cc)}$

To provide guidance on the application of the UNSW method, the methods of compaction are briefly described as follows: (1) no formal compaction — fill materials in the core were dumped in place, with no compaction, compaction by animal hooves, or compaction by travel of construction equipment only; (2) rolled, modest control — core materials were rolled but with poor control of moisture content (e.g., varying greater than ±2% of optimum water content) and (or) compacted in relatively thick layers; and (3) rolled, good control — core materials were compacted in thin layers, with good control of moisture content within ±2% of optimum water content and greater than 95% of Standard compaction. Hydraulic fill and puddle core dams are assigned $w_{E(cc)} = 1.0$, as their compaction method has already been taken into account by the zoning.

Conduits $w_{E(con)}$

The categories used to describe the degree of detailing incorporated into the design of conduits located through the embankment are described in Table 2. Conduits through the embankment include conduits above the level of the general foundation of the dam and conduits in trenches excavated through the foundation of the dam. Poor details of outlet conduits can include any of the following features: (1) no filter provided at the downstream end of the conduit; (2) outlet conduit located in a deep and narrow trench in soil or erodible rock, particularly with vertical or irregular sides; (3) corrugated metal formwork used for concrete surround, precluding good compaction; (4) poor conduit geometry such as overhangs, circular pipe with no support, poorly designed seepage cutoff collars, or other features that make compaction of the backfill around the conduit difficult; (5) no compaction or poorly compacted backfill; (6) old cast iron or other types of pipes in badly deteriorated condition or of unknown condition; (7) poor joint details, and no water stops or water stops deteriorated; (8) cracks in the outlet conduit, open joints, seepage into conduit; and (9) conduit founded on soil.

Typical USBR practice from 1950 to 1970 for the detailing of conduits includes (USBR 1977) no downstream filter surrounding the outlet conduit; special compaction around the outlet conduit with special materials and hand tampers; outlet conduits typically concrete formed in place with rectangular or horseshoe-shaped sections; concrete cutoff collars spaced at 15 feet (5 m); and trench slopes excavated at 1V:1H.

Foundation treatment $w_{E(ft)}$

The presence and treatment of both small-scale irregularities in the foundation and large-scale changes in abutment profile need to be considered, particularly those which affect most or all of the width of the dam core.

Observations of seepage $w_{E(obs)}$

The observations of seepage should incorporate an assessment of the full performance history of the dam and not just the current condition. Previous piping incidents may give indications of deficiencies in design and construction, and similar conditions may exist elsewhere in the dam. Except for the category of seepage emerging on the downstream slope, all of the other descriptions of leakage in Table 2 are for the seepage flows collected from the drainage systems of the dam or at the lowest part of the dam. The qualitative description of the neutral category “leakage steady, clear, or not observed” is intended to represent the leakage condition that would be expected to be normal (or typical) for the type and size of the dam being considered. The other two descriptions of “minor” leakage and “none or very small” leakage are intended to represent seepage conditions better than those of the typical dam. A higher category could be selected if pore pressures measured in the dam are shown to have sudden fluctuations in pressure or a steady increase in pressure which may tend to indicate active or impending piping conditions. However, this does not necessarily apply the other way, as satisfactory performance of the pore pressures only indicates piping is not occurring at the location of the piezometers. Allowance is made in the UNSW method to apply a value of $w_{E(obs)}$ within the range of 2–10 depending on the nature, severity, and location of any past piping episodes. This assessment should include piping events that may have occurred over the full life of the dam.

Piping through the foundation (Table 3)

Foundation filters $w_{F(filt)}$

There are two categories defined for the cases where no foundation filters are provided. In the worst case, foundation filters are not provided where it would be expected that foundation filters would be required, i.e., for dams constructed on permeable, erodible foundations. These cases are given the highest value of $w_{F(filt)}$, as shown in Table 3. Dams with no foundation filters on low-permeability and non-erodible foundations would not be expected to require foundation filters and so a lower weighting is suggested.

Foundation type (below cutoff) $w_{F(fnd)}$

The three categories of foundation below the “cutoff” of the dam are soil foundations; erodible rock foundations, with erodible materials present such as clay-filled joints or infilled karstic channels; and non-erodible rock foundations. The cutoff is either a cutoff trench or a sheetpile or slurry trench – diaphragm wall. Examples are shown in Fig. 1.

There should be a good basis for selecting the non-erodible rock category for describing a particular dam foundation, given that the weighting for non-erodible rock provides a reduction of 20 times compared with that for erodible rock. Intermediate values may be used.

Foundation cutoff type $w_{F(cts)}$ *and* $w_{F(ctr)}$

The two separate sets of weightings for the foundation cutoff type depend on whether the cutoff is on a soil or a rock foundation. For dams with cutoffs on soil foundations only, the foundation cutoff factors ($w_{F(cts)}$) for soil foundations should be used; for dams with cutoffs on rock foundations only, use $w_{F(ctr)}$. For dams where the cutoff is founded partly on soil foundations and partly on rock foundations (along the longitudinal axis of the dam), then the product of weighting factors of foundation × foundation ×

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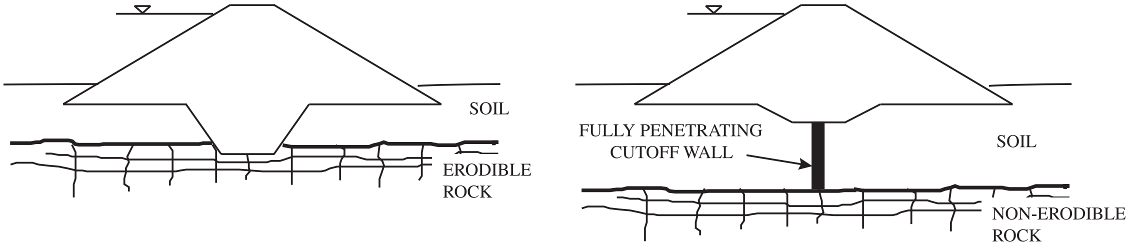
Fig. 1. Examples of foundation type below the cutoff.



FOUNDATION TYPE (below cutoff) = ERODIBLE ROCK

FOUNDATION TYPE (below cutoff) = NON-ERODIBLE ROCK

geology should be determined for both the soil and rock sections and the higher value obtained should be used, i.e., $w_{\mathrm{F(fnd)}}$ soil (type) × $w_{\mathrm{F(cts)}}$ (cutoff) × $w_{\mathrm{F(sg)}}$ (type), and $w_{\mathrm{F(fnd)}}$ rock (type) × $w_{\mathrm{F(ctr)}}$ (cutoff) × $w_{\mathrm{F(rg)}}$ (type).

Soil and rock geology $w_{F(sg)}$ and $w_{F(rg)}$

The intent of the classification of weighting factors is to apply high weighting factors to erodible soils and soluble, erodible, or open-jointed rock. Rock lithology has been used as the descriptor, because sometimes that is all that is known. Detailed should be used information where available, e.g., the basalt in a dam foundation may have few open joints, so a weighting factor of less than 5, say 1 or 2, may be applicable.

Observations of seepage and pore pressures $w_{F(obs)}$ and $w_{F(obp)}$

Only one of the weighting factors should be applied out of observations of seepage or pore pressures, selecting the worst case. Assessment of the observations of seepage and pore pressures should consider the full performance history of the dam and not just the current condition of the dam. All of the descriptions of leakage refer to either seepage flows emerging downstream of the dam or foundation seepage collected in the drainage systems of the dam. Seepage emerging from the drainage system of the dam would tend to indicate a potentially less hazardous seepage condition and therefore the weighting factors can be reduced slightly by a factor of say 0.75. The qualitative description of the neutral category “leakage steady, clear” can be considered the leakage that would be expected to be normal for the type of foundation geology and the size of the dam considered. The lower categories represent leakage conditions better than the typical conditions.

Piping from the embankment into the foundation (Table 4)

Foundation cutoff

If the cutoff trench penetrates both soil and rock, the product of weighting factors for foundation type × erosion-control measures × grouting of foundations × geology type should be determined for both the soil and rock characteristics and the highest value used, i.e., take the maximum of $w_{\mathrm{EF(fnd)}}$ soil × $w_{\mathrm{EF(ecm)}}$ × $w_{\mathrm{EF(gr)}}$ soil × $w_{\mathrm{EF(sg)}}$ or $w_{\mathrm{EF(fnd)}}$ rock × $w_{\mathrm{EF(ecm)}}$ × $w_{\mathrm{EF(gr)}}$ rock × $w_{\mathrm{EF(rg)}}$.

The following descriptions are given for guidance in applying the descriptive terms in the foundation cutoff categories: (1) deep and narrow cutoff trench — the cutoff trench would be considered deep if the trench is >3–5 m deep from the general foundation level and narrow if the width to depth ratio (W:D) is less than about 1.0, where the width is measured at the top of the cutoff trench; (2) shallow or no cutoff trench — a cutoff trench would be considered shallow if it is <2–3 m; and (3) average cutoff trench width and depth — depth 2–5 m and W:D > 1.0. The geology refers to the soil and rock in contact with the core materials, on the sides and base of the cutoff trench.

Erosion-control measures $w_{EF(ecm)}$

The erosion-control measures refer to the design and construction features used to protect the core materials within the cutoff trench from being eroded into the foundation. These measures can include slush concrete or shotcrete on rock foundations and filters located on the downstream side of the cutoff trench for soil or rock foundations.

The descriptive terms poor, average, or good foundation conditions refer to features in the foundation into which core materials can be eroded. For rock foundations, poor foundation conditions would include continuous open joints or bedding, or with clay infill or other erodible material, heavily fractured rock, karstic limestone features, or stress-relief joints in steep valleys or previously glaciated regions. Good foundation conditions would include tight, widely spaced joints with no weathered seams. For soil foundations, poor foundation conditions would include open-work gravels or other soils with voids and good foundation conditions would include fine-grained soils with no structures or soils where the filter retention criteria between the foundation soils and the core materials are met.

Observations of seepage $w_{EF(obs)}$

The comments for piping through the embankment apply also to piping from the embankment into the foundation.

Calibration of the weighting factors

General approach

The weighting factors represent how much more or less likely a dam will fail relative to the “average” dam. Quantifications of the weighting factors are based on the analysis of failures and accidents of embankment dams as described in Foster et al. (1998, 2000). The weighting factors were determined by comparing the characteristics of the dams that have experienced piping incidents with those of the dam population using the following calculation: weighting factor = (percentage of failure cases with the particular

Table 5. Weighting factors for the presence of embankment filters with piping through the embankment, $w_{E(filt)}$.

Description of embankment filters	No. of failures	% of failures	% of population	Weighting factor (based on statistics)*	Adopted weighting $w_{E(filt)}$
No embankment filter	8	100	40	2.5	2.0
Poor quality embankment filter present	0	0 (5)†	20‡	0 (0.25)§	0.2
Well-designed and well-constructed embankment filter present	0	0 (1)†	40‡	0 (0.025)§	0.02

Note: The failure and population statistics and weighting factors only apply to dam zoning types where embankment filters are usually present. These include earthfill with filter dams, zoned earthfill dams, zoned earthfill and rockfill dams, and central core earth and rockfill dams.

*Derived as (% of failures)/(% of population).

†An equivalent failure rate of 1% was assumed for dams with good filters and 5% for dams with poor filters for the purpose of estimating a weighting factor.

‡It is assumed that one third of the dams with filters present do not meet current standards in filter criteria or were susceptible to segregation during construction.

§Weighting factors are based on the assumed equivalent failure rate for the categories where filters are present.

characteristic)/(percentage of dam population with the particular characteristic).

Additional factors were added to take into account the dam characteristics which were not included in the dam incident database to take into account the performance of the dam and the degree of monitoring and surveillance of the dam. The weightings of other factors which are related or judged to be of similar significance were used as a basis to calibrate these other factors. The weighting factors were also checked by ensuring that the effect is neutral when the factors are applied to the dam population. This is possible by checking that the sum of the product of the weighting factors and the percent population for each of the factors is 100%, i.e., $\sum$(weighting factor × % population) = 100%.

A degree of judgement in relation to dam engineering principles was also used. Descriptions of the analysis and the assumptions used to derive the weighting factors are given in Foster et al. (1998, 2000) and Foster (1999). Some of the important points are given in the following sections.

Embankment filters $w_{E(filt)}$

The weighting factors for the presence or absence of embankment filters were determined directly from the failure and population statistics for the dam zoning types where embankment filters are normally present. The percentage of these dams with embankment filters is estimated to be 60%. For the purposes of estimating appropriate weighting factors, we assumed that of the 60% of dams with embankment filters, one third have poorly designed or constructed filters that do not meet current filter criteria, and two thirds meet current standards.

In the two failures where embankment filters were known to have been present, Ghattara Dam and Zoeknog Dam, piping occurred around the conduits. At Zoeknog Dam, the filter was not fully intercepting around the outlet conduit. This was likely also the case for Ghattara Dam, although there is insufficient information to prove this. These two cases therefore fall into the “no embankment filters present” category which implies there have been no failures by piping through dams where fully intercepting filters were present.

Weighting factors derived from the failure and population statistics for the presence of embankment filters are shown in Table 5. The values shown in the right-hand column of Table 5 are the weightings adopted for the assessment of relative likelihood of failure by piping. The weighting factors from the failure statistics for dams with embankment filters present are zero, as there have been no failures. An equivalent failure rate of 1% was assumed to estimate a weighting factor for the case where well-designed and well-constructed filters are present. This is a judgement which represents the generally accepted belief in the reliable performance of good quality filters downstream of the core in sealing concentrated leaks and preventing initiation of piping (Sherard and Dunnigan 1989; Peck 1990; Ripley 1983, 1984, 1986). An equivalent failure rate of 5% was assumed for dams with poor quality filters. This implies dams with poor quality filters are 10 times more likely to fail by piping than dams with good filters and 10 times less likely to fail than with dams with no filters. Dams with poor filters would be expected to have a lower probability of failure than dams with no filters, as the filter zone tends to act as a secondary core by limiting flows through the dam in the event of leakage through the core (Sherard and Dunnigan 1989; Peck 1990). A review by Vick (1997) of piping accidents to central core earth and rockfill dams showed dams with no filters experienced the largest flows through the damaged core.

Conduits $w_{E(con)}$

In about half of the piping failures, piping was known to have initiated around or near a conduit. Several categories were derived to describe the degree of detailing incorporated into design of the conduits, and these are described in a previous section. The estimated percentage of dams in the population that fall into each of the conduit descriptions and the assigned weighting factors were assessed. To calibrate the weighting factors, a conduit with many poor details was considered to be equivalent to a continuous zone of poor compaction, and an upper bound weighting of 5 was adopted using the weightings from core compaction as a baseline. This is consistent with other important factors such as zoning, where the worst case is about 5 times the average case. The lower bound weighting factor for dams with no outlet conduit through the embankment was assigned a factor of 0.5, assuming the historical probability of failure by piping may have been halved if the dams that failed by piping around the conduit had no conduit. The weighting factors of the intermediate categories were selected such that when they are applied to the population the result is neutral.

Observations of seepage $w_{E(obs)}$, $w_{F(obs)}$, *and* $w_{EF(obs)}$

The occurrence of past piping incidents or ongoing piping episodes is judged to be one of the most influential factors for predicting the likelihood of failure by piping. The worse-case condition where observations of muddy leakage and sudden increases in leakage have been observed is assumed to have a weighting factor 2 times higher than the highest weightings for any of the other factors. This gives a weighting factor of 10 for the worst observations of seepage and piping episodes. This weighting is considered to represent an upper bound, and allowance is made in the UNSW method to apply a factor within the range of 2–10 depending on the nature, severity, and location of any past piping episodes. The observation of sinkholes on the dam or sand boils in the foundations was assigned a lower weighting of 2, as they appear to be mainly associated with piping accidents rather than failures.

Monitoring and surveillance $w_{E(mon)}$, $w_{F(mon)}$, *and* $w_{EF(mon)}$

The frequency of inspections and measurements of seepage is included in recognition that more frequent monitoring and surveillance may be able to detect early stages of piping and measures taken to prevent the development of piping to failure. As discussed later in the paper, the time from the initiation of piping to breaching of the dam is often short (e.g., less than 6 h from the initial signs of muddy leakage to breaching), and so the likelihood of intervention is likely to be low even if the dam is monitored frequently. This is reflected in the low range of the weighting factors of only 4 times between the best and worst cases.

Justification for and limitations of the UNSW method

The UNSW method relies upon the assumption that the performance of embankment dams in the past is a guide to their performance in the future. This is reasonable given the following:

(1) The analysis upon which Table 1 is based was based on extensive surveys of dam failures and accidents by the International Commission on Large Dams (ICOLD) and represents over 11 000 dams and 300 000 dam-years of operation. Zoning of the population of dams was determined using a sample of more than 13% of the population. Table 1 allows for the higher incidence of failures on first filling, and through the zoning, for older types of dams.

(2) Dams are to a certain extent unique in that each has its own soil and geology, loading history, and details of design and construction. However, dam engineering standards, e.g., filter design criteria, and compaction density ratio and water content requirements are similar worldwide. The database and applicability of the UNSW method are to large dams, which are therefore mostly engineered to the standards of the day.

(3) The zoning categories in Table 1 are clearly linked to the degree of internal erosion control by the presence of filters and other features, upon which conventional dam engineering is based. The outcomes are consistent with what one would expect, e.g., dams with good internal erosion features have low frequencies of failure, and those with features which reduce the likelihood of breaching (e.g., high-permeability downstream rockfill zones) give low frequencies of failure and higher frequencies of accidents. The importance of zoning and filters have been recognised by many researchers, e.g., Sherard et al. (1963), Sherard (1973), and USBR (1977, 1989).

(4) There are precedents to use historic frequency of failures as a guide to the future performance in the assessment of the likelihood of failure of other complex geotechnical systems such as natural and constructed cut and fill slopes. Mostyn and Fell (1997) and Einstein (1997) give an overview of the methods and examples of their use.

The analysis of data (Foster et al. 1998; Foster 1999) shows that after the first 5 years the frequency of failure by piping is not very dependent on the age of the dam.

The extension of the UNSW method beyond application of the historic frequencies based on zoning relies on the analysis of the characteristics of the failures and accidents, and comparing these with the assessed characteristics of the population. Because the number of failures and accidents is relatively small, 50 failures and 167 accidents (Foster et al. 1998, 2000), data from all zoning categories and from first-filling and later failures have been combined. Therefore it has not been possible to prove that the values for the factors used in Tables 2–4 are statistically significant. However, it should be noted that, although the ranking and quantification of the factor are based on the analysis of the data, they are also determined by relation to published information on the erosion and piping and on the nature of geological environments. For example, reference has been made to the work of Lambe (1958), Sherard et al. (1963), Sherard (1953, 1973, 1985), Arulanandan and Perry (1983), Hanson and Robinson (1993), Charles et al. (1995), and Höeg et al. (1998), who discuss the effect of compaction density and water content, soil classification, foundation irregularities, and conduits on the likelihood of initiation and progression of piping. These have been combined with judgement from the authors to develop Tables 2–4. The factors for “observation of seepage” and “monitoring and surveillance” are based purely on judgement.

The following should be noted:

(1) The overall structure of the UNSW method and Tables 1–4 gives no one factor dominating the assessed relative likelihood of failure. This is consistent with the analysis of the data, and is also consistent with the observation that the failure case studies all had several “much more likely” or “more likely” factors present (Foster et al. 1998; Foster 1999). Consistent with this, high likelihood of failure can only be obtained when several of the factors are “much more likely” using the UNSW method.

(2) The UNSW method has been reviewed by the representatives of the sponsors, several of whom gave comments and suggestions for changes which were taken into account.

(3) The UNSW method has been used for a number of portfolio risk assessments in Australia and has given results that experienced dam engineers have been broadly comfortable with. In other words, the outputs are consistent with what experienced engineers judge to be reasonable. This does not say the results are proven in absolute terms, only that in relative terms they seem reasonable.

The limitations of the UNSW method include the following:

(1) The lack of rigorous statistical analysis to assess the interdependence of the weighting factors and the applicability of the hypothesis that the frequency of failures up to 1986 (in Table 1) is a guide to the likelihood of failures. This has not been possible because, as explained earlier, most failures include several factors with high weighting factors, so if the effect of one factor, e.g., compaction, is removed, the remaining samples are too small to allow analysis. Although ICOLD updated their failure statistics (ICOLD 1995), they did not reassess the accident statistics, so there is no basis for checking global performance since 1986.

(2) Failures on first filling are combined with later failures. The UNSW method allows for this in the base frequencies given in Table 1. Early in the study some work was done to see whether there was any difference in characteristics between the two groups. This was not done in a statistically rigorous way but showed little difference. Because of this, and the problems with splitting the relatively small number of failures and accidents for the analysis of the weighting factors, the decision was made to leave them as one group.

(3) As the weighting factors are often based on low numbers of accident and failure cases, some of the factors and the baseline annual frequencies of failure for the zoning categories are sensitive to the occurrence of only one or two piping failures for dams with a particular zoning category or some other characteristic. This may tend to either underestimate or overestimate the influence of these factors. However, attempts were made in the analysis of the weighting factors to highlight these cases and to check the reasonableness of the factors based on the expected susceptibility of the particular conditions for piping failure.

(4) The analysis of the weighting factors assumes the factors to be independent of each other; however, it is probable there is some degree of dependency between some of the factors. Therefore, when the weightings are multiplied together, some “doubling-up” of the weighting factors may occur and this may tend to overemphasise or underemphasise some factors. Any obvious cases of this doubling-up of factors were accounted for in the analysis and any remaining cases are considered unlikely to be large.

(5) The likelihoods of failure are based on large dams (>15 m height), so the UNSW method may tend to underestimate the likelihood of failure of piping if applied to smaller dams, which are more likely to be poorly constructed.

Factors affecting the warning time and ability to intervene to prevent failure

Case studies form a valuable means of obtaining guidance on the warning signs which may be evident prior to piping failures and accidents, and for the time to develop failure. These have a major influence on assessing whether intervention to prevent failure is possible or what warning time will be available to evacuate persons downstream. The following details the summary of observations. We recognise that when assessing an existing dam, the critical issue is whether monitoring and surveillance are sufficient to observe the onset of piping, and whether the observers are sufficiently skilled to react correctly to the warning signs. It is for this reason that the details of the incidents are included in Tables A1–A6 in Appendix 1 and in the summaries.

Observations during incidents

Piping through the embankment

Figure 2 summarizes the observations during incidents of piping through the embankment. An increase in leakage and muddy leakage were the most common observations made during both accident and failure cases. In approximately 30% of failure cases no observations were possible up to the failure because no eyewitnesses were present, e.g., failure occurred at night. Sinkholes were commonly observed in accidents (over 40% of cases) but not commonly observed in failures (10%). In failures, piping erosion tunnels progress back through the dam into direct connection with the reservoir and the sinkhole would form below the reservoir level and thus out of sight. Sinkholes observed on the crest or downstream slope of the dam in the accidents may indicate that limiting conditions of the piping erosion process have been reached or that collapse of the erosion roof of the tunnel has taken place. There have been very few piping incidents where changes in pore pressures in the dam were observed.

Piping through the foundation

Figure 3 summarizes the observations during incidents of piping through the foundation. Increases in leakage and muddy leakage were commonly observed during both failure and accident foundation piping cases. Sinkholes and sand boils were frequently observed in the accident cases, but rarely in the failure cases. As for embankment piping failures, the sinkhole forms out of sight below the reservoir surface. Von Thun (1996) notes that not all sand boils were related to retrogressive erosion piping and that some were only very localised surface features.

In all but one of the failure cases by piping through the foundation, the dams experienced seepage from the foundation emerging downstream of the dam. In one case, Baldwin Hills Reservoir, seepage was collected in a drainage system below the reservoir foundation. Previous piping incidents were experienced in only a few of the failure cases (Black Rock, Nanak Sagar, Ruahihi Canal, and Roxboro Municipal Lake dams). In all other cases, the seepage prior to the failure was described as clear with no evidence of piping. At Baldwin Hills Reservoir, which was closely monitored, there was a slight but detectable and consistent increase in seepage through the reservoir foundation floor drains for 12 months leading up to the failure. However, the measured seepage flow was approximately half of the maximum seepage flow recorded after first filling. At La Laguna Dam, there was also a slight increase in seepage flows over a 24 year period; however, 1 month prior to the failure the seepage flows exceeded the maximum ever recorded and the rate of increase of the seepage flows tended to accelerate prior to the failure.

The majority of accident cases by piping through the foundation involved recurring piping episodes usually over many years, and in only a few cases did it appear that an emergency situation eventuated (e.g., Upper Highline Reservoir and Caldeirao Dam).

Fig. 2. Observations during piping incidents, with piping through the embankment.

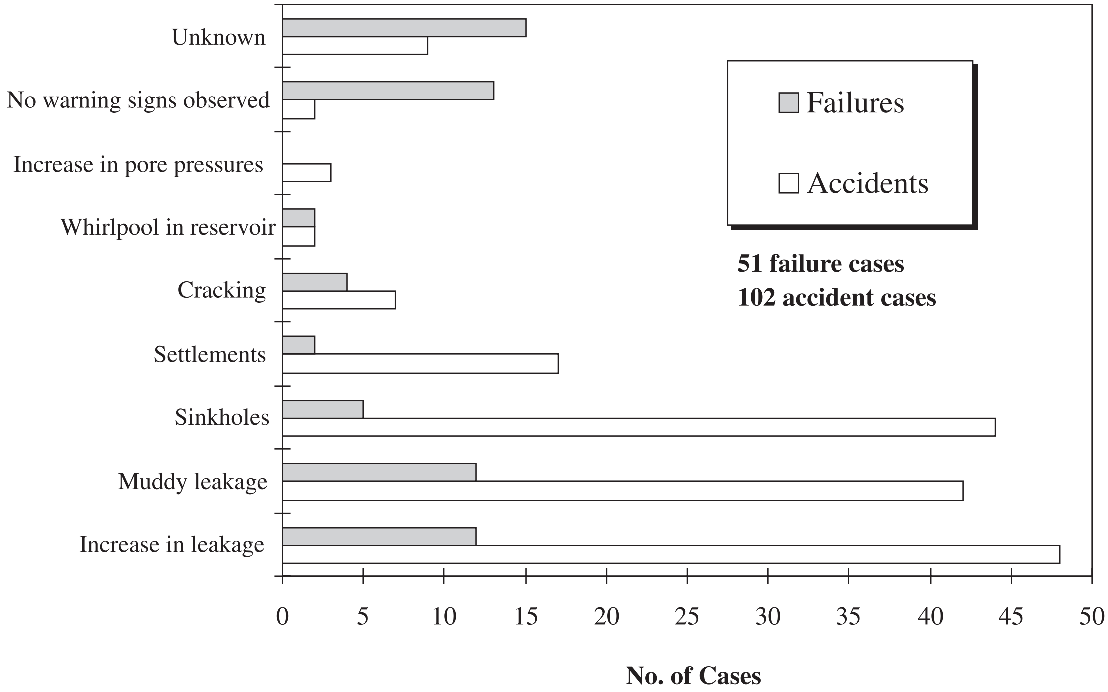
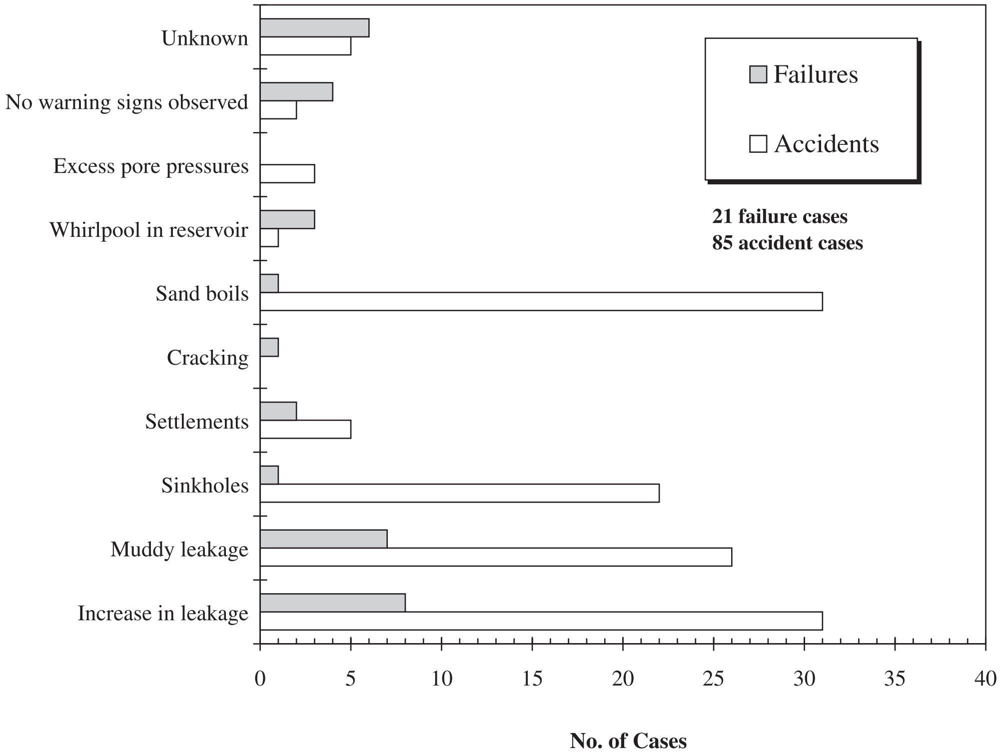


Fig. 3. Observations during piping incidents, with piping through the foundation.



Piping from embankment to foundation

For the failure cases, there is a wide range in the descriptions of long-term warning. At Teton Dam, there were no warning signs prior to the initiation of piping, apart from the appearance of minor leakages downstream of the dam several days before the failure. At Quail Creek Reservoir, there were recurring piping incidents from first filling up to the time of failure.

In the accident cases, the initial stages of piping tended to develop rapidly; however, after a while the flows from the

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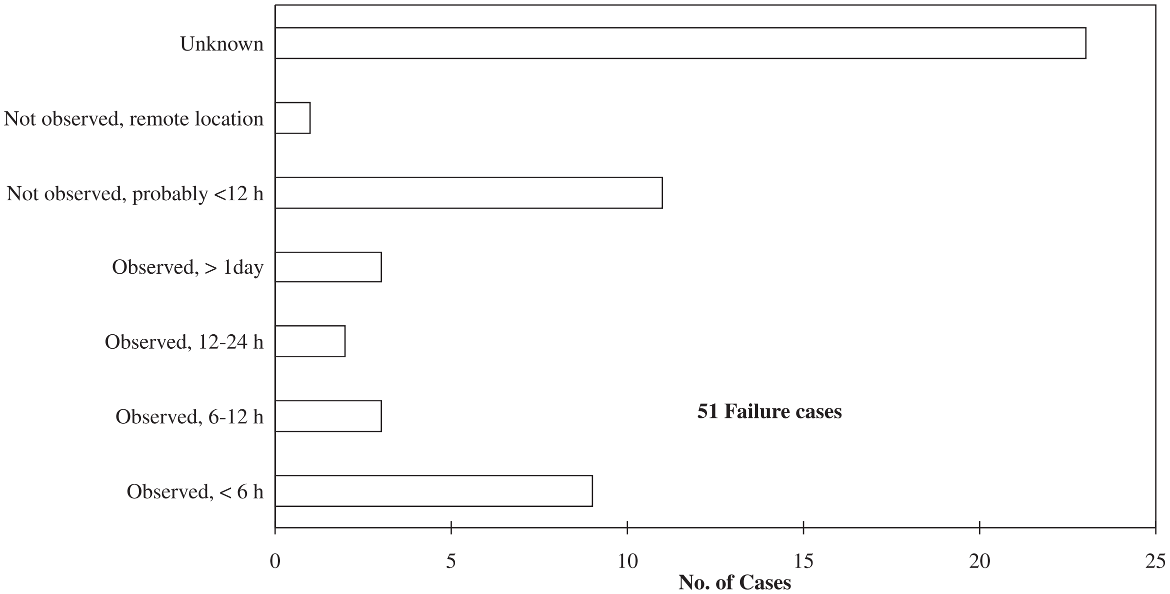
Fig. 4. Piping development time of failures by piping through the embankment.



concentrated leaks stabilized, allowing sufficient time (usually in the order of days) for remedial actions to be taken and to be effective. It is possible that in many of the accident cases the piping process was limited by the limited flow capacity through the open cracks in the bedrock, thereby slowing the erosion of the embankment materials.

Piping development time

Piping through the embankment

Figure 4 summarizes the times for development of failures by piping through the embankment. The piping development time is defined as the time from the first visual indication of initiation of piping (i.e., initial muddy leak) to the breaching of the embankment. In approximately 50% of the failure cases there was insufficient information in the failure descriptions to estimate the piping development time. In 11 cases the piping failure occurred overnight and the development of piping was not observed. However, it was evident from the description that inspections of the dam made the evening of the failure did not note any unusual observations. For these cases, it was assumed that the piping development time was probably less than 12 h. For the majority of cases where an estimate was available, the piping development time was less than 6 h and in some of these cases only 2–3 h. The piping development time was greater than 1 day in only one of the failure cases, that of Panshet Dam. In this case, muddy leakage was observed exiting the downstream toe of the dam reportedly 35 h prior to breaching of the dam.

Descriptions of the observations leading up to and during the piping incidents for all of the failure cases and for a select group of accident cases are given in Appendix 1. It is evident that in a few of the failure cases the dams were poorly maintained and remedial work was not carried out despite prior piping incidents (Blackbrook, Bilberry, and Kelly Barnes dams). Failures occurring during first filling of the reservoir generally occurred hours or weeks after filling of the reservoir and piping developed quite rapidly with very little warning. In roughly half of the failure cases occurring after first filling, the dams had suffered past piping incidents or increases in leakage prior to the failure (Ibra, Dale Dyke, Apishapa, Greenlick, Hatchtown, and Walter Bouldin dams). In other cases, concentrated leaks were present many years prior to the failure but the seepage tended to be steady and clear with time (Bila Desna, Hebron, Horse Creek, and Pampulha dams).

In many of the piping accident cases, the piping process appeared to have reached some limiting condition, allowing sufficient time to take remedial action. In these cases, the concentrated leaks initially developed rapidly, similar to failure cases, but the flows tended to stabilize, slowing the erosion of the embankment materials (examples include Wister, Hrinova, Martin Gonzalo, Table Rock Cove, and Scofield dams). In two of the accident cases, Suorva East and Songa dams, the piping process was self-healing and the leakage flows reduced prior to any remedial works being undertaken.

Piping through the foundation

Figure 5 summarizes the times for development of failures by piping through the foundation. In about 40% of the failure cases there was insufficient information in the incident descriptions to estimate the piping development time. The piping development time is less than 12 h in nine out of the 11 cases where it was possible to estimate. In five of these cases, piping developed rapidly in less than 6 h. In the two cases where the piping development time took longer than 12 h, Alamo Arroyo Site 2 Dam and Black Rock Dam, the development of piping took at least 2 days. At Alamo Arroyo Site 2 Dam, a 6–9 m wide and 180 m long tunnel developed through the foundation of the dam, draining the reservoir in 2 days without the embankment actually breaching. At Black Rock Dam, piping developed through the abutment of the dam, leading to settlements of the spillway and abutment over a 2 day period when a breach finally formed through the abutment.

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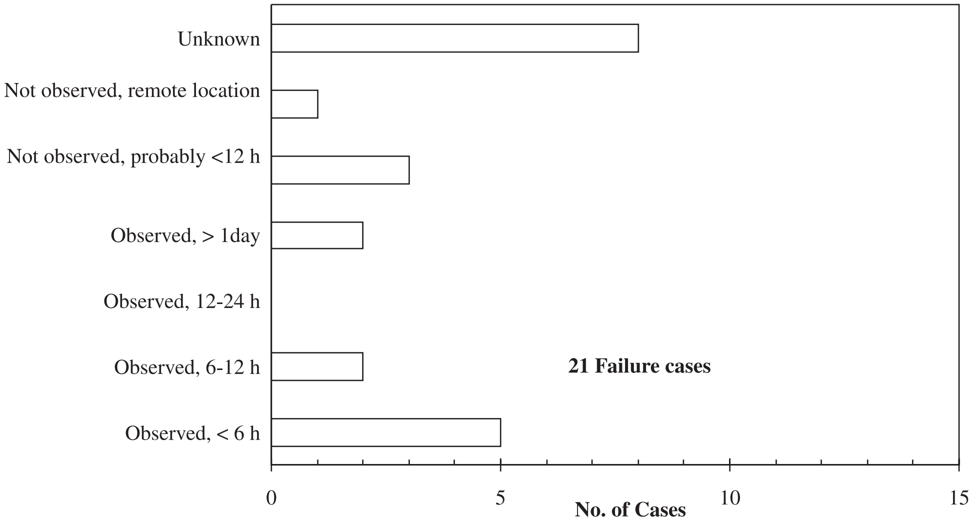
Fig. 5. Piping development time of failures by piping through the foundation.



Piping from the embankment into the foundation

The development times for piping failures from the embankment into the foundation were 3 h for Manivali Dam, 4 h for Teton Dam, and 12 h for Quail Creek Dam. All three cases involved piping of embankment materials into a rock foundation.

Conclusions

The UNSW method has been developed for estimating the relative likelihood of failure of embankment dams by piping. It is only suitable for preliminary assessments, as a ranking method for portfolio risk assessments to identify which dams to prioritise for more detailed studies, and for a check on event-tree methods. The results are expressed in terms of likelihood, meaning a qualitative measure of probability. We do not represent that the results are absolute estimates of probabilities.

The assessments made using the UNSW method will only be as good as the data upon which they are based. It is important to gather together all available information on the design, construction, and performance of the dam.

The UNSW method is meant only as an aid to judgement, and not as a substitute for sound engineering analysis and assessment.

Descriptions of failures show that piping develops rapidly. In the majority of failures, breaching of the dam occurred within 12 h from initial visual indication of piping developing, and in many cases this took less than 6 h. For the piping accidents, the emergency situation often lasted several days, with piping reaching a limiting condition, allowing sufficient time to draw the reservoir down or carry out remedial works to prevent breaching.

Acknowledgments

The support of the 17 sponsors of the research project, Dams Risk Assessment – Estimation of the Probability of Failure, and the Australian Research Council is acknowledged. The sponsors of the project are ACT Electricity and Water, Department of Land and Water Conservation, Electricity Corporation New Zealand, Goulburn Murray Water, Gutteridge Haskins and Davey (GHD), Hydro Electric Commission, Tasmania, Melbourne Water Corporation, NSW Department of Public Works and Services, NSW Dam Safety Committee, Pacific Power, Queensland Department of Natural Resources, Snowy Mountain Engineering Corporation, Snowy Mountain Hydro-electric Authority, South Australia Water Corporation, Sydney Water Corporation, Australian Water Technologies, and Western Australia Water Corporation. The assistance from other organisations that allowed access to their reports for the research project is also acknowledged. These include the United States Bureau of Reclamation (USBR), British Columbia Hydroelectric and Power Corporation (BC Hydro), Norwegian Geotechnical Institute, and Alberta Dam Safety. The assistance of Kurt Douglas is also gratefully acknowledged for his participation in the group meetings with the authors for the assessment of the population statistics and the assignment of the weighting factors.

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Appendix 1. Descriptions of warnings of piping failures and selected accidents.

This appendix is made up of six tables outlining the descriptions of warnings of piping failures and selected accidents.

Table A1. Descriptions of warnings of failures resulting from piping through the embankment.

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
First-filling failures								
Ahraura	India	2	26	1953	1953	Rapid first fill; seepage pressure not relieved near sluice gate (no rock toe); pressure buildup; piping	A 9 m rise in reservoir level 1 day prior to failure	Small leak initially observed 3 h prior to breach; seepage seen emerging at the downstream rock toe; leakage increased and scour hole formed on the downstream slope; a thatched roof thrown in the whirlpool in the reservoir washed through the scour hole
Battle River	Canada	0	14	1956	1956	Piping through embankment around bypass conduit, concentrated leak to breach in 18 h, no upstream blanket at location of failure	Dam closure 12 days prior to breach and water over spillway 7 days prior to breach; no other details available	A “boil” (about size of a man’s fist) observed on downstream slope adjacent to bypass pipe; the leak gradually increased during the night; a large volume of newly placed fill collapsed into whirlpool and the dam breached 18 h after the boil was first observed
Campbelltown Golf Course	Australia	1	10	1974	1974	Tunnel formed through dispersive embankment fill due to cracking over conduit trench following rapid filling	No details available	Initial leak observed on downstream slope adjacent to outlet pipe; leak increased to estimated 280–425 L/s 7 h later; water jetting out of 2 m diameter hole on downstream slope 10 h after initial leak first noticed; reservoir drained through piping tunnel
Dale Dyke	Great Britain	8	29	1864	1864	Most likely cause attributed to hydraulic fracture and internal erosion of thin puddle clay core into coarse shoulder fill with crest settlement and overtopping; Binnie (1981) attributed this to piping through the cutoff trench	Reportedly, a large spring issued from the foot of the dam where the breach occurred; a sinkhole had been observed in the stone pitching on the upstream slope several weeks or months prior to the failure	Longitudinal crack near the top of the downstream slope noticed 6 h prior to breach; crack widened from about 0.5 in. to 1 in. (1 in. = 25.4 mm); no descriptions of observed leakage in incident descriptions, but failure occurred at night
Ema	Brazil	13	18	1932	1940	ICOLD (1984) description suggests sliding of downstream slope due to piping	No details available	No details available
Fred Burr	United States	3	16	1947	1948	Failed on first filling when water 0.3 m below spillway; cause unknown but attributed to piping or slumping of embankment upon saturation	No details available	No details available

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Table A1 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning	
							Long term	Short term
Ghattara	Libya	1	38	1972	1977	Piping through embankment around conduit; rapid filling; dispersive embankment materials; probable poor compaction and no filters around conduit	Rapid filling of reservoir of 7 m in 3 days; no other details	Muddy water seen flooding the toe of the dam emerging from above the outlet conduit about 1.5 h prior to breaching; this area had been dry 1.5 h earlier
Ibra	Germany	6	10		1977	Piping along conduit due to inadequate connection of upstream membrane	On three previous test fillings, problems with connection of membrane to plinth next to intake structure; fluctuations in seepage through bottom drainage ranging from 27 to 80 L/s; on drawdown several large depressions observed in membrane	One day prior to breach, seepage from around outlet conduit increased considerably and water turned muddy; tunnel formed next to conduit
Kedar Nala	India	2	20	1964	1964	Very rapid first filling (9.1 m in 16 h); muddy concentrated leakage at downstream toe developed into piping tunnel which rapidly enlarged and breached dam; initial leak attributed to differential settlement of dam over closure section	Rapid first filling of reservoir starting 30 h prior to failure; no leakage or subsidence of dam observed prior to piping incident other than a few cracks on the crest of the dam	Early morning on day of failure, muddy water was observed jetting out at the downstream toe; flow estimated at 110–140 L/s; leak developed into tunnel emerging above level of downstream boulder toe which rapidly enlarged and dam breached at about 11 a.m.
La Escondida	Mexico	0	13	1970	1972	Formation of 50 pipes and eight breaches through embankment upon first rapid filling; dispersive clays used in embankment	No details available	Dam breached a few hours after first rapid filling of the reservoir; no other details available
Lake Cawndilla Outlet Regulator Embankment	Australia	0	12	1961	1962	Piping through dispersive embankment materials around conduit; poor compaction near conduit; arching across deep narrow conduit trench; piping leading to breach	No details available	No details available
Lake Francis (A)	United States	0	15	1899	1899	Rapid filling; flow through transverse settlement crack over steep right abutment leading to piping failure	Rapid first filling	Large settlement crack opened near and parallel to right abutment; large stream of water seen coming out of toe of dam adjacent to outlet pipe; several minutes later, water appeared on the downstream face; rapid development of piping to breach

Table A1 (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Little Deer Creek	United States	2	26	1962	1963	Piping of poorly compacted embankment materials into coarse rockfill toe drain; led to breach	One week prior to failure, there was “no water” at the measuring flume downstream of dam; no other details of performance of dam	No eyewitnesses to dam failure
Mafeteng	Lesotho	1	23	1988	1988	Piping through dispersive embankment materials along contact between embankment and concrete spillway wall; rapid first filling	Rapid filling of reservoir on the day before the failure	A leakage of muddy water observed at the lower part of the downstream slope adjacent to the spillway wall; the leak enlarged and at about 9.5 h after the initial leak was first observed it had progressed to full dam breach
Mena	Chile	13	17	1885	1888	ICOLD (1995) study gives cause of failure as piping through the embankment; Baab and Mermel (1968) attribute failure to steep slopes	No details available, but some reports indicate precarious conditions at the dam were known to certain responsible officials prior to the failure	No details available
Owen	United States	13	17	1915	1914	Leakage around outlet conduit caused partial failure	No details available	No details available
Panshet	India	3	49	1961	1961	Unfinished and unlined outlet conduit; gate stuck half open developed violent water-hammer; 1.4 m settlement of crest in 2 h; settlements probably due to piping through the embankment around conduit	Rapid first filling of reservoir; 37 m rise in 18 days	Steady seepage emerging from downstream rock toe (est. 140–200 L/s) 35 h prior to breach; settlements and cracks observed on crest over conduit trench 28 h prior to breach; rate of settlement increased and crest overtopped at subsided area
Piketberg	South Africa	0	12	1986	1986	Piping along conduit through dispersive fill on first filling; hydraulic fracture over conduit due to “mushroom” cross section shape	No details, except that the failure occurred 5 weeks after water was first pumped into reservoir	Major leakage suddenly appeared at downstream toe; all water from reservoir drained through piping tunnel in dam in 1 day
Ramsgate, Natal	South Africa	0	14	1984	1984	Several piping tunnels develop through embankment on first filling following cracking of dam due to settlement; dispersive embankment materials; tunnels enlarge to breach	Rapid filling of reservoir in 1 day	Several transverse cracks developed across the crest 24 h prior to failure; next morning crest of dam sagged where cracks had formed and water was emerging at several locations at downstream toe; flow increased during day and dam breached mid-afternoon

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Table A1 (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Senekal	South Africa	3	8	1974	1974	Piping through dispersive embankment core on first filling; 5 m high tunnel formed, emptied reservoir; only 3 m of water in reservoir at time of failure	Initial leak detected at downstream toe 1 week after water pumped into the reservoir	Initial leakage from two 40 mm diameter holes located at the downstream toe at shallow depth leading below the dam detected 4 days prior to failure; flow increased, developing into 5 m diameter tunnel which emptied reservoir
Sheep Creek	United States	3	18	1969	1970	During first rapid filling, piping developed around the outside of the service spillway pipe which passed through the dam, leading to breach; some difficulties in joining 3 m pipe lengths during construction	Rapid first filling	Some seepage observed along the outside of the spillway pipe at the stilling basin shortly after pipe started flowing; dam breached a few hours after spillway pipe went into operation
Stockton Creek	United States	2	29	1949	1950	Piping through embankment over steep abutment following rapid filling of reservoir	Rapid filling of the reservoir in 1 day	No eyewitnesses to the breach, but an inspection of the dam at 8 p.m. on the evening prior to failure noted nothing unusual; breach occurred early morning
Tupelo Bayou	United States	0	15	1973	1973	Piping through embankment during construction due to differential settlement cracking, resulting in breach	No details available	No details available
Zoeknog	South Africa	1	40	1992	1993	Piping through embankment around conduit on rapid first filling; dispersive embankment materials; poor detailing of conduit trench and filters	Failure occurred after reservoir level at 65% storage level for 3 weeks; no details of observations or monitoring prior to piping failure	Failure occurred at night; a few hours after a concentrated leak was discovered, a large tunnel formed and shortly afterwards the crest of the dam collapsed, resulting in a breach
Failure after first filling but less than 5 years of operation								
Apishapa	United States	2	35	1920	1923	Horizontal crack formed through dam due to differential settlement of upper and lower parts of embankment, leading to a rapid piping failure	After first filling, transverse and longitudinal cracks on crest and max. crest settlement of 0.76 m; on the day of the failure, labourers were repairing a small leak and sinkhole about 18 m away from breach location	Two hours prior to the breach no new cracks or subsidences were observed; an inspection 15 min prior to the breach observed a settlement at the water edge and a concentrated leak emerging on the downstream slope; backward erosion and collapse of crest in 15 min

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Table A1 (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Bila Desna	Czechoslovakia	0	18	1915	1916	Piping through embankment around outlet conduit; large quantity of muddy leakage following rapid filling leading to breach	Reservoir filled four times prior to failure; a leak of clear water emerged from the bottom of the outlet gallery at 0.7–3 L/s depending on the reservoir level; no remedial work carried out	Leak of clear water noticed near the exit from the outlet gallery; leakage increased in volume rapidly and turned muddy; dam breached 1.5 h after the initial observation of leakage
Blackbrook I	Great Britain	8	28	1797	1799	Internal erosion of poor quality puddle clay core into permeable shoulder fill leading to 0.5 m crest settlement and overtopping during flood	Dam leaked considerably prior to failure; crest settled by 46 cm	No description available
Greenlick	United States	0	19	1901	1904	Probable piping through embankment; leakage through embankment and foundation	Dam settled several feet during first spring due to thawing out of fill materials that had been placed frozen; excessive seepage through the dam and foundation; seepage through foundation had been increasing prior to failure	A concentrated leak was discovered on embankment on the morning of the day of the failure; breach occurred at about 10 p.m.
Hebron (A)	United States	0	17	1913	1914	Piping through embankment following rapid filling	Concentrated leak of about 30 L/s developed on downstream slope near outlet conduit on first filling; leakage flow remained constant	Heavy rainstorm filled reservoir; caretaker caught on one side of spillway and so no observations possible from 6 p.m. until breach occurred early morning at 2 a.m.
Hinds Lake	Canada	13	12	1980	1982	No description available (mode of failure assumed from ICOLD 1995 study)	No details available	No details available
Horse Creek, Colorado	United States	6	17	1912	1914	Seepage and piping through shale foundation leading to settlement of conduit, rupture, and (or) piping along conduit	On first filling, seepage along lower toe of dam; total seepage less than 30 L/s; did not increase on subsequent filling; slight seepage at lower end of conduit had been observed for some time without increase or signs of piping	Inspection of dam 10 h prior to breach did not note any increase in seepage along lower toe of dam or around outlet conduit; breach occurred at night and was not observed
Lyman (A)	United States	8	20	1913	1915	Piping through embankment at closure section which had been rapidly constructed	Dam had been carefully inspected during the day of the failure, at which time there was no evidence of cracking, settlements, or seepage	Breach occurred at night; incident descriptions give no times, but eyewitness accounts of incident suggest rapid development of tunnel and crest collapse leading to breach

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Table A1 (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Failure after 5 years of operation								
Avalon II	United States	4	18	1894	1904	Piping through the upstream earth core into the downstream rockfill zone; no embankment filters provided	Springs of large volume on river banks downstream of dam increasing in number and volume after construction due to seepage through limestone foundation	Description of incident not available
Bilberry	Great Britain	8	30	1845	1852	Internal erosion of thin puddle clay core into permeable shoulder fill resulting in 3 m crest settlement and overtopping during flood	On first filling in 1841, muddy leak developed through culvert; in 1843, leakage increased and water burst through culvert; a new leak developed in 1846, and leakage continued; a sinkhole developed on crest from 1846 to 1851; bank settled 3 m, and was not repaired	A flood filled the reservoir up to the level of the existing sinkhole and subsidence rapidly increased and crest was overtopped
Caulk Lake	United States	0	20	1950	1973	ICOLD (1984) description gives “complete structural failure of embankment. Probable cause is excessive development of excessive seepage forces as soft areas were observed prior to failure”	Soft areas on embankment observed prior to failure; no further details	No details available
Clandeboye	Great Britain	8	5	1888	1968	Collapse of old timber culvert causing rupture and settlement of embankment	No details available	No details available
Emery	United States	0	16	1850	1966	Piping of embankment materials into conduit through holes caused by corrosion or collapse of the conduit, and (or) uncontrolled seepage along conduit	No details available	No details available
Hatchtown (B)	United States	1	19	1908	1914	Piping through embankment adjacent to outlet works; outlet conduit reportedly had been dynamited to clear it 2 days prior to failure	On first filling, part of the downstream slope became saturated and started to slough dangerously; on following seasons, seepage continued but less than first filling; outlet works gate was reportedly dynamited 1 or 2 days prior to failure	A stream of muddy leakage about 150 mm in diameter first observed on downstream slope adjacent to the outlet conduit 5 h prior to breach; leak continued for 2 h and then progressive sloughing of the downstream slope commenced, leading to breach

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Table A1 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning	
							Long term	Short term
Kantalai	Sri Lanka	0	27	612	1986	British put in outlet pipes in 1875; believed to be initiator for piping; some downstream sloughing prior to fail (due to slope saturation?)	Four years prior to failure, construction of pumphouse on top of dam and dewatering from the intake well; believed this may have contributed to failure; no further details available	No details available
Kelly Barnes	United States	12	6	1899	1977	Failure attributed to slide of steep downstream slope probably associated with piping and (or) localized breach in crest	Continual seepage on downstream slope near point of exit of the spillway pipe; 5 years prior to failure, a large slide in the lower third of the downstream slope occurred in the same area as the later breach section	No eyewitnesses to dam breaching, as failure occurred at night
Lawn Lake	United States	2	8	1903	1982	Failure attributed to piping through embankment due to deterioration of lead caulking at outlet gate valve	Dam inspection 1 year prior to failure (when reservoir empty) noted some evidence of water flow from around the outlet pipe at the downstream end	Dam in remote location, thus no eyewitnesses to dam failure
Leeuw Gamka	South Africa	13	15	1920	1928	No description of incident available (piping through embankment mode of failure assumed from ICOLD 1995 study)	No details available	No details available
Mill Creek (California)	United States	12	20	1899	1957	Outlet pipe heavily corroded, allowing embankment material to pipe through outlet; a large blow hole developed in the upstream face more than 12 m diameter and 2.4–3 m deep	No details available	No details available
Pampulha	Brazil	6	18	1941	1954	Piping through embankment originating from seepage between drainage pipe and fracture in upstream concrete slab, leading to breach	Some seepage had been observed on the downstream slope for some time before failure; seepage is described as “not alarming and apparently in more or less stable volumes”	Sudden increase in seepage emerging on the downstream slope; developed into a concentrated jet with increasing turbidity over a 4 day period; roof of tunnel caved in, leading to breach; water drawdown not started until “imminent danger was pending”
Smartt Sindicate	South Africa	0	28	1912	1961	Piping developed through the dam at the contact between the old and new fill materials associated with a dam raising	No details available	Late evening water was heard running on the downstream slope of the embankment; breach occurred in the early morning hours
Toreson	United States	13	15	1898	1953	Cause of failure attributed to corrosion of the outlet pipe	No details available	No details available

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Table A1 (*concluded*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Trial Lake (dike)	United States	0	5	1925	1986	Foundation not thoroughly stripped during construction; contained rootholes and organics; piping along embankment–foundation interface	No details available	Breach not observed; no further details available
Utica	United States	0	21	1873	1902	Slides on downstream slope over 4 day period followed by piping through embankment, leading to breaching; steep downstream slope (1.5H:1V)	Small slips had occurred at various locations on the downstream slope for some years after construction; crest settlement of 0.9 m in 3 years	Progressive sliding of downstream slope over 4 day period; seepage emerging from the back scarp after initial slide; on the fourth day, two concentrated leaks developed which rapidly enlarged, leading to breach; reservoir unable to be lowered quickly
Walter Bouldin	United States	3	50	1967	1975	Muddy water flowing over powerhouse floor; piping along concrete–embankment interface; immediately prior to failure, very little seepage observed at downstream toe of dam except at the powerhouse excavation slopes adjacent to the backfill	Seepage problems through foundation of dikes after first filling; installation of relief wells, toe drains, and grout curtains; a piping incident had occurred in the foundation of west dike; instrumentation showed no adverse trends prior to failure	Failure occurred at night; inspection of dam in late evening noted nothing unusual; at 1:10 a.m. night guard observed muddy leakage flowing over powerhouse, and by about 1:45 a.m. breaching of crest commenced
Wheatland No. 1	United States	0	13	1893	1969	Actual cause of failure unknown; attributed to sliding downstream slope and (or) piping along conduit (possibly due to differential settlement of backfill used to install conduit 10 years earlier?)	No details available	No details available
Kaihua	Finland	0			1959	Piping along backfill to conduit; failure attributed to poor compaction around outlet works	No details available	No details available

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Table A2. Descriptions of warnings of accidents resulting from piping through the embankment.

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: Short term
First-filling incident								
Balderhead	Great Britain	5	48	1965	1967	Internal erosion of clay core into coarse filter following hydraulic fracture of narrow core, resulting in sinkholes on crest	During first year of reservoir filling, two increases in seepage measured from main underdrain, with maximum leakages of 35 and 60 L/s; alternating cloudy and clear seepage	A large sinkhole developed on the crest 3 months after maximum seepage and cloudy seepage was observed; seepage became clear and decreased to 10 L/s after 9 m drawdown
Hrinova (A)	Czechoslovakia	5	42	1965	1966	On first filling, piping of fines from core through filter into downstream rockfill zone; slumping of downstream slope; concentrated leaks on downstream slope increased from 4 to 100 L/s	Piping incident occurred after 1 month at full reservoir level	Sudden increase in seepage flow from drains from 1 to 100 L/s; cloudy seepage observed; reservoir was drawn down over approx. 2 weeks; seepage reduced to 20 L/s, then gradually reduced to <1 L/s after 3 months
Hyttejuvet	Norway	5	93	1965	1965	Hydraulic fracturing leading to internal erosion of narrow glacial core, resulting in sinkholes on crest and soft zones in core	On first filling, rapid increase in leakage from <2 L/s to 63 L/s over 15 days as reservoir reached within 7 m of full reservoir level; leakage was muddy with 0.1 g/L fines; leakage started to decrease while reservoir level continued to increase	On subsequent fillings after the first filling piping incident, leakage was lower at 10–20 L/s, but on some fillings the seepage was cloudy; a sinkhole appeared on the crest 6 years after the initial filling of the reservoir
Martin Gonzalo	Spain	7	54	1986	1987	Internal erosion of upstream membrane bedding layer into coarse drain, leading to sinkholes in upstream slope and 1000 L/s clear seepage	Very gradual increase in leakage at full reservoir level over a 6 month period from 5 to 9.5 L/s prior to piping incident	Sudden increase in leakage within 1 day from 9.5 L/s up to 1000 L/s; leakage mainly from drains but also through springs emerging on the downstream slope; reservoir level drawn down and seepage reduced to 170 L/s 9 days later
Matahina	New Zealand	5	85	1966	1967	Internal erosion of core into transition following formation of differential settlement cracks over steps in abutment; boulders in rockfill against abutment gave wide gaps for piping to occur		Abrupt increase in leakage measured from the drainage outlet from 70 to 570 L/s; water turned “slightly cloudy;” within a few hours the total seepage had reduced to 255 L/s and within 24 h the water was clear; a sinkhole appeared on crest 2 weeks later

Table A2 (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Table Rock Cove	United States	2	43	1927	1928	Diversion pipe ran through embankment; sagged at cutoff walls, ruptured pipe; blowout of downstream slope over conduit initiated major slide of downstream slope	Several weeks prior to the piping incident, leakage appeared in small quantities at several locations on the downstream slope; largest leakage from around the downstream end of the outlet conduit	Sudden blowout and geyser-like burst of water came from around the valve chamber; flow from the outlet cut deep narrow trench back into the dam for 45 m and a 100 m wide section of downstream slope slipped back to edge of crest; several days to draw water down
Viddalsvatn	Norway	5	80	1972	1972	Hydraulic fracturing and internal erosion of core; sudden increases in seepage with self-healing muddy leaks during first filling	On first filling, four sudden increases in leakage were observed with peak flows ranging from 50 to 140 L/s; the increases in leakage were initially muddy then cleared; leakages stabilized and reduced within several days	On second filling, leakage increased from <5 L/s to maximum of 210 L/s over 7 days and decreased back to 35 L/s after 1 week reservoir drawdown; two sinkholes appeared on the crest and upstream slope several days after the piping incident
Wister	United States	1	30	1948	1949	Piping tunnels developed through dispersive embankment materials upon first rapid filling		Small concentrated leak was observed on downstream slope carrying embankment fines; the leakage steadily increased, and 5 days later the flow was 570 L/s and still muddy; took additional 4 days for water level to fall below the entrance tunnels and leakage to stop
Incident after first filling, but less than 5 years of operation								
Rowallan	Australia	5	43	1967	1968	A 1.5 m diameter and 1.3 m deep sinkhole appeared on the upstream face adjacent to the spillway wall; large local loss of core material where core contact material was placed in direct contact with coarse filter (D_{15}/D_{85} = 30)	Five months prior to the appearance of the sinkhole, a small subsidence of about 300 mm was observed at the same location	A sinkhole appeared on the crest 12 months after the reservoir had been at full supply level

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Table A2. (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Scofield	United States	4	24	1926	1928	Internal erosion of core into downstream dumped-rockfill zone; large loss of core material; cavity 55 m in length; 1400–5000 L/s leak at toe	Transverse cracks developed across the crest adjacent to each of the abutments on first filling; complaints of water seeping through the dam made to officials at least 3 days prior to the piping incident	Afternoon prior to the incident, a large depression was discovered in the crest; by next morning, a large section of crest had caved in and seepage emerging from downstream rockfill est. at 1400–5600 L/s; sandbags placed for 2 days and leakage reduced to 140 L/s
Incident after 5 years of operation								
Bullileo	Chile	5	70	1945	1982	Internal erosion of poorly compacted core and transition materials into the downstream rockfill zone; irregularity in abutment at location of former construction road	A piping incident with cloudy seepage over a short duration and without increase occurred 32 years prior to the main piping incident; maximum seepage of 1000 L/s collected at the toe of the dam since first filling (mainly from foundation)	A leakage of “some hundreds” of litres per second which was cloudy was observed early morning and by midday increased to a maximum of about 8000 L/s; a sinkhole developed on the upstream slope; at midday, drawdown of the reservoir started and by next day seepage halved
Douglas	United States	2	12	1901	1990	New seepage at downstream toe; increase in seepage and turned cloudy; seepage through sandy layer in embankment or through gravel layer in foundation	No details available	A wet area appeared at the toe of the dam which was previously dry; after 10 days seepage increased to about 1 L/s and was cloudy; sand blanket placed over seepage and reservoir drawdown started; seepage decreased after reservoir level reduced a few feet
Greenbooth	Great Britain	8	35	1962	1983	Internal erosion of puddle core, resulting in formation of sinkhole	Seepage was observed downstream of the dam but was not measured; no cloudy leakage was observed prior to the appearance of the sinkhole	A depression suddenly appeared on the crest 21 years after first filling; the depression deepened to form a sinkhole over a 3 day period; reservoir level drawn down by 9.25 m over 8 day period

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Table A2. (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Juklavatn Secondary	Norway	5	25	1974	1982	Internal erosion of core material into filter and (or) bedrock, leading to 0.5 m × 0.2 m tunnel through core; poor quality filter	Erratic seepage flows experienced during filling of the reservoir in 1982; average leakage of 2–5 L/s, with bursts up to 12 L/s; bursts of leakage and high leakage (40–60 L/s) on subsequent fillings over a 10 year period after the 1982 piping event	When reservoir reached highest recorded level, leakage suddenly increased from 10 L/s to about 90 L/s in 2 days; the reservoir level was drawn down immediately and leakage reduced to 5 L/s 9 days later
Lluest Wen	Great Britain	8	20	1896	1969	Internal erosion of puddle clay core material into cracks in a 6 in. diameter cast iron drainage pipe leading to sinkhole	Sinkhole appeared on crest 73 years after construction; a subsidence of the crest had appeared in 1912	Sudden appearance of sinkhole on the crest of the dam; flow through the cracked drainpipe measured at 0.15 L/s steady and clear, but a deposit of clay was observed at the pipe outlet; took 20 days to reduce reservoir level by 6.1 m
MacMillan (B)	United States	4	16	1893	1937	Piping from embankment into downstream dumped rockfill; near failure; no embankment filter between earthfill and rockfill	In 1915, water eroded a large hole in the earthfill core which was filled quickly filled with sandbags	In the second piping incident in 1937, 2 days were spent sandbagging the whole length of the dam before the dam was stabilized
Paduli	Italy	11	19	1906	1925	Internal erosion of embankment materials; muddy seepage observed at several places on downstream slope at high reservoir levels; some settlements observed	Leakages on the downstream slope which turn muddy at high water levels have appeared from 1921 to 1974; continuing settlement of the dam at about 10 mm/year	
Sapins	France	2	16	1978	1988	Piping of embankment materials; progressive clogging of chimney drain, leading to saturation of parts of downstream slope resulting in shallow slip and initiation of backward erosion piping	Flow in horizontal drain always high and relatively constant at 10 L/s; flow from chimney drain reached a peak of 1.5 L/s before gradually reducing and stabilizing at 0.1–0.2 L/s 2 years later	Seepage carrying fines and a shallow slip were observed in the lower part of the downstream slope; rapid worsening of the situation in a matter of weeks prompted full reservoir drawdown and remedial work

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Table A2. (*concluded*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Songa	Norway	5	42	1962	1976	Internal erosion of broadly graded glacial core material into coarse filter; piping incidents on four occasions from 1976 to 1994; self-healing	Piping incidents in the form of sudden increases in leakage observed on three separate occasions in 1976, 1979, and 1991	In the 1994 piping episode, the leakage increased abruptly from a normal flow of 1.25 L/s to 107 L/s in about 20 min and reduced back to normal within 7 h
Sorpe	Germany	10	69	1935	1951	Leakage from cracked conduit caused internal erosion of upstream fill into cracks in concrete wall drainage system, leading to 0.7 m max. crest settlement; cracks due to World War II bombing; cracks up to 100 mm wide in core wall	Dam was bombed in World War II, damaging concrete core wall	In 1951, sudden increase in leakage from 40 L/s to more than 180 L/s into the inspection gallery of the core wall; seepage was muddy; grouting reduced seepages to 40–50 L/s, but piping episodes continued up to 1958 and crest settlement of 1.4 m
Suorva East	Sweden	5	50	1972	1983	Internal erosion of glacial core material into coarse filter D_{15} = 2.4 mm; muddy leakage up to 100 L/s; self-healing as leakage decreased by 75% prior to water level drawdown; upper part of core protected by only coarse gravel filter		Cloudy seepage of about 100 L/s was observed and at the same time a sinkhole formed on the dam crest; leakage had reduced by 75% prior to starting reservoir drawdown

Table A3. Descriptions of warnings of failures resulting from piping through the foundation.

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
First-filling failure								
Blyderivier	South Africa	13	22	1924	1922	No description of failure available; mode of failure from ICOLD (1995) causes	No details available	No details available
Alamo Arroyo Site 2	United States	3	21	1960	1960	Piping of very soft (SM–ML) saturated layer into underlying coarse gravel layer in foundation, resulting in 6–9 m wide tunnel through foundation 180 m long; drain reservoir in <2 days; did not breach	No details available	Piping tunnel developed through foundation; drained reservoir in 2 days; no other details on time for the development of piping
Jennings Creek Watershed No. 16	United States	2	17	1960	1964	Piping through residual materials in karst caverns in the dam foundation; embankment undermined near abutment and collapsed	“Dam functioned as designed” until failure; no other details available	Reservoir full for 2 weeks to 1 month prior to failure; no further details
Jennings Creek Watershed No.3	United States	2	21	1962	1963	Seepage through abutment eventually piped out residual materials in karstic caverns; dam drained and cavern(s) collapsed	No details available	Vortex developed in the reservoir above previously observed cave area; large hole blew out 23 m downstream of toe of dam; no further details
Lower Khajuri	India	13	16	1949	1949	Breached at junction with masonry wall; believed to be due to piping through foundation rock	No details available	No details available
Failure after first filling, but less than 5 years of operation								
Black Rock (A)	United States	11	21	1907	1909	Piping through alluvial sands under lava cap in abutments, leading to settlement in spillway and abutment; breach formed through abutment	Piping incident on opposite abutment on the previous day controlled by blanketing; no other details available	In morning, seepage emerging from abutment turned muddy and increased; whirlpools observed near shoreline; that evening spillway dropped 7 ft (1 ft = 0.3048 m) and seepage through abutment estimated at 140 000 L/s; over next 3 days seepage decreased from 50 000 to 14 000 L/s
Corpus Christi	United States	0	19	1930	1930	Seepage through foundation under sheetpile cutoffs which did not reach impervious clay; piping under and adjacent to spillway	Reservoir full 15–18 months prior to failure; seepage through the dam described as moderate and evenly distributed; no notable observations of spillway seepage or large flows or muddy flows from spillway weep holes were recorded	A man fishing on the dam observed water boiling up under the toe of spillway apron and whirlpool in reservoir; crack opened between embankment fill and spillway wall; dam breached while man went off to warn caretaker

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Table A3 (*continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Long term	Short term
Embalse Aromos	Chile	13	42	1979	1984	No failure description available; mode of failure assumed from ICOLD (1995) causes	No details available	No details available
Horse Creek, Colorado	United States	6	17	1912	1914	Seepage and piping through shale foundation, leading to settlement of conduit, rupture, and (or) piping along conduit	On first filling, seepage along lower toe of dam; total seepage less than 30 L/s did not increase on subsequent filling; slight seepage at lower end of conduit had been observed for some time without increase or signs of piping	Inspection of dam 10 h prior to breach did not note any increase in seepage along lower toe of dam or around outlet conduit; breach occurred at night and was not observed
Julesberg (B)	United States	6	18	1905	1911	Piping centres around a concentrated leak through limestone foundation	After first filling, leakage of 200 L/s at toe spread out over 2400 m of dam; largest leak of 30–40 L/s clear water; following fillings, leak continued and increased slightly; occasional large fish washed under dam; no remedial measures to reduce the leak	Failure occurred at night, and events leading up to breach not observed; section of embankment centred on the concentrated leak washed out completely; no indication of unusual activity on previous day
Log Falls	Canada	12	11	1921	1923	No description of failure available; ICOLD (1995) attributes cause of failure to piping through the foundation	No details available	No details available
Nanak Sagar	India	0	16	1962	1967	Piping through pervious foundation, leading to settlement of the crest and overtopping during a flood event	Seepage and boils had been observed continually downstream of toe of dam for 12 days prior to the failure; seepage treated by placing inverted filters and had started giving clear water	About 13 h prior to failure, a hairline crack appeared on the downstream slope; starting at 3.5 h prior to failure, boils of muddy water appeared which could not be controlled despite covering with filter; settlement of crest occurred and dam overtopped
Ruahihi Canal	New Zealand	2	9	1981	1981	Piping through highly erodible and dispersive volcanic foundation soils, leading to sliding of canal foundation and breaching	Piping and seepage problems on several fills located below the canal after first filling; extensive cracking and movements (up to 500 mm) of fill starting 1.5 months before and up to time of failure; piping tunnel formed through fill 1 month prior to failure	No eyewitnesses to the failure; cracks observed on the fill below the canal about 80 min prior to the failure
St-Lucien	Algeria	13	27	1861	1862	No descriptions available; ICOLD (1995) attributes failure to piping erosion in foundation	No details available	No details available

Table A3 (*concluded*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: Short term
Failure after 5 years of operation								
Baldwin Hills	United States	6	71	1951	1963	Differential settlement over fault movement, initiating piping through reservoir foundation progressing to embankment	Cracks in the dam and other signs of movement observed over 12 years of operation; slight but detectable and consistent increase in seepage through reservoir floor drains from 0.6–1.0 L/s over 12 month period leading up to the failure (initially 1.7 L/s)	Underdrain pipes "blowing like fire hoses" with muddy water 4 h prior to breach; reservoir drawdown initiated; muddy water observed emerging downstream from the east abutment 2.5 h prior to breach; leak steadily increased, leading to collapse of crest
La Laguna	Mexico	9	17	1912	1969	Piping through residual basaltic clays in foundation; concentrated leak leading to erosion of downstream slope and breaching in 5 h	Max. measured seepage on right abutment increased from 12 to 28 L/s over 24 year period; flows reached max. ever recorded 1 month prior to failure and continued to increase to 55 L/s; seepages emerging at several locations 10–20 m downstream of toe	Early morning, seepage at weir measured at 75 L/s and at 6 p.m. water under pressure issued from hole; concentrated leak increased, rapidly eroding downstream slope of dam; at 10:45 p.m. the cutoff wall was uncovered and a few minutes later breach opened
Lake Toxaway	United States	9	19	1902	1916	Piping through foundation; seepage through foundation rock fractures (which had flowed since first fill); probable defective bond between core wall and foundation	Small concentrated leak located at the downstream toe of dam since first filling; 9 days prior to failure, leak noticed to be larger but remained steady; reservoir 1 m higher than normal	Concentrated leak at the downstream toe turned muddy about noon; by about 6:30 p.m. the leak began caving and at 7 p.m. the dam started breaching
Roxboro Municipal Lake	United States	13	7	1955	1984	Piping underneath undrained spillway slab progressing to and beneath ogee spillway which subsequently collapsed; plans for repairs had been prepared but not carried out	State authorities noted signs of piping below the spillway slab months before the failure and repair plan had been prepared but repairs not carried out	Immediately before the failure, sagging of a secondary road bridge over the spillway was noted and a 6 m diameter vortex developed upstream of the ogee section; within a few minutes, the ogee section collapsed
Trial Lake (dike)	United States	0	5	1925	1986	Foundation not thoroughly stripped during construction; contained rootholes and organics; piping along embankment–foundation interface	No details available	Breach not observed; no further details available
El Salto	Bolivia	13	15		1976	No description of dam or incident available; assume piping through foundation from ICOLD (1995) causes	No details available	No details available

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Table A4. Descriptions of warnings of accidents resulting from piping through the foundation.

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Long term	Short term
First-filling incidents								
Bastusel	Sweden	5	40	1972	1972	Internal erosion of alluvial foundation soils probably into fractured bedrock, indicated by large grout takes at soil–rock contact	A few days after reservoir reached maximum water level, leakage of 35 L/s measured at weir downstream of left abutment; leakage slowly increased to 40 L/s in following 2 months	Leakage measured downstream of left abutment increased suddenly to 65 L/s; drawdown of water level by 2 m and leak decreased to 20 L/s; sinkhole suddenly appeared on the crest 2 weeks later
Bloemhoek	South Africa	5	21	1978	1978	Seepage through foundation in termite galleries; minor internal erosion may have occurred as indicated by deposition of fines in foundation drain	On first filling, seepage and boils developed downstream of left abutment; after 18 months, fourfold increase in seepage; remedial grouting reduced seepage from 2 to 0.5 L/s	Nine years after remedial grouting, seepage increased to 5 L/s and significant quantities of sediment observed in the toe drains
Logan Martin	United States	2	30	1964	1964	On first filling, piping through foundation; underseepage increased for 3 years then stabilized; piping of natural joint infill through limestone foundation	On first filling, springs and muddy seepage appeared in the river downstream of the dam	After 4 years of operation, concentrated leakage at the toe of the dam became muddy and increased 10–170 L/s, and a sinkhole formed on crest; leak reduced to 9.5 L/s and clear after remedial work
Tarbela	Pakistan	13	145	1974	1974	Four hundred sinkholes formed in upstream clay blanket due to internal erosion of broadly graded blanket material into open-work gravels in the reservoir foundation		After emptying reservoir after first filling, 362 sinkholes and 140 cracks had developed in the upstream blanket; sinkholes generally 0.3–4.6 m diameter; sinkholes redeveloped on subsequent fillings, but number decreased with time and ceased 12 years later
Washakie	United States	3	19	1935	1935	Seepage problems since first filling; sand boils and sinkholes, also sloughing; major sinkhole at downstream toe of dam in 1976	On first filling, seepage losses up to 1700 L/s through left abutment; slough developed adjacent to outlet works and sinkholes appeared upstream of dam; upstream blanket was placed	In 1976, a major sinkhole appeared at the downstream toe of the dam and pipe drains installed at the toe; piping episodes continued from 1977 to 1990, including seepage carrying sand emerging over pipe drains and sinkholes over drain moving upstream with time

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Table A4 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning	
							Long term	Short term
Incidents after first filling, but less than 5 years of operation								
Bent Run Dike	United States	6	35	1969	1971	Internal erosion of residual soils in foundation into underlying fractured sandstone resulting in formation of sinkholes in reservoir foundation and dike	Many sinkholes and depressions appeared in the asphalt lining of the reservoir foundation and leakages of 600–800 L/s at various discharges around the reservoir on first filling	Cavities and leakages continued on 2nd and 3rd filling, and each time asphalt lining repaired; from 1970 to 1983, cavities and leakages continued but to a lesser extent
Mill Creek, Washington	United States	1	44	1941	1945	Excessive seepage through pervious silt and conglomerate foundation, and piping of 575 m^3 of silt through foundation filter (piped silt possibly from foundation or embankment)	Severe seepage problems since first filling; 75% of stored water lost due to seepage in first 60 days; seepage areas downstream of dam; downstream toe saturated, and sinkholes in the reservoir foundation observed	Toe drains and relief wells constructed downstream of dam, but prior seepage problems continued and 575 m^3 of material lost through internal drainage system; seepage losses of 900 L/s on subsequent fillings
Upper Highline Reservoir	United States	0	26	1966	1967	Sand boil 30 m in diameter developed downstream of embankment; thick, muddy leakage flow		A sand boil developed downstream of the dam and by early morning of the following day the boil was 30 m in diameter with a flow of thick muddy water est. at 840 L/s; reservoir level was reduced from 15 to 9 m, and sand boil stopped flowing at a level of 10.6 m
Incidents after 5 years of operation								
Black Lake	United States	3	23	1967	1986	Internal erosion of sand pockets within the colluvial deposits in the abutment foundation	On first filling, considerable seepage up to 1600 L/s; sinkholes formed on right abutment and reservoir foundation, and whirlpools observed in reservoir; blanketing of upstream reservoir foundation largely ineffective and seepage problems continued	Piping episodes continued from 1986 to 1990, and seepage observed from left abutment and from around outlet works appeared milky at high reservoir levels

Table A4 (*concluded*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Long term	Short term
Caldeirao	Brazil	0	22	1947	1957	Continual small leakage through foundation became larger and began carrying fines when reservoir at high level	Small seepage emerging near downstream toe from foundation for many years prior to the piping incident; flow kept under observation	Ten years after filling, seepage observed to be muddy when reservoir was at maximum level; some days after, erosion of the material under the foundation was observed and progressed towards reservoir; erosion stopped by grouting; no movement of dam observed
Meeks Cabin	United States	3	57	1971	1986	Piping through left abutment foundation; seepage through glacial outwash deposits not cut off by cutoff trench; sinkholes upstream of left abutment and silt accumulations at seepage flumes	Since first filling, seepage emerging downstream from left abutment and small sinkholes observed at upstream toe of dam; horizontal drains installed and seepage measured at 32 L/s	After 14 years of operation, seepage downstream of left abutment migrated closer to downstream toe of dam and small slope failures occurred; accumulation of fine sand particles in seepage-collection system observed
Three Sisters	Canada	0	21	1952	1974	Sinkhole activity in foundation of reservoir due to internal erosion of sand and sandy silt layers into open-work gravels in reservoir foundation	On first filling, seepage and sand boils appeared in a band about 23 m width immediately downstream of toe; regular appearance of numerous sinkholes in reservoir foundation since filling; approx. 130 sinkholes observed in 9 year period	Sinkhole developed in downstream slope 29 years after operation; partial sheet pile curtain wall installed upstream of dam axis, but sinkhole activity in reservoir foundation continued
Uljua	Finland	5	16	1970	1990	Piping of glacial till foundation into fractured bedrock; erosion tunnel collapsed, forming large sinkholes on crest and reservoir floor	Seepage flow of about 0.8 L/s observed 100 m downstream of dam at end of tailrace tunnel since first filling; clear flow; 1 month after filling, sudden local leakages observed but were stopped by grouting	After 20 years, leakage turned muddy, flow increased to 30 L/s, and two sinkholes formed close to upstream toe of dam; 2 weeks later, a sinkhole suddenly appeared on the crest and leakage increased to 100 L/s; sinkhole filled and rockfill placed at downstream toe
Walter F. George Lock	United States	3	52	1963	1982	Piping through foundation through ungrouted construction piezometer holes upstream of power station	Sinkhole formed 120 m upstream of dam and measured 3.7 m × 5 m and 20 m deep; 3500 bags of concrete were dropped into sink until flow diminished, followed by 255 m^3 of gravel	Reoccurrence of sinkholes and sand boils downstream of dam since first filling; up to 1970, 30 sinkholes had developed

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Table A5. Descriptions of warnings of failures resulting from piping from the embankment into the foundation.

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Long term	Short term
First-filling failure								
Manivali	India	2	18	1975	1976	Piping of embankment materials, leading to crest settlement and overtopping; piping due to high pressures transmitted through jointed rock in foundation	Breach occurred 6 weeks after the start of filling the reservoir	Leakage at the downstream toe increased from 50 to 500 L/s and exit locations rose to the top of the rock toe; dam breached within 3 h after initial observation of muddy water at the downstream toe
Teton	United States	4	93	1976	1976	Piping of core into untreated joints in abutment cutoff trench leading to rapid erosion of core and breach in 4 h	No leaks observed for first 8 months of filling; several small springs observed 2 days prior to failure 400–600 m downstream of dam, totalling 6.3 L/s; on day before the failure, spring of clear water appeared on right abutment 75 m from downstream toe at 1.3 L/s	Muddy leak initially observed at 8:30 a.m. on right downstream toe est. at 570–850 L/s; by 10:30 a.m. leak at higher level and had increased to 420 L/s; headward erosion of downstream slope progressed back to crest in 40 min, leading to breach 4 h after initial observed leak
Failure after first filling, but less than 5 years of operation								
FP&L Martin Co. Dike	United States	0	10	1977	1979	Piping of fine sand in embankment into foundation soils, leading to breaching	Seepage at downstream toe was noted frequently prior to failure but was considered normal and not thought to be dangerous	No details available
Quail Creek	United States	3	24	1984	1988	Seepage through fractured foundation, leading to piping along embankment–foundation contact; erodible zone I material placed on foundation for full width of dam due to irregularities in foundation	Recurring piping episodes since first filling; steadily increasing concentrated leak at downstream toe; three periods of grouting temporarily reduced flows; sinkhole formed on downstream slope with water bubbling out of it; leakages treated with filter blankets	Leak of muddy water emerging from outside of an observation well at the downstream toe; 1.5 h later, upward muddy flow of about 1.8 m diameter; filter placed over discharge; flow turned horizontal and est. at 2000 L/s; rapid breach 14 h after initial leak

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Table A6. Descriptions of warnings of accidents resulting from piping from the embankment into the foundation.

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Long term	Short term
First-filling incident								
Brodhead	United States	1	33	1975	1984	Internal erosion of broadly graded glacial embankment materials into open joints in left abutment and (or) into coarse foundation filter drain; 190 m^3 of embankment material eroded	Flood-control dam with no permanent storage; in 9 years of service up to time of piping incident, dam had only experienced one or two low-level fillings each year	A large flood filled reservoir and maintained water in reservoir for 10 days; after reservoir was empty, a large sinkhole was found midway up the downstream slope; no evidence was found of any inlets or outlets to the concentrated leaks
Churchill Falls GJ-11A	Canada	4	21	1972	1972	Internal erosion of glacial core into open joints in bedrock and exiting into the downstream rockfill zone	Impounding of the reservoir 6 days prior to the incident	At 11:30 a.m., surveillance helicopter observed muddy water at toe of dyke close to spillway wing wall; at 8:45 p.m., a sinkhole reported on the downstream slope and from 9:30–12:00 p.m., hole doubled in size; drawdown emptied the reservoir in 10 days
Fontenelle	United States	3	42	1965	1965	Abutment seepage eroded 8000 m^3 of embankment material; poor treatment of open stress-relief joints in abutment	Large seepage areas 600 m downstream of dam on first filling; seepage from abutment rock up to 1 km downstream from dam est. at 2000 L/s; concentrated leaks and sloughing of fill materials adjacent to spillway chute on three occasions 2–4 months before incident	Wet spot on downstream slope noticed in morning; leak steadily increased and by next morning, flow increased to 600 L/s and 8000 m^3 of fill material eroded; flow stabilized with decreasing water level, but on 4th day, section of crest collapsed up to upstream edge
Yards Creek	United States	5	24	1965	1965	Dirty leakage (25–30 L/s) upon first rapid filling; internal erosion of core due to bypass of seepage water around embankment filters through bedrock joints (note D_{15} of filter = 0.2–0.3 mm)	Muddy leak of 30–38 L/s appeared abruptly at the downstream toe over a 92 m length; leakage alternately ran very dirty and clear in cycles of 1–2 days for several weeks while reservoir at high elevations; total estimated leakage of 106 L/s; core grouted	In the following year, a new muddy leak started and increased rapidly, reaching 1.5 L/s within a few hours; within a day or so, a small sinkhole appeared on the crest over the upstream filter; by the next day, the leak decreased to only approx. 0.25 L/s of clear water

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Table A6 (*Continued*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Long term	Short term
Incident after first filling, but less than 5 years of operation								
East Branch	United States	3	59	1952	1957	Heavily fractured foundation rock; seepage through open joints, under grout curtain, and into embankment drain (inadequate filters) initiates piping in embankment	Two years prior to incident, high flow of clear water discharging from the left abutment, 30 m downstream of toe (on opposite abutment to the piping incident)	Muddy water observed emerging from rock drain at downstream toe on right abutment; leak increased from 270 to 290 L/s in 12 h; flow getting muddier; 2 days later, started drawdown and pool lowered 7.3 m in 7 days; flows continued and further lowering 2 weeks later
Incident after 5 years of operation								
Hallby	Sweden	5	27	1970	1985	Internal erosion of glacial core material into bedrock joints; washout of clay-infilled joints	No details available	Sudden appearance of sinkhole on crest adjacent to spillway wing wall; at same time, flow increased suddenly from 0.33 to 3.33 L/s; water remained clear; reservoir level temporally lowered
LG 1 Cofferdam	Canada	4	19	1979	1989	Internal erosion of dumped glacial till core material into cobble and boulder foundation	Incident occurred when water level reached highest previously experienced, 3 months after dewatering started	Muddy water initially observed at toe of berm at downstream toe; cracks and sinkholes developed rapidly on berm and later on dam crest; dewatering was stopped on next day but flow continued to increase, reaching maximum of 1600 L/s, then reduced over 7 days
Lower Lliw	Great Britain	8	24	1867	1873	Internal erosion of puddle clay cutoff trench into fissured bedrock	"Trouble free service" for first 6 years of operation; seepage through drains under the downstream shoulder at 1.2–2.4 L/s, depending on rainfall; seepage attributed to natural springs	Seepage from drains under the downstream shoulder increased to highest previously observed (22 L/s) and was muddy; no other details available
Mogoto	South Africa	8	36	1924	1976	Piping of broadly graded fill materials into open-work colluvial foundation soils; concentrated leak at downstream toe took 3 days to plug; piping possibly initiated by upstream slip	Ongoing long-term settlements totalling 750 mm in 1976, with 170 mm in the period 1953–1976; sinkhole appeared on upstream slope 9 years prior to incident; waterline bulged upstream by about 600 mm directly opposite sinkhole	During a drilling investigation, plug of soil in former sinkhole dropped and continued to move downwards; at same time, a concentrated leak appeared at downstream toe, muddy and increasing; void found by drilling and grouting; took 3 days to seal the leak

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Table A6 (*Concluded*).

							Warning	
Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Long term	Short term
Wolf Creek	United States	1	61	1951	1967	Internal erosion of filling of solution channels in limestone and of embankment materials in cutoff trench into untreated limestone channels leading to sinkholes at downstream toe	Dam operated without any apparent distress for first 15 years of operation apart from a series of wet areas observed at downstream toe; small sinkhole found near downstream toe in 1967 investigation	Muddy flow observed from subsurface drainage pipes and from bedrock joint in tailrace downstream of powerhouse (when not in operation); 5 months later, sinkholes developed near downstream toe and muddy flows became more pronounced; reservoir drawn down

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Appendix H

Check Sheets for Dam Safety Expectations, Deficiencies and Priorities

Check Sheets for Dam Safety Expectations Deficiencies and Priorities

Deficiencies and non-conformances identified during the Dam Safety Review have been evaluated in accordance with the sample check sheet for Dam Safety Expectations Deficiencies and Priorities developed by BC MoE (May 2010). Deficiencies are classified into Actual Deficiencies and Potential Deficiencies and there is a variety of non-conformances. These classifications are described as follows.

Definitions of Deficiencies and Non-Conformances

1. Deficiencies
    a. Actual – An unacceptable dam performance condition has been confirmed, based on the CDA Guidelines, or other specified safety standard. Identification of an actual deficiency generally leads to an appropriate corrective action or directly to a capital improvement project:
        i. (An) Normal Load – Load which is expected to occur during the life of a dam.
        ii. (Au) Unlikely Load – Load which could occur under unusual load (large earthquake or flood).
    b. Potential – There is a reason to expect that an unacceptable condition might exist, but has not been confirmed. Identification of a potential deficiency generally leads to a Deficiency Investigation:
        i. (Pn) Normal Load – Load which is expected to occur during the life of a dam.
        ii. (Pu) Unlikely Load – Load which could occur under unusual load (large earthquake or flood).
        iii. (Pq) Quick – Potential deficiency that cannot be confirmed but can be readily eliminated by a specific action.
        iv. (Pd) Difficult - Potential deficiency that is difficult or impossible to prove or disprove.
2. Non-Conformances

    Established procedures, systems and instructions are not being followed, or, they are inadequate or inappropriate and should be revised:

    a. Operational (NCo), Maintenance (NCm), Surveillance (NCs).
    b. Information (NCi) – information is insufficient to confirm adequacy of dam or physical infrastructure for dam safety.
    c. Other Procedures (NCp) – other procedures, to be specified.

Table I2: **Dam Safety Expectations for the Stocking Lake Dam**

Dam Safety Expectations		Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
1.0	**Dam Safety Analysis**							
1.1	Records relevant to dam safety are available including design documents, historical instrument readings, inspection and testing reports, operational records and investigation results.			X			NCi	Limited inspection and operational records are available.
1.2	The Dam is classified appropriately in terms of the consequences of failure including life, environmental, cultural and third-party economic losses			X			NCo	Based on potential loss of life and economic consequences in the inundation zone it is recommended to increase the dam consequences classification to “High”.
1.3	Inundation study adequate to determine consequence classification. Flood and “sunny day” scenarios assessed.	X						Undertaken as part of this DSR.
1.4	Hazards external and internal to the dam have been defined.	X						Undertaken as part of this DSR.
1.5	The potential failure modes for the dam and the initial conditions downstream from the dam have been identified.	X						Undertaken as part of this DSR.
1.6	All other components of the water barrier (retaining walls, saddle dams, spillways, road embankments) are included in the dam safety management process.		X					
1.7	The MDE selected reflects current seismic understanding.	X						
1.8	The IDF is based on appropriate hydrological analyses.	X						
1.9	The dam is safely capable of passing flows as required for all applicable loading conditions (normal, winter, earthquake, and flood).	X						
1.10	The dam has adequate freeboard for all applicable operating conditions (normal, winter, earthquake, and flood).	X						
1.11	The analyses are current.	X						
1.12	The approach and exit channels of discharge facilities are adequately protected against erosion and free of any obstructions that could adversely affect the discharge capacity of the facilities.	X						The dam has a log boom.
1.13	The dams, abutments and foundations are not subject to unacceptable deformation or overstressing.	X						
1.14	Adequate filter and drainage facilities are provided to intercept and control the maximum anticipated seepage and to prevent internal erosion.			X	An			The dam is homogenous and susceptible to internal erosion.
1.15	Hydraulic gradients in the dams, abutments, foundations and along embedded structures are sufficiently low to prevent piping and instability.	X						
1.16	Slopes of an embankment have adequate protection against erosion, seepage, traffic, frost and burrowing animals	X						
1.17	Stability of reservoir slopes are evaluated under all conditions and unacceptable risk to public safety, the dam or its appurtenant structures is identified.	X						
1.18	The need for reservoir evacuation or emergency drawdown capability as a dam safety risk control measure has been assessed.			X				The reservoir does not have the ability to be drawn down rapidly.
2.0	**Operation, Maintenance and Surveillance**							
2.1	Responsibilities and authorities are clearly delegated within the organization for all dam safety activities.			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
2.2	Requirements for the safe operation, maintenance and surveillance of the dam are documented with sufficient information in accordance with the impacts of operation and the consequences of dam failure.			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
2.3	The OMS Manual is reviewed and updated periodically: when major changes to the structure, flow control equipment, operating conditions or company organizational structure and responsibilities have occurred.			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
2.4	Documented operating procedures for the dam and flow control equipment under normal, unusual and emergency conditions exist, are consistent with the OMS Manual and are followed.			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam.
	Operation							
2.5	Critical discharge facilities are able to operate under all expected conditions.	X						
a.	Flow control equipment is tested and is capable of operating as required.	X						

	Dam Safety Expectations	Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
b.	Normal and standby power sources, as well as local and remote controls, are tested.		X					
c.	Testing is on a defined schedule and test results are documented and reviewed.			X			NCo	No official testing records are available,
d.	Management of debris and ice is carried out to ensure operability of discharge facilities.	X		X				
2.6	Operating procedures take into account:							
a.	Outflow from upstream dams		X					
b.	Reservoir levels and rates of drawdown			X			NCo	No procedures for drawdown rates are available.
c.	Reservoir control and discharge during an emergency			X			NCo	No emergency procedures specific to Stocking Lake Dam are available.
d.	Reliable flood forecasting information	X						
e.	Operator safety			X			NCo	No safe work procedures were available.
	Maintenance							
2.7	The particular maintenance needs of critical components or subsystems, such as flow control systems, power supply, backup power, civil structures, drainage, public safety and security measures and communications and other infrastructure are identified.			X			NCm	Assumed to be a non-conformance as no supporting documentation provided.
2.8	Maintenance procedures are documented and followed to ensure that the dam remains in a safe and operational condition.			X			NCm	Assumed to be a non-conformance as no supporting documentation provided.
2.9	Maintenance activities are prioritized and carried out with due consideration to the consequences of failure, public safety and security.			X			NCm	Assumed to be a non-conformance as no supporting documentation provided.
	Surveillance							
2.10	Documented surveillance procedures for the dam and reservoir are followed to provide early identification and to allow for timely mitigation of conditions that might affect dam safety.	X						
2.11	The surveillance program provides regular monitoring of dam performance, as follows:							
a.	Actual and expected performances are compared to identify deviations.			X			NCs	Comparison of actual conditions to expected conditions documents were not available.
b.	Analysis of changes in performance, deviation from expected performance or the development of hazardous conditions.	X						
c.	Reservoir operations are confirmed to be in compliance with dam safety requirements.	X						
d.	Confirmation that adequate maintenance is being carried out.						NCs	Assumed to be a non-conformance as no supporting documentation provided.
2.12	The surveillance program has adequate quality assurance to maintain the integrity of data, inspection information, dam safety recommendations, training and response to unusual conditions.	X						
2.13	The frequency of inspection and monitoring activities reflects the consequences of failure, dam condition and past performance, rapidity of development of potential failure modes, access constraints due to weather or the season, regulatory requirements and security needs.	X						
2.14	Special inspections are undertaken following unusual events (if no unusual events then acknowledge that requirement to do so is documented in OMS).	X						
2.15	Training is provided so that inspectors understand the importance of their role, the value of good documentation, and the means to carry out their responsibilities effectively.			X			NCs	Assumed to be a non-conformance as no records of training are available.
2.16	Qualifications and training records of all individuals with responsibilities for dam safety activities are available and maintained.			X			NCs	Assumed to be a non-conformance as no records of training are available.
2.17	Procedures document how often instruments are read and by whom, where the instrument readings will be stored, how they will be processed, how they will be analyzed, what threshold values or limits are acceptable for triggering follow-up actions, what the follow-up actions should be and what instrument maintenance and calibration are necessary.			X			NCs	A OMS Manual needs to be prepared for Stocking Lake Dam
3.0	**Emergency Preparedness**							
3.1	An emergency management process is in place for the dam including emergency response procedures and emergency preparedness plans with a level of detail that is commensurate with the consequences of failure.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam

	Dam Safety Expectations	Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
3.2	The emergency response procedures outline the steps that the operations staff is to follow in the event of an emergency at the dam.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam
3.3	Documentation clearly states, in order of priority, the key roles and responsibilities, as well as the required notifications and contact information.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam
3.4	The emergency response procedures cover the full range of flood management planning, normal operating procedures and surveillance procedures.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam
3.5	The emergency management process ensures that effective emergency preparedness procedures are in place for use by external response agencies with responsibilities for public safety within the floodplain.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam
3.6	Roles and responsibilities of the dam owner and response agencies are defined.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam
3.7	Inundation maps and critical flood information are appropriate and are available to downstream response agencies.			X			NCp	Inundation maps included in this report should be incorporated into a DEP and provided to the downstream response agencies.
3.8	Exercises are carried out regularly to test the emergency procedures.			X			NCp	No documentation of training exercises is available.
3.9	Staff are adequately trained in the emergency procedures.			X			NCp	No documentation of training is available.
3.10	Emergency plans are updated regularly and updated pages are distributed to all plan holders in a controlled manner.			X			NCp	A Dam Emergency Plan (DEP) needs to be prepared for Stocking Lake Dam
4.0	**Dam Safety Review**							
4.1	A safety review of the dam ("Dam Safety Review") is carried out periodically based on the consequences of failure.	X						The CVRD commissioned this dam safety review. This is the first comprehensive dam safety review of this structure. Another dam safety review should be conducted in ten year (2028). The dam licenses should endeavour to implement the recommendations of this review before that time.
5.0	**Dam Safety Management System**							
5.1	The dam safety management system for the dam is in place incorporating:							
a.	Policies			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
b.	Responsibilities			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
c.	Plans and procedures including OMS, public safety and security			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
d.	Documentation			X			NCo	Documentation of inspections prior to 2016 are missing, other documentation is limited.
e.	Training and review			X			NCo	A OMS Manual needs to be prepared for Stocking Lake Dam
f.	Prioritization and correction of deficiencies and non-conformances	X						Prioritization of deficiencies are provided in this dam safety review.
g.	Supporting infrastructure	X						
5.2	Deficiencies are: documented, reviewed, and resolved in a timely manner. Decisions are justified and documented.			X			NCo	Prioritization of deficiencies are provided in this dam safety review.
5.3	Applicable regulations are met.			X			NCo	A OMS Manual & DEP needs to be prepared for Stocking Lake Dam

Appendix I

NDMP Risk Assessment Information Template

Public Safety Canada Sécurité publique Canada

Ottawa, Canada
K1A 0P8

UNCLASSIFIED

National Disaster Mitigation Program (NDMP)
Risk Assessment Information Template

Risk Event Details			
Start and End Date	Provide the start and end dates of the selected event, based on historical data.	Start Date: 20/08/2018	End Date: Ongoing
Severity of the Risk Event	Provide details about the risk, including: • Speed of onset and duration of event; • Level and type of damaged caused; • Insurable and non-insurable losses; and • Other details, as appropriate.	A comprehensive Dam Safety Review and Risk Assessment was undertaken of the Stocking Lake Dam in 2018 to meet the CVRD obligation as a water licensee under the BC Dam Safety Regulations. The dam safety review included a dam breach analysis, flood routing, inundation mapping and assessment of the performance of the dam structure to piping (internal erosion) failure and various meteorological and seismic hazards. Flood routing and inundation mapping indicates that hazards flow conditions downstream of the dam would occur within 3 to 10 hours of the initiation of dam breach. The results of the dam safety review and risk assessment indicated the following infrastructure is at risk in the event of a dam breach; 1. The TransCanada Highway 2. Water mains servicing the Town of Ladysmith and community of Saltair 3. Forestry service roads 4. S Watts Road 5. Chemainus Road Bridge over Stocking Creek 6. Fortis natural gas transmission line 7. Stocking Lake Creek Park 8. Southern Vancouver Island Railway	
Response During the Risk Event	Provide details on how the defined geographic area continued its essential operations while responding to the event.	N/A	

Recovery Method for the Risk Event	Provide details on how the defined geographic area recovered.	Recovery is anticipated to include the investigation, design and construction of a replacement dam structure that would meet the performance criteria contain within the BC Dam Safety Regulations, Canadian Dam Association Dam Safety Guidelines and Associated Technical Bulletins
Recovery Costs Related to the Risk Event	Provide details on the costs, in dollars, associated with implementing recovery strategies following the event.	Dam reconstruction and restoration of roads: $1,000,000 Potential additional costs
Recovery Time Related to the Risk Event	Provide details on the recovery time needed to return to normal operations following the event.	Unknown

Risk Event Identification and Overview	
Provide a qualitative description of the defined geographic area, including: • Watershed/community/region name(s); • Province/Territory; • Area type (i.e., city, township, watershed, organization, etc.); • Population size; • Population variances (e.g., significant change in population between summer and winter months); • Main economic areas of interest; • Special consideration areas (e.g., historical, cultural and natural resource areas); and an • Estimate of the annual operating budget of the area.	Watershed is the Stocking Creek watershed Ladysmith and Saltair Vancouver Island Region British Columbia Area type: Stocking Creek watershed Population Size of Saltair: 2,069 Population Variance: Unknown Main Economic interests: Forestry, Tourism, Agriculture Special Considerations: Stocking Creek Park, Stocking Lake Regional Park, Stocking Lake Trail Estimate of Annual Operating Budget: Unknown
Methodolgies, processes and analyses	
Provide the year in which the following processes/analyses were last completed and state the methodology(ies) used: • Hazard identification; • Vulnerability analysis; • Likelihood assessment; • Impact assessment; • Risk assessment; • Resiliency assessment; and/or • Climate change impact and/or adaptation assessment. *Note: It is recognized that many of the processes/analyses mentioned above may be included within one methodology.*	Analysis completed during the 2018 Comprehensive Dam Safety Review and Risk Assessment of the Stocking Lake Dam, prepared by Ecora Engineering & Resource Group Ltd. Report includes: Dam embankment stability analysis, liquefaction assessment, probabilistic piping assessment, dam breach assessment, dam hydrotechnical assessment including wind-wave analysis Hazards, vulnerability, likelihood, impact, risk are assigned as result of analysis.

Hazard Mapping

To complete this section:

- Obtain a map of the area that clearly indicates general land uses, neighbourhoods, landmarks, etc. For clarity throughout this exercise, it may be beneficial to omit any non-essential information from the map intended for use. Controlled photographs (e.g. aerial photography) can be used in place of or in addition to existing maps to avoid the cost of producing new maps.
- Place a grid over the maps/photographs of the area and assign row and column identifiers. This will help identify the specific area(s) that may be impacted, as well as additional information on the characteristics within and affecting the area.
- Identify where and how flood hazards may affect the defined geographic area.
- Identify the mapped areas that are most likely to be impacted by the identified flood hazard.

Map(s)/photograph(s) can also be used, where appropriate, to visually represent the information/prioritization being provided as part of this template.

Hazard identification and prioritization

List known or likely flood hazards to the defined geographic area in order of proposed priority. For example: (1) dyke breach overland flooding; (2) urban storm surge flooding ; and so on.	1. Dam breach of Stocking Lake Dam and overland flooding
Provide a rationale for each prioritization and the key information sources supporting this rationale.	1. The 2018 Comprehensive Dam Safety Review and Risk Assessment of the Stocking Lake Dam indicated that the dam in it's current form does not meet the performance criteria contain within the BC Dam Safety Regulations, Canadian Dam Association Dam Safety Guidelines and Associated Technical Bulletins and is at risk of structural failure due to internal erosion or a seismic event. 2018 Comprehensive Dam Safety Review and Risk Assessment of the Stocking Lake Dam, prepared by Ecora Engineering & Resource Group Ltd.

Risk Event Title

Identify the name/title of the risk. An example of a risk event name or title is: "A one-in-one hundred year flood following an extreme rain event."	Dam breach and overland flooding due to internal erosion caused by a 1 in 70 year event.

Type of Flood Hazard

Identify the type of flood hazard being described (e.g., riverine flooding, coastal inundation, urban run-off, etc.)	Riverine flooding and associated bank erosion. Failure and over topping of hydraulic structures.
Secondary hazards	
Describe any secondary effects resulting from the risk event (e.g., flooding that occurs following a hurricane).	Erosion and bank instabilities downstream of the dam failure due to elevated flows. Failure of road embankments where hydraulic structures are overwhelmed by breach flows.
Primary and secondary organizations for response	
Identify the primary organization(s) with a mandate related to a key element of a natural disaster emergency, and any supporting organization(s) that provide general or specialized assistance in response to a natural disaster emergency.	The Cowichen Valley Regional District, the Town of Ladysmith, the BC Ministry of Transportation & Infrastructure and Emergency Management BC would by the primary organizations with a mandate to respond to a natural disaster emergency at the subject site.

Risk Event Description	
Description of risk event, including risk statement and cause(s) of the event	
Provide a baseline description of the risk event, including: • Risk statement; • Context of the risk event; • Nature and scale of the risk event; • Lead-up to the risk event, including underlying cause and trigger/stimulus of the risk event; and • Any factors that could affect future events. *Note: The description entered here must be plausible in that factual information would support such a risk event.*	The primary risk event is the breach of Stocking Lake Dam due to internal (piping) erosion caused by a 1 in 70 year event. In the event of dam breach significant damage to public infrastructure would occur including damage to the TransCanada Highway, Fortis gas line supplying Victoria and water distribution lines servicing the Town of Ladysmith and the community of Saltair. Damage will extend to areas to the immediate proximity of Stocking Creek. The event would most likely occur in the spring freshet period when the lake levels and hydrostatic pressures within the dam are higher.

Location	
Provide details regarding the area impacted by the risk event such as: • Province(s)/territory(ies); • Region(s) or watershed(s); • Municipality(ies); • Community(ies); and so on.	Stocking Lake and Stocking Lake are located on the eastern side of Vancouver Island. Creek passes between the communities of Ladysmith, BC and Saltair, BC. A dam breach has the potential to disrupt transportation between the north and south parts of the island. Damage may also cause disruption of water supply to Saltair and Ladysmith. Damage to Fortis natural gas pipeline may disrupt supply of gas some areas on Vancouver Island
Natural environment considerations	
Document relevant physical or environmental characteristics of the defined geographic area.	The Stocking Lake watershed is heavily forested around the creek, logging has taken place in areas close proximity. Elevation of catchment varies from 640 m to sea level. Development and infrastructure threatened by a dam breach is primarily in the lower bounds of the catchment.
Meteorological conditions	
Identify the relevant meteorological conditions that may influence the outcome of the risk event.	Relevant meteorological conditions may include: - High snowpack in the Stocking Lake watershed - High temperatures as snow thaws - Extreme rainfall - Extreme rain on snow

Seasonal conditions	
Identify the relevant seasonal changes that may influence the outcome of the risk assessment of a particular risk event.	Relevant seasonal conditions may include: - Extreme precipitation - Wood debris in the dam spillway - Changing watershed conditions due wildfire, logging and other factors
Nature and vulnerability	
Document key elements related to the affected population, including: • Population density; • Vulnerable populations (identify these on the hazard map from step 7); • Degree of urbanization; • Key local infrastructure in the defined geographic area; • Economic and political considerations; and • Other elements, as deemed pertinent to the defined geographic area.	Population density for Saltair: 305.1 people per square km. Hazardous area is identified on hazards maps included with 2018 Comprehensive Dam Safety Review and Risk Assessment completed by Ecora. Area around creek is mostly rural with development only existing in the lower ranges of the catchment. Key local infrastructure: 1. The TransCanada Highway 2. Water mains servicing Town of Lady Smith and community of Saltair 3. Forestry Service Roads, 4. S Watts road 5. Chemainus Road Bridge over Stocking Creek 6. Fortis natural gas transmission line 7. Southern Vancouver Island Railway Economic and political considerations: A dam breach will impact the major infrastructure providing services between north and southern areas of Vancouver Island. Breach would impact local parks.

Asset inventory	
Identify the asset inventory of the defined geographic area, including: • Critical assets; • Cultural or historical assets; • Commercial assets; and • Other area assets, as applicable to the defined geographic area. Key asset-related information should also be provided, including: • Location on the hazard map (from step 7); • Size; • Structure replacement cost; • Content value; • Displacement costs; • Importance rating and rationale; • Vulnerability rating and reason; and • Average daily cost to operate. A total estimated value of physical assets in the area should also be provided.	Key Local assets that are within the high hazard area include: 1. The TransCanada Highway 2. Water distribution mains for Town of Ladysmith and community of Saltair 3. Local forestry roads 4. S Watts Road 5. Chemainus Road Bridge over Stocking Creek 6. Fortis natural gas transmission line 7. Stocking Lake Creek Park Possible further damage from overland damage to areas in medium hazard areas. No detailed cost estimate has taken place, however total impact cost is estimated to be between $3 million and $30 million for total reconstruction costs based on the scope of infrastructure. Daily cost to operate infrastructure is unknown and it is unknown on how long infrastructure will be out of service
Other assumptions, variability and/or relevant information	
Identify any assumptions made in describing the risk event; define details regarding any areas of uncertainty or unpredictability around the risk event; and supply any supplemental information, as applicable.	A breach of the dam could be the result of a number of scenarios and thus it is difficult to say which scenario would be the first that causes dam failure. As per Canadian Dam Association (CDA) guidelines the most conservative scenario was considered. Dam breach analysis conservatively assumed that spillway was blocked by debris resulting in an over-topping event that causes dam failure. Some variation between the modeled breach and a real breach may exist due to variations in terrain that may not entirely captured in the digital terrain model used.
Existing Risk Treatment Measures	
Identify existing risk treatment measures that are currently in place within the defined geographic area to mitigate the risk event, and describe the sufficiency of these risk treatment measures.	It is anticipated that culverts and bridge on Stocking Creek would at most be sized for a 200-year flood event. It is anticipated that dam breach peak flow will be much greater than the 200-year event and as such it is expected that the infrastructure will fail during a dam breach.

Likelihood Assessment

Return Period

Identify the time period during which the risk event might occur. For example, the risk event described is expected to occur once every X number of years. Applicants are asked to provide the X value for the risk event.	Three risk events that could result in failure of the dam structure were assessed, namely; failure due to internal (piping) erosion, failure due to over topping by a meteorological event and failure due to a seismic event. Failure of the dam structure due to; internal erosion was assessed to be a 1 in 70 year event, over topping by a meteorological event was assessed to be greater than 1 in 5000 years and, a seismic event greater than 1 in 100 years but less than 1 in 500 years.

Period of interest

Applicants are asked to determine and identify the likelihood rating (i.e. period of interest) for the risk event described by using the likelihood rating scale within the table below.

Likelihood Rating	Definition	
5	The event is expected and may be triggered by conditions expected over a 30 year period.	3
4	The event is expected and may be triggered by conditions expected over a 30 - 50 year period.	
3	The event is expected and may be triggered by conditions expected over a 50 - 500 year period.	
2	The event is expected and may be triggered by conditions expected over a 500 - 5000 year period.	
1	The event is possible and may be triggered by conditions exceeding a period of 5000 years.	

Provide any other relevant information, notes or comments relating to the likelihood assessment, as applicable.	Failure of the dam structure due to either internal erosion or a seismic event is expected by conditions expected over a 50 - 500 year period.

Impacts/Consequences Assessment

There are 12 impacts categories within 5 impact classes rated on a scale of 1 (least impacts) to 5 (greatest impact). Conduct an assessment of the impacts associated with the risk event, and assign one risk rating for each category. Additional information may be provided for each of the categories in the supplemental fields provided.

A) People and societal impacts

	Risk Rating	Definition	Assigned risk rating
Fatalities	5	Could result in more than 50 fatalities	2
	4	Could result in 10 - 49 fatalities	
	3	Could result in 5 - 9 fatalities	
	2	Could result in 1 - 4 fatalities	
	1	Not likely to result in fatalities	
Supplemental information (optional)	No Permanent Population at Risk (PAR) was identified within the dam inundation zone, however transient population such as road users would be in the inundation zone that could result in fatalities.		
Injuries	5	Injuries, illness and/or psychological disablements cannot be addressed by local, regional, or provincial/territorial healthcare resources; federal support or intervention is required	2
	4	Injuries, illnesses and/or psychological disablements cannot be addressed by local or regional healthcare resources; provincial/territorial healthcare support or intervention is required.	
	3	Injuries, illnesses and/or psychological disablements cannot be addressed by local or regional healthcare resources additional healthcare support or intervention is required from other regions, and supplementary support could be required from the province/territory	
	2	Injuries, illnesses and/or psychological disablements cannot be addressed by local resources through local facilities; healthcare support is required from other areas such as an adjacent area(ies)/municipality(ies) within the region	
	1	Any injuries, illnesses, and/or psychological disablements can be addressed by local resources through local facilities; available resources can meet the demand for care	
Supplemental information (optional)	Closest hospital to impacted area is Nanaimo Regional General Hospital approximately 30 km away. Hospital is expected to have sufficient resources.		

		Risk Rating	Definition	Assigned risk rating
Displacement	Percentage of displaced individuals	5	> 15% of total local population	1
		4	10 - 14.9% of total local population	
		3	5 - 9.9% of total local population	
		2	2 - 4.9% of total local population	
		1	0 - 1.9% of total local population	
	Duration of displacement	5	> 26 weeks (6 months)	4
		4	4 weeks - 26 weeks (6 months)	
		3	1 week - 4 weeks	
		2	72 hours - 168 hours (1 week)	
		1	Less than 72 hours	
Supplemental information (optional)		The primary impact will be to a temporary population and thus it is expect to only affect a very small fraction of the population. The duration of displacement could be a number weeks as road access is restored and other infrastructure is repaired.		
B) Environmental impacts				
		5	> 75% of flora or fauna impacted or 1 or more ecosystems significantly impaired; Air quality has significantly deteriorated; Water quality is significantly lower than normal or water level is > 3 meters above highest natural level; Soil quality or quantity is significantly lower (i.e., significant soil loss, evidence of lethal soil contamination) than normal; > 15% of local area is affected	2
		4	40 - 74.9% of flora or fauna impacted or 1 or more ecosystems considerably impaired; Air quality has considerably deteriorated; Water quality is considerably lower than normal or water level is 2 - 2.9 meters above highest natural level; Soil quality or quantity is moderately lower than normal; 10 - 14.9% of local area is affected	
		3	10 - 39.9% of flora or fauna impacted or 1 1 or more ecosystems moderately impaired; Air quality has moderately deteriorated; Water quality is moderately lower than normal or water level is 1 - 2 meters above highest natural level; Soil quality is moderately lower than normal; 6 - 9.9 % of area affected	

	2	< 10 % of flora or fauna impacted or little or no impact to any ecosystems; Little to no impact to air quality and/or soil quality or quantity; Water quality is slightly lower than normal, or water level is less than 0.9 meters above highest natural level and increased for less than 24 hours; 3 - 5.9 % of local area is affected	
	1	Little to no impact to flora or fauna, any ecosystems, air quality, water quality or quantity, or to soil quality or quantity; 0 - 2.9 % of local area is affected	
Supplemental information (optional)	Elevated water levels are expected for a period of less than 24 hours as the flood wave moves downstream.		

C) Local economic impacts

	Risk Rating	Definition	Assigned risk rating
	5	> 15 % of local economy impacted	5
	4	10 - 14.9 % of local economy impacted	
	3	6 - 9.9 % of local economy impacted	
	2	3 - 5.9 % of local economy impacted	
	1	0 - 2.9 % of local economy impacted	
Supplemental information (optional)	Impacts to the roads will impact the ability to access either side of the creek with the potential to impact a large section of the community.		

D) Local infrastructure impacts			
	Risk Rating	**Definition**	**Assigned risk rating**
Transportation	5	Local activity stopped for more than 72 hours; > 20% of local population affected; lost access to local area and/or delivery of crucial service or product; or having an international level impact	2
	4	Local activity stopped for 48 - 71 hours; 10 - 19.9% of local population affected; significantly reduced access to local area and/or delivery of crucial service or product; or having a national level impact	
	3	Local activity stopped for 25 - 47 hours; 5 - 9.9% of local population affected; moderately reduced access to local area and/or delivery of crucial service or product; or having a provincial/territorial level impact	
	2	Local activity stopped for 13 - 24 hours; 2 - 4.9% of local population affected; minor reduction in access to local area and/or delivery of crucial service or product; or having a regional level impact	
	1	Local activity stopped for 0 - 12 hours; 0 - 1.9% of local population affected; little to no reduction in access to local area and/or delivery of crucial service or product	
Supplemental information (optional)	The impacts to infrastructure are expect to have impacts on a regional level. Traffic moving between areas to the north and south parts of Vancouver Island will be disrupted.		
Energy and Utilities	5	Duration of impacts > 72 hours; > 20% of local population without service or product; or having an international level impact	5
	4	Duration of impact 48 - 71 hours; 10 - 19.9% of local population without service or product; or having a national impact	
	3	Duration of impact 25 - 47 hours; 5 - 9.9% of local population without service or product; or having a provincial/territorial level impact	
	2	Duration of impact 13 - 24 hours; 2 - 4.9% of local population without service or product; or having a regional level impact	
	1	Local activity stopped for 0 - 12 hours; 0 - 1.9% of local population affected; little to no reduction in access to local area and/or delivery of crucial service or product	

Supplemental information (optional)	Duration of impacts are expected to be greater than 72 hours. Infrastructure will need repairs which will take more than three days to fully rehabilitate. Possible disruption of natural gas supply to southern Vancouver Island.		
Information and Communications Technology	5	Service unavailable for > 72 hours; > 20 % of local population without service; or having an international level impact	2
	4	Service unavailable for 48 - 71 hours; 10 - 19.9 % of local population without service; or having a national level impact	
	3	Service unavailable for 25 - 47 hours; 5 - 9.9 % of local population without service; or having a provincial/territorial level impact	
	2	Service unavailable for 13 - 24 hours; 2 - 4.9 % of local population without service; or having a regional level impact	
	1	Service unavailable for 0 - 12 hours; 0 - 1.9 % of local population without service	
Supplemental information (optional)			
Health, Food, and Water	5	Inability to access potable water, food, sanitation services, or healthcare services for > 72 hours; non - essential services cancelled; > 20 % of local population impacted; or having an international level impact	5
	4	Inability to access potable water, food, sanitation services, or healthcare services for 48 - 72 hours; major delays for nonessential services; 10 - 19.9 % of local population impacted; or having a national level impact	
	3	Inability to access potable water, food, sanitation services, or healthcare services for 25 - 48 hours; moderate delays for nonessential services; 5 - 9.9 % of local population impacted; or having a provincial/territorial level impact	
	2	Inability to access potable water, food, sanitation services, or healthcare services for 13 - 24 hours; minor delays for nonessential; 2 - 4.9 % of local population impacted; or having a regional level impact	
	1	Inability to access potable water, food, sanitation services, or healthcare services for 0 - 12 hours; 0 - 1.9 % of local population impacted	

Supplemental information (optional)			
Safety and Security	5	> 20 % of local population impacted; loss of intelligence or defence assets or systems for > 72 hours; or having an international level impact	2
	4	10 - 19.9 % of local population impacted; loss of intelligence or defence assets or systems for 48 – 71 hours; or having a national level impact	
	3	5 - 9.9 % of local population impacted; loss of intelligence or defence assets or systems for 25 – 47 hours; or having a provincial/territorial level impact	
	2	2 - 4.9 % of local population impacted; loss of intelligence or defence assets or systems for 13 – 24 hours; or having a regional level impact	
	1	0 - 1.9 % of local population impacted; loss of intelligence or defence assets or systems for 0 – 12 hours	
Supplemental information (optional)			

E) Public sensitivity impacts

	Risk Rating	Definition	Assigned risk rating
	5	Sustained, long term loss in reputation/public perception of public institutions and/or sustained, long term loss of trust and confidence in public institutions; or having an international level impact	3
	4	Significant loss in reputation/public perception of public institutions and/or significant loss of trust and confidence in public institutions; significant resistance; or having a national level impact	
	3	Some loss in reputation/public perception of public institutions and/or some loss of trust and confidence in public institutions; escalating resistance	
	2	Isolated/minor, recoverable set - back in reputation, public perception, trust, and/or confidence of public institutions	
	1	No impact on reputation, public perception, trust, and/or confidence of public institutions	
Supplemental information (optional)			

Confidence Assessment

Based on the table below, indicate the level of confidence regarding the information entered in the risk assessment information template in the "Confidence Level Assigned" column. Confidence levels are language - based and range from A to E (A=most confident to E=least confident).

Confidence Level	Definition	Confidence Level Assigned
A	Very high degree of confidence Risk assessment used to inform the risk assessment information template was evidence - based on a thorough knowledge of the natural hazard risk event; leveraged a significant quantity of high - quality data that was quantitative and qualitative in nature; leveraged a wide variety of data and information including from historical records, geospatial and other information sources; and the risk assessment and analysis processes were completed by a multidisciplinary team with subject matter experts (i.e., a wide array of experts and knowledgeable individuals on the specific natural hazard and its consequences) Assessment of impacts considered a significant number of existing/known mitigation measures	
B	High degree of confidence Risk assessment used to inform the risk assessment information template was evidence - based on a thorough knowledge of the natural hazard risk event; leveraged a significant quantity of data that was quantitative and qualitative in nature; leveraged a wide variety of data and information including from historical records, geospatial and other information sources; and the risk assessment and analysis processes were completed by a multidisciplinary team with some subject matter expertise (i.e., a wide array of experts and knowledgeable individuals on the specific natural hazard and its consequences) Assessment of impacts considered a significant number of potential mitigation measures	

C	Moderate confidence Risk assessment used to inform the risk assessment information template was moderately evidence - based from a considerable amount of knowledge of the natural hazard risk event; leveraged a considerable quantity of data that was quantitative and/or qualitative in nature; leveraged a considerable amount of data and information including from historical records, geospatial and other information sources; and the risk assessment and analysis processes were completed by a moderately sized multidisciplinary team, incorporating some subject matter experts (i.e., a wide array of experts and knowledgeable individuals on the specific natural hazard and its consequences) Assessment of impacts considered a large number of potential mitigation measures	B
D	Low confidence Risk assessment used to inform the risk assessment information template was based on a relatively small amount of knowledge of the natural hazard risk event; leveraged a relatively small quantity of quantitative and/or qualitative data that was largely historical in nature; may have leveraged some geospatial information or information from other sources (i.e., databases, key risk and resilience methodologies); and the risk assessment and analysis processes were completed by a small team that may or may not have incorporated subject matter experts (i.e., did not include a wide array of experts and knowledgeable individuals on the specific natural hazard and its consequences). Assessment of impacts considered a relatively small number of potential mitigation measures	
E	Very low confidence Risk assessment used to inform the risk assessment information template was not evidence - based; leveraged a small quantity of information and/or data relating to the natural risk hazard and risk event; primary qualitative information used with little to no quantitative data or information; and the risk assessment and analysis processes were completed by an individual or small group of individuals little subject matter expertise (i.e., did not include a wide array of experts and knowledgeable individuals on the specific natural hazard and its consequences). Assessment of impacts did not consider existing or potential mitigation measures	

Rationale for level of confidence

Provide the rationale for the selected confidence level, including any references or sources to support the level assigned.	Intrusive investigations were undertaken of the existing dam structure to provide inputs for engineering analysis undertaken as part of the Comprehensible Dam Safety Review and Risk Assessment. The Risk Assessment considered multiple risk events including the probabilistic (5th Generation) seismic hazard model developed by the Geological Survey of Canada (GSC) (Halchuk, Adams and Allen, 2015) that forms the basis of the seismic design provisions of the 2015 National Building Code of Canada (NBCC, 2015). Only one mitigation measure was considered as rehabilitation of the existing dam structure is the most logical option.

Key Information Sources	
Identify all supporting documentation and information sources for qualitative and quantitative data used to identify risk events, develop the risk event description, and assess impacts and likelihood. This ensures credibility and validity of risk information presented as well as enables referencing back to decision points at any point in time. Clearly identify unclassified and classified information.	Comprehensive Dam Safety Review and Risk Assessment of the Stocking Lake Dam, prepared by Ecora Engineering & Resource Group Ltd. in draft form 2018. Unclassified
Description of the risk analysis team	
List and describe the type and level of experience of each individual who was involved with the completion of the risk assessment and risk analysis used to inform the information contained within this risk assessment information template.	Michael J. Laws, P.Eng. Senior Geotechnical & Dam Safety Engineer Dr. Adrian Chantler, P.Eng. Senior Hydrotechnical Engineer

Appendix J

Dam Safety Assurance Statement

APPENDIX C1: DAM SAFETY REVIEW ASSURANCE STATEMENT – WATER RESERVOIR DAMS

Note: This statement is to be read and completed in conjunction with the current *APEGBC Professional Practice Guidelines – Legislated Dam Safety Reviews in British Columbia*, ("APEGBC Guidelines") and is to be provided for dam safety review reports for the purposes of the *Dam Safety Regulation*, BC Reg. 40/2016 as amended. Italicized words are defined in the APEGBC Guidelines.

To: The Owner(s) Date: November 28, 2018

Cowichan Valley Regional District

Name

175 Ingram Street

Duncan, BC V9L 1N8

Address

With reference to the *Dam Safety Regulation*, B.C. Reg. 40/2016 as amended.

For the dam:

UTM (Location): 440052 E 5422991 N (Zone 10)

Located at (Description): Banon Forest Service Road, Ladysmith, BC

Name of dam or description: Stocking Lake Dam

Provincial dam number: D720127-00

Dam function: Domestic Water Supply

Owned by: Cowichan Valley Regional District

(the "Dam")

Current Dam classification is:

Check one

☐ Low
☑ Significant
☐ High
☐ Very High
☐ Extreme

The undersigned hereby gives assurance that he/she is a Qualified Professional Engineer.

I have signed, sealed and dated the attached dam safety review report on the Dam in accordance with the APEGBC Guidelines. That report must be read in conjunction with this Statement. In preparing that report I have:

Check to the left of applicable items (see Guideline Section 3.2):

✓ 1. Collected and reviewed available and relevant background information, documentation and data

✓ 2. Understood the current classification for the Dam, including performance expectations

✓ 3. Undertaken an initial facility review

✓ 4. Reviewed and assessed the Dam safety management obligations and procedures

✓ 5. Reviewed the condition of the Dam, reservoir and relevant upstream and downstream portions of the river

✓ 6. Interviewed operations and maintenance personnel

✓ 7. Reviewed available maintenance records, the Operations, Maintenance and Surveillance (OMS) Manual and the Dam Emergency Plan

✓ 8. Confirmed proper functioning of flow control equipment

✓ 9. After the above, reassess the consequence classification, including the identification of required dam safety criteria

✓ 10. Carried out a dam safety analysis based on the classification in 9. above

✓ 11. Evaluated facility performance

✓ 12. Identified, characterized and determined the severity of deficiencies in the safe operation of the Dam and non-conformances in dam safety management system

✓ 13. Recommended and prioritized actions to be taken in relation to deficiencies and non-conformances

✓ 14. Prepared a dam safety review report for submittal to the regulatory authority by the Owner and reviewed the report with the Owner

✓ 15. The dam safety review report has been reviewed in meeting the intent of APEGBC Bylaw 14(b)(2)

Based on my dam safety review, the current dam classification is:

Check one

☐ Appropriate

☑ Should be reviewed and amended

I undertook the following type of dam safety review:

Check one

☐ Audit

☑ Comprehensive

☐ Detailed design-based multi-disciplinary

☐ Comprehensive, detailed design and performance

I hereby give my assurance that, based on the attached dam safety review report, at this point in time:

Check one

☐ The Dam is reasonably safe in that the dam safety review did not reveal any unsafe or unacceptable conditions in relation to the design, construction, maintenance and operation of the Dam as set out in the attached dam safety review report

☐ The Dam is reasonably safe but the dam safety review did reveal non-conformances with the *Dam Safety Regulation* as set out in section(s) ____ of the attached dam safety review report.

☐ The Dam is reasonably safe but the dam safety review did reveal deficiencies and non-conformances as set out in section(s) ____ of the attached dam safety review report.

☑ The Dam is not safe in that the dam safety review did reveal deficiencies and/or non-conformances which require urgent action as set out in section(s) ____ of the attached dam safety review report.
12.5, 14 and 15

Michael J. Laws, P.Eng.
Name

November 28, 2018
Date

Signature

579 Lawrence Avenue, Kelowna, BC V1Y 6L8
Address

250.469.9757
Telephone

M. J. LAWS
36691

(Affix Professional Seal here)

If the Qualified Professional Engineer is a member of a firm, complete the following:

I am a member of the firm Ecora Engineering & Resource Group Ltd.
and I sign this letter on behalf of the firm. (Print name of firm)

Appendix K

Statement of General Conditions – Geotechnical

Standard of Care

Ecora Engineering and Resource Group Ltd. (Ecora) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report

This report and the recommendations contained in it are intended for the sole use of Ecora's Client. Ecora does not accept any responsibility for the accuracy of any of the data, the analyses or the recommendations contained or referenced in the report when the report is used or relied upon by any party other than Ecora's Client unless otherwise authorized in writing by Ecora. Any unauthorized use of the report is at the sole risk of the user. 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.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of Ecora. Additional copies of the report, if required, may be obtained upon request.

Alternate Report Format

Where Ecora submits both electronic file and hard copy versions of reports, drawings and other project-related documents, only the signed and/or sealed versions shall be considered final and legally binding. The original signed and/or sealed version archived by Ecora shall be deemed to be the original for the Project. Both electronic file and hard copy versions of Ecora's deliverables shall not, under any circumstances, no matter who owns or uses them, be altered by any party except Ecora.

Soil, Rock and Groundwater Conditions

Classification and identification of soils, rocks and geological units have been based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Ecora does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities such as traffic, excavation, groundwater level lowering, pile driving, blasting on the site or on adjacent sites. Excavation may expose the soils to climatic elements such as freeze/thaw and wet /dry cycles and/or mechanical disturbance which can cause severe deterioration. Unless otherwise indicated the soil must be protected from these changes during construction.

Environmental and Regulatory Issues

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Sample Disposal

Ecora will dispose all soil and rock samples for 30 days following issue of this report. Further storage or transfer of samples can be made at the Client's expense upon written request, otherwise samples will be discarded.

Construction Services

During construction, Ecora should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Ecora's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Ecora's report. Adequate field review, observation and testing during construction are necessary for Ecora to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Ecora's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Job Site Safety

Ecora is responsible only for the activities of our employees on the jobsite. The presence of Ecora's personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that Ecora never occupy a position of control of the site. The Client undertakes to inform Ecora of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general.

Changed Conditions and Drainage

Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Ecora be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Ecora be employed to visit the site with sufficient frequency to detect if conditions have changed significantly. Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Ecora takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

Services of Sub consultants and Contractors

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. Ecora may arrange the hiring of these services as a convenience to our Clients. As these services are for the Client's benefit, the Client agrees to hold the Company harmless and to indemnify and defend Ecora from and against all claims arising through such hiring's to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.