Regional District of Okanagan-Similkameen

ISSUED FOR USE

DAM SAFETY REVIEW — BIG MEADOW LAKE DAM

K13101459.001

December 17, 2010

EBA Engineering Consultants Ltd.
p. 250.862.4832 • f. 250.862.2941
150, 1715 Dickson Avenue • Kelowna, British Columbia V1Y 9G6 • CANADA

TABLE OF CONTENTS

PAGE

TABLE OF CONTENTS

FIGURES

PHOTOGRAPHS

APPENDICES

1.0 INTRODUCTION

1.1 GENERAL

EBA, A Tetra Tech Company (EBA) was engaged by the Regional District of Okanagan-Similkameen (RDOS) to undertake dam safety reviews of its four Naramata area dams, namely;

- Big Meadow Lake Dam
- Elinor Lake North (Saddle) Dam
- Elinor (Eleanor) Lake South Dam
- Naramata Lake Dam

The four dams form three interconnected reservoirs that have provided a historical upland source of potable water to the Township of Naramata. The dams were originally constructed during the first half of the twentieth century by the Naramata Irrigation District (NID), which has been subsequently incorporated into the RDOS. With the recent commissioning of a new water treatment facility in the township that draws water from Lake Okanagan, the dams are no longer required for the supply of potable water and the RDOS is considering maintaining these facilities for irrigation purposes only.

This report presents the technical findings of the Big Meadow Lake Dam Safety Review (DSR) and it is understood that this is the first dam safety review of this facility. The technical findings of the dam safety reviews for the other Naramata dams are presented in companion reports, with the key findings for each dam safety review presented in a summary report.

The Dam Safety Review was undertaken in general accordance with the requirements of the British Columbia Water Act (1998), the British Columbia Ministry of Environment (BC MoE) Dam Safety Review Guidelines (May 2010), the Canadian Dam Association (CDA) Dam Safety Guidelines (2007), the Interim Consequence Classification Policy For Dams in British Columbia (February 2010) and the BC Dam Safety Regulation (February 2000). It is noted that the BC Regulations take precedence over the CDA Guidelines.

1.2 SITE DESCRIPTION

Big Meadow Lake Dam is situated within a bowl shaped feature near the headwaters of the Chute Creek catchment, approximately 13 km to the northeast of Naramata Township.

Reference to iMap on the BC MoE Water Stewardship website indicates that the dam is approximately 256 m long and 6.7 m high at its maximum height with a design crest elevation of 1613.9 m above mean sea level. Vehicle access to the dam is provided via Arawana Road, which extends off North Naramata Road to the southwest.

A diversion structure is situated downstream of Big Meadow Lake Dam, which can divert flow from Chute Creek into the downstream Elinor Lake reservoir.

A location plan showing the position of the dam relative to the other Naramata dams and Lake Okanagan is attached as Figure 1.

2.0 SCOPE OF WORK

EBA's scope of work for the Dam Safety Review was outlined in our proposal, dated June 30, 2010, which was accepted by the RDOS. In summary, the study included the following tasks:

- Background review;
- Site reconnaissance;
- Review of consequence classification;
- Hydrotechnical analysis including hydrological analysis, flood routing and hydraulics;
- Geotechnical assessment, including embankment stability and seepage;
- Review of Operation, Maintenance and Surveillance Manual;
- Review of Emergency Preparedness Plan;
- Review of any public safety management strategies;
- Assessment of compliance with previous reviews;
- Assessment of compliance with CDA Principles; and
- Development of conclusions and recommendations.

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

3.0 BACKGROUND REVIEW

3.1 SOURCES OF INFORMATION

The following sources of background information were reviewed prior to the site reconnaissance:

- Historic air photos;
- Readily available published sources of geological data;
- RDOS files and discussions with RDOS staff familiar with the site; and
- British Columbia Ministry of the Environment (BC MoE) Dam Safety Branch files;

The search of BC MoE files was undertaken by RDOS and provided to EBA; therefore this has been considered one combined source of information. We understand that this search may have only been of information held at MoE files in Penticton and didn't include a search of MoE files in Victoria which we understand to be a very good archive of dam information for all of British Columbia.

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

3.2 HISTORICAL AERIAL PHOTOS

The review of historical aerial photographs of the Naramata area held by the Geography Department of the University of British Columbia (UBC) included aerial photographs for the years 1938, 1959, 1969, 1970, 1972, 1985 and 1992.

3.3 GEOLOGICAL SETTING

Reference to the Geological Survey of Canada Map Surficial Geology Kooteney Lake (1984) indicates that natural subsoil conditions at all four dam sites are anticipated to comprise Sandy Till overlying crystalline metamorphic bedrock

The Sandy Till is described as a olive grey, grey to pale grey, weakly calcareous to calcareous loamy sand, sandy loam and loam, generally gravelly, cobbly or bouldery. It is mainly massive but may contain lenses of stratified sediments. It occurs as a blanket deposit with surface relief due to the shape of the underlying surface. The thickness of the soil unit varies from up to 30 m in the valley bottoms to no more than 5 m thick. Clast lithlogies reflect local bedrock which comprises mainly crystalline metamorphic and granitic rock.

The surficial geology of the Naramata dams' area is shown on the attached Figure 1.

3.4 SEISMICITY

In terms of Table 4.1.8.4.A of the National Building Code (NBC), a seismic site classification of Class C "Very Dense Soil and Soft Rock" is considered appropriate for the four Naramata dam sites.

Reference Peak Ground Accelerations (PGA) and Spectral Accelerations ($S_a(T)$) as obtained from the Earthquakes Canada website (http://earthquakescanada.nrcan.gc.ca) for a "Class C" site and a 1/2475 year earthquake return period at the location of the dams are provided in Table 1 below.

TABLE 1: REFERENCE (CLASS C) DESIGN PGA AND S_A FOR 1/2475 YEAR RETURN PERIOD

Structure	PGA	S_a (0.2)	S_a (0.5)	S_a (1.0)	S_a (2.0)
Big Meadow Lake Dam	0.138g	0.278	0.175	0.101	0.060

Reference Peak Ground Accelerations (PGA) Spectral and Spectral Accelerations ($S_a(T)$) for other earthquake return periods are provided on the attached Figure 2.

3.5 EXISTING DRAWINGS

A review of the provided existing drawing for the dam, as summarized in Appendix A1 indicates the following details.

- The dam has a design crest elevation of 5295 feet (1613.9 m), crest width of 12 feet (3.7 m), an embankment length of 817 feet (249 m) and a maximum embankment height of 7.5 m measured from the downstream toe to the crest, which is close to the registered (iMap) length and height of 256 m and 6.7 m respectively.

- The upstream and downstream slopes have a design profile of 3H:1V and 2.5H:1V respectively.
- The spillway structure has a design sill elevation of 5289 feet (1612.1 m).
- The low level outlet has an inlet elevation of 5271.4 feet (1606.7 m) and an outlet elevation of 5270.3 feet (1606.4 m); however, it is unclear whether this was the level of the original concrete culvert or of the 18 inch diameter steel pipe that was grouted inside the original concrete culvert in 1952.

3.6 DESIGN AND CONSTRUCTION

There is limited information available with respect to the design and construction of Big Meadow Lake Dam. The information available at the time of the dam safety review on the construction and history of the dam is listed in Appendix A. Review of the existing drawings indicates that it is a granular earthfill embankment dam with a central concrete core wall.

There is some uncertainty as to when Big Meadow Lake Dam was constructed. The existing drawings suggest that is was originally constructed in 1920; however, the dam information board states that it was constructed in 1933. A review of the oldest available aerial photography from 1938 indicates that the embankment had been constructed; however, it appears that the reservoir had not been filled yet suggesting that the 1933 date is probably the correct time of construction.

The original embankment was designed with 2H:1V downstream and upstream slopes with modifications undertaken in 1952 comprising flattening to a 3H:1V upstream slope and 2.5H:1V downstream slope, raising the core wall 12 inches (305 mm) and an 18 inch (457.2 mm) diameter steel discharge conduit was grouted into the original concrete culvert.

After reconstruction in 1952, sloughing and seepage occurred at the toe of the embankment adjacent to the low level outlet structure and subsequently, a rock toe drain was installed to collect the seepage. In 1964, a new sliding gate control was installed as the original gate had been undermined and was inoperative.

Following inspection of the dam in 1991, which noted significant seepage along the toe of the embankment, the installation of a 1.0 m deep granular toe drain incorporating a 0.3 m diameter perforated pipe was recommended; however, it is unknown if these works were undertaken and there was no evidence of these works observed during the dam inspection.

During the Okanagan Mountain Park fire of 2003, a fire guard was constructed to the northwest of the dam resulting in some minor excavation into the downstream face (resulting in a loss of freeboard) to provide vehicle access around the spillway structure. Following repair works to the embankment, an area of significant seepage was noted at the toe of the embankment towards the left abutment and a 0.3 m wide x 0.3 m deep, 25 mm drain rock toe drain was recommended to be constructed in 2004 to intercept the seepage in this area. It is assumed that the shallow toe drainage observed during the dam inspection was as a result of this recommendation.

A plan of Big Meadow Lake Dam, embankment profile and section at the maximum height of the dam are shown on the attached Figures 3 and 4.

3.7 DAM INSPECTION REPORTS

A review was undertaken of available inspection reports, listed in Appendix A undertaken by BC MoE, RDOS and engineering consultants. Key point's from EBA's review of the dam inspection reports are as follows;

- After reconstruction of the dam in 1952, sloughing and seepage was observed at the toe of the embankment adjacent to the low level outlet structure and, subsequently, a rock toe was installed.
- Seepage adjacent to the low level outlet structure on the downstream face of the dam was observed during inspections undertaken in 1977 and 1991.
- The installation of a toe drain incorporating a perforated drain pipe was recommended in 1991 to intercept observed seepage however it is unknown if these works were undertaken.
- Repairs to the embankment were undertaken in 2004 to restore freeboard that was lost due to the construction of a temporary access road to bypass the spillway structure during the 2003 Okanagan Mountain Park fire.
- Seepage was reported at the toe of the embankment adjacent to the left abutment during a 2004 inspection and installation of a toe drain was recommended.

4.0 SITE RECONNAISSANCE

A site reconnaissance of the Big Meadow Lake Dam was conducted by EBA on September 16, 2010. EBA's site representatives were Dr. Adrian Chantler, Ph.D., P.Eng., Mr. Bob Patrick, P.Eng and Mr. Michael J. Laws. They were accompanied by Mr. Alfred E. Hartviksen, P.Eng. and Mr. David Carlson of RDOS.

EBA inspected the crest, upstream slope, downstream slope, and downstream toe area and spillway structure of the dam. Photos 1 through 15 show the Big Meadow Lake Dam at the time of the site reconnaissance. The observations made during this inspection are presented in Appendix B. Key observations are as follows:

- The reservoir level was approximately 2.0 m below the dam crest at the time of the inspection (Photo 4).
- The downstream and upstream slopes are approximately 2.5H:1V and 3H:1V respectively.
- The stop logs were removed from the spillway channel and cut up at the time of the inspection (Photo 7).

- Some minor honeycombing was observed in the concrete surface of the spillway structure (Photo 7).
- Clear seepage was observed along the left hand side (LHS) of the spillway structure on the downstream face
- Clear seepage was observed along the right hand side (RHS) of the low level outlet structure on the downstream face (Photo 10).
- Woody debris had accumulated in the weir downstream of the low level outlet structure.
- Minor rutting from vehicle movement was noted along the dam crest.
- Some loss of freeboard (estimated to be in the order of 0.3 m) of the embankment was noted at the left abutment most likely due to the construction of the temporary access road during the Okanagan Mountain Park fire (Photo 12).
- Erosion and rutting was observed on the downstream face above the low level outlet structure, from ATV or Skidoo traffic (Photo 11).
- Some scrubby vegetation is growing on the left hand side of the downstream face (Photo 15).
- Noticeable clear seepage (estimated to be in the order of 1000 to 2000 l/min) was observed from the LHS and RHS toe drains (Big-O pipe) into the low level outlet channel (Photo 15).
- Beaching was noted along the crest of the upstream face and some minor woody debris accumulation (Photo 4).
- Erosion, causing oversteepening of upstream face of the dam embankment, and woody debris accumulation was noted adjacent to the right abutment (Photo 13).

5.0 CONSEQUENCE CLASSIFICATION

The Dam Safety Guidelines published by the Canadian Dam Association (CDA Guidelines, 2007) and the Interim Consequence Classification Policy For Dams in British Columbia (February 2010) were reviewed to confirm the current BC MoE consequence classification of High for the Big Meadow Lake Dam as found on the BC MoE Water Stewardship website. The two systems are similar, but the CDA defines the classifications in greater detail. The High Consequence Dam Class in the BC Dam Safety Regulation has been subdivided into High (High) and High (Low), which are equivalent to Very High and High in the CDA classification.

A comparison of the two sets of guidelines is provided in Table 3.

Downstream of Big Meadow Lake Dam, along Chute Creek, there is a main paved arterial rural road (North Naramata Road), a paved residential road (Indian Rock Road), and several residential properties at the creek's outlet into Lake Okanagan, as shown on the attached Photo 16. Economic losses, including dam replacement and downstream rehabilitation costs, in the event of a dam failure could be in the $1M to $10M range. It is anticipated the significant environmental losses would also occur particularly in the lower reaches of Chute Creek, however it is considered probable that restoration would be possible. The potential loss of life could be in the 1 to 10 range (see Table 3). Therefore, this places the dam in the High (Low) category of the BC Dam Safety Regulation and the High category of the CDA Guidelines.

The 2007 CDA Dam Safety Review Guidelines provides suggested design flood and earthquake levels as a function of dam consequence classification as reproduced in Table 2 below.

TABLE 2: SUGGESTED DESIGN FLOOD AND EARTHQUAKE LEVEL

Dam Consequence Classification (CDA)	Annual Exceedance Probability	
	Inflow Design Flood	EQ Design Ground Motion
Low	1/100	1/500
Significant	Between 1/100 and 1/1000	1/1000
High	1/3 between 1/1000 and PMF	1/2500
Very High	2/3 between 1/1000 and PMF	1/5000
Extreme	Probable Maximum Flood (PMF)	1/10,000

TABLE 3: COMPARISON OF DAM CONSEQUENCE CLASSIFICATIONS

Dam Classification from BC Dam Safety Reg.	Loss of Life		Economic and Social Losses		Environmental and Cultural Losses		Dam Classification from CDA 2007	IDF from CDA 2007
	BC Reg.	CDA	BC Reg.	CDA	BC Regulation	CDA		
Very High	>100	>100	>$100M Very High Infrastructure; Public, Commercial, Residential	Extreme – Critical Infrastructure or Service	Nationally and Provincially Important Habitat and Site – Restoration Chance Low	Major Loss of Critical Habitat – No Restoration Possible	Extreme	PMF
High (High)	10-100	10-100	$10M – 100M Substantial Infrastructure Public Commercial	Very High – Important Infrastructure or Services	Nationally and Provincially Important Habitat and Site – Restoration Chance High	Significant Loss of Critical Habitat – Restoration Possible	Very High	2/3 between 1/1000 year and PMF
High (Low)	1-10	1-10	$1M – 10M Same as above	High – Infrastructure, Public Transit and Commercial	Same as above	Significant Loss of Important Habitat – Restoration Possible	High	1/3 between 1/1000 year and PMF
Low	Some Possible	Unspecified	$100K - $1M Limited Infrastructure; Public, Commercial	Temporary and Infrequent	Regionally Important Habitat and Sites – Restoration Chance High	No Significant Loss of Habitat – Restoration Possible	Significant	Between 1/100 and 1/1000 year
Very Low	Minimal	0	<$100K Minimal	Low	No Significant Loss of Habitat or Sites	Minimal Short Term Loss	Low	1/100 year

6.0 FAILURE MODES ASSESSMENT

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 (earthquake, 16% total).

The percentages presented above reflect the characteristics of that database, not the likelihood of those failures developing at Big Meadow 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 either has insufficient capacity to discharge flood flows, either due to inadequate size or blockage with debris. Embankment overtopping is addressed in the hydrotechnical assessment presented in Section 8.0.

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 rock fill 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 7.0.

d. Slope instability. Gravitational and seepage forces can cause instability in earth dams when they exceed the available shear strength of the soil. Slope stability of the dam is discussed further in Section 7.0.

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

For the Big Meadow Lake Dam, the following failure modes are considered to be plausible:

- Overtopping – The spillway may be undersized for the design flood event.
- Piping through the embankment or foundation – The absence of a downstream filter or foundation treatment of the soils beneath the concrete core wall may place the dam at risk of a piping failure.

- Downstream slope instability – High water levels within the dam, as evident by seepage at the toe of the downstream slope of the dam, increases the risk of downstream slope instability, either during static conditions or during a significant seismic event.
- Soil Liquefaction – The upstream shell of the dam is constructed with sand & gravel which may be prone to deformation when subjected to the design earthquake.

7.0 GEOTECHNICAL ASSESSMENT

7.1 GENERAL

The scope of work for the Big Meadow Lake Dam Safety Review in EBA's proposal did not include a detailed intrusive geotechnical assessment (e.g. drilling, sampling, testing, etc.) to confirm the nature of the existing embankment materials. This assessment is based on observations during the site reconnaissance, available data on the existing dam, published geological data, published geotechnical and EBA's engineering judgement and, therefore, should be considered preliminary in nature. The objective of this approach is to identify potential geotechnical issues so that any detailed geotechnical assessment can be tailored to the particular issue.

The following subjects will be discussed in this section;

- Embankment Seepage;
- Embankment Stability;
- Liquefaction; and
- Potential for Piping.

7.2 GEOTECHNICAL PARAMATERS ESTIMATION

Reference has been made to several publications that provide typical values of geotechnical parameters for a range of different soil types, namely Craig (1992) which provides typical ranges of hydraulic conductivities in Table 2.1 which is reproduced as Table 4 below; Bowles (1988) which provides representative values of angle of internal friction in Table 2-6 which is reproduced as Table 5 below; and Ardiaca (2009) that provides typical geotechnical strength parameters for concrete.

TABLE 4: 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 5: REPRESENTATIVE VALUES FOR ANGLE OF INTERNAL FRICTION Ø FROM BOWLES (1988)	
Soil Type	**Angle of Internal Friction ø**
Gravel	
Medium Size	40 – 50°
Sandy	35 – 50°
Sand	
Loose Dry	27 – 35°
Loose Saturated	27 – 35°
Dense dry	43 – 50°
Dense saturated	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 were utilized in the various analyses as summarized in Table 6 below.

TABLE 6:SUMMARY OF PARAMETERS UTILIZED IN GEOTECHNICAL ANLAYSES BIG MEADOW LAKE DAM					
Material	**Soil Parameters**				
	c' (kPa)	**φ' (°)**	**γ_{unsat} (kN/m^3)**	**γ_{sat} (kN/m^3)**	**k (m/s)**
Concrete Core	365	35	24	24	-
Sand & Gravel Embankment Fill	0.5[1]	35	19	20	1 x 10^{-3}
Granular Till Foundation	0.5[1]	42	21	21.5	1 x 10^{-5}
Toe Berm Material	0.5[1]	38	21	21.5	5 x 10^{-3}

[1] Small cohesion value given to granular soils for numerical stability of model utilized.
c' = Effective Cohesion Intercept.
φ' = Internal Angle of Friction.
γ_{unsat} = Unsaturated Unit Weight of Soil.
γ_{sat} = Saturated Unit Weight of Soil.
k = Hydraulic Conductivity.

7.3 SEEPAGE

Seepage at the downstream toe of Big Meadow Lake Dam has been a commonly observed phenomenon; however, there has been very limited actually quantification and documentation of these flows during the history of the dams operation. At the time of our site inspection the toe seepage was estimated to be in the order of 1000 to 2000 l/min.

Previous inspections of the dam have generally concluded that these flows are primarily as a result of seepage through the dam foundation and around the core wall through the embankment abutment, which would be expected given the nature of the foundation and abutment ground conditions, e.g. relatively free draining granular till.

Steady state seepages estimates were calculated using the two-dimensional finite element analysis program Plaxis.

The toe seepage calculated using the parameters summarized in Table 4 appears to underestimate the magnitude of toe seepage observed during the site inspection and that previous documented, suggesting that either the concrete core wall is cracked or there is a preferential seepage path such as a zone of higher permeability material in the dam foundation. Therefore a second case was considered in the analysis namely, a cracked concrete core wall, which was modelled by giving the core wall a hydraulic conductivity of 1×10^{-5} m/s. This resulted in a calculated magnitude of toe seepage similar to that observed during the site inspection.

The rate of toe seepage calculated at the dam location for both cases is summarized in Table 7 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 height is less.

TABLE 7: ESTIMATED RATE OF TOE SEEPAGE FOR BIG MEADOW DAM

Reservoir Level	Case	Calculated Toe Seepage
1612.09 m	Uncracked Concrete Core Wall	6.09 m3/day/m (4.23 litres/min/m)
1612.09 m	Cracked Concrete Core Wall	12.35 m3/day/m (8.58 litres/min/m)

The flow fields from the steady state seepage analysis of the dams are shown on the attached Figures 5 and 6.

7.4 EMBANKMENT STABILITY REVIEW

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 Tables 8 and 9 below.

TABLE 8: FACTORS OF SAFTEY FOR SLOPE 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 9: FACTORS OF SAFTEY FOR SLOPE STABILITY– SEISMIC ASSESSMENT

Loading Conditions	Minimum Factor of Safety	Slope
Pseudo-static	1	Upstream and Downstream
Post-earthquake	1.2-1.3	Upstream and Downstream

The interim Consequence Classification Policy for Dams in British Columbia (2010) permits the minimum design earthquake level for earth dams constructed prior to 2008 to be assessed in accordance with the criteria of the 1999 CDA Dam Safety Review

Guidelines. However, it recommends that dam owners move towards the design criteria provided in the 2007 CDA Dam Safety Review Guidelines and, therefore, this is the criteria that has been applied in this safety review.

Methodology

As no detailed borehole logs or construction records are available for the dam, the stability review of the embankment was undertaken based on existing drawings of the dam, published geological maps and typical engineering properties of the materials used in the embankment construction.

The CDA Technical Bulletin, Geotechnical Consideration for Dam Safety, recommends a staged approach with respect to assessing the seismic stability of earth dams, 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 peak ground acceleration for the earthquake return period under consideration. Should the embankment have a factor of safety in excess of 1.0 for this loading it is considered not to undergo any deformation during the design earthquake and therefore no further analysis is required. Should a factor of safety of less than 1.0 be obtain 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. Newmark (1965), Bray (2007), etc) approach is recommend as the second stage of analysis to confirm that the embankment has adequate freeboard post the design earthquake event deformation. Should the second stage of analysis yield unfavourable results then a series of more sophisticated analysis approaches (e.g. Finite Element Analysis) are recommended. As this assessment is considered preliminary in nature, only the first two stages of analysis have been considered for this dam safety review as there are too many unknowns to undertake a more sophisticated type of analysis.

Static and pseudo-static seismic global stability factors of safety for the existing embankments were calculated using the two-dimensional Finite Element analysis program Plaxis.

Global stability factors of safety were obtained through use of the phi-c (strength) reduction calculation approach where the strength parameters of the soils are incrementally reduced until failure occurs, with the ratio of the initially available soil strength and that at failure equating to the factor of safety. The theoretical failure body is shown in the output as an area of discoloration.

Pore water pressures in the dam were determined by undertaking a steady state seepage analysis in the initial conditions calculation phase of the Plaxis analysis of each dam assuming the reservoir level was at the spillway sill.

For the post earthquake residual shear strength soil case, it was assumed that the sand and gravel comprising shell of the dam, which was determined to be fully saturated in the seepage analysis, could liquefy. The undrained residual shear strength (S_r) of the soil was estimated in accordance with Figure 88 of Idriss and Boulanger (2008).

The stability review of the Big Meadow Lake Dam was considered at its maximum height with the geometry and soil profile of the model based on the embankment cross section shown attached Figure 4. The results of the analysis indicate that the downstream slope of the dam is marginally stable for the cracked core case under both static and seismic loading conditions. Therefore this case was reanalyzed with the inclusion of a free draining toe berm on the downstream slope of the dam which resulted in significantly improved factors of safety. The results of the analysis are summarized are in Table 10 below and presented on the attached Figures 7 to 13 inclusive.

TABLE 10: FACTORS OF SAFETY – SLOPE STABILITY ASSESSMENT BIG MEADOW LAKE DAM

Loading Conditions	Core Wall Condition	Calculated Factor of Safety	Slope
Static long-term (steady state seepage, normal reservoir level)	Uncracked	1.75[1]	Downstream & Upstream
Static long-term (steady state seepage, normal reservoir level)	Cracked	0.93[2]	Downstream
Addition of free draining toe berm static long-term (steady state seepage, normal reservoir level)	Cracked	1.96	Downstream
Full or partial rapid drawdown[3]	N/A	N/A	Upstream
Seismic pseudo-static (PGA, steady state seepage, normal reservoir level)	Uncracked	1.04	Downstream
Seismic pseudo-static (PGA, steady state seepage, normal reservoir level)	Cracked	<0.93[2]	Downstream
Addition of free draining toe berm seismic pseudo-static (PGA, steady state seepage, normal reservoir level)	Cracked	1.92	Downstream
Seismic pseudo-static (PGA, steady state seepage, normal reservoir level)	Uncracked	1.06	Upstream
Post seismic residual shear strength (steady state seepage, normal reservoir level)	Cracked	1.01[3]	Downstream & Upstream

1 Upstream shell greater than 1.75.

2. Localised toe failure.

3. Not considered an applicable loading condition as the upstream face is constructed of free draining material.

4. Upstream shell greater than 1.01.

7.5 LIQUEFACTION & POST SEISMIC DEFORMATION

No suitable historical geotechnical data (e.g. boreholes' with insitu testing) is available for this dam, to accurately quantify if there is a risk of the embankment undergoing deformation as a result of soil liquefaction during the design seismic event.

The shell of the Big Meadow Lake Dam comprises sand and gravel that could potentially be susceptible to liquefaction when subject to strong ground motion. As no geotechnical data is available on this dam, we have undertaken a preliminary liquefaction analysis utilizing lower bound material parameters and loading conditions.

It is considered likely that the materials that comprise the shell of the dam were end dumped with little or no compaction and are therefore somewhere between a loose to compact consistency. Conservatively assuming that a total stress over effective stress ratio of 2 exists for each dam, a cyclic shear stress ratio (CSR) of approximately 0.09 can be calculated for the design earthquake event.

Using the semi-empirical method developed by Tokimatsu & Seed (1987) of CSR versus, $(N_1)_{60}$ (insitu standard penetration test results) and volumetric strain, a volumetric strain of 5% would occur during the design earthquake for a loose consistency soil with a $(N_1)_{60}$ equal to 5 (loose material), while a volumetric strain of 0.1% would occur during the design earthquake event for a compact consistency soil with a $(N_1)_{60}$ equal to 10 (compact material).

Therefore, vertical settlements of no greater than 400 mm and 10 mm for a loose and compact soil profile respectively are estimated for the sand and gravel shell of Big Meadow Lake Dam for the design earthquake event, which is well within the available freeboard of this dam.

Given the depositional nature (e.g. glacial) of the dam's foundation materials there is considered to be very low risk of the dams' foundations undergoing liquefaction during the design seismic event.

The post seismic residual shear strength stability analysis resulted in a factor of safety just above unity suggesting that the dam slopes are likely to undergo some lateral deformation as a result of the design earthquake, assuming the saturated sands and gravels in the dam's shell are susceptible to liquefaction. Estimating the magnitude of this lateral deformation would require undertaking a more detailed liquefaction assessment based on the results of a field investigation and a dynamic finite element analysis which is beyond the scope of this dam safety review.

7.6 POTENTIAL FOR PIPING

The condition of the Big Meadow Lake Dam presents a challenge in that the dam has performed reasonably well since it's upgrades during the 1950's, with no known reported occurrences of turbid seepage since these works were completed. This is significant given that there does not appear to be a reliable filter in place. Piping more frequently occurs within five years of first filling; however, there are many examples of dams where the effects of piping were only observed many years after first filling as presented in Foster et al. (2000b). Therefore, an apparent lack of a reliable filter supports the conclusion that the Big Meadow Lake Dam could potentially still develop piping failure, despite its history of relatively good performance.

Piping is typically accompanied by seepage containing suspended fines and sand. The seepage can be turbid (e.g., discoloured by suspended fines) and silt and sand is typically deposited at the toe of the dam where the seepage exits from the dam fill or foundation.

EBA 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 as presented in Foster et al. (2000b). This paper is included in Appendix C 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 EBA's application of the UNSW method, the total annual likelihood of piping failure under current conditions for the Big Meadow Lake Dam is 1.91 x 10^{-4} (1 in 5236 years). 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 C.

While this figure implies a high degree of accuracy, it is not possible to accurately estimate the likelihood of failure for the Big Meadow Lake Dam given what is currently known about it. The implied accuracy is due to the statistics used in the Foster et al. (2000b) study. This probability confirms EBA's intuition that, while the performance of the Big Meadow Lake Dam to date is encouraging, there is still a small probability that piping failure could develop even if they experience the same loading conditions in the future as that they have been subjected to in the past. The results of this assessment will be considered further in Section 10.0 – Dam Safety Management.

Currently the calculated probability of a dam failure to occur due to piping is greater than the upper limit of the ALARP zone (see Section 9.6) shown on Figure 16 for Big Meadow Lake Dam. The following key points should be kept in mind when considering the impact of the results of the UNSW assessment presented herein:

- There has been a well documented history of toe seepage at Big Meadow Lake Dam. The construction of a toe berm incorporating a filter and drainage system at the dam or segments of dam where the seepage has occurred, would likely result in a 40% improvement of the values given above for the estimated probability of piping failure of the dam.
- Currently seepage monitoring at Big Meadow Lake Dam has been poorly documented. An improved monitoring program where seepage monitoring is well documented would result in a 20% improvement in the values given above for the estimated probability of piping failure of the dam.

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, as discussed in Section 7.6, is estimated from Foster et al. (2000b) to be more than ten times higher than the currently estimated probability. This will again be greater than the upper limit of the ALARP zone (see Section 9.6) shown on Figure 16, and would require some study and possible rehabilitative measures to be taken.

8.0 HYDROTECHNICAL ASSESSMENT

The technical findings of the hydrotechnical assessment for the Big Meadow Dam are contained in the companion hydrotechnical report

The simulation and results from the hydrotechnical report for the Inflow Design Flood (IDF) are summarized in Table 11 below.

TABLE 11: FLOOD ROUTING RESULTS

Reservoir	Spillway Crest Elev. (m)	Spillway Crest Length (m)	Dam Crest Elev. (m)	Peak Inflow (m^3/s)	Peak Water Elev. (m)	Freeboard Elev. (m)	Peak Storage Volume (m^3)	Peak Outflow (m^3/s)
Big Meadow	1612.09	5.8	1613.92	5.53	1612.7	1.22	477,000	4.87

The analysis of routing flows through the dam indicates that the existing spillway for Big Meadow Lake Dam is able to pass the routed IDF. The freeboard (the vertical distance between the maximum water level and the dam crest) calculated for Big Meadow Lake Dam is greater than the minimum requirement of 1.0 m.

9.0 DAM SAFETY MANAGEMENT SYSTEM

9.1 GENERAL

Dam safety management can be generally described to have 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 Plans; and,
- Management of Public Safety.

A general schematic of a dam safety management system is presented in Figure 14. EBA has assessed the dam safety management system in place for Big Meadow Lake Dam and the results of this assessment are presented in this section.

9.2 REVIEW OF 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).

No OMS Manual has been prepared for this facility, however due to the interconnected nature of the dams it is suggested that the OMS manual for the Naramata Lake Dam could be upgraded to include the Big Meadow Lake Dam.

9.3 REVIEW OF EMERGENCY PREPAREDNESS PLAN

EBA has reviewed the document, Naramata Water System Emergency Response plan dated August 2007 prepared by the RDOS:

In general the content of the document appears to be more focussed on water quality issues than on dam safety and does not conform to BC MoE's minimum requirements.

Detailed revision of the Emergency Preparedness Plan (EPP) is a significant undertaking and was not part of the agreed upon scope of work. EBA has noted the following areas for improvement in the EPP:

- As the EPP is intended to be a "living" document, document control measures should be taken that include revision number, date, and circulation list as a minimum. Emergency contact information will change continually and the EPP will have to be regularly updated to reflect these changes. Finally, the name of the person responsible for keeping the EPP current should be indicated in the EPP.
- The EPP should be updated to include a description and location in latitude and longitude of Big Meadow Lake Dam.
- The plan attached to the EPP should show the location of Big Meadow Lake Dam and the potentially inundated areas caused by failure of the dam.
- The EPP should be updated to include a description of how access is achieved via road to Big Meadow Lake Dam, including alternate routes. Ideally this should be presented on a plan.
- There is no mention of the landowners and residents who are located in the potential inundation zone below Big Meadow Lake Dam. A map showing where the residents are located and their emergency contact information should be included. Specific attention should be paid to any residents who may need assistance (e.g. have restricted mobility) to relocate out of a potential inundation zone.
- The EPP provides the details of multiple contractors and consultants to contact in the event of an emergency. However, no specific mention is made of the nature of the potential emergencies and what actions could be required. The EPP should be revised to include the potential failure modes discussed herein and potential actions required (e.g. unblock the spillway culverts during heavy rainfall events, start pumping the reservoir down in the event of a developing piping failure). EBA can provide RDOS with assistance in this matter under a separate scope of work. RDOS should verify that the local contractors have the resources to respond to these events quickly.

In reviewing the EPP we would recommend that RDOS make themselves familiar with the contents of the BC MoE Emergency Preparedness Plan Template.

9.4 PUBLIC SAFETY MANAGEMENT

The 2007 CDA Guidelines contain a draft Technical Bulletin on Public Safety and Security around Dams. Public safety and security around dams is an emerging topic in the dam safety community in both Canada, and the world, that the CDA is leading. As this is an emerging topic, it is not surprising that there is no Public Safety or Security Management Plan in place. However, given the nature of the Big Meadow Lake Dam area, public interaction with the dams potentially presents ongoing problems.

During EBA's inspection of Big Meadow Lake Dam it was noted that there are no restrictions on public interaction with the dams and plenty of evidence of ground disturbance and vandalism were noted.

RDOS should undertake a review of their dam security and implement improvements. It is envisioned that typical improvements would include but not be limited to:

- Improved signage advising of the importance of the structures. It is assumed that many back country users would ignore or damage such signage. We would note that the BC MoE is about to specify minimum signage requirement for dams situated on crown land in an pending amendment to the BC Dam Safety Regulation, an example of the proposed BC MoE signage requirement is attached as Figure 15;
- Education of back country users, e.g. local ATV clubs, as to the importance of the dam and consequence of its failure;
- Obstruction's such as gates and large boulders, will need to be placed to restrict access and protective critical dam components;
- Regular maintenance program that identifies and rectifies any damage done to any of the dams during routine inspections; and,
- Any instrumentation that comprises an essential component of dam safety management should be installed in secured manholes or a locking valve box.

9.5 DAM SAFETY EXPECTATIONS ASSESSMENT

A Dam Safety expectations assessment has been undertaken of the Big Meadow Lake Dam using the sample check sheet for Dam Safety Expectation, Deficiencies and Priorities as prepared by the BC MoE (May 2010) as presented in Appendix D.

The Dam Safety Expectations are divided into five categories:

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

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

Analysis and Assessment

There are four potential deficiencies and three non-conformances in this category.

Operations, Maintenance and Surveillance

There are twenty-two non-conformances in this category, which all could be easily resolved with the preparation of a OMS manual for this facility.

Emergency Preparedness

There is one potential deficiency and six non-conformances in this category. Four of the non-conformances could be easily resolved by updating the EPP for this facility.

Dam Safety Review

There are no deficiencies nor non-conformances in this category. By commissioning this Dam Safety Review, RDOS conforms to the dam safety expectations for this category.

Dam Safety Management System

There are six non-conformances in this category, five of which could be easily resolved by preparing a OMS manual and updating the EPP for this facility.

9.6 ASSESSMENT OF DAM SAFETY BASED ON ALARP PRINCIPAL

9.6.1 General

Management of dam safety is the cornerstone of managing the liability associated with potential risk of dam failure. Societal tolerances for loss of life have generally been decreasing through the years.

In the case of the Big Meadow Lake Dam, given the findings to date of this dam safety review, these questions need to be asked:

- "How safe is safe enough?"; and,
- "How does RDOS balance equity and efficiency?"

The first question deals with tolerance of risk of failure and defining a frequency or probability of failure beyond which it isn't practical to be concerned about. The second question deals with how to balance risk tolerance with financial costs associated with reducing risk.

The 2007 CDA Guidelines introduced the "ALARP" principal to the Canadian Dam Safety community with regard to tolerable risk. ALARP stands for **A**s **L**ow **A**s **R**easonably **P**racticable. This principal is demonstrated in Figure 16 which relates magnitude of loss of life to probability of loss of life. This chart shows the suggested relationship between probability of occurrence, potential loss of life and varying degrees of risk tolerability (broadly acceptable, ALARP and unacceptable).

EBA cannot advise RDOS and other stakeholders (e.g., the community, utility owners, BC MoT, BC MoE dam safety) what their tolerance for risk of loss of life is. The level of risk accepted by the RDOS and the stakeholders is up to them. Therefore, this section has been prepared to illustrate what generally accepted risk tolerance is within the dam community in Canada, as defined by the CDA.

EBA has applied the ALARP principal to the deficiencies and non-conformances identified during the dam safety review and the results of this assessment are presented in the following sections. For the purposes of this assessment, EBA has assumed that the maximum number of deaths that could occur is ten. This magnitude of loss of life is the maximum for a High (Low) Consequence classification dam.

9.6.2 Stability of Embankment Slopes

Undertaking a probabilistic stability assessment of dams is not in the typical scope of a dam safety review given that current CDA acceptance criteria for stability is based on accepted minimum factors of safety, therefore it is currently not possible to predict a probability of failure causing loss of life associated for the dams at this time. A probabilistic stability assessment would require undertaking an intrusive investigation (e.g., drilling and in situ testing) to asses the variability of the embankment materials to enable a probabilistic assessment of failure and reservoir release through static and seismic stability analyses.

9.6.3 Piping Failure

EBA has considered the probability of failure due to a piping event as discussed in Section 7.6. The results of this semi-qualitative assessment are an annual probability of piping failure. The probability of one or more people being downstream of the Big Meadow Lake Dam when a flood wave from dam failure passes down the valley is considerably less than unity.

Additionally, the probability of one or more people being killed while being in the path of the flood wave is also considerably less than unity. The probability of one or more people being killed by the flood wave is the product of all three probabilities as below.

$$P_{\text{loss of life}} = P_{\text{piping failure}} \text{ x } P_{\text{persons in way}} \text{ x } P_{\text{persons in way being killed}}$$

Assuming that no more than ten people could ever be killed downstream of the dam by a flood wave caused by dam failure due to piping, the maximum probability of failure causing loss of life would equal to the values presented in Section 7.6 and would plot within the "Intolerable" zone as shown on Figure 16. This depends on the assumption that the probability of people being in the path of a floodwave and being killed by it is certain, e.g., probability of 1.0. From a practical perspective, recognizing a reduction in the probability of loss of life associated with the latter two individual probabilities in the equation above, the probability of loss of life would still likely be within the "Intolerable" zone. EBA has made the conservative decision to assume the probability of ten people being downstream of the dams and being killed by the flooding is 1.0 (certainty).

There has been a well documented history of toe seepage at Big Meadow Lake Dam. The construction of a toe berm incorporating a filter and drainage system at the dams or segment of the dam where the seepage has occurred and an improved monitoring program where seepage monitoring is well documented, would likely result in a 55% improvement in the values given in Section 7.6 for the estimated probability of piping failure of Big Meadow Lake Dam which would result in this dam moving into the "ALARP" zone as shown on the attached Figure 16.

10.0 CONCLUSIONS

The conclusions reached during the Dam Safety Review of the Big Meadow Lake Dam are presented as follows for each area of review:

Background Review

- There is no site specific subsurface information available for EBA's review.
- There is no design information and limited as-built construction documentation. It is probable that the dam was initially constructed in 1933, with major upgrades to the dam completed in 1952.
- The dam was designed as a homogenous earthfill dam with a central concrete core wall.
- Historical seepage has been observed at the downstream toe of the embankment and has generally been reported as clear; however, the volume of flow has not been quantified.

Site Reconnaissance

- Clear seepage was observed along left hand side of spillway structure on the downstream face.
- Clear seepage was observed along right hand side of low level outlet structure on the downstream face.
- Woody debris had accumulated in weir downstream of low level outlet structure.
- Minor rutting from vehicle movement was noted along the dam crest.
- Some loss of freeboard of the embankment was noted at the left abutment most likely due to the construction of the temporary access road during the Okanagan Mountain Park fire.
- Erosion and rutting was observed on downstream face above the low level outlet structure from ATV or Skidoo traffic.
- Some scrubby vegetation is growing on left hand side of downstream face.
- Noticeable clear seepage was observed from the LHS and RHS toe drains (Big-O pipe) into low level outlet channel.
- Erosion, over steepening of upstream face of the dam embankment and woody debris accumulation was noted adjacent to the right abutment.
- The reservoir side slopes appear stable.

Consequence Classification

- The Big Meadow Lake Dam is classified as a High Consequence Dam according to CDA Guidelines and High-Low according got BC MoE classification guidelines.

Failure Mode Assessment

- The plausible failure modes for the dam are overtopping, piping through the embankment and foundation, downstream slope instability and soil liquefaction of the upstream slope.

Geotechnical Assessment

- In general, the seepage flow field patterns determined by the steady state seepage analysis assuming a cracked core concur with the historical observations of seepage at the embankment toe.
- The results of the preliminary stability analysis indicate that the downstream toe of the dam is marginal stable when subjected to excess seepage or the design seismic event.
- The magnitude of potential vertical settlements estimated as a result of soil liquefaction for the design earthquake event is well within the available freeboard for the dam.
- Given the depositional nature (e.g. glacial) of the dam foundation there is considered to be no risk of the dam foundation undergoing liquefaction during the design seismic event.
- The post seismic residual shear strength stability analysis resulted in a factor of safety just above unity suggesting that the dam is likely to undergo some lateral deformation as a result of the design earthquake, assuming the saturated sands and gravels in the dam's shell are susceptible to liquefaction.
- A probabilistic piping risk assessment was conducted using a published method. A probability of piping failure developing of 1.91×10^{-4} was calculated.
- Currently seepage monitoring at the dam has been poorly documented. An improved monitoring program where seepage monitoring is well documented would result in a reduction of the probability of piping failure developing by 20% for the dam.
- There has been a well documented history of toe seepage at the dam. The construction of a toe berm incorporating a filter and drainage system at the dam or segments of the dam where the seepage has occurred, would likely result in a 40% reduction of the values of the estimated probability of piping failure develop for the dam.

Hydrotechnical Assessment

- Analysis indicates that existing dam is able to pass the IDF with an available freeboard of 1.22 m, which is greater than the minimum requirement of 1.0 m.

Dam Safety Management

- A OMS manual needs to be prepared for this facility.
- The EPP should be modified to include additional information to ensure that it is reflective of the current state of practice for dam safety management.

- The only dam safety security issue appears to be vandalism to the dam downstream face and crest from recreational vehicle traffic.
- The potential for piping failure causing loss of life is currently in the unacceptable zone of the ALARP chart suggested by CDA Guidelines.
- EBA cannot advise RDOS on what their corporate tolerances are for risk of loss of life. This also applies to the citizens of Naramata and all other stakeholders.
- No instrumentation is installed in the dam.
- Improving the inspection documentation to include quantify seepage rates and include comments on clarity of seepage would decrease the probability of piping failure.

11.0 RECOMMENDATIONS

The priority (high, medium or low) for each item is given in brackets after each recommendation.

Background Review

- RDOS should continue to look for background information on the design and construction of the Big Meadow Lake Dam such as, but not limited to, design reports and construction records including quality control testing results. This should not only include a search of RDOS archives but also BC MoE archives in Victoria (Low).

Site Reconnaissance

- The area of upstream erosion of the embankment and woody debris accumulation adjacent to the left abutment should be cleaned out and protected with rip-rap (High).
- The woody debris should be cleared out of the weir downstream of the low level outlet structure (High).
- The scrubby vegetation on the right half of the downstream face of the dam should be cleared (Medium).
- RDOS should commission a topographical survey of the dam to confirm that it has sufficient freeboard and that it has maintained its design slopes. At the same time all dam features e.g. spillway structure, locations of seepage etc should also be picked up. The survey could be used to prepare an updated plan of the dam to be incorporated in an OMS manual. Should the survey indicate that there has been a loss of freeboard this will require reinstatement (High).

Consequence Classification

- There are no recommendations from this area of review.

Failure Mode Assessment

- There are no recommendations from this area of review.

Geotechnical Assessment

- An intrusive geotechnical investigation and topographical survey of the dam is required to more accurately quantify the liquefaction potential, static stability and seismic stability of the embankment, which is currently not considered within CDA criteria based on the results of the preliminary stability assessment. Should unacceptable factors of safety be obtain following this work it is envisioned that the construction of a toe berm will be required to improve dam stability (High).
- Construction of a toe berm incorporating a filter and drainage system at the dam or segment of dam where the seepage has historically occurred should be evaluated (Medium).
- An improved seepage monitoring program should be implemented which in conjunction with the construction of a toe berm incorporating a filter and drainage system would reduce the probability of piping failure occurring to within the "ALARP" zone (High).

Hydrotechnical Assessment

- If stop logs are to be utilized, the design flood calculations should be revised. It is recommended that stop logs are not in place during the spring freshet (High).

Dam Safety Management

- A OMS manual needs to be prepared for this facility (High).
- The EPP needs updating to conform to current dam safety expectation (Medium).
- There is no instrumentation installed to monitor the performance of the dam. As a minimum one piezometer should be installed in the downstream slope and instrumentation installed or a procedure developed to quantify the volume of toe seepage. In-situ testing, sampling, laboratory testing and a formal borehole log should be prepared of the piezometer installed at the dam to provided "as-built" information on the dam and assist in an future engineering assessment (Medium).
- RDOS should undertake a review of dam security and implement improvements (Medium).

12.0 LIMITATIONS OF REPORT

This report and its contents are intended for the sole use of the Regional District of Okanagan-Similkameen and their agents. EBA does not accept any responsibility for the accuracy of any of the data, the analysis or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than the Regional District of Okanagan-Similkameen, or for any Project other than the proposed development at the subject site. Any such unauthorized use of this report is at the sole risk of the user. Use of this report is subject to the Terms and Conditions stated in EBA's Services Agreement and in the General Conditions provided in Appendix E of this report.

13.0 CLOSURE

EBA trust this report meets your present requirement. Do not hesitate to contact any of the undersigned should there be any questions or comments.

EBA Engineering Consultants Ltd.

Dec 17, 2010

Report Prepared by:
Michael Laws, BE (Civil), BSc (Geology)
Project Manager
Engineering Practice
e. mlaws@eba.ca
t. 250.862.3026 x230

Reviewed by:
Bob Patrick, P.Eng
Principal Engineer
Engineering Practice
e. bpatrick@eba.ca
t. 250.756.2256

/tmkp

REFERENCES

Ardiaca, D.H., 2009. Mohr-Coulomb Parameters for Modelling of Concrete Structures. Plaxis Bulletin Issue 25 Spring 2009.

Bowles, J.E., 1988. Foundation Analysis and Design – Forth Edition.. McGraw-Hill Publishing Co.

British Columbia Ministry of Environment, 1998. Dam Safety Guidelines – Inspection and Maintenance of Dams.

British Columbia Ministry of Environment, 2010. Dam Safety Review Guidelines. – Version 2.

British Columbia Ministry of Environment, 2010. Interim Consequence Classification Policy for Dams in British Columbia.

Canadian Dam Association, 1997. Dam Safety Guidelines.

Canadian Dam Association, 2002. Dam Safety Review Workshop, 2002 CDA Conference, Victoria, British Columbia.

Canadian Dam Association, 2004. Public Safety Around Dams Workshop, 2004 CDA Conference, Ottawa, Ontario.

Canadian Dam Association, 2007. Dam Safety Guidelines.

Canadian Dam Association, 2007. Technical Bulletin – Dam Safety Analysis and Assessment.

Canadian Dam Association, 2007. Technical Bulletin – Geotechnical Considerations for Dam Safety.

Canadian Dam Association, 2007. Technical Bulletin – Hydrotechnical Considerations for Dam Safety.

Canadian Dam Association, 2007. Technical Bulletin – Inundation, Consequences and Classification for Dam Safety.

Canadian Dam Association, 2007. Technical Bulletin – Seismic Hazard Considerations for Dam Safety

Craig, R.F., 1992. Soil Mechanics – Fifth Edition. Chapman & Hall.

Foster, M., Fell, R. and Spannagle, M., 2000. A method for assessing the relative likelihood of failure of embankment dams by piping. Can. Geotech. 5., Vol. 37, pp 1000-1024.

Foster, M., Fell, R. and Spannagle, M., 2000. The statistics of embankment dam failures. Can. Geotech. 5., Vol. 37, pp 1000-1024.

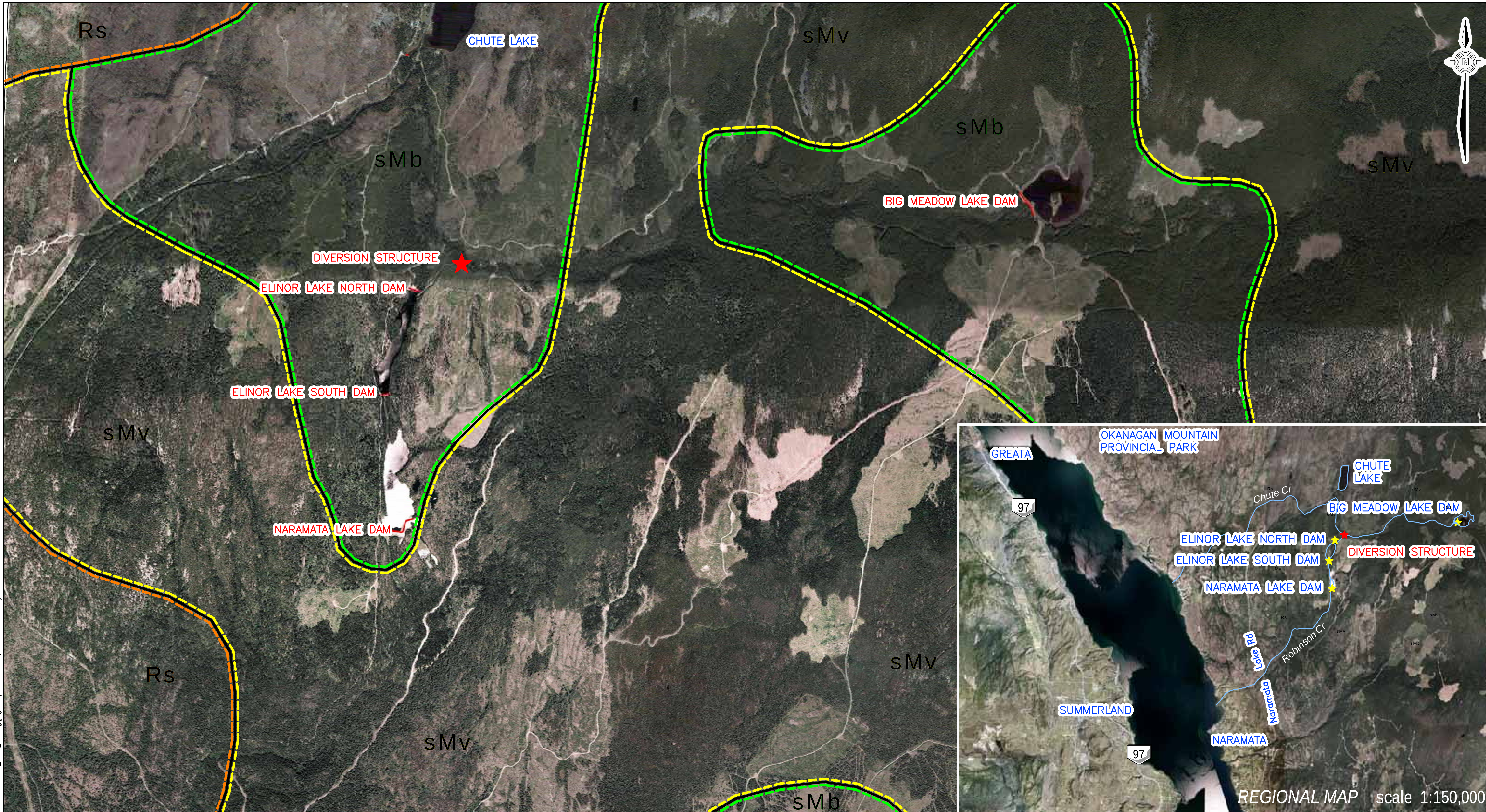
Idriss, I.M. and Boulanger, R.W., 2008. Soil Liquefaction During Earthquakes. Earthquake Engineering Research Institute Monograph 12.

Kramer, S.L, 1996. Geotechnical Earthquake Engineering. Prentice Hall.

Queen's Printer, 2003. Water Act – British Columbia Dam Safety Regulation, BC Reg. 44/2000.

Tokimatsu, K. and Seed, H.B., 1987. Evaluation of settlements in sand due to earthquake shaking. J. of Geotechnical Engineering, ASCE. Vol. 113(8): 861-878.

FIGURES



C:\Kelowna\Drafting\k131\K13101459.001\Big Meadow K13101459.001_FIGS_R0.dwg [Figure 1] December 14, 2010 - 9:51am lmajetich

NOTES

Image from Google Earth Pro
Imagery Dates March 4, 2004 - September 26, 2005
Surficial Geology from GEOLOGICAL SURVEY OF CANADA (1984)
Surficial Geology (1984)
Map Kootenay Lake

SCALE 1:30,000

200 0 200 600m 1.2km

SURFICIAL GEOLOGY

Rs , ROCK: Crystalline metamorphic, acidic igneous, quartzite, argillite, marble, greenstone, phyllite, greywacke, limestone, dolomite and sandstone. Areas mapped as rock consist dominantly of rock at the surface but include minor areas of rock covered by a veneer of colluvium and till. Rs: Rock characterized by steep slopes or exposed by modern stream.

sMb ,sMv, SANDY TILL: Olive grey, grey and pale grey, weakly calcareous to non-calcareous loamy sand, sandy loam and loam. Generally gravelly, cobbly or bouldery. Mainly massive but locally contains lenses of stratified sediments. Clast lithologies reflect local bedrock which is chiefly crystalline metamorphic and granitic in character. Locally includes unmapped areas of alluvial, glaciofluvial and glaciolacustrine deposits and areas of rock. Locally in valley bottoms till may be as thick as 30 m but generally it is no more than 5 m thick. Occurs as a blanket with surface relief due to the general shape of the underlying surface or deposit; sMb: thickness up to 5 m; sMv: thin and discontinuous with thickness up to 2 m.

CLIENT	BIG MEADOW LAKE DAM SAFETY REVIEW			
REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN	Location Plan & Surficial Geology			
EBA Engineering Consultants Ltd.	PROJECT NO. K13101459.001	DWN LM	CKD MJL	REV 0
	OFFICE EBA-KELOWNA	DATE December 14, 2010		Figure 1

2005 National Building Code Seismic Hazard Calculation

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

Requested by: Michael J Laws, EBA Engineering Consultants Ltd — October 18, 2010

Site Coordinates: 49.6675 North 119.5023 West

User File Reference: Naramata Dams

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

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.278	0.175	0.101	0.060	0.138

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2005 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.062	0.138	0.191
Sa(0.5)	0.044	0.089	0.122
Sa(1.0)	0.026	0.054	0.073
Sa(2.0)	0.015	0.032	0.043
PGA	0.034	0.073	0.098

References

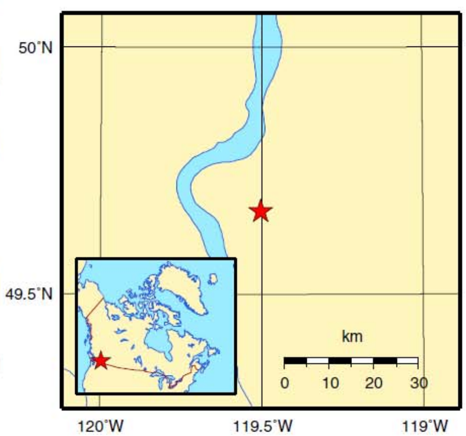
National Building Code of Canada 2005 NRCC no. 47666; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2005, Structural Commentaries NRCC no. 48192
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Grid values to be used with the 2005 National Building Code of Canada (in preparation)

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

Aussi disponible en français



Natural Resources Canada — Ressources naturelles Canada

Canada

NOTES

From the Earthquakes Canada website (http://earthquakescanada.nrcan.gc.ca)

CLIENT

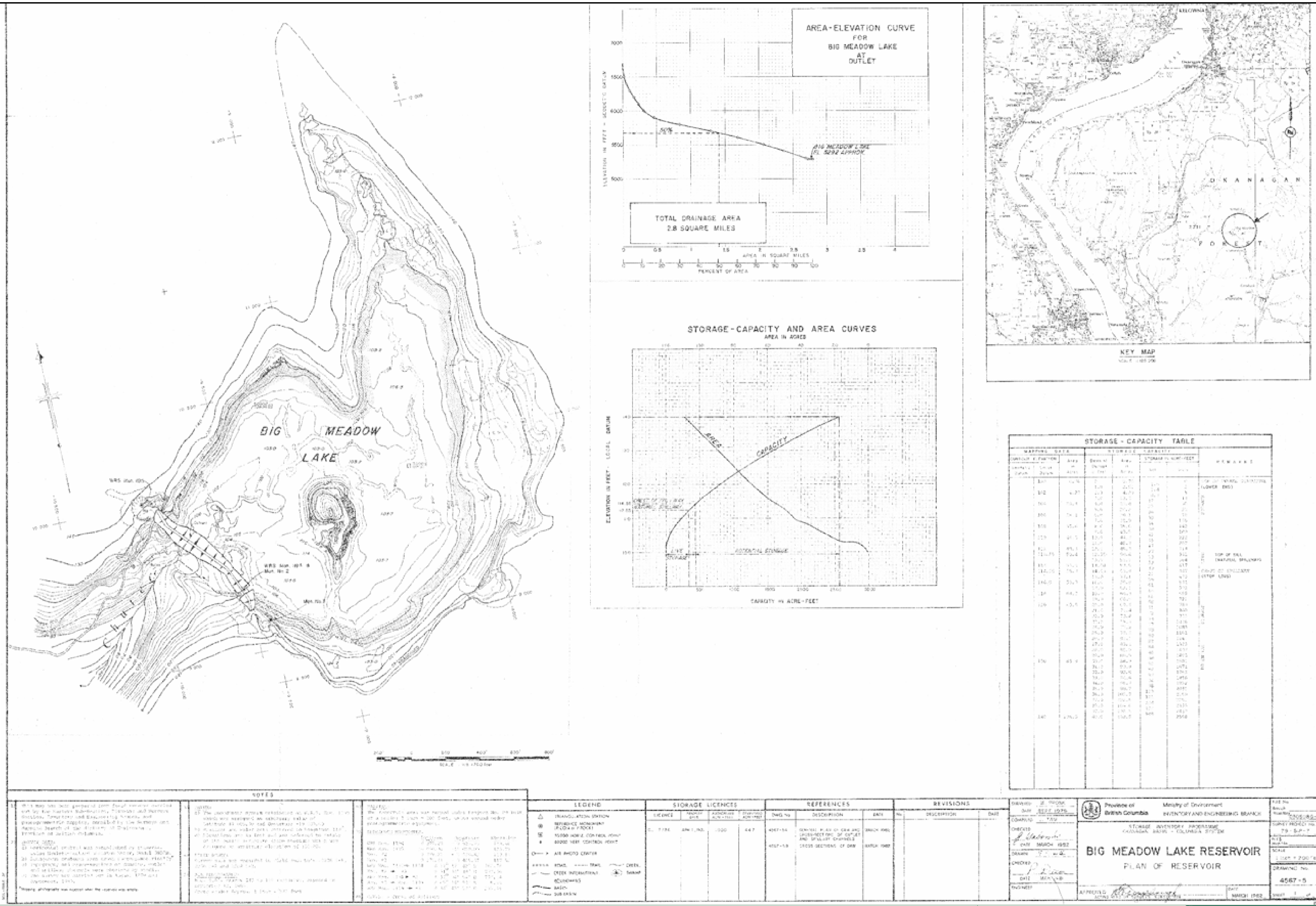
REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Reference Peak Ground Accelerations (PGA) and Spectral Accelerations ($S_a(T)$)

PROJECT NO.	DWN	CKD	REV	
K13101459.001	MJL	BP	0	Figure 2
OFFICE: EBA-KELOWNA	DATE: December 14, 2010			



CLIENT

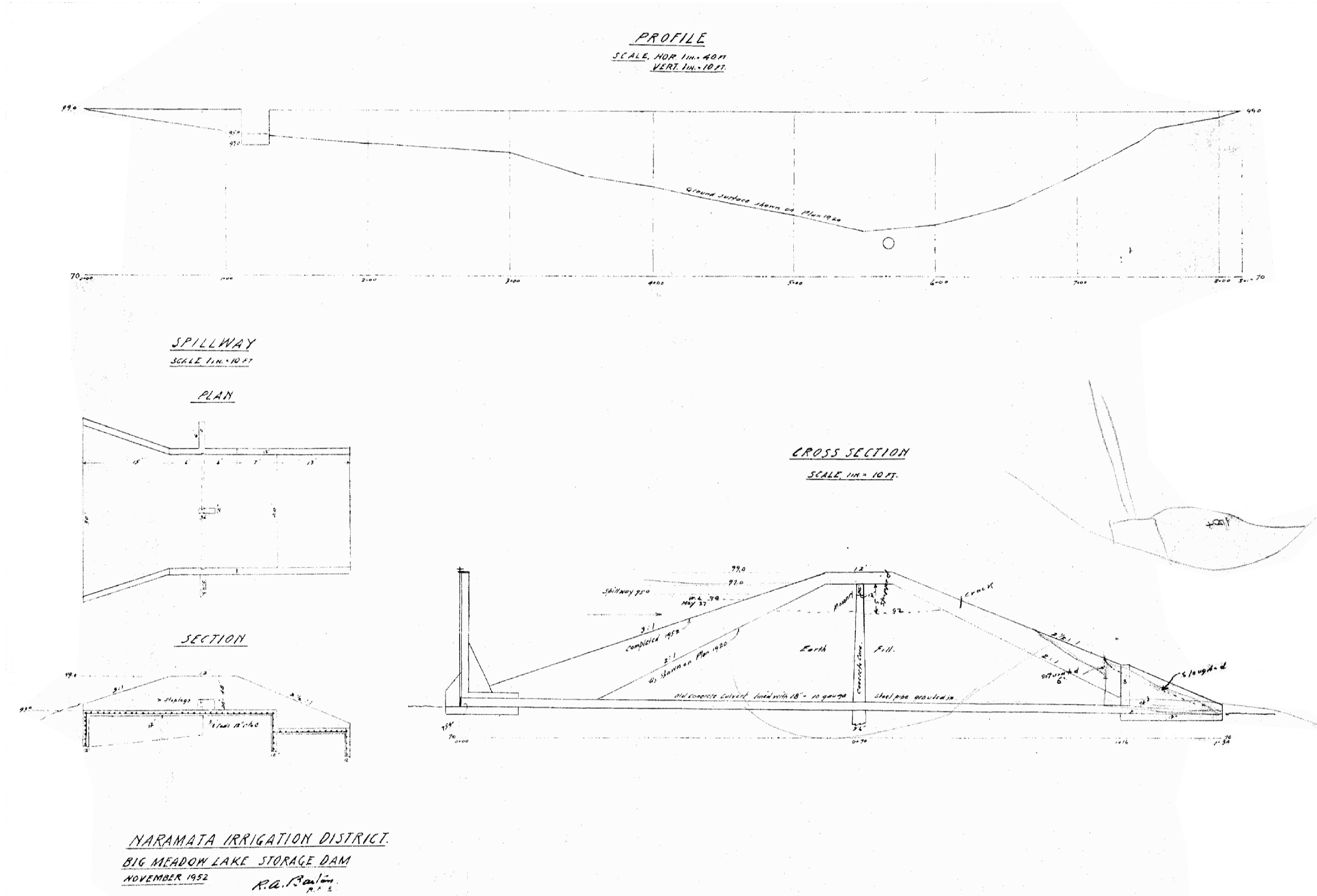
RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Big Meadow Lake Dam
Plan of Reservoir

PROJECT NO.	DWN	CKD	REV	
K13101459.001	LM	MJL	0	**Figure 3**
OFFICE	DATE			
EBA-KELOWNA	December 14, 2010			



CLIENT

RDOS OKANAGAN-SIMILKAMEEN

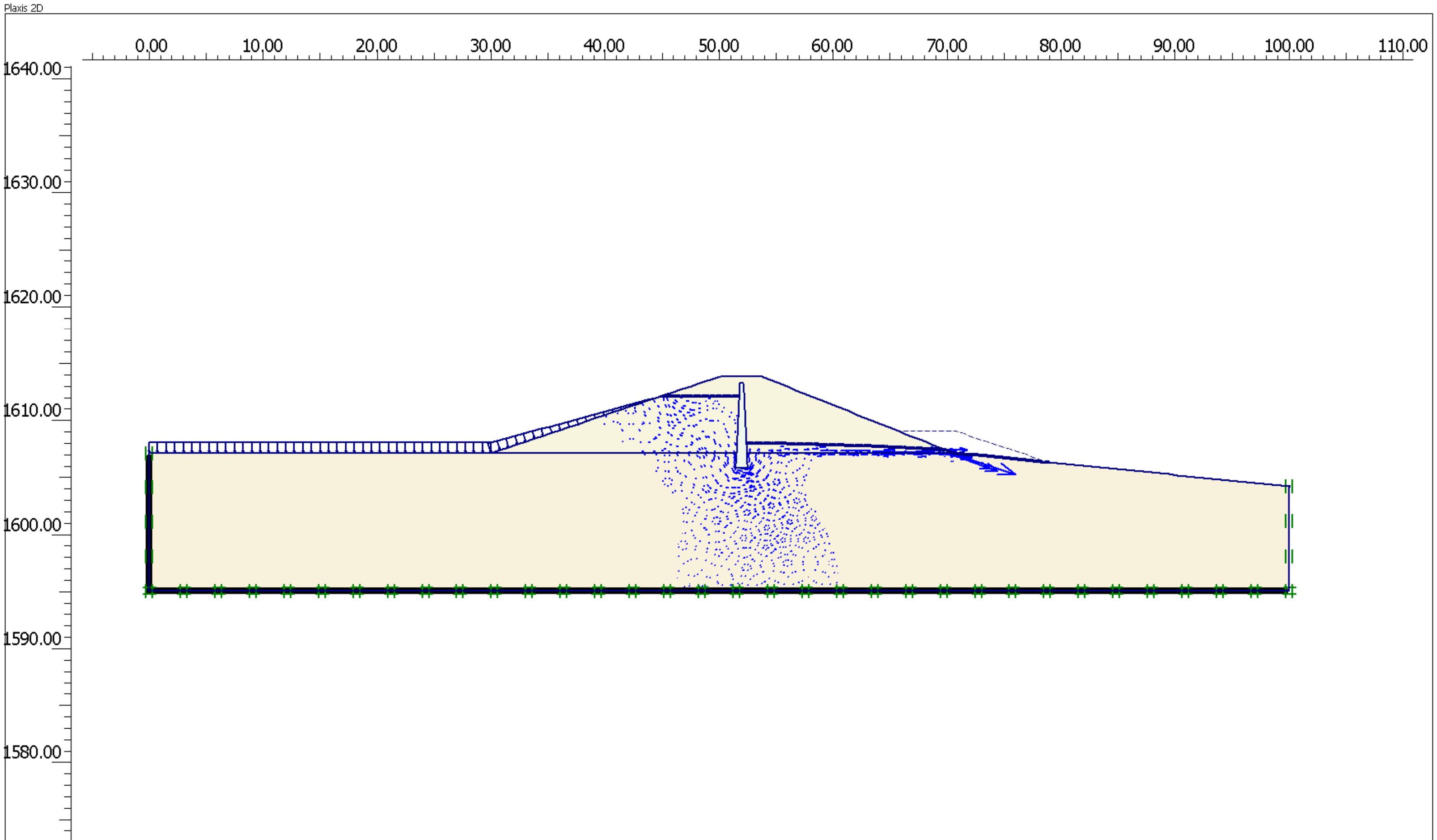
BIG MEADOW LAKE DAM SAFETY REVIEW

Big Meadow Lake Dam
Profile, Embankment Section & Spillway Details

EBA Engineering Consultants Ltd.

PROJECT NO.	DWN	CKD	REV
K13101459.001	LM	MJL	0
OFFICE	DATE		
EBA-KELOWNA	December 14, 2010		

Figure 4



Flow field

Extreme velocity 12.34 m/day

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description: Big Meadow Lake Dam	
Version 9.0.1.1182	Project name / Date: 11/12/10	User name: EBA Engineering Consultants Ltd.

NOTES

Toe seepage of 6.09 m^3/day/m (4.23 l/min/m) calculated.

CLIENT

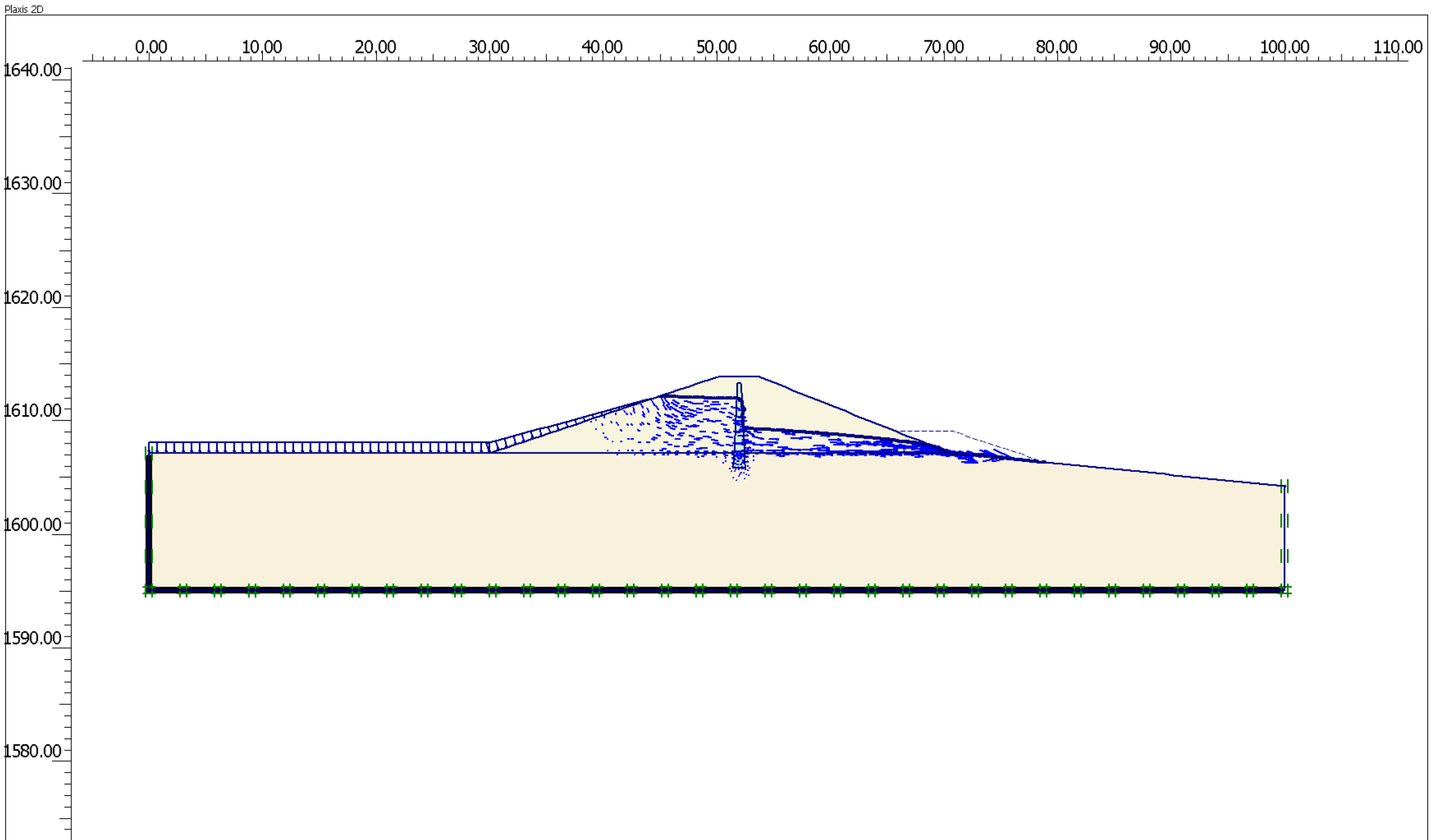
RDOS REGIONAL DISTRICT OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Uncracked Core Wall
Steady State Seepage Analysis Flow Field
Reservoir Level 1612.1 m

PROJECT NO.	DWN	CKD	REV	
K13101459.001	MJL	RP	0	Figure 5
OFFICE: EBA-KELOWNA	DATE: December 11, 2010			



Flow field

Extreme velocity 31.48 m/day

PLAXIS Finite Element Code for Soil and Rock Analyses	Project description Big Meadow Lake Dam		
	Project name	Date 11/12/10	User name EBA Engineering Consultants Ltd.

Version 9.0.1.1182

NOTES

Toe seepage of 12.35 m³/day/m (8.58 l/min/m) calculated.

CLIENT

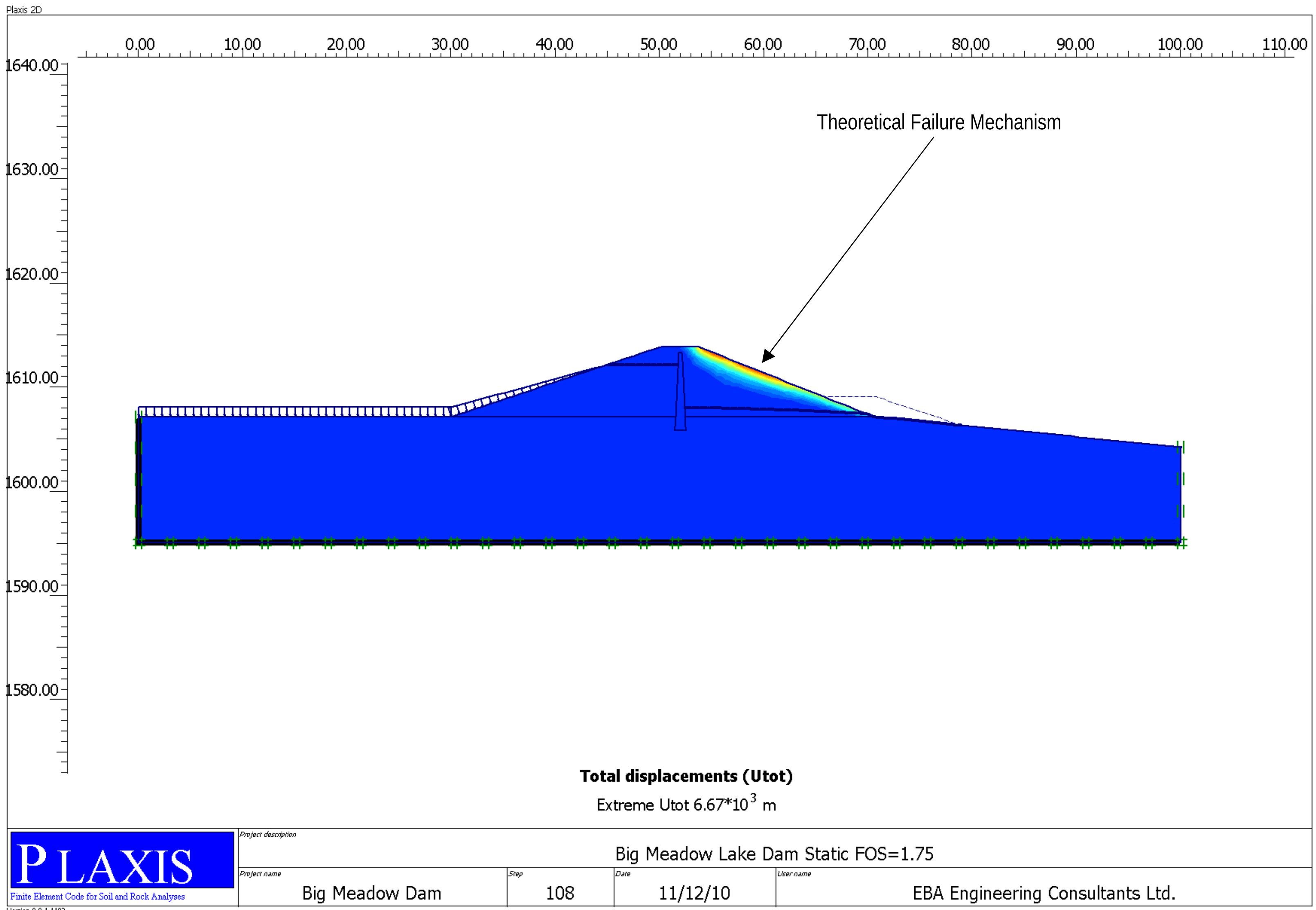
REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Cracked Core Wall
Steady State Seepage Analysis Flow Field
Reservoir Level 1612.1 m

PROJECT NO.	DWN	CKD	REV	
K13101459.001	MJL	RP	0	Figure 6
OFFICE	DATE			
EBA-KELOWNA	December 11, 2010			



NOTES

The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.

CLIENT

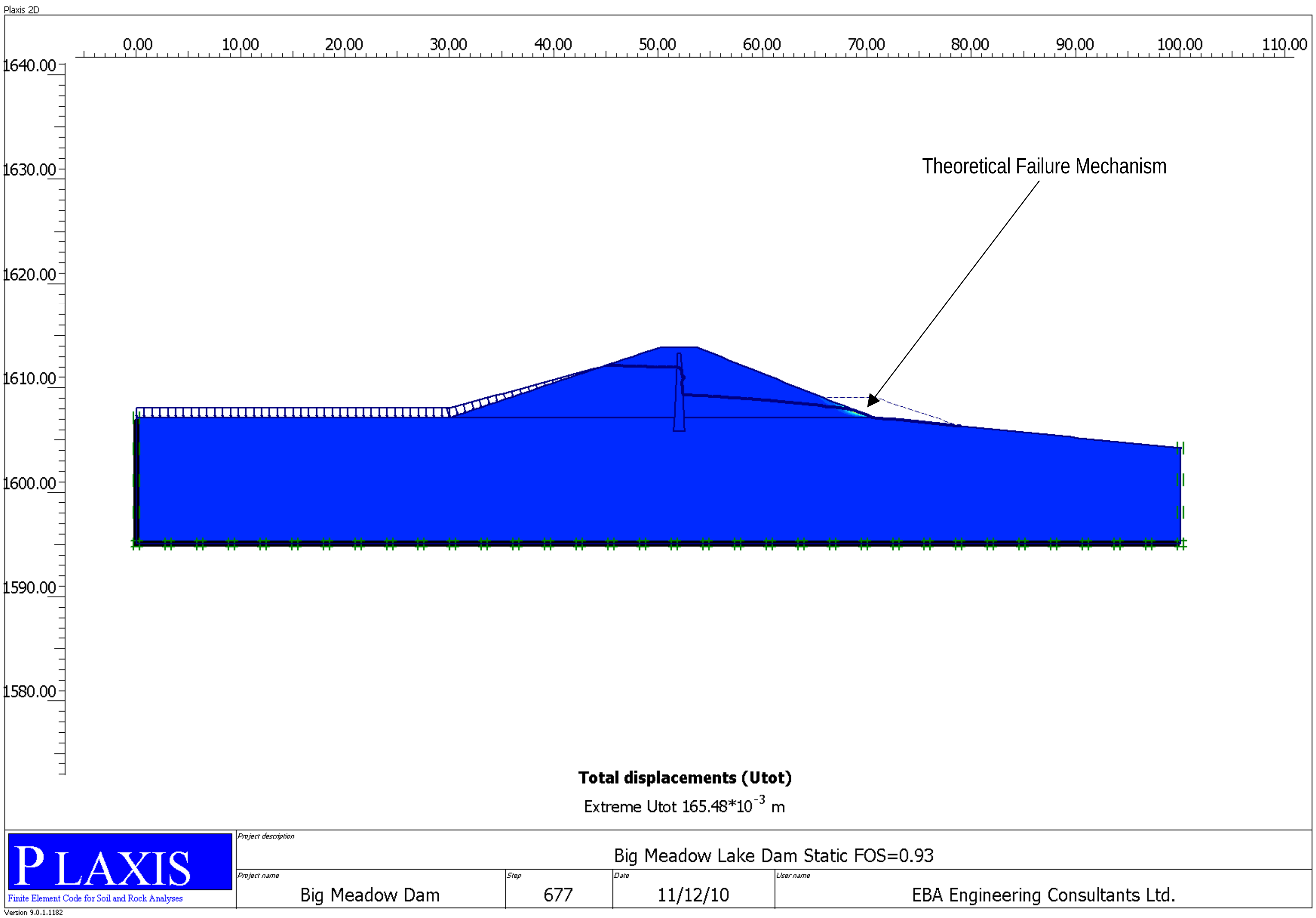
REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Uncracked Core Wall
Static Stability Analysis
Reservoir Level 1612.1 m

PROJECT NO.	DWN	CKD	REV	
K13101459	MJL	RP	0	Figure 7
OFFICE	DATE			
EBA-KELOWNA	December 11, 2010			



NOTES

The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.

CLIENT

REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

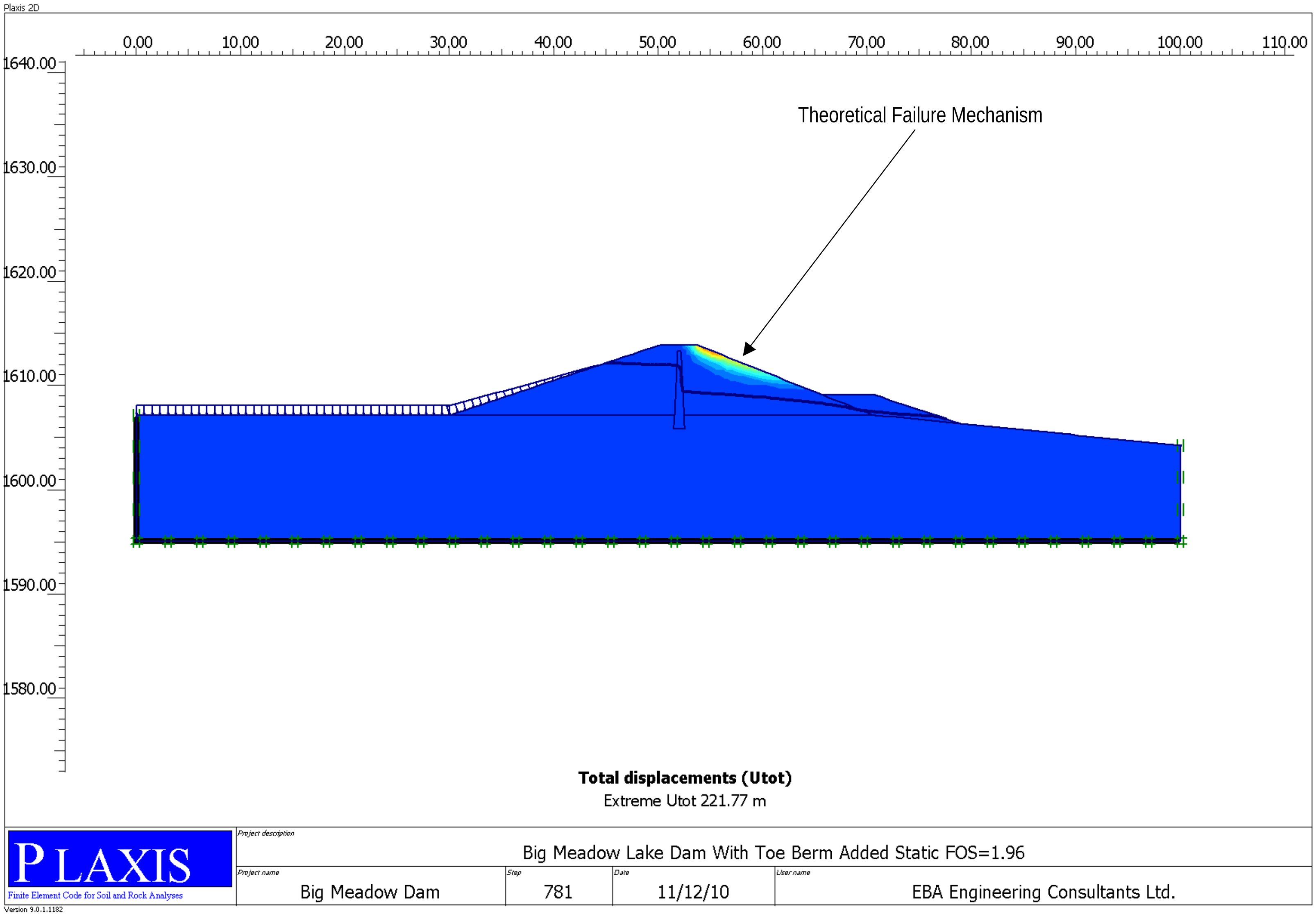
BIG MEADOW LAKE DAM SAFETY REVIEW

Cracked Core Wall Static Stability Analysis Reservoir Level 1612.1 m

PROJECT NO.	DWN	CKD	REV
K13101459	MJL	RP	0

OFFICE	DATE
EBA-KELOWNA	December 11, 2010

Figure 8



NOTES

The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.

CLIENT

REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

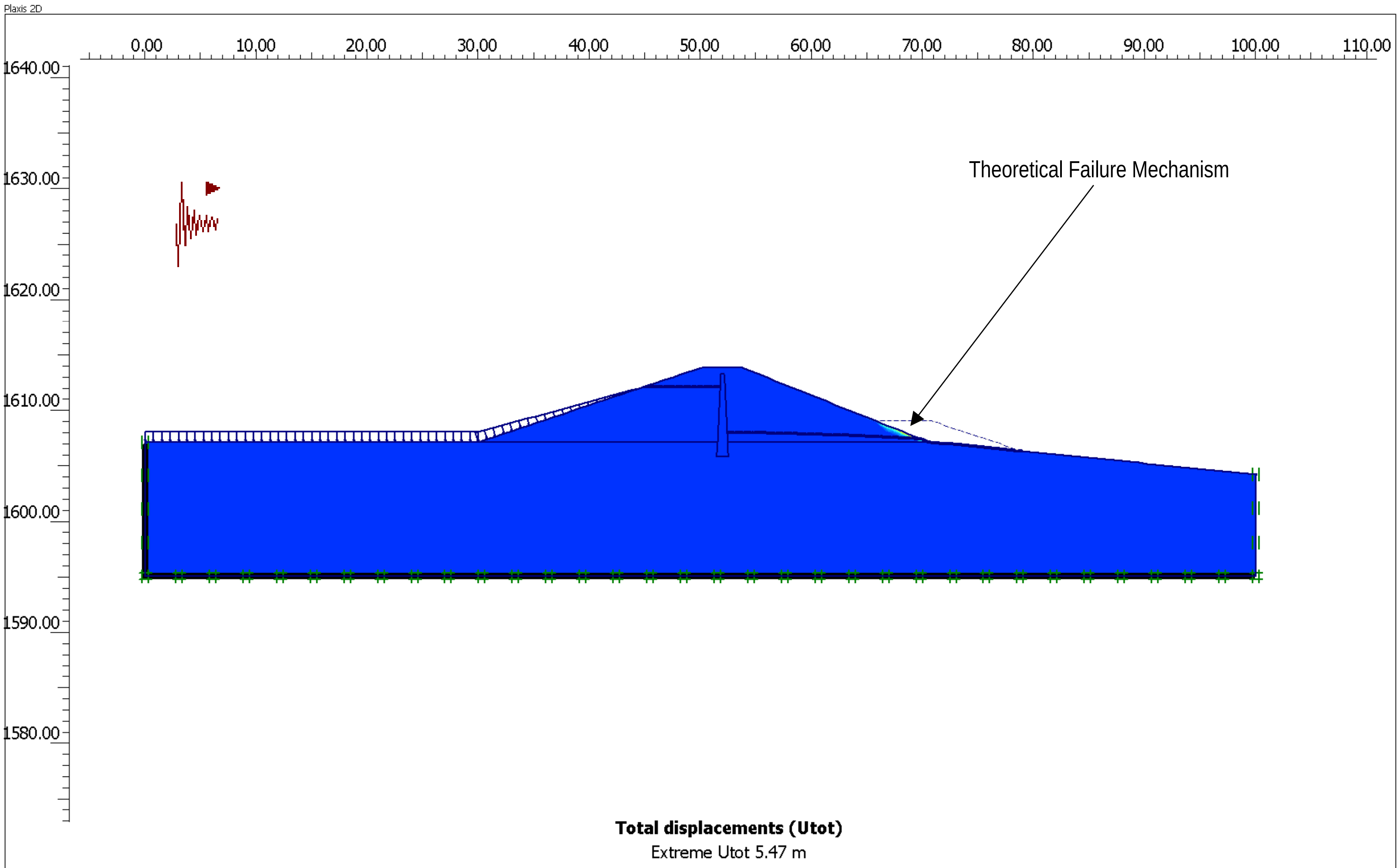
EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Cracked Core Wall With Toe Berm Added
Static Stability Analysis
Reservoir Level 1612.1 m

PROJECT NO.	DWN	CKD	REV
K13101459	MJL	RP	0
OFFICE	DATE		
EBA-KELOWNA	December 11, 2010		

Figure 9



PLAXIS Finite Element Code for Soil and Rock Analyses	Project description			
	Big Meadow Lake Dam Downstream Seismic FOS=1.04			
	Project name: Big Meadow Dam	Step: 231	Date: 11/12/10	User name: EBA Engineering Consultants Ltd.

Version 9.0.1.1182

NOTES

The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.

CLIENT

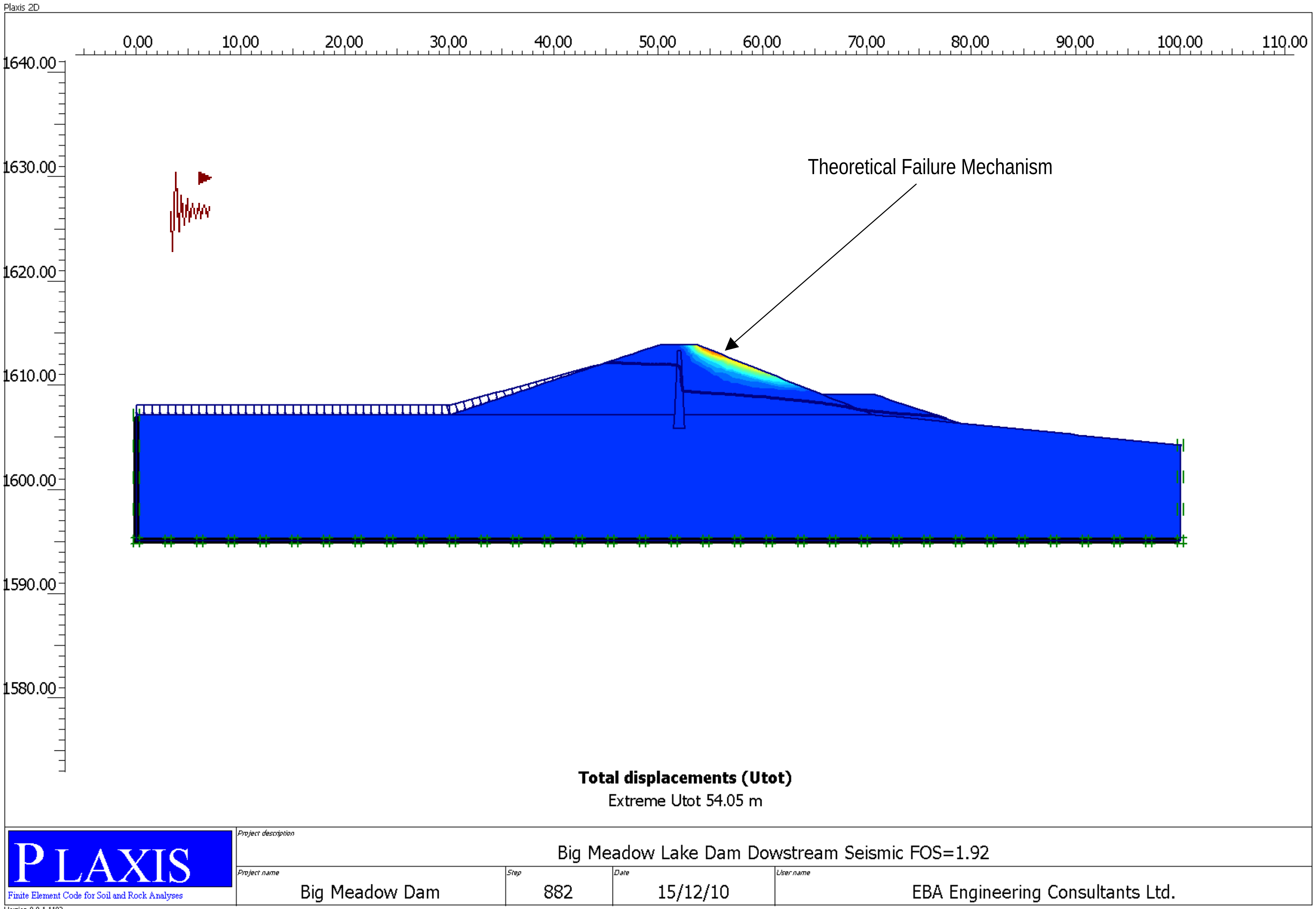
REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Uncracked Core Wall
Pseudo-Static Seismic Stability Analysis
Downstream Earthquake

PROJECT NO.	DWN	CKD	REV	
K13101459	MJL	RP	0	Figure 10
OFFICE: EBA-KELOWNA	DATE: December 11, 2010			



NOTES

The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.

CLIENT

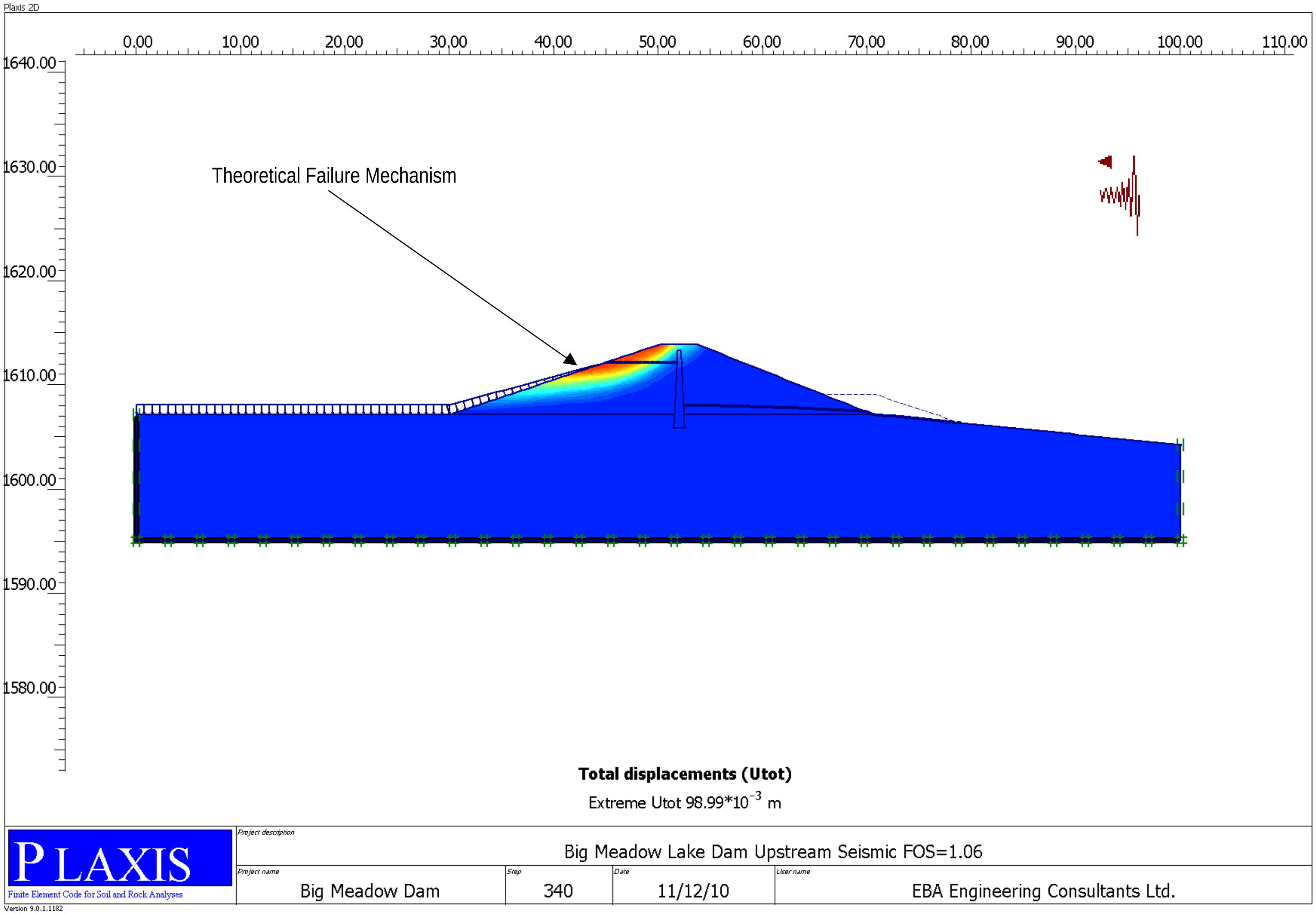
RDOS Regional District Okanagan-Similkameen

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

**Cracked Core Wall with Toe Berm Added
Pseudo-Static Seismic Stability Analysis
Downstream Earthquake**

PROJECT NO.	DWN	CKD	REV	
K13101459	MJL	RP	0	Figure 11
OFFICE	DATE			
EBA-KELOWNA	December 15, 2010			



NOTES

The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.

CLIENT

REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

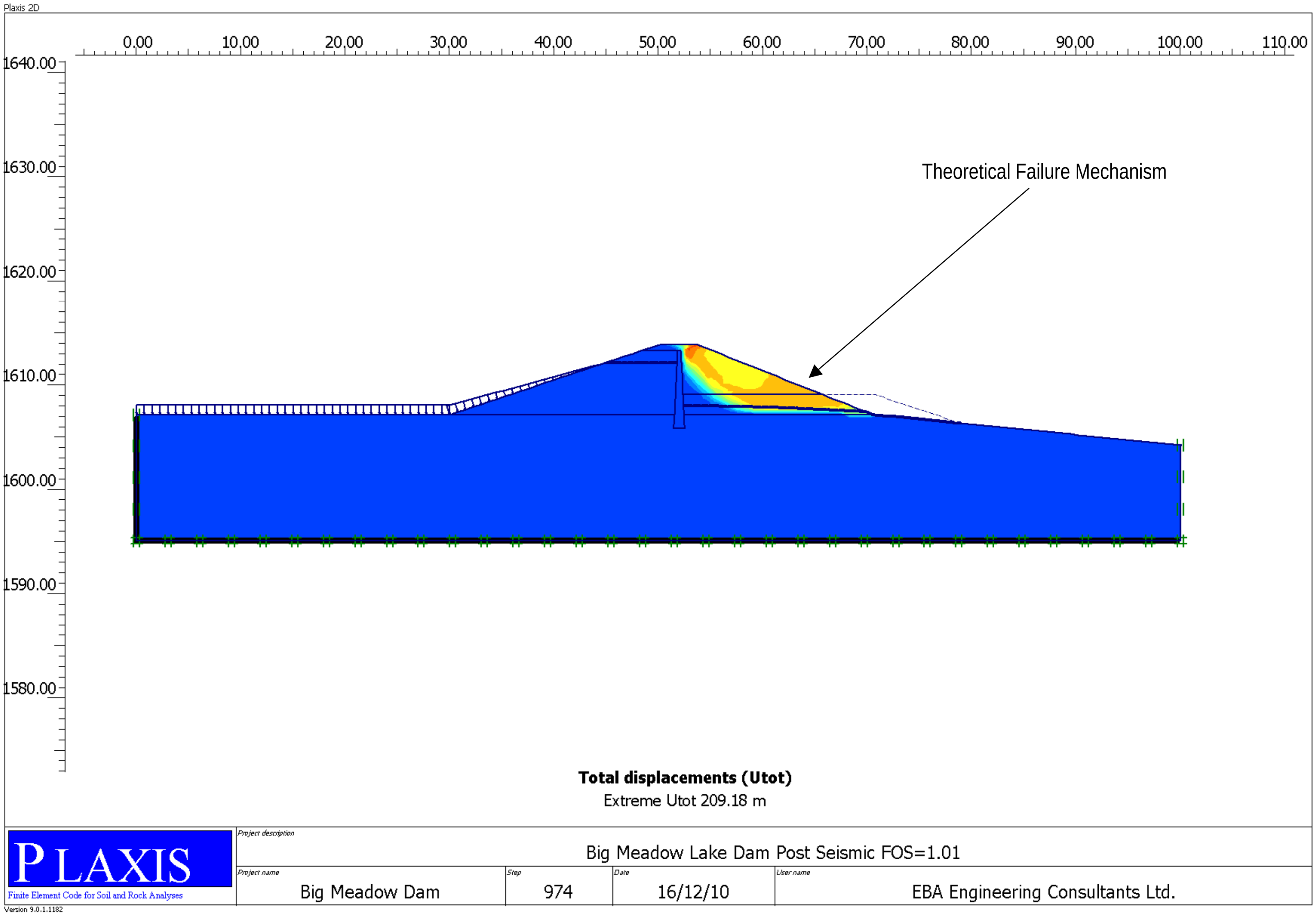
EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Uncracked Core Wall
Pseudo-Static Seismic Stability Analysis
Upstream Earthquake

PROJECT NO.	DWN	CKD	REV
K13101459	MJL	RP	0
OFFICE	DATE		
EBA-KELOWNA	December 11, 2010		

Figure 12



NOTES

1. The phi-c reduction procedure in PLAXIS© generates additional, large, non-physical displacements with the shape of the deformed mesh and pattern of the displacements giving an indication of the shape of the theoretical failure mechanism.
2. Undrained residual shear strength of soil, S_r estimated for an equivalent clean sand SPT corrected blow count of 10 from Figure 88 of Idriss & Boulanger (2008).

CLIENT

REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

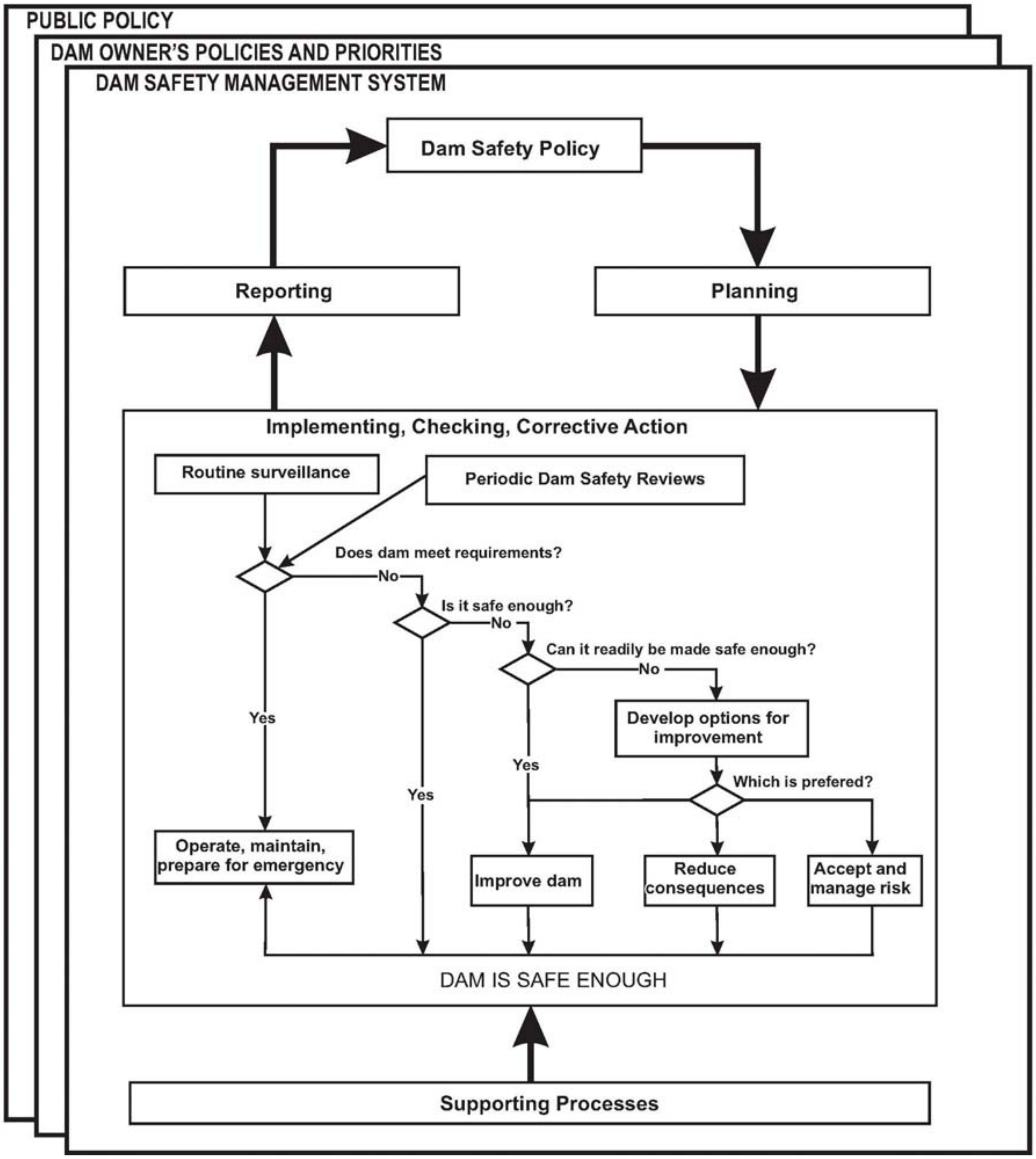
BIG MEADOW LAKE DAM SAFETY REVIEW

Residual Shear Strength of Liquefied Soil
Post Seismic Stability Analysis

PROJECT NO.	DWN	CKD	REV
K13101459	MJL	RP	0

OFFICE	DATE
EBA-KELOWNA	December 16, 2010

Figure 13



CLIENT

REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Dam Safety Management System

PROJECT NO.	DWN	CKD	REV
K13101459.001	TP	MJL	0
OFFICE	DATE		
EBA-Kelowna	December 14, 2010		

Figure 14

NOTES

From the BC MoE report, Response to Recommendations Contained in the Report: "Review of the Testalinden Dam Failure" (July 2010), October 2010.

CLIENT

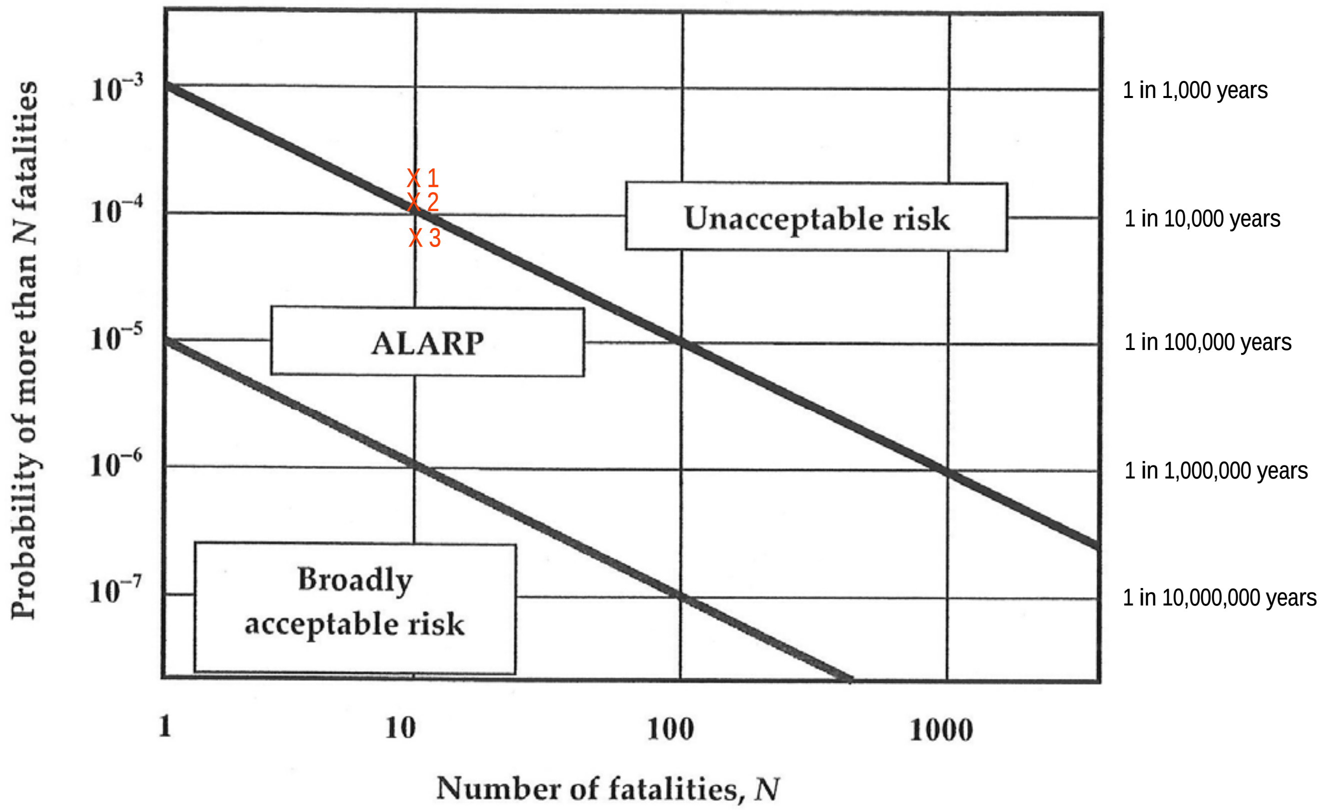
REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Proposed BC MoE
Dam Signage Requirements

PROJECT NO.	DWN	CKD	REV	
K13101459	MJL	RP	0	Figure 15
OFFICE	DATE			
EBA-KELOWNA	December 13, 2010			



Calculated Annual Likelihood of Piping Failure:

1. Big Meadow Lake Dam existing.
2. Big Meadow Lake Dam with a toe berm incorporating a filter.
3. Big Meadow Lake Dam with a toe berm incorporating a filter and an improved seepage monitoring program.

CLIENT

REGIONAL DISTRICT RDOS OKANAGAN-SIMILKAMEEN

EBA Engineering Consultants Ltd.

BIG MEADOW LAKE DAM SAFETY REVIEW

Societal Risk Criteria for Dam Safety

PROJECT NO.	DWN	CKD	REV
K13101459.001	TP	MJL	0
OFFICE	DATE		
EBA-Kelowna	December 14, 2010		

Figure 16

PHOTOGRAPHS

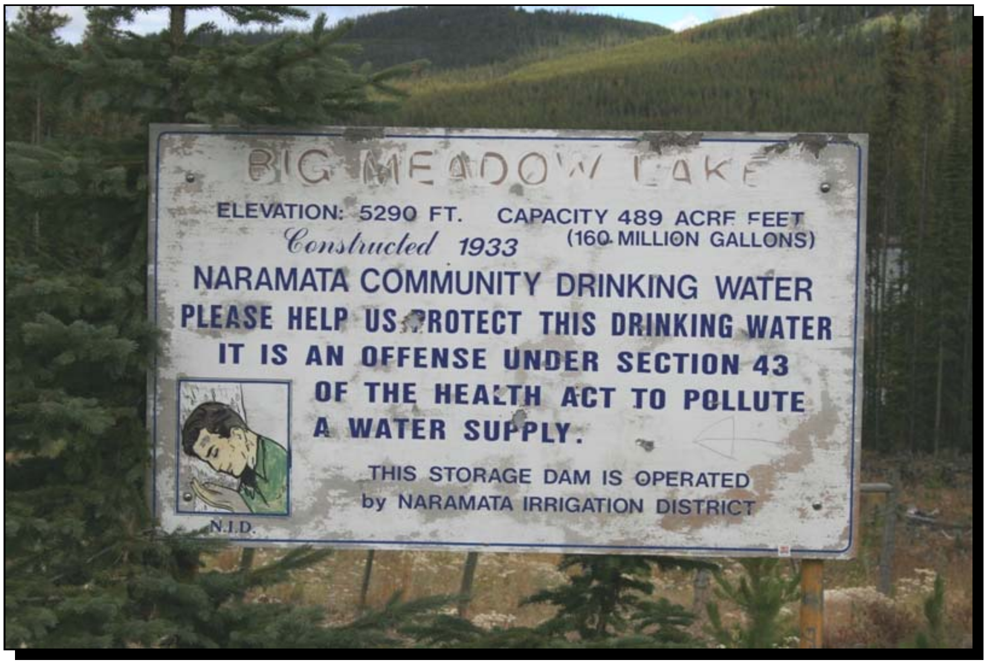


Photo 1
Big Meadow Lake Dam — Information Board

Photo 2
Big Meadow Lake Dam — Downstream Face from left abutment

Photo 3
Big Meadow Lake Dam — Vegetation growing on left side of Downstream Face

Photo 4
Big Meadow Lake Dam — Upstream Face from right abutment

Photo 5
Big Meadow Lake Dam — Intake Structure Stem Guide

Photo 6
Big Meadow Lake Dam — Gate Hoist Head-block

Photo 7
Big Meadow Lake Dam — Spillway structure with Stop logs removed

Photo 8
Big Meadow Lake Dam — Spillway structure downstream view

Photo 9
Big Meadow Lake Dam — Low-level outlet structure

Photo 10
Big Meadow Lake Dam — Seepage from right of low-level outlet structure

Photo 11
Big Meadow Lake Dam — ATV tracks on Downstream Face adjacent to low-level outlet structure

Photo 12
Big Meadow Lake Dam — Left abutment loss of freeboard due to vehicle traffic

Photo 13
Big Meadow Lake Dam — Erosion of upstream face and accumulation of debris adjacent to right abutment

Photo 14
Big Meadow Lake Dam — Wood debris in Weir downstream of low-level outlet

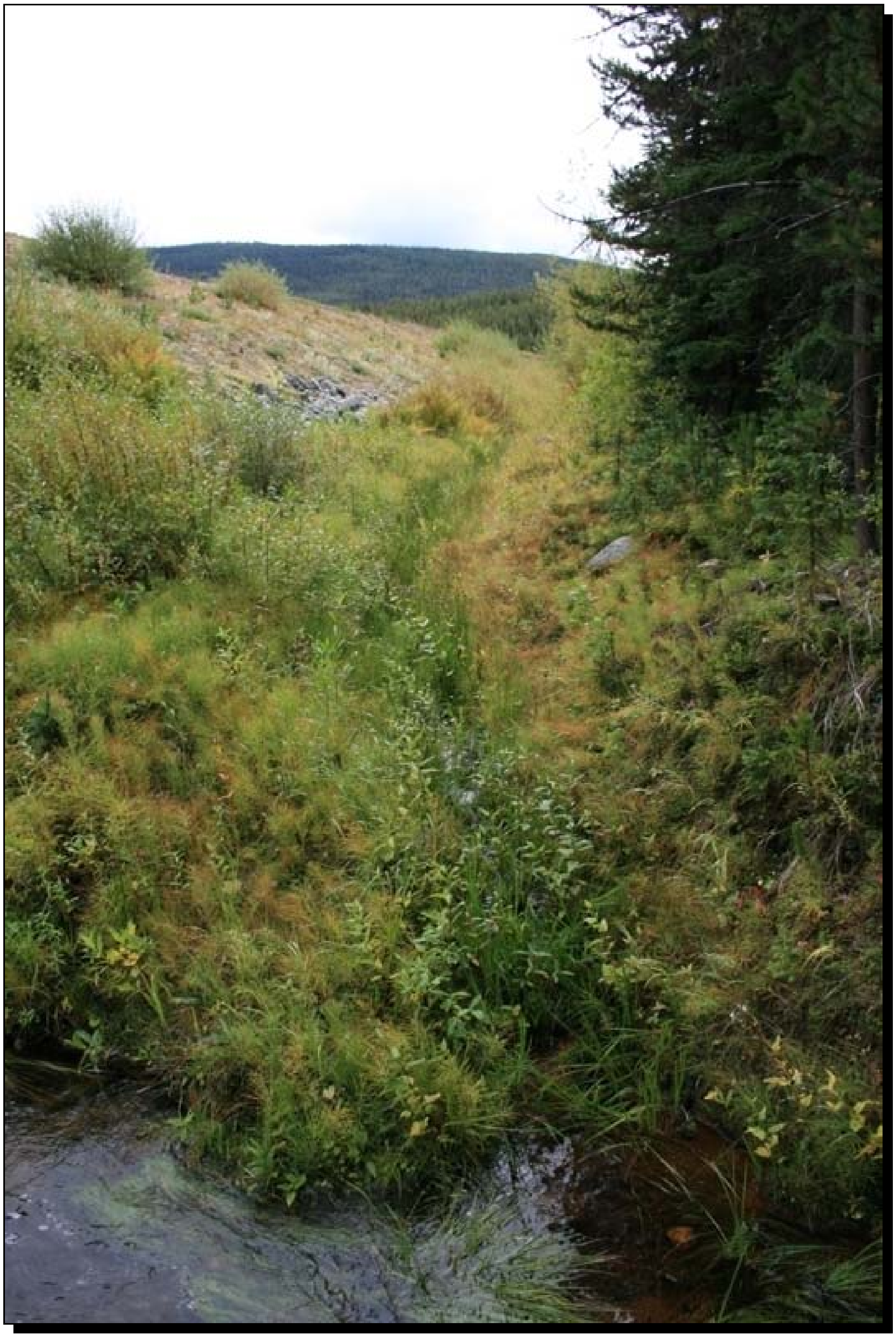


Photo 15
Big Meadow Lake Dam — Toe drain left-hand side of downstream face

Photo 16
Chute Creek Outlet

APPENDIX A

APPENDIX A BACKGROUND INFORMATION REVIEW

APPENDIX A: BACKGROUND INFORMATION REVIEW

SOURCES OF BACKGROUND INFORMATION REVIEWED FOR 2010 DAM SAFETY REVIEW

APPENDIX A1: DRAWINGS

General

- Stanley Associated Engineering Ltd, Chute Lake Diversion – Existing Structure, October 1993.

Big Meadow Dam

- Unknown, Big Meadow Lake Storage Dam, November 1952.
- BC Department of Lands, Forests and Water Resources, Big Meadow Reservoir – Plan of Storage, April, 8 1963.
- BC Department of Lands, Forests and Water Resources, Big Meadow Dam – Details of Repairs to Culvert Gate & Outlet, September, 19 1966.
- BC MOE, Big Meadow Lake Reservoir – Plan of Reservoir, March 1982.

APPENDIX A2: REPORTS AND INSPCETIONS

RDOS conducted a record check at EBA's request to see what records or documents they might have that could be of use for the Dam Safety Review. The following records were found:

• Big Meadow – Water Rights Branch Dept of Lands and Forests letter	1961 Aug 16
• Big Meadow – UMA Letter Field Inspection of Water Reservoir Storage Dams	1977 Jul 11
• Big Meadow – Storage Capacity Table Reservoir Inventory No. 5 420acre-ft	1979 Apr 26
• Big Meadow – MoE Memo – Re-establishing Mapping Control Targets	1980 Jul 9
• Big Meadow MoE Letter	1980 Oct 23
• Big Meadow MoE Letter	1982 May 21
• Big Meadow MoE Letter and Dam Inspection Report	1983 Dec 7
• Big Meadow MoE Letter Construction of Access Road to Big Meadow Dam	1985 Sep 26
• Big Meadow MoE Letter and Dam Inspection Report	1987 Oct 28
• Big Meadow MoE Letter and Dam Inspection Report	1990 Jul 23
• Big Meadow MoE Inspection Letter Report	1991 Aug 27
• Big Meadow — Golder Report Geotechnical Consultation – Seepage in area of recent dam repair	2004 Jul 14
• Big Meadow MoF Memo – Seepage Issues	2004 Jul 28
• Big Meadow MoF Memo – Repair of Big Meadow Lake Dam	2009 Oct 16

APPENDIX B

APPENDIX B DAM INSPECTION NOTES

APPENDIX B: SITE INSPECTION OBSERVATIONS OF BIG MEADOW LAKE DAM

GENERAL DESCRIPTION OF DAM

Date:	September 16, 2010	Attendees:	AG (EBA), MJL (EBA), RP(EBA), AEH (RDOS), DC (RDOS)
Weather:	Sunny, Clear to Cloudy	Location:	11U 322175 m E 5505870 m N
Length:	249 m	Outlet type:	Drop inlet culvert
Max. Height	7.5 m	Sluice gate:	Slide gate with inclined stem hoist
Crest Elevation	1613.92 m	Spillway:	Small chute channel
Crest Width:	3.6 m	Spillway Crest Elevation:	1612.09 m
Water Level:	No reading taken, reservoir drawn down ~2 m from crest.	Downstream slope angle:	2.5H:1V (22°)
		Upstream slope angle:	3H:1V (18.5°)
Appurtenances:			

OBSERVATIONS

Location	Observation
Left Abutment	Dam information sign badly deteriorated
Spillway Structure	Log boom in place
Spillway Structure	Stop logs removed
Spillway Structure	Loss of fines from surface of spillway concrete
Spillway Structure	Seepage along left hand side of spillway structure on downstream face, flowing clear
Low Level Outlet Structure	Seepage along right hand side of low level outlet structure on downstream face, flowing clear
Low Level Outlet Channel	Wood debris accumulation in weir downstream of low level outlet structure
Crest	Some loss of freeboard at left abutment due to vehicle traffic
Crest	Minor rutting from vehicle movement along crest
Downstream Face	Erosion and rutting on downstream face above low level outlet structure from ATV or Skidoo traffic
Downstream Face	Scrubby vegetation on left hand side of downstream face
Downstream Face	Noticeable seepage from LHS and RHS toe drains (Big-O pipe) into low level outlet channel, flowing clear
Upstream Face	Beaching along the crest of the upstream face and some minor wood debris accumulation
Upstream Face	Erosion, over steepening of upstream face and wood debris accumulation adjacent to the right abutment

APPENDIX C

APPENDIX C UNSW PIPING FAILURE RISK ASSESSMENT

APPENDIX C: 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:

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

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 Naramata Dams 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 for the Naramata Dams EBA has assumed the following zoning categories as defined in Table 1 from the Foster et al. (2000b);

- Big Meadow Lake Dam, Earthfill with core wall.
- Elinor North (Saddle) Lake Dam, Central core earth and rockfill.
- Elinor South Lake Dam, Central core earth and rockfill.
- Naramata Lake Dam, Zoned earthfill.

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

The average annual probability of failure presented in Table C were 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 presented below.

TABLE C1: CALCULATION OF ANNUAL LIKELIHOOD OF PIPING FAILURE — BIG MEADOW DAM

Piping Failure Mode	Zoning Category	Average Annual Probability of Failure	Overall Weighting Facture	Weighted Likelihood of Piping Failure
Piping through embankment (P_e)	Earthfill with core wall	$P_e = 8 \times 10^{-6}$	$w_E = 8.0$	$P_e \times w_E = 64 \times 10^{-6}$
Piping through the foundation (P_f)	Earthfill with core wall	$P_f = 19 \times 10^{-6}$	$w_F = 6.0$	$P_f \times w_F = 114 \times 10^{-6}$
Piping from embankment into foundation (P_{ef})	Earthfill with core wall	$P_{ef} = 4 \times 10^{-6}$	$w_{EF} = 3.2$	$P_{ef} \times w_{EF} = 12.8 \times 10^{-6}$
Annual Likelihood of Piping Failure (P_p)				$\mathbf{P_p = 1.91 \times 10^{-4}}$

TABLE C1.1 WEIGHTING FACTORS FOR PIPING THROUGH THE EMBANKMENT MODE OF FAILURE — CALCULATION OF w_E

Factor	Big Meadow Dam	Weighting	Comment
Embankment Filters	No filter	2.0	Drawings indicate that there is no filter in the dam.
Core geological origin	N/A Concrete core wall	1.0	Condition of concrete unknown therefore neutral weighting used.
Core soil type	N/A Concrete core wall	1.0	Condition of concrete unknown therefore neutral weighting used.
Compaction	N/A Concrete core wall	1.0	Condition of concrete unknown therefore neutral weighting used.
Conduits	Conduit through embankment, typical detail.	1.0	Typical conduit details with cutoff collars.
Foundation treatment	Untreated vertical faces or overhangs in core foundation.	2.0	Previous inspections have suggested seepage around sides of core wall.
Observations of seepage	Seepage emerging on downstream slope.	2.0	Seepage rate not available, however previous inspections have indicated seepage clear.
Monitoring and surveillance	Inspections weekly	1.0	Weekly documented inspections (weather permitting), irregular seepage observations.
	w_E, product of individual weighting factors	8.00	

TABLE C1.2 WEIGHTING FACTORS FOR PIPING THROUGH THE FOUNDATION MODE OF FAILURE — CALCULATION OF w_F

Factor	Big Meadow Lake Dam	Weighting	Comment
Filters	No foundation filter	1.0	Unknown if a foundation filter was required or not, seems unlikely
Foundation below cut off	Soil foundation.	5.0	Foundation soil's granular glacial moraine deposits.
Cutoff (soil foundation)	Shallow cutoff trench	1.2	Drawings indicate a shallow cutoff trench, no dimensions provided.
Soil geology, below cutoff	Glacial	0.5	Foundation soil's granular glacial moraine deposits.

TABLE C1.2 WEIGHTING FACTORS FOR PIPING THROUGH THE FOUNDATION MODE OF FAILURE — CALCULATION OF w_F

Factor	Big Meadow Lake Dam	Weighting	Comment
Observations of seepage	Seepage emerging on downstream slope.	2.0	Seepage rate not available, however previous inspections have indicated seepage clear.
Observations of pore pressures	Unknown	1.0	No pore pressure measurement, assumed to be high pressures due to no grouting
Monitoring and surveillance	Inspections weekly	1.0	Weekly documented inspections (weather permitting), irregular seepage observations.
	wF, product of individual weighting factors	6.00	

TABLE C1.3 WEIGHTING FACTORS FOR PIPING FROM THE EMBANKMENT INTO THE FOUNDATION MODE OF FAILURE — CALCULATION OF w_{EF}

Factor	Big Meadow Lake Dam	Weighting	Comment
Filters	Factor doesn't influence piping through foundation	1	
Foundation cut off trench	Shallow or none	0.8	Drawings show a narrow foundation cut off.
Foundation	Founding on or partly on soil foundations.	0.5	Foundation soil's granular glacial moraine deposits.
Erosion control measures of foundation	No erosion-control measures, good foundation conditions.	1.0	None provided for in design, foundation soil's granular glacial moraine deposits
Grouting	Soil foundation only, not applicable	1.0	
Soil geology types	Glacial	2.0	Foundation soil's granular glacial moraine deposits.
Core geological origin	N/A Concrete core wall	1.0	Condition of concrete unknown therefore neutral weighting used.
Core soil type	N/A Concrete core wall	1.0	Condition of concrete unknown therefore neutral weighting used.
Core compaction	N/A Concrete core wall	1.0	Condition of concrete unknown therefore neutral weighting used.
Foundation treatment	Untreated vertical faces or overhangs in core foundation.	2.0	Previous inspections have suggested seepage around sides of core wall.
Observations of seepage	Seepage emerging on downstream slope.	2.0	Seepage rate not available, however previous inspections have indicated seepage clear.
Monitoring and surveillance	Inspections weekly	1.0	Weekly documented inspections, irregular seepage observations.
	w_{EF}, product of individual weighting factors	3.20	

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.

M. Foster. URS, Level 3, 116 Miller St., North Sydney, Australia 2060.
R. Fell. School of Civil and Environmental Engineering, University of New South Wales, Sydney, Australia 2052.
M. Spannagle. Department of Land and Water Conservation, GPO Box 39, Sydney, Australia 2001.

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 ($\times 10^{-6}$)			Average annual P_f ($\times 10^{-6}$)			Average annual P_{ef} ($\times 10^{-6}$)	
Zoning category	Average P_{Te} ($\times 10^{-3}$)	First 5 years operation	After 5 years operation	Average P_{Tf} ($\times 10^{-3}$)	First 5 years operation	After 5 years operation	Average P_{Tef} ($\times 10^{-3}$)	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 $w_{E(filt)}$		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 $w_{E(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil $w_{E(cst)}$	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 $w_{E(cc)}$	No formal compaction (5)	Rolled, modest control (1.2)	Puddle, hydraulic fill (1.0)		Rolled, good control (0.5)
Conduits $w_{E(con)}$	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 $w_{E(ft)}$	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 $w_{E(obs)}$	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 $w_{E(mon)}$	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, $w_E = w_{E(filt)} \times w_{E(cgo)} \times w_{E(cst)} \times w_{E(cc)} \times w_{E(con)} \times w_{E(ft)} \times w_{E(obs)} \times w_{E(mon)}$ (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:

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

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.

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 $w_{F(filt)}$		No foundation filter present when required (1.2)	No foundation filter (1.0)	Foundation filter(s) present (0.8)	
Foundation (below cutoff) $w_{F(fnd)}$	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) $w_{F(cts)}$		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) $w_{F(ctr)}$	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) $w_{F(sg)}$	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) $w_{F(rg)}$	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 $w_{F(obs)}$	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 $w_{F(obp)}$	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 $w_{F(mon)}$	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: $w_{E(x)}$ is piping through the embankment, $w_{F(x)}$ is piping through the foundation, and $w_{EF(x)}$ 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, $w_{E(filt)}$, 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

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 $w_{EF(filt)}$	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 $w_{EF(cot)}$	Deep and narrow cutoff trench (1.5)		Average cutoff trench width and depth (1.0)	Shallow or no cutoff trench (0.8)	
Foundation $w_{EF(fnd)}$		Founding on or partly on rock foundations (1.5)			Founding on or partly on soil foundations (0.5)
Erosion-control measures of core foundation $w_{EF(ecm)}$	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 $w_{EF(gr)}$		No grouting on rock foundations (1.3)	Soil foundation only, not applicable (1.0)	Rock foundations grouted (0.8)	
Soil geology types $w_{EF(sg)}$	Colluvial (5)	Glacial (2)		Residual (0.8)	Alluvial, aeolian, lacustrine, marine, volcanic (0.5)
Rock geology types $w_{EF(rg)}$	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 $w_{EF(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil type $w_{EF(cst)}$	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 $w_{EF(cc)}$	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 $w_{EF(ft)}$	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 $w_{EF(obs)}$	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 $w_{EF(mon)}$	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.

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 $w_{\mathrm{E(cc)}} = 1.0$, 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 $w_{\mathrm{E(obs)}}$ 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 $w_{\mathrm{F(filt)}}$, as shown in Table 3. Dams with no foundation filters on low-permeability and nonerodible foundations would not be expected to require foundation filters and so a lower weighting is suggested.

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

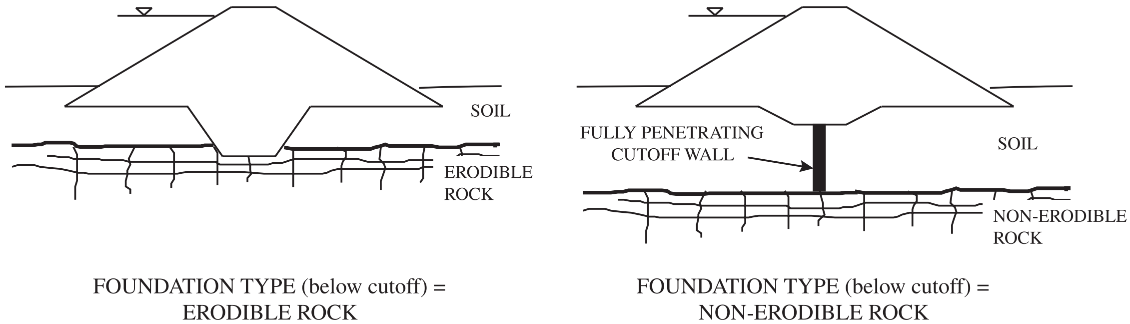
The three categories of foundation below the "cutoff" of the dam are soil foundations; erodible rock foundations, with erodible materials present such as clay-filled joints or infilled karstic channels; and non-erodible rock foundations. The cutoff is either a cutoff trench or a sheetpile or slurry trench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

There should be a good basis for selecting the nonerodible 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 ($w_{\mathrm{F(cts)}}$) for soil foundations should be used; for dams with cutoffs on rock foundations only, use $w_{\mathrm{F(ctr)}}$. 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 ×

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



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.

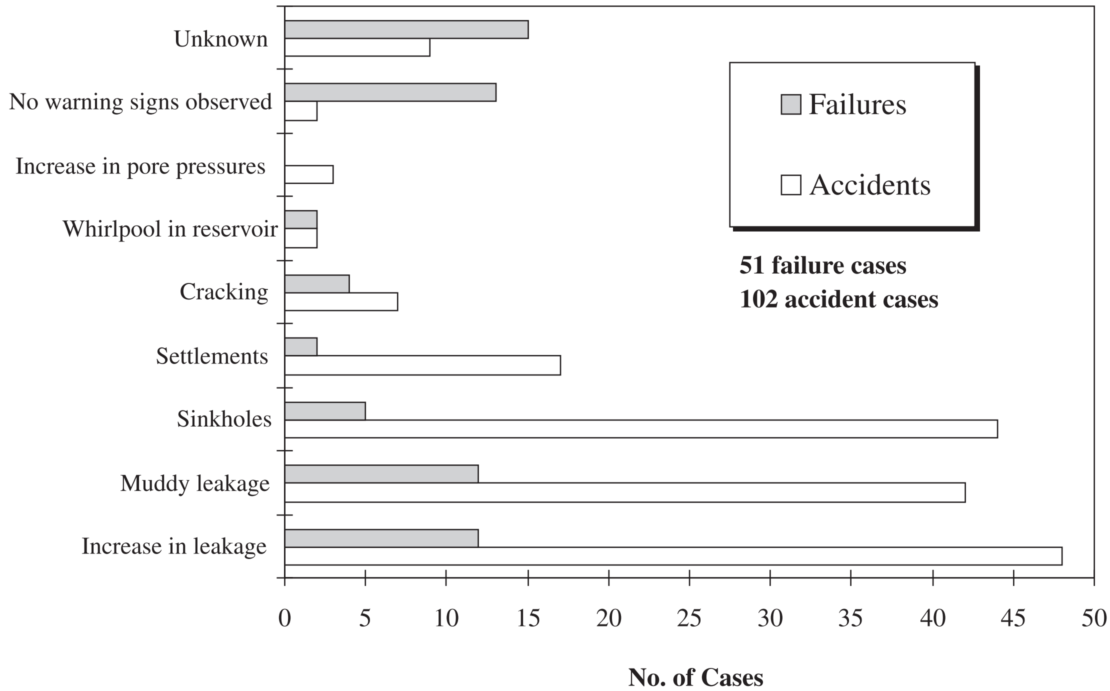
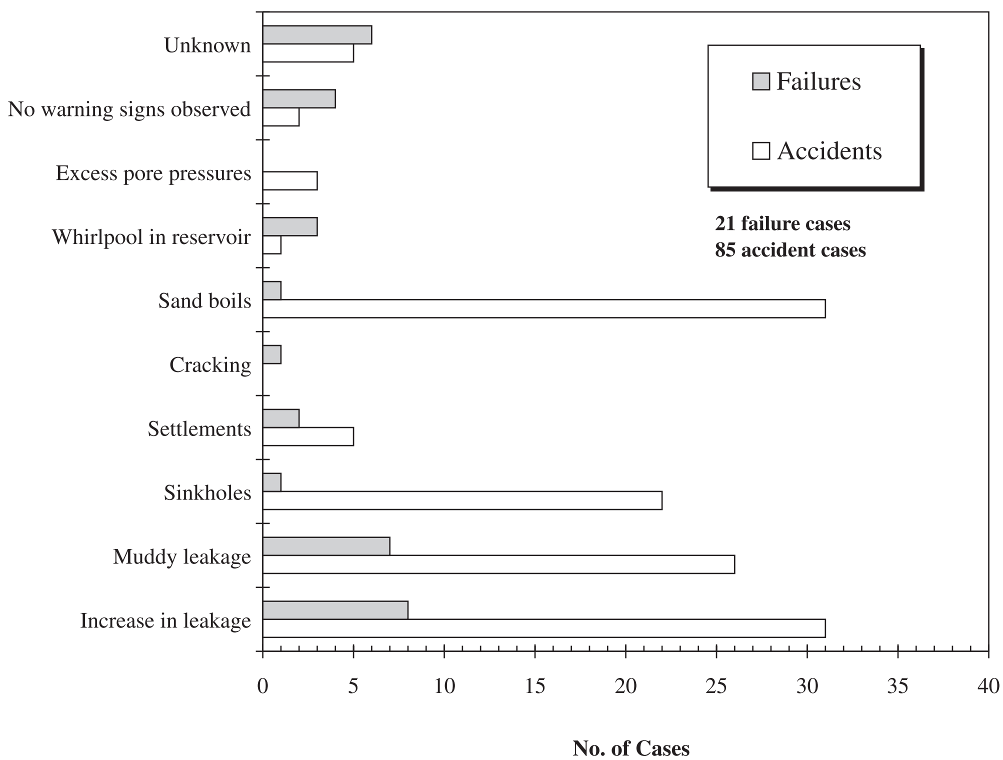


Fig. 3. Observations during piping incidents, with piping through the foundation.

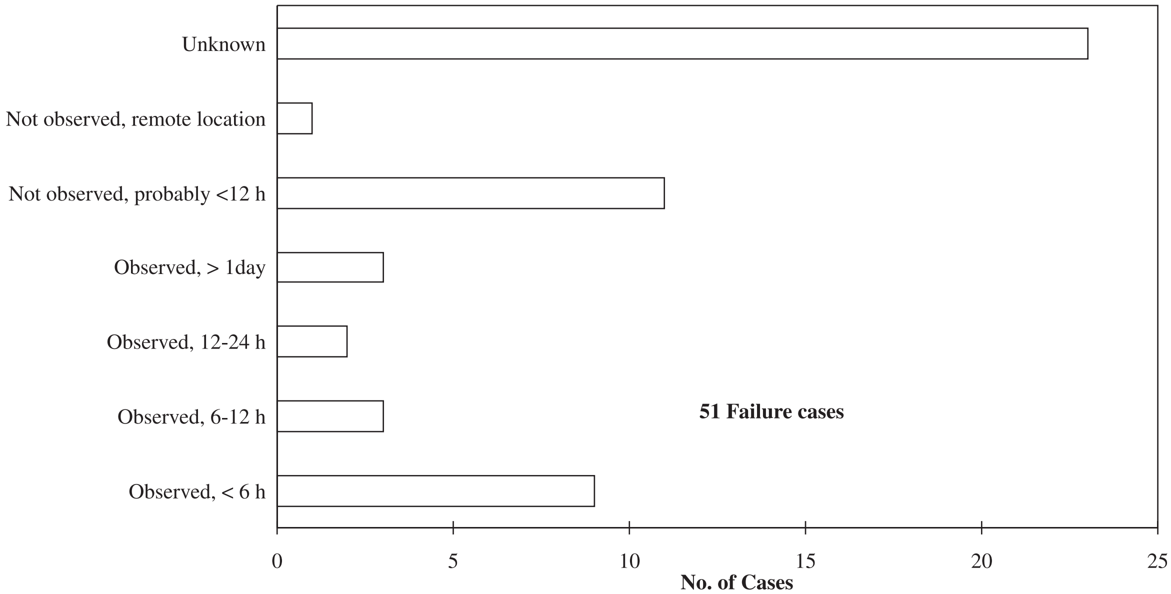


Piping from embankment to foundation

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

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.

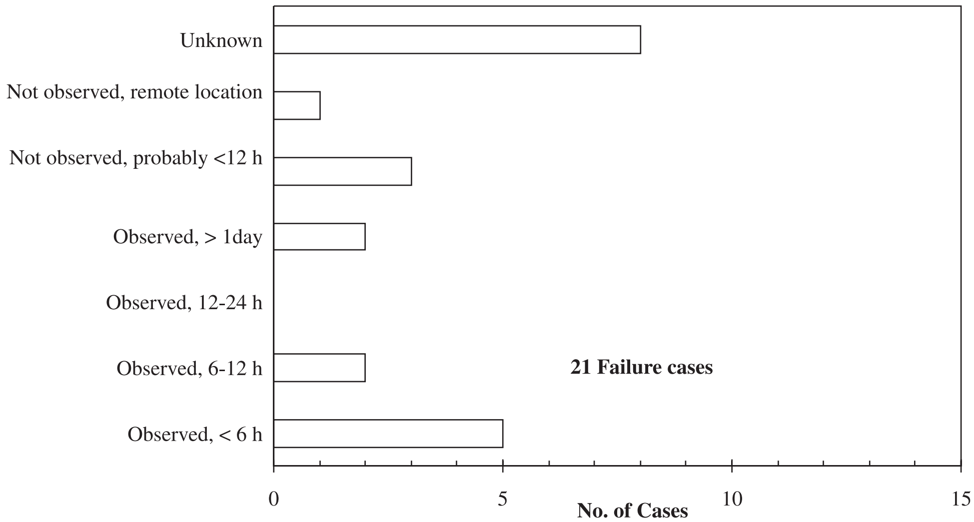
Descriptions of the observations leading up to and during the piping incidents for all of the failure cases and for a select group of accident cases are given in Appendix 1. It is evident that in a few of the failure cases the dams were poorly maintained and remedial work was not carried out despite 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.

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.

References

Arulanandan, K., and Perry, E.B. 1983. Erosion in relation to filter design criteria in earth dams. Journal of the Geotechnical Engineering Division, ASCE, **109**(GT5): 682–698.

Baab, A.O., and Mermel, T.W. 1968. Catalog of Dam Disasters, Failures and Accidents. US Department of Interior, Bureau of Reclamation, Washington, D.C.

Binnie, G.M. 1981. Early Victorian Water Engineers. Thomas Telford, London.

Charles, J.A., Tedd, P., and Holton, I.R. 1995. Internal erosion in clay cores of British dams. *In* Research and Development in the Field of Dams, 7–9 Sept. Crans-Montana, Switzerland, Swiss National Committee on Large Dams, pp. 59–70.

Einstein, H.H. 1997. Landslide risk – systematic approaches to assessment and management. *In* Landslide Risk Assessment, Honolulu, Hawaii. *Edited by* D.M. Cruden and R. Fell. A.A. Balkema, Rotterdam, The Netherlands, pp. 25–50.

Foster, M.A. 1999. The probability of failure of embankment dams by internal erosion and piping. Ph.D. thesis, School of Civil and Environmental Engineering, University of New South Wales, Sydney, Australia.

Foster, M., Fell, R., and Spannagle, M. 1998. Analysis of embankment dam incidents. UNICIV Report No. R-374, School of Civil and Environmental Engineering, University of New South Wales, Sydney, Australia. ISBN 85841 349 3.
Foster, M., Fell, R., and Spannagle, M. 2000. The statistics of embankment dam failures and accidents. Canadian Geotechnical Journal, **37**: 1000–1024.
Hanson, G.J., and Robinson, K.M. 1993. The influence of soil moisture and compaction on spillway erosion. Transactions of the ASAE, **36**(5): 1349–1352.
Höeg, K., Johansen, P.M., Kjaernsli, B., and Torblaa, I. 1998. Internal erosion in embankment dams. Research Project Sponsored by Norwegian Electricity Federation (EnFO). (In Norwegian with English summary.)
ICOLD. 1983. Deterioration of dams and reservoirs. International Commission on Large Dams (ICOLD). Paris.
ICOLD. 1984. World register of dams, 3rd updating. International Commission on Large Dams (ICOLD). Paris.
ICOLD. 1995. Dam failures statistical analysis. International Commission on Large Dams (ICOLD), Bulletin 99.
Johansen, P., Vick, S., and Rikartson, K. 1997. Risk analyses of three Norwegian rockfill dams. *In* Hydropower '97. Proceedings of the 3rd International Conference on Hydropower, Trondheim, Norway 30 June – 2 July. *Edited by* E. Broch, D.K. Lysne, N. Flatabö, and E. Helland-Hansen. A.A. Balkema, Rotterdam, The Netherlands, pp. 431–442.
Lambe, T.W. 1958. The engineering behaviour of compacted clay. Journal of the Soil Mechanics and Foundations Division, ASCE, **84**(SM2): 1655-1–1655-35.
Landon-Jones, I., Wellington, N.B., and Bell, G. 1996. Risk assessment of Prospect Dam. Australian National Committee on Large Dams (ANCOLD), ANCOLD Bulletin, Issue 103, pp. 16–31.
Mostyn, G.R., and Fell, R. 1997. Quantitative and semiquantitative estimation of the probability of landsliding. *In* Landslide Risk Assessment, Honolulu, Hawaii. *Edited by* D.M. Cruden and R. Fell. A.A. Balkema, Rotterdam, The Netherlands, pp. 297–316.
Peck, R. 1990. Interface between core and downstream filter. *In* Proceedings of the H. Bolton Seed Memorial Symposium. BiTech Publishers, Vancouver, Vol. 2, pp. 237–251.
Ripley, C.F. 1983. Discussion of: Design of filters for clay cores of dams. Journal of Geotechnical Engineering, ASCE, **109**(9): 1193–1195.
Ripley, C.F. 1984. Discussion of: Progress in rockfill dams. Journal of Geotechnical Engineering, ASCE, **114**(2): 236–240.
Ripley, C.F. 1986. Internal stability of granular filters: Discussion. Canadian Geotechnical Journal, **23**: 255–258.
Sherard, J.L. 1953. Influence of soil properties and construction methods on the performance of homogeneous earth dams. Ph.D. thesis, Harvard University, Boston, Mass.
Sherard, J.L. 1973. Embankment dam cracking. *In* Embankment dam engineering. *Edited by* R.C. Hirschfeld and S.J. Poulos. John Wiley & Sons, New York, pp. 271–353.
Sherard, J.L. 1985. Hydraulic fracturing in embankment dams. *In* Proceedings, Symposium on Seepage and Leakage from Dams and Impoundments. *Edited by* R.L. Volpe and W.E. Kelly. American Society of Civil Engineers, New York, pp.115–141.
Sherard, J.L., and Dunnigan, L.P. 1989. Critical filters for impervious soils. Journal of Geotechnical Engineering, ASCE, **115**(7): 927–947.
Sherard, J.L., Woodward, R.J., Gizienski, S.F., and Clevenger, W.A. 1963. Earth and earth-rock dams. John Wiley & Sons, New York.
USBR. 1977. Design of small dams. United States Bureau of Reclamation (USBR), United States Department of the Interior, Denver, Colo.
USBR. 1989. Design standards no. 13 — embankment dams. United States Bureau of Reclamation, United States Department of the Interior, Denver, Colo.
Vanmarke, E.H. 1977. Probabilistic modeling of soil profiles. Journal of the Geotechnical Engineering Division, ASCE, **103**(GT11): 1227–1246.
Vick, S.G. 1997. Internal erosion risk analysis for rockfill dams. Unpublished notes submitted to the USBR Seepage/Piping Risk Assessment Workshop, United States Bureau of Reclamation (USBR), United States Department of the Interior, Denver, Colo., pp. 1–24.
Von Thun, J.L. 1996. Understanding seepage and piping failures — the no. 1 dam safety problem in the west. *In* Proceedings of Association of State Dam Safety Officials, (ASDSO) Western Regional Conference, Lake Tahoe, Nev., pp. 1–24.

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.

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning	
							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

Table A1 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A1 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A1 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A1 (*continued*).

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								
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 sink-hole 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

Table A1 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A1 (*concluded*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

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*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A2. (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning	
							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

Table A2. (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A2. (*concluded*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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.

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A3 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of failure	Description of incident	Warning: Long term	Warning: 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

Table A4. Descriptions of warnings of accidents resulting from piping through the foundation.

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning: Long term	Warning: 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

Table A4 (*continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning: Long term	Warning: 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*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning: Long term	Warning: 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

Table A5. Descriptions of warnings of failures resulting from piping from the embankment into the foundation.

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning: Long term	Warning: 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

Table A6. Descriptions of warnings of accidents resulting from piping from the embankment into the foundation.

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning: Long term	Warning: 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

Table A6 (*Continued*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning: Long term	Warning: 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

Table A6 (*Concluded*).

Name of dam	Country	Dam zoning	Height (m)	Year completed	Year of incident	Description of incident	Warning	
							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

APPENDIX D

APPENDIX D DAM SAFETY EXPECTATIONS ASSESSMENT

APPENDIX D 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 prepared by BC MoE (May 2010). Deficiencies are classified into Actual Deficiencies and Potential Deficiencies and there are 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, BC Dam Safety Regulations 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 D DAM SAFETY EXPECTATIONS — BIG MEADOW LAKE DAM

	DAM SAFETY EXPECTATIONS	Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
1	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	There was no formal as-built information, construction records and only limited drawings of the dam available during this review. It is recommended that RDOS undertake a record search of all suitable information archives, i.e. BCMoE Dam Safety Branch in Victoria.
1.2	The Dam is classified appropriately in terms of the consequences of failure including life, environmental, cultural and third-party economic losses (As of 2009 consequence classifications for dams in BC are based on the BC Regulation and CDA Guidelines – see Interim Consequence Classification Policy, February 2009 on the Dam Safety web site)	X						
1.3	Inundation study adequate to determine consequence classification. Flood and "sunny day" scenarios assessed.			X		Pd		No inundation study has been undertaken, however infrastructure and dwellings downstream of the dam are confined to a relatively small area and it is therefore easily quantifiable so an inundation study is considered unlikely to result in a change to dam consequence classification.
1.4	Hazards external and internal to the dam have been defined	X						
1.5	The potential failure modes for the dam and the initial conditions downstream from the dam have been identified	X						
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, flood)	X						
1.10	The dam has adequate freeboard for all applicable operating conditions (normal, winter, earthquake, flood)			X		Pu		Some apparent loss of freeboard at the left abutment due to vehicle activity. Topographical survey of embankment required to confirm freeboard.
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			NCm	Wood debris was observed in the weir downstream of the low level outlet.
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		Pn		The dam was designed without a filter. Seepage at the toe of the embankment has been a well documented phenomena with previous toe drainage repairs having not permanently resolving this problem. The design of a toe berm incorporating a filter should be investigated.
1.15	Hydraulic gradients in the dams, abutments, foundations and along embedded structures are sufficiently low to prevent piping and instability			X		Pn/Pu		Seepage at the toe of the embankment has resulted in historical sloughing at the toe. Current stability analysis indicates that the toe is just within acceptable criteria for static and seismic loading. Analysis should be re-conducted based parameters derived from an intrusive borehole investigation at the site so that the nature and engineering parameters can be better defined. Should the re-analysis concluded that the toe is of marginal stability the design of a toe berm incorporating a filter should be investigated.
1.16	Slopes of an embankment have adequate protection against erosion, seepage, traffic, frost and burrowing animals			X			NCo	Dam exhibits erosion from vehicle traffic.
1.17	Stability of reservoir slopes are evaluated under all conditions and unacceptable risk to	X						Reservoir sides slopes are relatively gently sloping therefore present no perceived risk.

TABLE D DAM SAFETY EXPECTATIONS — BIG MEADOW LAKE DAM

	DAM SAFETY EXPECTATIONS	Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
	public safety, the dam or its appurtenant structures is identified.							
1.18	The need for reservoir evacuation or emergency drawdown capability as a dam safety risk control measure has been assessed.			X				
2	Operation, Maintenance and Surveillance							
2.1	Responsibilities and authorities are clearly delegated within the organization for all dam safety activities			X			NCo	No OMS has been prepared for the facility.
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	No OMS has been prepared for the facility.
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	No OMS has been prepared for the facility.
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	No OMS has been prepared for the facility.
	Operation							
2.5	Critical discharge facilities are able to operate under all expected conditions.							
a.	Flow control equipment are tested and are capable of operating as required.	X						It is understood that this is undertaken as part of RDOS weekly inspections.
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 OMS has been prepared for the facility.
d.	Management of debris and ice is carried out to ensure operability of discharge facilities			X			NCo	No OMS has been prepared for the facility.
2.6	Operating procedures take into account:							
a.	Outflow from upstream dams		X					
b.	Reservoir levels and rates of drawdown			X			NCo	No OMS has been prepared for the facility.
c.	Reservoir control and discharge during an emergency			X			NCo	No OMS has been prepared for the facility.
d.	Reliable flood forecasting information			X			NCo	No OMS has been prepared for the facility.
e.	Operator safety			X			NCo	No OMS has been prepared for the facility.
	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 have been identified			X			NCm	No OMS has been prepared for the facility.
2.8	Maintenance procedures are documented and followed to ensure that the dam remains in a safe and operational condition			X			NCm	No OMS has been prepared for the facility.
2.9	Maintenance activities are prioritized and carried out with due consideration to the consequences of failure, public safety and security			X			NCm	No OMS has been prepared for the facility.
	Surveillance							
2.10	Documented surveillance procedures for the dam and reservoir are followed to provide			X			NCs	No OMS has been prepared for the facility.

TABLE D	DAM SAFETY EXPECTATIONS — BIG MEADOW LAKE DAM							
	DAM SAFETY EXPECTATIONS	Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
	early identification and to allow for timely mitigation of conditions that might affect dam safety							
2.11	The surveillance program provides regular monitoring of dam performance, as follows:							
a.	Actual and expected performance are compared to identify deviations			X			NCs	No OMS has been prepared for the facility. No instrumentation installed in dam to monitor performance. Installation of piezometers in the embankment and abutment are recommended, and instrumentation to monitor toe seepage.
b.	Analysis of changes in performance, deviation from expected performance or the development of hazardous conditions			X			NCs	No OMS has been prepared for the facility. No instrumentation installed in dam to monitor performance. Installation of piezometers in the embankment and abutment are recommended, and monitoring of toe seepage.
c.	Reservoir operations are confirmed to be in compliance with dam safety requirements							
d.	Confirmation that adequate maintenance is being carried out			X			NCs	Assumed to be a non-conformance, no maintenance documentation was 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			NCs	No OMS has been prepared for the facility.
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						Dams inspected weekly, weather permitting and documented.
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			NCs	No OMS has been prepared for the facility.
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 there is no indication if regular dam safety training is provided to the inspector(s). As a minimum RDOS staff responsible for the EPP should attend BCMoe seminars on dam safety and inspections (understood to be provided annually in most areas of BC) as well as considering enrolling in other applicable training course put out on by others (i.e. USBR Training Aids for Dam Saftey or Similar).
2.16	Qualifications and training records of all individuals with responsibilities for dam safety activities are available and maintained			X			NCs	As Assumed to be a non-conformance as there is no indication if regular dam safety training is provided to the inspector(s).
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	No OMS has been prepared for the facility. No instrumentation installed in dam to monitor performance. Installation of piezometers in the embankment and abutment are recommended, and instrumentation to monitor toe seepage.
3	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	EPP needs to include a list of landowners in the potential downstream inundation zone and their emergency contact information.
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						
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	EPP needs to include a list of landowners in the potential downstream inundation zone and their emergency contact information.
3.4	The emergency response procedures cover the full range of flood management planning, normal operating procedures and surveillance procedures	X		X				

TABLE D DAM SAFETY EXPECTATIONS — BIG MEADOW LAKE DAM

	DAM SAFETY EXPECTATIONS	Yes	N/A	No	Deficiencies		Non-Conformances	Comments
					Actual	Potential		
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	EPP needs to include a list of landowners in the potential downstream inundation zone and their emergency contact information.
3.6	Roles and responsibilities of the dam owner and response agencies are defined.	X						
3.7	Inundation maps and critical flood information are appropriate and are available to downstream response agencies.			X		Pd		No inundation study has been undertaken, however infrastructure and dwellings downstream of the dam are confined to a relatively small area and it is therefore easily quantifiable so an inundation study is considered unlikely to result in a change to dam consequence classification.
3.8	Exercises are carried out regularly to test the emergency procedures.			X			NCp	Assumed to be a non-conformance, no documentation that exercises have been undertaken was provided.
3.9	Staff are adequately trained in the emergency procedures.			X			NCp	Assumed to be a non-conformance, no documentation that staff have been undertaken training was provided.
3.10	Emergency plans are updated regularly and updated pages are distributed to all plan holders in a controlled manner.			X			NCp	Assumed, the EPP was prepared it 2007, it is reasonable to assume some organizational changes have occurred since then.
4	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						RDOS commissioned this dam safety review. This is the first Dam Safety review for this structure. Another Dam Safety Review should be conducted in seven years (2017), however RDOS should endeavor to implement the recommendations of this review before that time.
5	Dam Safety Management System							
5.1	The dam safety management system for the dam is in place incorporating:							
a.	policies,			X			NCp	RDOS should formalize all the existing elements of dam safety management into one over-arching strategy. This would include preparing the OMS and EPP documentation, identifying the chain of organizational responsibility for the Big Meadow Lake Dam (a Dam Safety Management organizational chart) and implementing a document control system for the various elements of dam safety management.
b.	responsibilities,			X			NCp	
c.	plans and procedures including OMS, public safety and security,			X			NCp	
d.	documentation,			X			NCp	
e.	training and review,			X			NCp	
f.	prioritization and correction of deficiencies and non-conformances,	X						Prioritization and corrections of deficiencies and non-conformances are documented in this Dam Safety Review.
g.	supporting infrastructure			X			NCs	No instrumentation installed in dam.
5.2	Deficiencies are documented, reviewed and resolved in a timely manner. Decisions are justified and documented	X						Deficiencies are documented in this Dam Safety Review.
5.3	Applicable regulations are met	X						

APPENDIX E

APPENDIX E GEOTECHNICAL REPORT — GENERAL CONDITIONS

GEOTECHNICAL REPORT – GENERAL CONDITIONS

This report incorporates and is subject to these "General Conditions".

1.0 USE OF REPORT AND OWNERSHIP

This geotechnical report pertains to a specific site, a specific development and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for types of development other than that to which it refers. Any variation from the site or development would necessitate a supplementary geotechnical assessment.

This report and the recommendations contained in it are intended for the sole use of EBA's Client. EBA 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 EBA's Client unless otherwise authorized in writing by EBA. Any unauthorized use of the report is at the sole risk of the user.

This report is subject to copyright and shall not be reproduced either wholly or in part without the prior, written permission of EBA. Additional copies of the report, if required, may be obtained upon request.

2.0 ALTERNATE REPORT FORMAT

Where EBA submits both electronic file and hard copy versions of reports, drawings and other project-related documents and deliverables (collectively termed EBA's instruments of professional service), only the signed and/or sealed versions shall be considered final and legally binding. The original signed and/or sealed version archived by EBA shall be deemed to be the original for the Project.

Both electronic file and hard copy versions of EBA's instruments of professional service shall not, under any circumstances, no matter who owns or uses them, be altered by any party except EBA. EBA's instruments of professional service will be used only and exactly as submitted by EBA.

Electronic files submitted by EBA have been prepared and submitted using specific software and hardware systems. EBA makes no representation about the compatibility of these files with the Client's current or future software and hardware systems.

3.0 ENVIRONMENTAL AND REGULATORY ISSUES

Unless stipulated in the report, EBA has not been retained to investigate, address or consider and has not investigated, addressed or considered any environmental or regulatory issues associated with development on the subject site.

4.0 NATURE AND EXACTNESS OF SOIL AND ROCK DESCRIPTIONS

Classification and identification of soils and rocks are based upon commonly accepted systems and methods employed in professional geotechnical practice. This report contains descriptions of the systems and methods used. Where deviations from the system or method prevail, they are specifically mentioned.

Classification and identification of geological units are judgmental in nature as to both type and condition. EBA does not warrant conditions represented herein as exact, but infers accuracy only to the extent that is common in practice.

Where subsurface conditions encountered during development are different from those described in this report, qualified geotechnical personnel should revisit the site and review recommendations in light of the actual conditions encountered.

5.0 LOGS OF TESTHOLES

The testhole logs are a compilation of conditions and classification of soils and rocks as obtained from field observations and laboratory testing of selected samples. Soil and rock zones have been interpreted. Change from one geological zone to the other, indicated on the logs as a distinct line, can be, in fact, transitional. The extent of transition is interpretive. Any circumstance which requires precise definition of soil or rock zone transition elevations may require further investigation and review.

6.0 STRATIGRAPHIC AND GEOLOGICAL INFORMATION

The stratigraphic and geological information indicated on drawings contained in this report are inferred from logs of test holes and/or soil/rock exposures. Stratigraphy is known only at the locations of the test hole or exposure. Actual geology and stratigraphy between test holes and/or exposures may vary from that shown on these drawings. Natural variations in geological conditions are inherent and are a function of the historic environment. EBA does not represent the conditions illustrated as exact but recognizes that variations will exist. Where knowledge of more precise locations of geological units is necessary, additional investigation and review may be necessary.

7.0 SURFACE WATER AND GROUNDWATER CONDITIONS

Surface and groundwater conditions mentioned in this report are those observed at the times recorded in the report. These conditions vary with geological detail between observation sites; annual, seasonal and special meteorologic conditions; and with development activity. Interpretation of water conditions from observations and records is judgemental and constitutes an evaluation of circumstances as influenced by geology, meteorology and development activity. Deviations from these observations may occur during the course of development activities.

8.0 PROTECTION OF EXPOSED GROUND

Excavation and construction operations expose geological materials to climatic elements (freeze/thaw, wet/dry) and/or mechanical disturbance which can cause severe deterioration. Unless otherwise specifically indicated in this report, the walls and floors of excavations must be protected from the elements, particularly moisture, desiccation, frost action and construction traffic.

9.0 SUPPORT OF ADJACENT GROUND AND STRUCTURES

Unless otherwise specifically advised, support of ground and structures adjacent to the anticipated construction and preservation of adjacent ground and structures from the adverse impact of construction activity is required.

10.0 INFLUENCE OF CONSTRUCTION ACTIVITY

There is a direct correlation between construction activity and structural performance of adjacent buildings and other installations. The influence of all anticipated construction activities should be considered by the contractor, owner, architect and prime engineer in consultation with a geotechnical engineer when the final design and construction techniques are known.

11.0 OBSERVATIONS DURING CONSTRUCTION

Because of the nature of geological deposits, the judgmental nature of geotechnical engineering, as well as the potential of adverse circumstances arising from construction activity, observations during site preparation, excavation and construction should be carried out by a geotechnical engineer. These observations may then serve as the basis for confirmation and/or alteration of geotechnical recommendations or design guidelines presented herein.

12.0 DRAINAGE SYSTEMS

Where temporary or permanent drainage systems are installed within or around a structure, the systems which will be installed must protect the structure from loss of ground due to internal erosion and must be designed so as to assure continued performance of the drains. Specific design detail of such systems should be developed or reviewed by the geotechnical engineer. Unless otherwise specified, it is a condition of this report that effective temporary and permanent drainage systems are required and that they must be considered in relation to project purpose and function.

13.0 BEARING CAPACITY

Design bearing capacities, loads and allowable stresses quoted in this report relate to a specific soil or rock type and condition. Construction activity and environmental circumstances can materially change the condition of soil or rock. The elevation at which a soil or rock type occurs is variable. It is a requirement of this report that structural elements be founded in and/or upon geological materials of the type and in the condition assumed. Sufficient observations should be made by qualified geotechnical personnel during construction to assure that the soil and/or rock conditions assumed in this report in fact exist at the site.

14.0 SAMPLES

EBA will retain all soil and rock samples for 30 days after this report is issued. Further storage or transfer of samples can be made at the Client's expense upon written request, otherwise samples will be discarded.

15.0 INFORMATION PROVIDED TO EBA BY OTHERS

During the performance of the work and the preparation of the report, EBA may rely on information provided by persons other than the Client. While EBA endeavours to verify the accuracy of such information when instructed to do so by the Client, EBA accepts no responsibility for the accuracy or the reliability of such information which may affect the report.