The City of Nanaimo

ISSUED FOR USE

SEISMIC HAZARD ASSESSMENT
MIDDLE AND LOWER CHASE DAMS

N13101249

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EXECUTIVE SUMMARY

The City of Nanaimo (City) retained EBA Engineering Consultants Ltd. (EBA) to conduct a seismic hazard assessment for the Middle and Lower Chase Dams. This work was requested by the City in a Request for Proposal dated December 2008.

The Middle and Lower Chase Dams are located in the southern part of the City and were constructed circa 1911 to provide coal washing water during the early 20th century coal mining era of Nanaimo. Middle and Lower Chase Dams are 13 and 24 m high and 50 and 77 m long, respectively. Both dams are generally comprised of a central concrete core wall buttressed by rock fill slopes constructed upstream and downstream of the concrete wall. Additional fill was placed on the downstream side of each dam in subsequent construction episodes

Middle and Lower Chase Dams have become part of an urban park (Colliery Dams Park) since the end of the coal mining era of Nanaimo and the area downstream of the dams has been urbanized. A Dam Safety Review conducted on these dams in 2003 recommended that a seismic hazard assessment be conducted to assess if the dams are able to continue to safely impound their reservoirs in the event of a significant seismic event.

It is understood that in 2010, the City will release a request for proposal for a detailed flood inundation study associated with various flood emergencies, including those related to the seismic hazard assessment discussed herein. This will be a key study, the results of which will be required to place the findings of this seismic hazard study in to perspective.

A background information review was conducted using readily available historical information. The findings of the review were supplemented by engineering judgement and experience with aging dams of the same vintage as the Middle and Lower Chase Dams. The findings of the background review were used to prepare model input parameters, model geometry and to provide perspective when interpreting the results of the analysis.

Consequence classifications of the two subject dams were reviewed as part of this assessment. EBA concluded that the consequence classification is either at the upper end of High-Low or at the low end of High-High classification categories used by the Dam Safety Branch of the British Columbia Ministry of the Environment (BCMOE). Uncertainty exists in assessing the number of lives that could be lost in the event of failure of one or both dams (under 10 versus over 10 respectively). This matter cannot be resolved until the 2010 flood inundation study is completed.

The magnitude of seismic event selected for use in this assessment corresponds to a 1:3,000 year event (1.6% chance of occurring in 50 years). The BCMOE Dam Safety Branch advised EBA during this study that the 1:3,000 year event established in the 2003 Dam Safety Review would be accepted as the design seismic event for assessing the seismic response of the dams in their existing condition. However, should the City select rehabilitation or reconstruction of the dams, a higher return period seismic event (i.e. more severe ground shaking) may need to be selected. A specialist subconsultant, CAN Engineering Ltd., was hired to provide ground motion input parameters for the seismic modelling phase of this work.

The predicted horizontal deformations for the top of the concrete wall in each dam during the design seismic event were estimated to range from 0.360 m to 0.924 m for the Middle Chase Dam and from 0.055 m to 0.065 m for the Lower Chase Dam, depending on the scenario analyzed. The level of accuracy afforded by this analysis, given the nature of the input parameters, does not warrant millimetre accuracy. The estimated range of validity of these results is +/- 50%. The concrete wall exposed on the upstream face of Middle Chase Dam is expected to topple during the design seismic event with development of an overtopping failure and uncontrolled discharge of the Middle Chase reservoir. The rate of discharge and the capacity of the Lower Chase Dam spillway will determine if Lower Chase Dam is overtopped or not. This will be assessed during the 2010 flood inundation study.

Additional analyses were conducted for lower return period seismic events (i.e. less severe ground shaking) to assess the likelihood of wall toppling at Middle Chase Dam. The results of this analysis concluded that wall toppling could occur during a seismic event with a return period corresponding to an approximate 15% chance in 50 years.

The ALARP principle advocated by the Canadian Dam Association (CDA) was used to assess the potential for loss of life to provide the City with guidance on whether seismic hazards posed by the subject dams justify any mitigative measures. ALARP stands for "As Low As Reasonably Practical". The ALARP Principle is used to assess whether probability and magnitude of loss of life associated with a dam failure is within the CDA's view of societal tolerances. The risk of loss of life caused by failure of one or both dams during the design seismic event is described as Unacceptable using the ALARP principle. This assessment is for 10 fatalities. The City will have to decide if the ALARP Principle as advocated by the CDA is consistent with the City's risk tolerances. Circumstances that exacerbate this situation are the likelihood that a smaller return seismic event could cause a similar failure and inundation or that the 2010 inundation study concludes that more than 10 fatalities could occur.

A piping assessment was also conducted to assess the likelihood of piping developing under current conditions and under post seismic conditions. This assessment indicated that the current risk of a piping failure developing in either dam is ALARP. The presence of a wooden conduit through to Middle Chase Dam core wall is a potential risk under static loading due to the continued deterioration of the wood. However, the risk of piping failure developing in Lower Chase Dam after the design seismic event (or even one of smaller magnitude) is unacceptable according to the ALARP principle. The risk of post seismic piping failure of Middle Chase Dam will be unacceptable due to the undecommissioned wooden conduit; however, the analysis indicates Middle Chase Dam will likely fail during the earthquake. This assessment is for the case of 10 fatalities. The deformation of the concrete core walls during the seismic event and associated increase in seepage would largely be responsible for this increased risk of piping failure at Lower Chase Dam.

The City's post seismic performance expectations, the budget for such work and the social and environmental value of the Colliery Dams Park will to a large part determine what measures are appropriate for addressing the seismic hazards posed by the existing dams. It is our view that there are three general options the City has to address the seismic hazard risk posed by the subject dams:

- Option 1 - Eliminate the seismic hazards by removing the dams;

- Option 2 - Conduct seismic upgrades to the existing dams that bring the dams to a state where they safely impound their reservoirs during and shortly after the design seismic event but will need an engineering inspection immediately thereafter to assess the damage that has occurred, possibly followed by major maintenance or removal and, if necessary, evacuation of the potential inundation area; or
- Option 3 – Bring the impoundments into a state where not only do the dams safely impound the reservoirs during and after the design seismic event, but also require minimal maintenance after the design seismic event. This will require construction of new dams or extensive improvement of the fill in the existing dams with jet grouting or other in-situ treatment.

Based on the proximity of the subject dams to a downstream urban area combined with the findings of this seismic hazard assessment it is concluded that relying solely on evacuation of the inundation zone will be insufficient to prevent loss of life. Additional insight into the extent and effects of inundation caused by failure of one or both of the subject dams will be gained by completing the 2010 inundation study.

From a risk management perspective and upon consideration of the presence of a school, residences and a daycare within the inundation zone, the most practical and socially palateable option for addressing the seismic hazard risks posed by the subject dams is Option 1 – Dam Removal. Depending on the influence of other social and environmental factors and the risk tolerance of stakeholders (e.g. affected residents, school board, general public), the City may wish to accept some future risk and select Option #2. Discussions with the City have indicated that constructing new dams as part of Option #3 is not considered to be an option at this time.

1.0 INTRODUCTION

The City of Nanaimo (City) has retained EBA Engineering Consultants Ltd. (EBA) to conduct a seismic hazard assessment for the Middle and Lower Chase Dams. This work was requested by the City in the Request for Proposal dated December 2008. EBA's proposal for this work was dated January 19, 2009 and the City authorized this work under PO #516718, dated February 24, 2009.

The Middle and Lower Chase Dams are located in the southern part of the City as shown in Figure 1. They were constructed circa 1911 to provide coal washing water during the early 20th century coal mining era of Nanaimo. The Chase River Valley is the site of a series of dams, progressing upstream from Lower Chase Dam, listed as follows:

- Lower Chase Dam (earth and rock fill dam with concrete core wall);
- Middle Chase Dam (earth and rock fill dam with concrete core wall);
- Reservoir #1 (concrete gravity dam); and
- Upper Chase Dam (very small earth fill dam, with an upstream concrete retaining wall to support road fill rather than impoundment of water, see EBA report dated 2005).

There is an additional dam, the Harewood Dam, on a tributary to the Chase River, uphill from the Chase River Dam. However, the spillway discharge and an uncontrolled discharge from its reservoir would enter the Chase River Valley downstream of Lower Chase Dam. As this study is for the Middle and Lower Chase Dams, the response of the Reservoir #1 Dam and Harewood Dam under the design seismic event has not been considered in this study. The seismic response of the Upper Chase Dam was assessed by EBA in 2005. The results of this assessment are discussed briefly later in this report.

Middle and Lower Chase Dams are 13 and 24 m high and 50 m and 77 m long respectively. Both dams are generally comprised of a central concrete core wall buttressed by rock fill slopes constructed upstream and downstream of the concrete wall. Additional fill was placed on the downstream side of each dam in subsequent construction episodes. The location of the subject dam is presented in Figure 1.

Middle and Lower Chase Dams have become part of an urban park, Colliery Dams Park, since the end of the coal mining era of Nanaimo and the area downstream of the dams has been urbanized through construction of city streets, residences, a school and a daycare. A Dam Safety Review conducted on these dams in 2003 (Golder 2003) recommended that a seismic hazard assessment be conducted to assess if the dams are able to continue to safely impound their reservoirs in the event of a significant seismic event.

EBA retained Herold Engineering Ltd. (Herold), a Nanaimo based structural engineering consulting firm to assist with structural engineering matters for this project.

This report is subject to the General Conditions presents in Appendix A.

2.0 SCOPE OF WORK

The scope of work for this project is to complete a seismic hazard analysis of both the Middle and Lower Chase Dams in their current state and determine if they meet current Dam Safety requirements. Additionally, EBA will also provide the City with an assessment of whether upgrading the dams is practical and economical and, if so, what options are practical as well as if other measures such as risk management or dam removal should be considered if the result of this study is that one or both of the dams do not meet current Dam Safety requirements.

EBA's scope of work was presented in our proposal dated January 19, 2009 and included the following tasks:

- Task 1 - Project initiation meeting;
- Task 2 - Review of background information;
- Task 3 - Detailed inspection of dams and limited subsurface inspection;
- Task 4 - Structural assessment;
- Task 5 - Develop seismic criteria;
- Task 6 - Develop parameters for use in analysis;
- Task 7 - Assess seismic stability of each dam;
- Task 8 - Prepare conceptual designs and costs for upgrading each option;
- Task 9 - Discuss results with BCMoE Dam Safety branch; and
- Task 10 - Reporting.

Details on the scope of each task are presented in our proposal. The results of each task are discussed in the following sections.

Some of the work presented herein is based on engineering judgement and estimates of the extent of inundation associated with uncontrolled discharge from each of the subject dams. To date, the only inundation study that has been conducted is one where a massive flood event occurs that causes failure of all dams. The results of this assessment are not completely applicable to the study presented herein, although they do provide some valuable insight for the purposes of this assignment. In 2010, the City will release a request for proposals for a more detailed flood inundation study associated with various flood emergencies, including those related to the seismic hazard assessment discussed herein. This will be a key study, the results of which will be required to put the findings of this seismic hazard study in to perspective.

3.0 BACKGROUND INFORMATION

3.1 GENERAL

A background information review was conducted at the outset of this assignment to establish what was known to date about the subject dams. Readily available local historical accounts provided the basis for some deduction on the dam construction since there is generally a paucity of information on the actual construction and historic operation of the subject dams.

3.2 SOURCES OF INFORMATION

EBA reviewed the following key sources of background information prior to the project initiation meeting:

- 2008 Annual Dam Inspection report, EBA;
- 2003 Dam Safety Review reports for Middle and Lower Chase Dams, Golder, March 2004;
- Operations, Maintenance and Surveillance (OMS) Manual for Chase River Dams, Golder, Rev. 1, April 2004;
- Data Books for Middle and Lower Chase Dams, EBA, May 1992;
- Site investigation report for Nanaimo Dams, Golder Associates Ltd., 1978;
- Bathymetric and topographic survey data of both dams and reservoirs;
- Various rehabilitation design drawings for Middle and Lower Chase Dams, Willis Cunliffe and Tait, July 1978;
- Archival photos of provincial dam safety inspections of Middle and Lower Chase Dams (1977 to 1982 and 1976 to 1981 for Middle and Lower Chase Dams respectively); and
- Chase River Dams, Phase 1 of Incremental Damage Assessment, Water Management Consultants Inc., 2002. EBA Note: this is a study that models breach of every dam on the Chase River system during the Probable Maximum Flood, PMF.

Design and construction records are not available for the Middle and Lower Chase Dams. Although Middle Chase Dam was rehabilitated in 1980, only design drawings are available for this work. Although there are some construction photographs, as-built records are unavailable. The construction photos are in the City files for these dams and were reviewed by EBA as part of this work. EBA discussed the 1980 rehabilitation of Middle Chase Dam and modifications to Lower Chase Dam with Mr. Douglas Anderson, P.Eng., the resident engineer for this work in 1980.

Additionally, EBA reviewed the following sources of historical information on the coal mining history of Nanaimo to gain a greater understanding of the historical setting of the dam construction:

- Boss Whistle, (Bowen, 2002);
- Coal Mine Underground Workings Atlas for Nanaimo, (Coal Mine Atlas, Pacific Spatial Systems Inc., 2004); and
- Various online archives available to the public via the internet.

These documents provide background information on the ownership and sequence of operations of various coal mines in Nanaimo that are directly and indirectly related to the subject dams. The Coal Mine Atlas also provides information on historic coal mining activities and associated railways which are relevant to this study.

3.3 DESCRIPTION OF DAMS

Both the Middle and Lower Chase Dams are located in the Chase River Valley within a City park known as Colliery Dams Park. The Chase River Valley is narrow and steep sided within the park boundaries with exposed bedrock and till on the sideslopes. Some thin colluvium has been noted at the base of the valley slopes, as discussed later in this report. Soft/loose unconsolidated channel deposits are present at the base of the valley between the two dams and downstream of Lower Chase Dam; they are of limited lateral extent due to the narrow and steep sided nature of the river valley. Additionally, they appear to only be present when the valley bottom flattens below steeper reaches of the channel where bedrock is exposed. Borehole investigations were completed at both dams in 1978 and information on the downstream shell material present in each dam in 1978 was obtained.

A brief description of the two dams is presented in the following paragraphs.

<u>Middle Chase Dam</u>

Middle Chase Dam is a rock fill dam with a vertical, 0.6 m wide, concrete core wall. The dam is approximately 13 m high and has a crest length and width of approximately 50 and 5 m respectively. The embankment sideslopes are approximately 2.5H:1V and 1.6H:1V for the downstream and upstream slopes respectively. There are no records that verify whether or not there is steel reinforcing within the concrete core wall.

The downstream shell of Middle Chase Dam was substantially excavated in 1980 as part of a dam rehabilitation program conducted at that time. The purpose of the excavation was primarily to locate a low level conduit believed to be constructed within the dam. While the low level conduit was not located in 1980, the 2003 Dam Safety Review correctly questioned the impact should a low level conduit be in place as it was intuitive that there should be one. At the outset of this project, EBA assumed that it was a wood stave conduit like those present within the Lower Chase Dam and other dams of similar vintage in the Nanaimo region such as Westwood Lake Dam. It is reasonable to conclude that the wood conduit was cast directly through the concrete core wall.

Based on historical construction records and photographs the backfill placed in the excavation in 1980 is compacted, pit run sand and gravel. Additionally, some foundation drains were placed to address historical seepage which was believed to be primarily through the bedrock. Plan views of Middle Chase Dam are presented in Figures 2 and 3 which show satellite imagery (Google Earth) and surveyed topography. A cross-section of Middle Chase Dam is presented in Figure 4.

Lower Chase Dam

Lower Chase Dam is a rock fill dam with a 1.2 m thick, vertical, concrete core wall. Mine/process waste material was placed on the downstream face of the dam sometime after construction with an additional toe berm/filter layer placed over the lower half of the downstream face in 1980. The dam is approximately 24 m high and has a crest length and width of 77 and 10 m respectively. The upstream and downstream embankment sideslopes are 2.2H:1V and 1.5H:1V respectively. Two wood stave conduits were constructed through the dam. It is reasonable to conclude that they were cast through the concrete core wall. These conduits were backfilled with concrete as part of the 1980 rehabilitation works. Additionally, the concrete valve chamber which drained into these conduits was backfilled with concrete as part of the 1980 rehabilitation works, effectively sealing the entrance to the conduits and any seepage path between the wood and the concrete. There was no indication if the wood stave conduit was deteriorated or rotted.

Review of the dam details in plan view indicates that the left abutment (north) is wider than the right (south) abutment and also that the downstream crest of the embankment fill is not parallel to the concrete wall. This is discussed further in Section 3.4. The 1978 investigation also included a number of test pits, including one (TP-4, 1978) excavated at the downstream edge of the concrete core wall. The excavation of TP-4 (1978) exposed a thickening of the concrete wall from 0.3 m wide to 1.2 m wide at a depth of 0.6 m below crest elevation. The 1978 boreholes and test pits encountered a layer of loose sand, gravel, cinders and ash overlying rock fill downstream of the concrete core wall. Plan views of Lower Chase Dam are presented in Figures 5 and 6 which show satellite imagery (Google Earth) and surveyed topography. A cross-section of Lower Chase Dam is presented in Figure 7. The zonation within Lower Chase Dam presented in Figure 7 has been inferred from the 1978 borehole and test pit logs which are also shown on Figure 7.

3.4 HISTORICAL SETTING OF DAM CONSTRUCTION

Through review of the background information referenced in this report, EBA has deduced the following:

- The subject dams were likely constructed by Western Fuel Corporation (Western Fuel) after 1904. In 1903, Western Fuel purchased a number of the Nanaimo area mines which included several of the mines within the current city limits, including the Harewood Mine, the Nanaimo #1 Esplanade Mine (Bowen, 2002). Later, the Wakesiah Colliery was opened by Western Fuel in 1918 (Bowen, 2002);

- In 1904, the Todd Bay Portland Cement Quarry (now Butchart Gardens) opened near Victoria which initiated the start of concrete construction for the Nanaimo coal mining industry (e.g., Morden Mine head frame, most likely the Harewood Dam, and the subject dams). It appears that the Middle and Lower Chase Dams and associated concrete structures are likely among the earliest concrete structures constructed in Nanaimo. The likelihood of poor concrete construction practices being used in the construction of the subject dams is considered by EBA to be high;
- The former presence of a railway line from the Harewood Mine to the coal wharves and its proximity to the subject dams and presence of spur lines from this railway extending to each dam (EBA, 1992) suggests the dams were built with rock fill from the Harewood Mine that was transported to the dam sites via train. This partially explains why previous documents (EBA, 1992) referred to both dams as former railway crossing sites constructed by the Harewood Mine;
- Although the conveyance of coal was mechanized through the use of coal fired surface trains and electric trains for underground use at the time of construction, the excavation of blasted bedrock for adits and shafts was still very much focused on hand excavation and hand loading of carts (Bowen, 2002). The rock fill that was used as fill material was most likely mined by hand and loaded into tram cars by hand and therefore likely did not have particles much larger than 0.6 m diameter;
- The majority, if not all, of Western Fuel's coal was shipped from the Nanaimo coal wharves which were located to the south of the current downtown harbour front area (Bowen, 2002). The coal was delivered to these wharves by Western Fuel trains from most of their mines. Coal washing was conducted at the coal wharf site as indicated by a photograph within the Coal Mine Atlas (Pacific Geospatial Systems Inc, 2004). The nearest practical source of fresh, coal wash water that could be economically conveyed to the coal wharves was from the Chase River. EBA has assumed that sea water would not have been used to wash coal due to its salinity and the need for pumping. This supports the previous documentation (EBA, 1992) that the dams were constructed to provide coal washing water;
- An online excerpt from the Nanaimo archives indicates that there were two original dams on Chase River at the southwest corner of Harewood Road to supply water to the City of Nanaimo. Western Fuel started to build a dam downstream of the city dams for their own purposes as they expanded their operations. The dam was completed May 1, 1911. The first gasoline cement mixer in Nanaimo was used to construct the dam. It is not known whether or not these dams also supplied drinking water;
- The observation of an old timber crib structure in the downstream shell of the Middle Chase Dam in 1980 (archived BCMoE Dam Safety Branch inspections reports) indicates that there may have been older dams constructed at both sites prior to construction of the original dams. The Nanaimo coal wharves were in operation before 1900 (Bowen, 2002) and would have required a source of coal wash water, presumably from the Chase River. Consolidation of many mines under a single corporate ownership

in 1903 may have required greater volumes of wash water at the coal wharves. It stands to reason that Western Fuel may have upgraded their coal wash water supply infrastructure at this time. It also stands to reason that a low level conduit would have been constructed through both dams to maximize the use of stored water from both reservoirs; and

- Upon development of the Wakesiah Mine by Western Fuel in 1918 (Bowen, 2002), the railway used to transport the coal from the mine to the coal wharves was constructed over the Lower Chase Dam. The railway is shown in historical maps in the Coal Mine Atlas (Pacific Geospatial Systems, 2004). It appears that additional fill was added to Lower Chase Dam to permit a railway crossing at an azimuth not parallel with the concrete core wall. This appears to be the reason why the downstream crest of the Lower Chase Dam is not parallel to the concrete wall.

3.5 FILL AND FOUNDATION MATERIALS

The fill and foundation materials within and below each dam are known primarily through the contents of the 1978 borehole logs and to a lesser extent, the 1978 test pits. Both the 1978 borehole logs and test pit logs are presented in Appendix B. Fill and foundation materials are discussed in the following sections.

3.5.1 Fill Materials Within the Dams

There are five general material types within the subject dams:

- Concrete in the vertical core wall and spillway;
- Rock fill on either side of the concrete walls from the original construction work in 1911 (original rock fill);
- Compacted pit run in the downstream shell of the Middle Chase Dam;
- Cinders, ash and sand and gravel on the downstream side of the concrete core wall on top of the original rock fill at the Lower Chase Dam; and
- Timber in the low level conduit.

Each material type is discussed in further detail in the following paragraphs.

<u>Concrete</u>

Historical concrete construction near the turn of the 20th century on Vancouver Island was of variable quality. Discussions with Herold Engineering (the structural engineer for this project) indicated that it was possible that a mass concrete structure, or supported wall in the case of the Chase River Dams, could have been constructed without steel reinforcement. Steel reinforcing in the Chase River Dams is discussed in more detail later in this report.

The most prominent concrete coal mining structures of this era still standing in the Nanaimo area that EBA is aware of, aside from the subject dams, is the Morden Colliery head frame (constructed in 1913) and the Harewood Dam (constructed in 1911). Herold has worked on the Morden head frame and have observed that the quality of the concrete and construction in general is poor. The Morden head frame was reinforced with steel, which stands to reason as it was a hoisting structure with tensile loads in some of its members. While the materials within the Morden head frame and the Chase River Dams cannot be directly related or compared, it is worthy of mention that the Morden head frame was designed by experts brought in from Europe and is viewed to be representative the state of the art in reinforced concrete construction in the early 1900s in the Nanaimo area. By comparison, the Harewood Dam appears to be in much better shape and may be more reflective of the quality of a mass concrete structure constructed in the early 20th century in Nanaimo.

The sequencing of concrete placement within the concrete walls is unknown. It is likely the concrete was placed in lifts which means there are likely various concrete construction "cold" joints present throughout the walls. Review of photographs taken during the 1980 Middle Chase rehabilitation works indicates that planks were used as form work but construction joints were not visible. It is not known if the wall was constructed prior to fill placement or if the wall was constructed in segments followed by fill placement to permit access for subsequent concrete work. EBA considers the latter construction method to be more likely due to the decreased need for extensive formwork and access scaffolds.

The quality of concrete at depth is unknown for both dams; a review of the photographs taken during rehabilitation of Middle Chase Dam in 1980 indicated no visible segregation or honey-combing of the concrete exposed on the downstream face of the wall. However such features are visible on the concrete spillway walls at Lower Chase Dam. EBA believes it to be prudent to assume the quality of the concrete core wall construction in both dams is poor by today's standards.

Discussions with Mr. Douglas Anderson, P.Eng., who was the engineer for the 1980 rehabilitation works, indicated that he could not recall if the Middle Chase Dam concrete wall had any steel reinforcement. The rehabilitation drawings did not show if the wall had any reinforcing steel. A hole was blasted through the concrete wall at the Middle Chase Dam in 1950, reportedly to allow for additional discharge capacity during a severe storm event. The design drawings for repair of this hole did not indicate if the wall was reinforced or not. EBA considers it to be reasonable to assume that if steel was present it would have been indicated on the drawings. The result of the background information review is inconclusive on whether or not the concrete core wall was reinforced.

Smooth bar steel reinforcement is visible on the spillway walls at Lower Chase Dam where deterioration of the concrete has spalled some of the cover off of the steel. However, the age of this structure is unknown and it can not be confirmed whether or not this structure was constructed at the same time as the dam. Extensive honey-combing of the concrete is present at this location as well.

Rock Fill

The rock fill used in dam construction was most likely derived from the operating Harewood Mine as discussed earlier in this section. The gradation of the waste rock mined in Nanaimo at the turn of the 20th century would be variable, but would generally have a top size no greater than 0.6 m as previously discussed in Section 3.4. Photos of the rock fill on the upstream and downstream side of the Middle Chase Dam during the 1980 rehabilitation works, indicates that the maximum rock fill particle size is generally 0.6 m or smaller. The waste rock would also have appreciable cobble, gravel, sand and silt contents. It seems reasonable to conclude that the larger particles would be loaded first and the smaller particles loaded last which would result in segregation of the waste rock and formation of pockets of highly variable gradation throughout the rock fill mass. This variability was inferred from the data collected in the 1978 drilling program. The mineralogy of the rock fill particles was not indicated in the 1978 drilling report. The history of the dam construction discussed in Section 3.4 suggests sedimentary rock existed from the Harewood Mine was used to construct the upstream and downstream shells.

The method of rock fill placement and construction is not known; however, based on the review of the 1978 borehole logs and understanding the historical context of construction, it is very likely that the rock fill was end-dumped down the south valley walls and, at best, spread by surface mine labourers. There would have been no compaction aside from the force of gravity and impact of larger particles being dropped into place. Finer grained materials would have likely segregated from the coarser particles during loading of the trams and train cars at the mine, and the dumping, resulting in a segregated and highly variable rock fill mass.

The Middle Chase Dam downstream shell was substantially excavated in 1980 in an attempt to locate a low level conduit within the dam. A section of the original downstream shell fill was left in place adjacent to the left abutment next to the spillway. The backfill material placed in 1980 was compacted with a walk behind compactor. Compaction testing was conducted at this time but no records are available for review. EBA believes it is reasonable to conclude that the fill material placed in 1980 was compacted to a dense state. However, a thin section of original, uncompacted rock fill was been left in place between the deepest section of the dam and the spillway, which is founded on bedrock.

The sand, gravel, cinders and ash that are present on the downstream side of Lower Chase Dam appear to have been placed during construction of the Wakesiah Colliery railway around 1918. Based on the observations and testing conducted during the 1978 drilling investigation, it appears this additional fill was end-dumped without any compaction. Finer grained materials, such as the sand, gravel, cinders and ash, tend to experience less compaction during dumping than coarse rock fill does. Therefore, it tends to be much looser when placed without compaction.

3.5.2 Foundation Materials

The foundation materials beneath the Middle and Lower Chase Dams appear to be either bedrock in the case of Middle Chase Dam or till overlying bedrock in the case of Lower Chase Dam. This assessment is based on the limited information contained in the 1978 borehole logs and conditions visible on the valley slopes in the vicinity of each dam reported in previous documents.

4.0 FIELD WORK

4.1 GENERAL

EBA conducted the following field activities as part of this assessment:

- Initial site visit and structural inspection of the walls at both dams;
- Supplementary dam inspections when they were not snow covered, including review of other historical structures and conditions within the Chase River Valley within Colliery Dam Park as well as the urbanized area downstream of the subject dams;
- Diving inspections of the upstream slopes of both dams; and
- Excavating shallow test pits by hand on the downstream slope and abutments of both dams.

Photographs taken during the various phases of field work are attached to this report.

4.2 INSPECTIONS

4.2.1 Initial Site Visit and Structural Inspections

EBA completed a field investigation of the Lower and Middle Chase River dams and the area surrounding the dams on February 26, 2009. A detailed inspection of the subject dams was to be conducted as this time but heavy snow cover from a storm the previous evening precluded this inspection. There was a thin covering of ice on both reservoirs at this time due to the cold weather conditions.

Field work included the following activities:

- Schmidt Hammer rebound testing by EBA to estimate the approximate compressive strength of the concrete walls of the Lower and Middle Chase Dams;
- A rebar survey by Herold of the aforementioned walls using a digital handheld rebar detector; and
- Bathymetric profiling along sections perpendicular to the dams.

Christopher Wintle, E.I.T., of EBA performed the Schmidt Hammer rebound testing of the concrete walls on February 26, 2009 to estimate compressive strength of the concrete walls. Access for the testing was obtained using a Zodiac inflatable boat, as presented in Photo 3. This test employs a handheld rebound hammer that releases a spring-loaded mass towards the concrete surface. The rebound force of the mass is measured by the handheld device which presents the user a rebound number. A chart is provided by the manufacturer to correlate the rebound number to a concrete compressive strength.

EBA measured the concrete strength at 18 locations on the upstream face of the concrete core wall at Lower Chase Dam. Nine of these locations were on what appeared to be a newer section of the concrete wall constructed on the original wall. The remaining nine locations were on the lower, original wall. Concrete strengths were recorded at ten locations along the Middle Chase Dam. EBA noted the surface of the original concrete walls along both dams had varying levels and extents of spalling and loose concrete, with the most deterioration at Middle Chase Dam. To take into account the inherent variability of the Schmidt Hammer test and the added potential for error from the deteriorated face, sample locations were carefully selected and smoothed prior to testing and three rebound tests were performed at each location within 50 mm of each other. A summary of the Schmidt Hammer testing is presented in Appendix C.

Craig Work, E.I.T., of Herold completed the rebar survey of the two dams on February 26, 2009 while EBA was conducting the Schmidt Hammer testing. The variability of Schmidt Hammer reading is generally accepted as +/- 20%. Additionally, to account for spatial variability EBA tested nine locations on each wall with a total of three readings per location as indicated in Appendix C. The rebar survey involved scanning the walls of the dams using a Profometer 5+ Rebar Scanner manufactured by Proceq. According to Herold, the device has been used successfully in many foundation wall inspections, yielding readings with up to 200 mm concrete cover on reinforcing steel. Beyond 200 mm, however, the accuracy of this device is unknown. This machine is adversely affected by deteriorated or rough concrete surface quality.

Profometer readings were taken at five or more locations along each dam. No rebar was found in the exposed portions of the Lower and Middle Chase Dam walls just above the reservoir level. It should be noted, however, that these walls ranged from 600 mm to 1200 mm thick, which may place any rebar in the wall outside the 200 mm testing range of the device. However, it is understood from Herold that steel reinforcement in concrete, even in the early 1900s, was typically placed as close as possible to the face of the concrete structure, while maintaining a minimum cover thickness, to provide maximum reinforcement in flexure.

In the case of the Middle Chase Dam where the concrete wall is 600 mm thick, the profometer would have sensed 1/3 of the wall thickness. As no rebar was noted, it is reasonable to conclude there is no rebar in the original concrete wall. By extension and due to the Profometer not detecting any reinforcing steel, the thicker Lower Chase Dam wall has been assumed to be unreinforced as well.

Scans of the newer concrete wall at Middle Chase Dam indicated that there was 4.8 mm diameter steel in the wall spaced approximately 150 mm on center in each direction which appears to be dowel used in 1980 to connect the new upper section of the wall with the original concrete wall.

Simplified bathymetric data was collected by EBA using a lead line and measurements of the distance from the upstream wall of each dam. This data compared to the bathymetric data initially provided by the City and it was observed that the City's bathymetric data was incorrect. The City initiated an additional survey of reservoir bathymetry for both reservoirs and provided EBA with the revised data by the City. Additional topographic surveying downstream of each dam was also conducted to provide more general information on the slope of the former channel beneath the dam through extrapolation from downstream of the dam to upstream of the dam at the reservoir bottom.

Photographs of the two dams at the time of the February 26, 2009 inspections are presented in Photos 1 to 5.

4.2.2 Supplementary Site Inspection

An inspection of the dams and areas downstream of each dam was conducted on May 9, 2009 by Chris Gräpel, P.Eng. In general, the condition of both dams do not appear to have changed appreciably since the 2008 inspections conducted by EBA. The vegetation growing on the downstream slopes of both dams had been cut within the previous month before the May 2009 inspections. Photos 6 through 30 were taken on May 9 and 10, 2009.

A brief summary of observations made at each dam that are of particular note for this study are presented as follows:

<u>Middle Chase Dam</u>

- Sandstone and conglomerate bedrock is visible at both abutments of the dam, upstream of the dam at various locations at the edge of the reservoir and on the invert of the spillway as shown in Photos 7 through 10. Bedrock is also visible at the downstream toe of the dam;
- The bedrock appears to be overlain with a dense, sandy gravelly till, or diamictin, in some locations;
- The original concrete wall has experienced surface deterioration up to 25 mm (1 inch) over its entire upstream face as shown in Photos 2, 8 and 9;
- The upstream face of the concrete wall is generally vertical;
- Inspection of the original wall contact with the bedrock at the left abutment (left and right are defined by looking downstream) indicates that the concrete wall was constructed underneath a bedrock overhang. This is indicative of poor foundation preparation practices albeit, in a noncritical location;
- The extent of excavation and reconstruction of the downstream shell and slope of the dam is evident. A thin zone of original fill was left in place approximately 2 to 3 m wide at the slope face to permit safe working conditions during the 1980 excavation and backfilling works. This feature is shown in Photos 11, 12 and 13;

- Downstream of Middle Chase Dam the Chase River Valley is generally bedrock controlled with bedrock outcrops present at various locations between the downstream toe of Middle Chase Dam and the upstream limit of the Lower Chase Dam reservoir as shown in Photo 15; and
- Seepage exiting the toe of the dam is collected in a concrete catch basin that houses a v-notch weir that is used to record seepage rates.

Lower Chase Dam

- The concrete wall at the upstream face of the dam appears to be in good condition with a vertical upstream face. There are some voids which indicate weathering and possibly construction cold joints;
- Review of the concrete wall (Photos 19, 21 and 22) as it approaches the park areas on the upstream side of the dam indicates that it is straight. The various walls associated with the widened areas on the left and right sides of the reservoir appear to be from subsequent episodes of construction;
- Honey-combing (i.e., voids between segregated cement paste covered aggregate) is evident in the spillway retaining walls. Construction cold joints are also present. The compressive strength of the concrete is higher then what is present in Middle Chase Dam. However, the cold joints and honeycombing are still present in Lower Chase Dam;
- Shallow deformation of the downstream face of the dam is evident with deflection cracking of the asphalt walkway near the downstream crest of the dam as shown in Photo 26. The downstream face of the dam has experienced shallow slope instability, possibly in response to concentrated water flow over the downstream crest of the dam. It appears that there is an erosion channel on the upper slope of the dam;
- The fence placed at the downstream crest of the dam does not appear to have deflected appreciably since installation but the fence is also curved slightly to match the downstream crest of the dam which could mask minor deflections;
- There are several large trees growing out the dam fill near both abutments. The trees near the right abutment are leaning in an upslope direction;
- Ash, cinders, sand and gravel are visible in the face of the upper slope of the dam;
- The downstream toe berm/filter layer has an insloped bench at the top that appears to have been designed to catch water flowing down the face of the dam as shown in Photo 25;
- There are some limited toe failures approximately 0.5 m high at the downstream toe of the dam as shown in Photo 27. These features do appear to be recently active. It appears that these are due to historic seepage exiting from the dam. Seepage was not noted to be present at this elevation;

- Bedrock is visible downstream of the dam with a maximum outcrop elevation on the right valley slope that corresponds to approximately half way between the top of the toe berm and crest of the dam. There is an unquantified thickness of sand, gravel and silt colluvium over the lower quarter of the natural river valley slope but its maximum thickness appeared to be no more than 2 m thick;
- The base of the Chase River Valley downstream of the Lower Chase Dam is approximately 6 m wide and is the location of soft, wet, unconsolidated deposits of unknown thickness; and
- Downstream of Lower Chase Dam, the Chase River Valley turns sharply to the right to where the stream from Harewood Lake (and the Harewood Dam spillway) enters chase River.

<u>Additional Areas Upstream and Downstream of Lower Chase Dam</u>

Additional observations of interest to this study were made while walking around Colliery Dam Park on May 9 and 10, 2009. An old railway or pipeline bridge is located to the northwest of the Lower Chase Dam. The age of this structure is unknown; however, the condition of the concrete piers is of interest. The following observations were made:

- The concrete piers are cast to have sloping sides (estimated 1H:40V batter) that are visually obvious;
- There is much concrete segregation which has created extensive zones of honey-combing;
- Cold joints are evident in numerous locations with honey-combing immediately above them. The depth of honey-combing in some locations approaches 75 mm; and
- There are no visible signs of reinforcing steel being used in these piers but a detailed rebar survey was not conducted.

A brief reconnaissance of the school and daycare grounds adjacent to the Chase River was also conducted on May 10, 2009. The following observations were made:

- The playing field where the largest body of students would be located when not in the school building(s) are located to the south of the school grounds, well away from the Chase River, although potentially not outside of the inundation zone associated with one or more dam failures resulting in uncontrolled discharge; and
- The closest building to the Chase River Valley on the school grounds is a maintenance garage. However, the elevation difference between the classroom buildings, the day care and the crest of the Chase River channel was visually estimated with a hand level to be less than 0.3 m and those structures are located approximately 50 from the banks of the Chase River.

The John Barsby High School and Little Ferns Daycare buildings are illustrated in Photos 28 and 29.

EBA briefly viewed various single family and multifamily residences along the banks of the Chase River downstream of the dams in the Howard Avenue to Bruce Avenue area. There are at least four houses and multifamily units that could be impacted by floodwaters caused by an uncontrolled discharge from one or more dams, especially one house north of the Chase River on Howard Avenue. The 2010 flood inundation study should request provision of greater resolution on which buildings would be affected by a seismically induced failure of one or both dams.

The third parties downstream of the dams, be they residents, children in school or at the daycare, pedestrians, motorists on the streets of highway, passengers or operators of railroad traffic or any utility owners or operators that could be affected by an uncontrolled discharge are hereafter referred to as downstream stakeholders.

EBA briefly viewed the Bruce Avenue Bridge crossing of the Chase River and noted that the channel narrows beneath the bridge due to the rip rap armoured abutments. This could either cause erosion of the abutments or cause a backwater effect that increases flood water level, or some combined there of. Additionally, there are signs indicating that a Terasen natural gas pipeline is located on the upstream side of the bridge. The depth of burial of the pipeline is unknown. The Bruce Avenue Bridge is illustrated in Photo 30.

4.3 DIVING INSPECTIONS

A diver conducted an underwater inspection of the Lower Chase Dam on February 27, 2009. Mr. Gräpel, P.Eng., attended the inspection and advised the diver what to look for. In general, visibility was poor which precluded any photographs being taken with an underwater camera. However, the diver was able to confirm in several locations that the upstream shell consisted of compact to dense sand, gravel and cobbles and possibly some small boulders (0.3 to 0.6 m size). The upstream shell was mantled with about 0.2 m of lakebed sediments which hindered vision.

A diver conducted a search for the Middle Chase Dam low level conduit inlet on May 10, 2009. The inlet to the low level conduit was located approximately 20 m from the face of the dam, 9 m from the right bridge abutment. The low level conduit appears to pass generally below the patched area where a hole was blasted through the concrete wall in the early 1950s. This may have been the site of a valve stem.

The low level conduit appears to be located on the left abutment of the dam, possibly on a natural bedrock ledge, and appears to pass beneath the wedge of original fill that was left in place during the 1980 excavation. The seepage noted at this location over the years is likely in part from the abandoned low level conduit.

The condition of the exposed wood of the low level conduit intake was very poor. The inlet of the low level conduit was rotted and partially collapsed. Approximately 1 m from the inlet the low level conduit was encased in an unknown thickness of concrete. Thereafter, it was buried in lakebed sediments. Disturbance of the lakebed sediments prohibited accurate measurement of the length of conduit not encased in concrete. The approximate location of the inlet is presented in Figure 3. The area is covered with easily disturbed silts which quickly clouded any visibility.

4.4 APRIL 9, 2009 TEST PITTING

Christopher Wintle, E.I.T. of EBA completed a total of 8 hand excavated test pits on April 9, 2009. Test pits were excavated using a spade, pick axe and post hole digger. Five of the eight test pits were excavated along the downstream side of the Lower Chase Dam and three test pits were excavated along the downstream side of the Middle Chase Dam. Test pit locations excavated by EBA are presented on Figures 3 and 6. Test pits were excavated to depths ranging from 0.80 m to 1.30 m, terminating either at the maximum practical reach of the equipment or when increasing soil density prohibited further excavation by hand. Test pit logs are presented in Appendix C.

5.0 DISCUSSION ON AVAILABLE INFORMATION

The purpose of the background review and field work conducted by EBA was to compile as much as could be practically and reliably obtained.

EBA's reasons for not conducting a drilling investigation as part of this assessment were provided in our proposal dated January 19, 2009 and are summarized as follows:

- The rock fill located on the upstream side of either dam can not be practically (i.e., cost effectively) investigated which will require properties of this rock fill mass to be estimated even if a conventional drilling program is undertaken. The strength and stiffness of these materials will have a significant influence on the deformations experienced by the concrete walls. Due to anticipated variability of the original rock fill, a range of values will need to be estimated;
- There is sufficient subsurface information to permit assessment of the static stability of the dam (i.e., the geometry, material types and water levels within the dams are sufficiently defined);
- EBA considered a number of subsurface investigation techniques to assess the stiffness of the dam fill materials and concluded that the risk of the difficult subsurface conditions (e.g., coarse rock fill, voids up to 0.3 m in width) would result in an unacceptably high risk that the expense of the investigation may not yield reliable results. Furthermore, due to the known variability of the fill materials, such an investigation would encounter significant variability in the few measurements that could be reliably made. This would result in a high degree of reliance on properties based on engineering judgement, experience and the 1978 borehole logs;
- The fill placed in the Middle Chase Dam in the 1980s is understood to be relatively uniform and was placed and compacted under controlled conditions. As a result, estimation of material properties for this material should be relatively uncomplicated; and

- The refinement of material properties beyond the levels afforded by engineering judgement, experience and the 1978 borehole logs using expensive investigation and testing methods, that will be potentially unreliable, will not change the City's options for practically addressing the hazards posed by these dams during or after design seismic loading.

The first consideration in assessing the reviewed background information is that the dams were constructed shortly after the turn of the century by a mining company. Furthermore, seismic loading was most likely not considered. Up until the late 1960s most dams in the Vancouver Island area were designed assuming peak horizontal acceleration of 0.1 g, far less than the required standards of today. Based on the understanding of the construction, the two subject dams in their current state will experience appreciable damage during the design seismic event. This condition will need to be rectified by either seismic rehabilitation or removal.

The methods of construction bear highlighting at this juncture. All indications are that while the rock fill may have been transported to site by train, the method of placement was likely conducted with some combination of end dumping of hand carts and some spreading by labourers. Compaction of the rock fill would not have been achieved through any other means than self weight impact upon placing. The lack of compaction will result in a rock fill mass that deforms more readily than a modern compacted rock fill mass.

The available historical information from a context wider than one limited to the dams indicates that concrete was a new construction material in the Nanaimo region when the dams were built. Records from the City of Nanaimo archives indicate that the concrete in the dams was made with the first gasoline powered cement mixer in the City. Observations made during the various phases of field work indicate zones of very poor durability at construction cold joints, poor durability of exposed concrete faces, extensive honey-combing of concrete in various structures and relatively low compressive strength in Middle Chase Dam. These observations support EBA's initial judgement that although the concrete walls may have performed adequately to date, the quality of the concrete is not of the level that would justify drilling through the walls to assess conditions. It is EBA's opinion that an unacceptably high likelihood that extensive zones of poor durability and/or honey-combing on the upstream and downstream sides of the walls could be interconnected by a borehole if drilling was conducted under full reservoir conditions.

The rebar survey conducted by Herold indicated there was no rebar within about 200 mm of the upstream face of the concrete walls for either dam. In the case of Middle Chase Dam with its 600 mm thick wall, the reinforcing steel sensor would have sensed to within 100 mm of the centre of the wall. If reinforcing steel was present within the original Middle Chase Dam wall, it would likely have been sensed by Herold during the reinforcing steel survey. Similarly, there does not appear to be steel reinforcing in the Lower Chase Dam wall either. Steel reinforcing was noted in the concrete works at the Lower Chase spillway under a thin concrete cover. Herold advised EBA that reinforcing steel may not have been included in such a structure at the turn of the century and the concrete had a high likelihood of being poorly constructed as is the case at the Morden Colliery head frame.

Given EBA's experience with aging dam structures on Vancouver Island and using engineering judgement combined with the findings of the background information review and field work discussed in Sections 3.0 and 4.0 , the following assumptions for both dams can be made for the analysis:

- The rock fill is in a generally compact state. Compact is a term developed decades ago to describe soils which were not loose or dense. The correct use of modern compaction methods would produce a dense material. The actual density will vary and the analysis will consider a range of densities for the analysis to account for this variability;
- The low seepage rates and free draining nature of the rock fill in the downstream shell is such that the water level within the downstream shell is very low;
- The concrete walls are in poor structural condition due to the combination of low strength and durability of the concrete caused by lack of familiarity with concrete construction for dams, poor construction cold joints and poor concrete placement and consolidation techniques; and
- The concrete walls are unreinforced which means that large permanent deformations can occur once the walls crack upon seismic loading due to the lack of ductility in the wall system.

6.0 ASSESSMENT OF MAGNITUDE OF DESIGN SEISMIC EVENT

6.1 FAILURE MODE ASSESSMENT

The following failure modes during or after the design seismic event are considered to be possible for the subject dams in their current state:

- <u>Failure Mode #1</u> - Complete or partial loss of reservoir during seismic event due to seismic loading;
- <u>Failure Mode #2</u> - Cascade overtopping failure from a dam failure further upstream during or after the design seismic event;
- <u>Failure Mode #3</u> - Significant post seismic event deformations that eventually cause complete or partial loss of the reservoir after the design seismic event; and
- <u>Failure Mode #4</u> - Perception of impending failure after the design seismic event.

Excessive deformations of either dam include settlement of the crest, instability of the upstream or downstream slopes or excessive deflection of the concrete walls below the crest of the dam that compromises their ability to hold water in a safe manner or causes them to experience shear or toppling failure and lose concrete-to-concrete contact at any point along the height of the wall.

It is EBA's opinion that the failure modes described above may occur in response to a seismic event with a return period less than the design seismic event. Although this study is focused on the design seismic event, the actual return period of an event that results in failure may be less than the return period of the design seismic event.

Each of the aforementioned failure modes and their impact on loss of life are discussed briefly in the following paragraphs. Discussions regarding potential for loss of life associated with inundation are presented in generalities as the inundation zone associated with failure of one or both the subject dams due to a seismic event (rather than a PMF) is unknown at this time.

Failure Mode #1 - This failure mode involves the initiation of an uncontrolled discharge during the seismic event, a condition which is exacerbated during the remainder of the seismic event. A partial uncontrolled discharge could be caused by excessive deflections of a portion of the concrete core walls in either dam that results in severe cracking and/or toppling of an upper portion of the wall. Alternatively, instability of either the upstream of downstream slope could result in reduced support for the concrete wall, initiating toppling failure or excessive cracking and deformations of the wall that could initiate an uncontrolled discharge.

This mode of failure will occur at a time when it is not practical to provide sufficient warning to initiate evacuation of the potential inundation zone. The potential loss of life associated with this mode of failure could be a high percentage of the population at risk. The extent of uncontrolled discharge (and by extension, loss of life) will depend on the depth at which the concrete wall either fails by toppling or shear and the degree of downstream shell erosion.

Failure during the design seismic event is considered to be most likely for Middle Chase Dam.

Failure Mode #2 - Cascade dam failure (i.e., overtopping failure of a dam caused by uncontrolled discharge from a dam located further upstream) is a complicated topic. The only dams upstream of Middle and Lower Chases Dams are the Reservoir #1 and Upper Chase Dam. It is understood that the Reservoir #1 dam was retrofitted for seismic loading in the late 1990s. More recently, the Upper Chase Dam was studied by EBA in a seismic hazard assessment in 2004 and 2005 and was found to be stable during the seismic event with some potential for post seismic instability due to the buried large diameter waterlines beneath the road at the crest of the dam. Provided the risks associated with the waterlines have been addressed as recommended by EBA in 2005, failure of Upper Chase Dam after a seismic event is considered to be unlikely.

Seismic hazards related to one or both of these two dams failing upstream of Middle Chase Dam is not part of the scope of this assessment. However, failure of one or both of Reservoir #1 Dam and Upper Chase Dam is not considered to be as likely as a rapidly progressing uncontrolled discharge from Middle Chase Dam causing overtopping failure of Lower Chase Dam. If this failure mode were to occur (without monitoring and an evacuation plan in place) during or very shortly after the seismic event the loss of life could be a high percentage of the population at risk.

Failure Mode #3 - This failure mode is more likely to be due to excessive seepage caused by damage to the wall that reduces the stability of the downstream slope. Post seismic event instability of the downstream slope could cause deflection or toppling of a damaged section of the wall, initiating an uncontrolled discharge. The potential loss of life can be mitigated if post seismic event monitoring protocols with a post seismic emergency response plan and proper evacuation procedures are established and followed. If effectively administered, the loss of life associated with this failure mode could be as low as zero.

Failure Mode #4 - In this case, there is no failure, but given what is known about the dams, it is EBA's opinion that it would be prudent to view them as being on the verge of failure in their current condition after the design seismic event. The potential for failure well after the seismic event could cause perceived safety issues with re-inhabiting or continuing to inhabit the potential inundation zone.

All of these failure modes are "sunny day" failures, meaning they could occur without a storm event or associated increase in reservoir level. The seismic event is considered a "sunny day" event. The confirmed presence of the low level conduit in Middle Chase Dam constitutes a risk of a "sunny day" piping failure without an initiating seismic event. This could in turn lead to an overtopping failure of Lower Chase Dam. This is discussed further in Section 9.2.2.

The following supplementary recommendations are made as they relate indirectly to the seismic hazard study discussed herein:

- The seismic hazard assessment for Reservoir #1 Dam should be reviewed in light of the 2004 revisions (i.e. increases) made by the Geologic Survey of Canada to the frequency - magnitude relationship for seismic events on Vancouver Island and Lower Mainland area of British Columbia; and
- The seismic stability of Harewood Dam and potential for uncontrolled discharge should be evaluated. An uncontrolled discharge from Harewood Dam in response to a significant seismic event could contribute to the potential zone of inundation associated with partial or complete failure of one or both of the subject dams.

6.2 CONSEQUENCE CLASSIFICATION

The Consequence Classification for Middle and Lower Chase Dams were set as High in the 2003 Dam Safety Review (Golder, 2004a, 2004b). This consequence classification was based on the 1999 Dam Safety Guidelines prepared by the CDA (CDA Guidelines, 1999). In 2007, the CDA revised their consequence classification system which has required some reconsideration on how dams are classified in British Columbia.

The BCMoE Dam Safety branch currently has the Middle and Lower Chase River Dams classified as High-High Consequence Dams using the draft Interim Consequence Classification Policy presented in Appendix D. The BCMoE High-High consequence classification corresponds to a 2007 CDA Consequence classification of Very High. The primary reason for this classification is that BCMoE feels that between 10 and 100 people could die in the event of a failure of this dam.

EBA and the City reviewed the consequence classification for these structures in 2008 and concluded that they were a High – Low classification which implies that 1 to 10 people would die in the event of a failure of either dam. This recommendation has not yet been accepted by BCMoE.

A qualitative assessment of the extent of inundation from loss of all or part of the impoundments for Middle and Lower Chase Dam or Lower Chase Dam alone would suggest that the consequences of an uncontrolled discharge will include the following:

- Wash out of the Howard Avenue crossing of the Chase River;
- Inundation of a portion or all of the low lying area between Howard and Bruce Avenue which inlcudes the school and daycare buildings, both of which are located within 50 m of the Chase River;
- Potential inundation of various residences from the upstream side of Howard Avenue to approximately Park Avenue. Downstream of Park Avenue the Chase River Valley appears to deepen enough to contain the flood caused by failure of Middle and/or Lower Chase Dams. This should be verified by the 2010 inundation study;
- Potential wash out of the E&N railway crossing of the Chase River; and
- Potential wash out of the #1 TransCanada Highway crossing of the Chase River.

It is reasonable to assume that a rapid rise of swiftly flowing water would occur in the relatively low lying area between Howard Avenue and Bruce Street. The depth and velocity of flood water associated with failure of either of the subject dams alone, or in concert with other structures, due to a seismic event is unknown. These two variables are important for estimating the likelihood of loss of life. Additionally, the depth and velocity of the water released from one or both subject dam reservoirs, plus any contributions from Reservoir #1 or Harewood Lake, would be a function of the rate of uncontrolled discharge from each impoundment. The impact of the Chase River channel geometry downstream of Lower Chase Dam may also attenuate the flood flow rates. These effects will need to be considered during the upcoming flood inundation modeling assignment to be commissioned by the City in early 2010. The proximity of a school and a daycare to the Chase River downstream of the subject dams heightens the need for this assessment.

Based on the aforementioned observations and considerations, and given the unknown extent of flood inundation, the consequence classification is either at the upper end of High-Low or at the low end of High-High. This matter cannot be resolved until the 2010 flood inundation study is completed.

6.3 RETURN PERIOD OF DESIGN SEISMIC EVENT

The consequence classification of a dam determines the design requirements. In the case of seismic loading or spillway design, the consequence classification determines the return periods of the seismic and flood events respectively. With the latest edition of the CDA Guidelines (CDA, 2007), there has been an increase in the severity of seismic and flood events recommended for design and for assessment. The previous version of the CDA guidelines (CDA, 1999) upon which the 2003 Dam Safety Review was based, supported a High Consequence Classification and the 1:3,000 year return period recommended in the last dam safety review (Golder, 2003). The 2007 CDA Guidelines use a different system for establishing a more conservative design criteria. The BCMoE classification of High-High discussed in Section 6.2 relates to a CDA (2007) classification of Very High which now in turn requires the use of a design seismic event with a 1:5,000 year return period. This creates a dilemma in that while the Water Act (2000) and BCMoE regulations govern in the province of British Columbia, they do not recommend design return periods for seismic events.

The draft Interim Consequence Classification document prepared by BCMoE (included in Appendix D) attempts to bridge the gap between the current Canadian Dam Association Dam Safety Guidelines (2007) and the Water Act (2000). The draft policy includes a key statement "An important distinction to note is that Dam Safety Regulation classifications are for dam owner requirements and the CDA Guidelines classifications are for dam design criteria". Therefore, the draft Interim Consequences Classification document applies to owners of existing dams for operations considerations such as this study. Engineering design for seismic rehabilitation of such dams would be governed by the 2007 CDA Guidelines with regard to return period of design seismic event. The seismic hazard assessment presented herein is subject to the contents of the draft Interim Consequence Classification document. Furthermore, given that the primary purpose of a Consequence Classification is to establish the return period of the design seismic event (in this case), BCMoE has advised EBA that the 1:3,000 design seismic event established during the 2003 Dam Safety Review would be accepted.

EBA discussed seismic design considerations from a structural perspective with Herold Engineering. A new school would have to be designed to withstand a 1:2,475 year seismic event. It is not known if the school or daycare structures have been assessed or upgraded to withstand the 1:2,475 year seismic event.

It is important to recognize that a 1:3,000 or 1:2,475 year event will not occur in 3,000 or 2,475 years. A different way of presenting this return period is in terms of a percent chance in 50 years. A 1:2,475 event has a 2 percent chance of occurring in 50 years. Similarly, the 1:3,000 year event has a 1.6 percent chance of occurring in 50 years.

The 2007 CDA Guidelines Table 6-1 Suggested Design Flood and Earthquake Levels (presented in Appendix D as part of the Interim Consequence Classification Policy for Dams in BC), Note 6 says that the Earthquake Design Ground Motions must be justified to demonstrate conformance to societal norms of acceptable risk. Justification can be provided with the help of failure modes analysis focused on the particular modes that can contribute to failure initiated by a seismic event. If the justification can not be provided, the EDGM (design seismic event) should be 1:10,000 (return period). The implications of this statement are discussed in the following paragraph.

Ground shaking associated with a 1:3,000 year seismic event will likely result in unacceptable damage to either dam that will likely need to be addressed in some manner after completion of this study and the 2010 inundation study, especially given the proximity of the various downstream stakeholders to the Chase River. The design of the rehabilitative measures may have to use the 1:5,000 year seismic event as it will involve modification of the dam. Depending on the City's, and other stakeholder's, tolerance for risk, there may be some justification for the 1:10,000 year event due to the proximity of the school and daycare to the Chase River. The upcoming flood inundation study to be commissioned by the City in 2010 is expected to provide further refinement of the Consequence Classification.

7.0 SEISMIC LOADING INPUT FOR ANALYSIS

EBA retained the services of Dr. Anderson, P.Geo, P.Eng. of CAN Engineering Ltd. (CAN) to conduct a study of the seismic loading anticipated from a 1:3,000 year seismic event. Dr. Anderson's report is presented in Appendix E.

The seismic loading modeled by EBA during the analysis and modeling phase of this assignment was based on the CAN report.

8.0 ANALYSIS INPUT

8.1 GENERAL

The three general factors that affect the static stability of a dam are as follows:

- Geometry of slope and interfaces;
- Shear strength of materials within or beneath the dam; and
- Porewater pressures within the dam.

In the case of seismic stability, the deformability of the various materials is also of concern, in addition to the potential for liquefaction which is a form of strain weakening due to dynamic loading.

Each of these properties is discussed in the following sections.

8.2 GEOMETRY

The analysis cross sections used in the modeling phase of this assignment are presented in Figures 4 and 7 for the Middle and Lower Chase Dams respectively. These figures are based on the information available from the review of background information and the results of the site reconnaissance (including diving) work conducted by or for EBA. With regard to the Lower Chase Dam, the depths to rock fill encountered in the 1978 borehole and test pit logs are noted on Figure 7. The steepest slope of rock fill within the downstream shell that could be inferred from this data was selected and this slope generally matched the upstream slope of the rock fill upstream of the concrete core wall. The selected slope of the rock fill within the downstream shell also extended to a point just inside of the downstream toe of the dam which matches the available historic information in that rock fill was not visible over the lower portion of the slope before the toe berm/filter layer was added in 1980.

The basal geometry and height of wall in each dam was estimated using topographic and bathymetric survey data and the limited number of 1978 boreholes that intercepted bedrock or till beneath the dams. The soft, wet unconsolidated alluvium downstream of Lower Chase Dam appears to be in a relatively flat lying area at the toe of the dam. The steeper section of valley bottom beneath the dam would likely not have such deposits based on our observations elsewhere in the Chase Dam Valley.

8.3 SHEAR STRENGTH AND STIFFNESS

A parametric approach to the modeling analysis was briefly described in EBA's proposal dated January, 2009. A three step parametric analysis method was selected. In this method EBA selected material properties of the fill materials to correspond to the following descriptors:

- Best Case Scenario – the best material properties that could be reasonably expected given what is known about the dams;
- Reasonable Worst Case Scenario – the reasonably worst material properties that could be expected; and
- Most Likely Case Scenario – The material properties that the available data and engineering judgment indicates are most likely to be present.

This approach was selected given what is known about the dams, the consequences of failure and the practical consideration that the City will have to implement some manner of rehabilitation program for the dams and it was judged that budget would be better spent on design and analysis of the proposed rehabilitation measures. The justification for this approach was presented in our proposal dated January 19, 2009.

Review of the background information for the various materials indicated the following for each material:

<u>Original 1904 Vintage Rock Fill</u>

The 1978 drilling investigation included Standard Penetration Testing (SPT) which consists of driving a standard dimension sampler a specified distance using a specified hammer weight dropped a specified height at a specified rate. For all of the specifications involved in an SPT test, it is affected by many factors, not the least of which is the size of the particles it is driven into. The SPT was designed for sand and the tests where the SPT was successfully conducted indicate that the tested zones contain an appreciable amount of fine gravel, sand and silt. The presence of cobbles and particles up to about 0.6 m within the fill places a limitation on the usefulness of the SPT test as indicated by some tests meeting refusal with minimal sampler penetration. However, when the tests are considered, the general range of the completed SPT test results (known as "N values") fall into the density description of "compact" (N values between 10 and 30). Furthermore, the maximum blow count corresponds to the upper limit of "compact" (i.e., N= 30), the minimum blow count corresponds to the approximate lower limit of the compact zone (i.e., N=9) and the mean and median blow count corresponds to the mid point of "compact" (i.e., N=15) as noted on EBA's Geotechnical Description Terms sheet in Appendix B. There is a large body of empirical correlations used to relate SPT "N" values to material properties which is the cornerstone of EBA's method for preparation of input parameters for the modeling phase.

EBA selected the following "N" values for the various scenarios used in the analysis:

- Best Case Scenario – Rock fill behaves like material with an SPT "N" value of 30 (highest value, upper end of "compact" range);
- Reasonable Worst Case Scenario – Rock fill behaves like material with an SPT "N" value of 9 (lowest value, upper level of loose range); and
- Most Likely Case Scenario – Rock fill behaves like material with an SPT "N" value of 15 (mean of "compact" range).

This range of values applies to all original rock fill zones within the two dams.

<u>1918 Railway Fill Placed at Lower Chase Dam</u>

The cinders, slag, sand and gravel appear to have been end-dumped into place with no compaction based on the low SPT "N" values (N=1 and 4) for this material. The relatively fine grained nature of these soils would not have experienced the impact of placement that the rock fill particles experienced during placement; as such, the 1918 fill is generally "very loose" to "loose", based on the 1978 SPT blow counts. Loose to very loose soils, and even those of the lower range of "compact" are more likely to experience significant deformations during a seismic event. If they were saturated they would likely experience loss of strength known as "liquefaction". However, this is unlikely for the majority of the mass of this material within Lower Chase Dam as they are almost entirely above any water table or seepage except for those at the valley bottom fill contact. The water table within the dams is discussed in Section 8.4.

Using a similar logic as was used for the original rock fill, EBA selected the following "N" values for the various scenarios used in the analysis:

- Best Case Scenario – 1918 fill behaves like material with an SPT "N" value of 4 (highest value);
- Reasonable Worst Case Scenario – 1918 fill behaves like material with an SPT "N" value of 1 (lowest value); and

- Most Likely Case Scenario – 1918 fill behaves like material with an SPT "N" value of 3.

These low values are consistent with EBA's experience in investigation and assessing historical dams constructed as part of Cumberland coal mining activities and elsewhere on Vancouver Island using end-dumped sand and gravel fill.

<u>1980 Fill Placed in Downstream Shell of Middle and Lower Chase Dams</u>

The 1980 fill placed in the downstream shell of Middle Chase Dam was reasonably high quality fill that was placed in thin lifts, compacted with a walk behind vibratory compactor and was density tested. Although no records are available for review, it is expected that a compaction standard of 95% of the maximum dry density achieved using moisture-density relationship testing with standard effort (95% Standard Proctor) would have been used to guide density testing. This suggests the fill is in a dense state.

Using a similar logic as described above, EBA selected the following "N" values for the various scenarios used in the analysis using our engineering judgment and experience:

- Best Case Scenario – 1980 fill behaves like dense material with an SPT "N" value of 40;
- Reasonable Worst Case Scenario – 1980 fill behaves like compact material with an SPT "N" value of 25; and
- Most Likely Case Scenario – 1980 fill behaves like material with an SPT "N" value of 30 (boundary between compact and dense).

This range of values are also considered to be representative of the filter zone placed on the lower slope of Lower Chase Dam which was also compacted sand and gravel.

<u>Concrete Core Walls</u>

The concrete core wall properties are discussed in Section 9.3.

<u>Bedrock or Till Foundation</u>

The lack of any sign of cracking, settlement or distortion of the concrete walls indicates they are on a firm competent foundation, mostly likely bedrock at Middle Chase Dam and till overlying bedrock or bedrock beneath Lower Chase Dam. It is possible that the valley bottom beneath the Lower Chase Dam may have some unconsolidated channel deposits and alluvium beneath the upstream and downstream shells. However, due to the generally V-shaped nature of the valley, there will be an arching and three dimensional effect of the abutment-fill contact which will minimize the destabilizing effect of any valley bottom alluvium.

8.4 POREWATER PRESSURES

There is no porewater pressure data available for either dam. However, it is EBA's opinion based on experience and judgement that pore pressures are low within the downstream shells of both dams for the following reasons:

- The downstream shells of each dam are relatively free draining which would not support development of a high phreatic surface (e.g., water table within the downstream shell) or high pore pressures; and
- Seepage rates through the dams are relatively low, based on review of v-notch weir data from each dam. The water levels within the upstream shell have been assumed to be equal to lake level which is reasonable for a coarse rock fill shell and concrete core wall founded on bedrock or bedrock and till.

In addition to these considerations, Middle Chase Dam has a number of under drains constructed beneath the 1980 fill to collect seepage in addition to the relatively free draining sand and gravel fill in the downstream shell. Lower Chase Dam has a rock fill shell covered with a mix of free draining cinders, slag, sand and gravel and a sand and gravel filter design to act as a drain.

Seepage rates for Lower Chase Dam are low and the majority of the seepage at Middle Chase Dam is believed to be either from bedrock discontinuities or the low level conduit left in place at the left abutment of the dam. These low seepage rates do not introduce enough water into the downstream shell of the dam to support high pore pressures

Based on these considerations, the groundwater levels within the downstream shell of each dam under normal reservoir level conditions are anticipated to be generally within 0.5 m above the base of the dam.

9.0 ANALYSIS

9.1 GENERAL

The analysis conducted by EBA included consideration of the seismic response of both dams. EBA also conducted a semi-quantitative piping risk assessment starting with the current condition of the subject dams under static loading.

9.2 PIPING RISK ASSESSMENT

9.2.1 General

The potential for piping developing after a significant seismic event has been assessed by EBA. To provide a basis for comparison, a piping risk assessment for the current condition of Middle and Lower Chase Dams has been conducted as well.

9.2.2 Potential for Piping Under Current Conditions

The condition of the Middle and Lower Chase Dams presents a challenge in that each dam has performed reasonably well since the late 1970s (and most likely earlier) with no reported or available reports of occurrences of turbid seepage. The condition of the dams in the late 1970s indicates that dam performance prior to that appears to have been generally satisfactory although records are not available prior to 1970.

The dams have different scenarios for a piping failure to develop under current conditions. In the case of Middle Chase Dam, the confirmation of the existence of the low level conduit at the left abutment raises the concern that continued deterioration of the wooden low level conduit pipe will eventually result in a sudden increase in seepage through the concrete wall. The downstream shell of Middle Chase Dam consists mostly of compacted sand and gravel, which could experience piping erosion under high gradients and high rates of seepage. However, the fill at the left abutment above the low level conduit is original rock fill left in place during the 1980 repairs. Depending on the rapidity of increase of seepage around/through the low level conduit passing through the concrete core wall, a significant portion of the downstream shell could be eroded with subsequent initiation of downstream slope instability which would increase the potential for toppling of the deteriorate concrete wall due to loss of support. An uncontrolled discharge of the Middle Chase reservoir could cause an overtopping failure of Lower Chase Dam as discussed in Section 6.1. This is a “sunny day” failure mechanism that could occur with little forewarning. Piping through the foundation appears to be unlikely due to the competency of the bedrock visible upstream and downstream of the dam.

The occurrence of a significant seismic event could initiate a piping failure at Middle Chase Dam similar to what has been described for non-seismic conditions, but more due to deflection of the wood stave pipe within the concrete wall.

In the case of Lower Chase Dam, the low level conduits were backfilled with concrete and the valve chamber at their inlets was backfilled with concrete. Therefore, there is limited potential for seepage to pass through concrete core wall due to any continued deterioration off the wooden low level conduits. Finally, the majority of the fill in contact with the abutments and concrete core wall that could experience piping from seepage through or beneath the core wall is rock fill. A sand and gravel filter/toe berm has been constructed at the toe of the dam to stabilize and most likely filter the slag, cinders, sand and gravel that EBA believes was placed over the rock fill around 1918. In the unlikely event of an increase in seepage through or beneath the concrete core wall (with the wall founded on competent bedrock) under current conditions, the sand and gravel toe berm/filter would inhibit or prevent transfer of a significant amount of fines from within the body of the dam.

EBA has used a probabilistic method, the University of New South Wales (UNSW) method, for assessing the relative likelihood of failure of the dams by piping in their current condition as presented in Foster et al. (2000). This paper is included in Appendix F 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, without a seismic event occurring, is as follows:

- Middle Chase Dam – 1.85 x 10^{-5} per annum (or 1:54,054 years); and
- Lower Chase Dam – 3.08 x 10^{-5} per annum (or 1:32,467 years, for wall founded on bedrock case).

This methodology indicates Lower Chase Dam is more susceptible to piping then Middle Chase Dam. However, because there appears to be a timber conduit passing though the Middle Chase Dam wall, the Middle Chase Dam has a higher potential for piping.

These probabilities are 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 weighing factors for each piping mode with justification is presented in Appendix F. While these figures imply a high degree of accuracy, it is not possible to estimate the likelihood of failure for either dam this accurately given what is known about the dams. The implied accuracy is due to the statistics used in the Foster et al. (2006) study. This probability confirms EBA's intuition that, while the good performance to date of either dam is encouraging, there is still a small probability that it could develop a piping failure even if it experiences the same loading conditions in the future as it has in the past.

The case of Middle Chase Dam, the presence of the old deteriorating timber conduit and the sand and gravel fill in the downstream shell present the primary risk of piping, even under static loading. The probability of piping presented earlier reflects the clear and relatively steady seepage rate recorded at the Middle Chase Dam v-notch weir. An increase of seepage or the development of turbid seepage without a seismic event would raise this probability of piping failure by a factor varying from 2 to 10 as per the UNSW method. In the case of Lower Chase Dam, the potential for the wall being founded on till or bedrock with infilling or poorly indurated/weathered zones is the primary concern. The probability of piping calculated herein is for the assumed case that the concrete wall is founded on bedrock. The possibility that the concrete wall is founded on till appears to be remote given the anticipated long-term performance of the dam with high gradients across the base of the concrete wall.

9.2.3 Potential for Piping After a Significant Seismic Event

In the event of a significant seismic event, the integrity of the dams could be compromised by increased seepage through the body of the dam due to cracking of the concrete wall or shifting of the dam foundations. In this event, the dams would be impounding water under significantly different conditions than they currently are; therefore, the past 100 years or so of generally satisfactory service could not be relied upon to continue in the future. The probability of a piping failure being able to develop would increase accordingly. EBA anticipates that the concrete walls will either deform in a manner that causes or exacerbates cracking at construction cold joints or weaker sections where concrete honeycombing is present. It is reasonable to conclude that seepage rates through the wall would significantly increase due to wall damage experienced during a seismic event. The magnitude of the seismic event that causes wall damage and increased seepage may be less severe than the design seismic event.

Piping more frequently occurs within five years of first filling (Foster et al, 2000); however, there are many examples of dams where the effects of piping were only observed many years after first filling. In the case of a significant seismic event, the satisfactory time record of dam performance would then start at the day of the significant seismic event, not the date of first filling about 100 years ago, or after the significant seismic event that occurred in 1946. The 2003 Dam Safety Review report indicated that the peak ground acceleration experienced by both dams to date was about 0.03 g, due to distance to the earthquake epicentre. Additionally, since that time 63 years have passed which is a long time for additional wood deterioration to occur in the low level conduit.

The probability of a piping failure developing at each in the first five years after a significant seismic event is estimate during the UNSW method to be as follows:

- Middle Chase Dam – 1.59 x 10^{-4} per annum (or 1:6,289 years); and
- Lower Chase Dam – 2.78 x 10^{-4} per annum (or 1:3,597 years, for wall founded on bedrock case).

In general, the occurrence of a significant seismic event, not necessarily as large as the design seismic event, increases the probability of piping failure predicted by the method of an order of magnitude.

An important distinction to be made is that this assessment has only modified the factor relating to the age of the dam to one where the dam is considered to be less than five years of age. An increase of seepage or the development of turbid seepage would raise this probability of piping failure by a factor varying from 2 to 10 as per the UNSW method. This confirms EBA's intuition that a severe seismic event would tend to increase the potential for a piping failure developing.

9.3 SEISMIC RESPONSE ANALYSIS

Seismic response of the Middle and Lower Chase Dams were carried out using the computer program FLAC v 6.0 (Itasca, 2009). These dynamic analyses were performed in time domain using the earthquake motions provided by CAN for the 1:3,000 year design event (see Section 7.0 and Appendix E).

Figure 8 presents the FLAC layout and material zones used in the models for both dams. Non-linear behaviour of the rock fill, sand and gravel, as well as cinders/slag zones were simulated using the UBCSAND constitutive model (e.g. Byrne et al., 2004). This model has been developed by Dr. Byrne, EBA's external seismic reviewer for this project, at UBC, primarily for simulating the seismic behaviour of liquefiable sand; however, its general features allow the use of it for materials that will not undergo liquefaction as well.

The primary input parameter for this constitutive model is the SPT "N" value. Values selected for the three cases (Reasonable Worst, Most Likely, and Best) are described in Section 8.3. For the rock fill zone, the SPT "N" was used as a parameter to define the shear modulus number and friction angle at failure (Saboya and Byrne, 1993). The resulting values are shown in Table 1:

TABLE 1: FRICTION ANGLE AND SHEAR MODULUS NUMBER BASED ON SPT "N" VALUES

	SPT Number			Friction Angle at Failure (°)			Shear Modulus Number		
Material	Worst	Most Likely	Best	Worst	Most Likely	Best	Worst	Most Likely	Best
Rock Fill	9	15	30	41	42	43	675	800	1010
Sand/Gravel	25	30	40	38	38	39	950	1010	1110
Cinder/Slag	1	3	4	33	33	33	325	470	515

The shear modulus number is a means to reflect the dependence of shear stiffness of a material with increased confining stress. The shear stiffness has been estimated based on the SPT N values but is also known to increase with confining stress. Therefore, materials at the base of each dam under the load (or confining stress) of the overlying fill will tend to be stiffer.

The concrete wall was modelled using beam elements with elastic and inelastic (moment capacity) behaviour. Structural properties of the wall were provided by Herold. Most importantly for the dynamic analyses, the effective moment of inertia of the wall was selected as 20% of the value for the gross section; and, the damping ratio was set to 10%. Young's modulus values of 17.3 and 21.9 GPa were used for the concrete walls of the Middle and Lower Chase Dams, respectively, based on Schmidt hammer test results.

Figures 9 and 10 present contours of horizontal displacement and the deformed shape of the concrete wall (when modelled as an elastic member i.e. does not crack) for the Middle and Lower Dams, respectively, at the end of shaking. Although displacements larger than 0.5 m are predicted, the contour plots are limited to +/- 0.5 m to clearly show the movement zones within the upstream/downstream shells Deflections predicted for the top of the concrete wall are also presented on Figures 9 and 10.

Figure 11 presents contours of total displacement (i.e., resultant of horizontal and vertical components, limited to + 0.5 m), deformed shape of the concrete wall and its top deflection (m) for the Most Likely case.

As illustrated by contours of horizontal displacement (Figures 9 & 10), and total displacement (Figure 11), the deformation mechanisms of the two dams are as follows:

a) Middle Chase Dam: large and deep-seated deformation of the upstream shell, concrete wall, crest and the upper section of the downstream shell in an upstream direction.

b) Lower Chase Dam: large deformation of the cinder/slag zone located under the rail tracks and consequently the toe berm in a downstream direction; and, relatively small and shallow-seated deformation of the upstream shell and concrete wall in an upstream direction.

The predicted deformations for the top of the concrete wall (as shown on Figures 9 & 10) range from 0.360 to 0.924 m for the Middle Chase Dam and from 0.055 to 0.065 m for the Lower Chase Dam, depending on the scenario analyzed. The level of accuracy afforded by this analysis, given the nature of the inputs does not warrant millimetre accuracy. The estimated range of validity of these results is +/- 50%.

Effect of inelastic (in other words: elastic perfectly plastic) behaviour of the concrete wall was considered using bending moment capacities of 150 and 600 kN/m for the 0.6 and 1.2 m thick walls of the Middle and Lower Chase Dams, respectively. Beam elements with defined bending moment capacity will yield and deform indefinitely when/if the bending moment applied to the element during earthquake shaking exceeds the capacity. Figure 12 presents the deformations predicted in case of inelastic behaviour of the wall for the Most Likely case for both dams. The deformations predicted for the top of the walls were increased by a factor of about 3 to values of 1.5 and 0.2 m for the Middle and Lower Chase Dams, respectively. The mechanisms of deformations, however, were similar to the elastic case. Figure 13 shows the bending moment diagram at the end of shaking and the depth at which the maximum bending movement exists.

Figures 14 and 15 present the time histories of horizontal displacements predicted for the top of the walls relative to the input motion applied at the base (bottom) of the model. As illustrated, for both dams, the wall movement builds up in the upstream direction with almost no downstream relative movement during shaking.

The final deformed mesh at the end of the modelled design seismic event is presented for both dams on Figure 16.

10.0 DISCUSSION

10.1 GENERAL

The background review, field work and analysis conducted to date indicate that the Middle and Lower Chase Dams are complex structures due to the nature of the following factors:

- Age;
- State of concrete construction practices in Nanaimo when these dams were built; and
- The varying methods of fill placement used in their initial construction and subsequent modifications.

This complexity inherent in the dam structures is exacerbated when the seismic and post seismic response of these dams and the presence of a downstream stakeholders in an urban environment is considered.

The discussion presented within this section will put the results of the previous analyses into context with regard to what options the City has for addressing the seismic hazards presented by the subject dams.

It is of paramount importance for the City to be aware that it is EBA's opinion that a seismic event of lesser magnitude than the design seismic event could cause significant damage to the dams which could have similar outcomes to what is described in this report. Determination of the seismic event which could trigger a failure was outside the scope of this assignment.

10.2 ALARP PRINCIPAL

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 though the years.

For the Middle and Lower Chase River Dams, the following questions need to be asked:

- "How safe is safe enough?"; and
- "How does one 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 regards to tolerable risk. ALARP stands for **A**s **L**ow **A**s **R**easonably **P**racticable. This principal is demonstrated in Figure 17 which relates magnitude of loss of life to probability of loss of life. This chart shows the suggested relationship between the probability of occurrence, potential loss of life and varying degrees of risk tolerance is within the dam community in Canada, as defined by the CDA. EBA can not define the City's tolerance for loss of life, therefore, it is up to the City to decide if the ALARP limits shown on Figure 17 are acceptable or not.

EBA has applied the ALARP principal to the results of this assessment presented herein. The probability of one or more people being killed by the flood wave from failure of one or both dams is the product of three probabilities as below.

$$\mathbf{P}_{\text{loss of life}} = \mathbf{P}_{\text{failure}} \times \mathbf{P}_{\text{persons in way}} \times \mathbf{P}_{\text{persons in way being killed}}$$

"P" refers to "Probability".

For the purposes of this assessment, EBA has assumed that the maximum number of deaths that could occur is ten as discussed in Section 4.3. This loss of life estimate could be revised upon completion of the flood inundation study the City will initiate in early 2010.

Based on the ALARP Principal, the ALARP range of probability for ten fatalities is bounded by $1x10^{-6}$ to $1x10^{-4}$ per annum, or between a 1:1,000,000 and a 1:10,000 per year event.

10.3 RISK ASSESSMENT OF PIPING POTENTIAL

The potential for a piping failure developing under current conditions and after a significant seismic event is present on Figure 17 for an assumed ten lives lost in the event of a failure.

The dams in their current condition fall within the ALARP zone. As discussed in Section 9.2, the presence of the timber low level conduit within the left abutment and concrete core wall of Middle Chase Dam presents a potential piping risk that could increase under static loading conditions if the rate of seepage increases or if the seepage becomes turbid. Should that occur, the piping risk for Middle Chase Dam without seismic loading could increase by a factor of 2 to 10, approaching or entering the Unacceptable Zone presented in Figure 17.

For post seismic conditions, both dams fall above the ALARP zone in the Unacceptable zone. This rating does not include consideration of any increased or turbid seepage. The reduction in the risks of piping failure for a post seismic event will be briefly discussed in Section 11.0 where the candidate rehabilitative measures are presented.

10.4 INTERPRETATION OF SEISMIC RESPONSE

The analysis results discussed in Section 9.3 and presented in Figures 9 through 16 indicate that the effect of the design seismic event will vary between the two subject dams. Both dams have upstream shells that have a lower top elevation than the downstream shells. This condition is more pronounced in the case of Middle Chase Dam as shown in Figures 4 and 7. This creates an imbalance in the forces available to support the concrete wall in that the upstream shell will provide less support to the concrete wall than the downstream shell. Additionally this results in higher dynamic loading of the concrete wall from the fill on the downstream side of the dam when the ground is being accelerated in an upstream direction. The effect of this imbalance is a net upstream deflection of the top of the concrete walls in both dams with the most severe deformations being experienced at Middle Chase Dam.

The concrete walls are most likely unreinforced and the quality of concrete construction appears to be poor as indicated by other historical concrete structures in Collier Dams Park and the known state of concrete construction practice in Nanaimo when these dams were constructed. Based on the results of the analysis and EBA's judgement, the loading and deformations experienced by the concrete walls in both dams during the design seismic event is such that cracks will readily form in the following areas:

- Construction cold joints;
- Locations where significant honeycombing has reduced the effective structural wall thickness;
- Zones of poor quality concrete; and

- Locations where zones of loose or reduced stiffness fill are present such as zones of segregated fines within the original rock fill mass(es).

The modelling conducted by EBA was to gain insight into the nature of deformations. Exact predictions of wall deflection and depth of cracking are not possible given the uncertainties and variability's associated with the subject dams. However, the modelled response of the dams to seismic loading associated with the design seismic event can be used with engineering judgement to provide practical predictions of dam behaviour that can then be used to prepare conceptual designs to address/mitigate the seismic hazards associated with each dam.

A comparative analysis for the Most Likely Case using elastic and inelastic behaviour for the concrete walls was conducted for each dam. For small transient deflections in the order of 1 or 2 cm it would be justifiable to use an elastic model where no permanent deformations were modeled. However, the magnitude of wall deformation observed in both cases lead to the conclusion that the flexural strains experienced by the wall would exceed the tensile and possibly the compressive strength of the concrete as the wall cyclically deflected under the seismic loading. Once cracked, the concrete wall would then deform more significantly above the crack with an accumulation of movement in an upstream direction as shown in Figures 14 and 15. It is for this reason that inelastic wall properties were used which resulted in large permanent deformations.

The seismic response of each dam and the most likely mode of failure are discussed in the following paragraphs.

<u>Middle Chase Dam</u>

In the case of Middle Chase Dam, the wall appears to crack three to four seconds into the modelled seismic event and is gradually deflected in an upstream direction throughout the rest of the seismic event. This is evident as the top of wall movement diverges from that of the base movement. Although the model predicts a top of wall maximum deflection of about 1.5 m using inelastic concrete parameters (1.47 m as shown on Figure 12), in reality the upstream wall would topple into the reservoir shortly after cracking and likely relatively early during the design seismic event. The maximum depth of persistent cracking appears to be approximately 8 m below the crest of the wall (as shown on Figure 13) which corresponds to the point of maximum bending moment experienced by the wall. This corresponds to about 2.0 to 2.5 m below the top of the upstream rock fill. Given the state of deterioration of the Middle Chase Dam wall, the lack of steel reinforcement, it is expected that portions of the concrete wall above the rock fill buttress will topple completely with severe cracking or opening of cold construction joints occurring to 2.0 to 2.5 m below the top of the rock fill. Cracking below this depth would occur but is anticipated to be less severe than what would occur above due to the increased confinement at depth provided by the upstream rock fill berm.

The base of the toppling failure will likely be below normal operating level of the reservoir which means the amount of seepage that starts to pass through the dam after a toppling failure, even without counting seepage through cracks, will be very high with the potential to saturate the downstream shell of the dam in a very short period of time resulting in failure and uncontrolled discharge. More importantly, the crest of the dam would be left unsupported upon toppling failure for the remaining 15 to 20 seconds of the modelled seismic event. As shown in Figure 9, upstream horizontal deflections of the crest fill between 0.3 and 0.5 m would occur which extend to the downstream crest. The modelled deformations are presented in the deformed mesh shown on Figure 16. The depth of these movements is such that a breach would be likely to occur during the modelled seismic event or within an hour, if not minutes, after completion of the modelled seismic event (i.e., Failure Mode #1 discussed in Section 6.1). The hydrodynamics of a breach and overtopping erosion were not modelled using the software used in this assessment. However, although the downstream shell would not experience any significant damage at depth due to the seismic event, it could be quickly washed away to a depth of at least 8 m below the current top of the wall.

An additional analysis was undertaken to assess the impact of the wall toppling on the performance of the fill behind the wall at Middle Chase Dam. This effect was modelled by deleting a portion of the wall from the FLAC model when cracking was initiated in the model. The results were that greater lateral deflections to greater depths occurred. This confirms EBA's intuition that wall toppling would exacerbate the deformations experienced by the fill within the dam during the remainder of the seismic event. In this modified case, the deformations of the fill were of sufficient magnitude that the reservoir would start to over top the dam before the seismic event was over.

A second series of additional analyses was undertaken to assess the impact of less severe seismic events on Middle Chase Dam. It is recalled that the 2003 Dam Safety Review report (Golder 2004a) indicated that the 1946 earthquake near Campbell River resulted in the dam experiencing peak horizontal ground accelerations in the order of 0.03g (3% of gravity). The degree of deformation of the concrete wall is not known. The results of the initial analysis are presented as follows:

- 0.1g (10% of gravity) – 85 mm total horizontal deflection; and
- 0.2g (20% gravity) – 177 mm total horizontal deflection.

The potential for toppling of the Middle Chase Dam wall upon experiencing modeled seismic events with peak ground accelerations of 0.1 to 0.2 g is unclear. However, assuming the concrete remains rigid above the basal crack (assumed to be 8 m below crest of wall); the vertical force of the cracked wall will be located within the middle third of the wall section for the 0.1g case and just inside of the middle third of the wall section for the 0.2g case. This leads EBA to conclude the following:

- The wall has a remote chance of toppling some time after the 0.1g peak ground acceleration seismic event; and

- The wall could possibly topple near the end of a seismic event with a 0.2 g peak ground acceleration.

The potential for wall toppling depends on a number of factors such as cold joint bond strength, segregation, variability in concrete strength. Given the potential for variability in these features and given the overall poor state of the Middle Chase Dam concrete wall it is reasonable to conclude that any prediction on the exact level of ground shaking required to topple the Middle Chase Wall and at what time during the seismic event will be subject to a low degree of reliability. It is EBA's opinion that any possibility of wall toppling during the seismic event should be accompanied by the expectation that sufficient fill deformations will occur that will permit overtopping during the late stages of the seismic event or shortly after it ends.

The data available from the Pacific Geoscience Centre in Sidney, BC indicates that a 1:475 year seismic event (10% chance in 50 years) will have a peak ground acceleration of 0.27g and a 1:100 year event (40% chance in 50 years) will have a peak ground acceleration of 0.13g. Given the variability associated with the Middle Chase Dam concrete wall, it is not reasonable to predict the exact return period of the seismic event that will result in toppling. However, seismic events generation peak ground accelerations of 0.1g and 0.2 g will occur with a 15% and just over 40% chance in 50 years respectively.

<u>Lower Chase Dam</u>

In the case of Lower Chase Dam, the wall appears to crack nine to ten seconds into the modelled seismic event and is gradually deflected in an upstream direction throughout the rest of the seismic event. The increased time to cracking with respect to what was modelled for Middle Chase Dam reflects not only the increased thickness of the wall at Lower Chase Dam but also the overall greater degree of confinement provided by the increased height of the rock fill berm. The model predicts a top of wall maximum deflection of about 7 cm using inelastic concrete parameters. The smaller magnitude of movement indicates that height of concrete wall that is not supported on the upstream side will not topple. The reduced deflections compared to Middle Chase Dam are due to the shorter height of unsupported wall, the softer response of the fill upstream of the wall and, possibly, the amount of loose crest fill movement in a downstream direction as the downstream slope deforms under seismic loading.

The difference in response between the downstream fill in both dams is described as follows. In the case of Middle Chase Dam, the 1980 fill that was apparently densely compacted in place tends to induce a much higher cyclic load than the loose 1918 fill and the underlying compact original rock fill at Lower Chase Dam which tends to absorb or dampen the seismic energy input into the fill. The density of the downstream shell fill in Middle Chase Dam transfers the seismic energy more efficiently from the foundation into the concrete wall.

With regard to deflection induced wall cracking, the maximum depth of persistent cracking appears to be approximately 9.5 m below the top of the wall (as shown on Figure 13) which corresponds to the point of maximum bending moment experienced by the wall which corresponds to about 7.0 m below the top of the upstream rock fill, well below normal operating water level. The permanent deflection in an upstream direction means that the portion of the wall will tend to rotate upstream. However, the relatively small magnitude of deflection qualitatively indicates that the degree, extent and magnitude of cracking and corresponding rate of seepage would not be as great as what would occur at Middle Chase Dam. However, it would represent a significant increase in seepage. The downstream slope of the Lower Chase Dam, especially the 1918 railway fill, will experience significant movements during the seismic event, in the order of 1.0 to 2.0 m, due to its loose state. The underlying rock fill will not experience as much deformation, generally less than 0.2 m of horizontal movement. Therefore, the primary effect of downstream slope movement will be distortion of the 1918 fill with crest settlement of over 0.5 m and, more importantly, distortion causing loss of continuity to the 1980 filter zone. The modelled deformations are presented in the deformed mesh shown on Figure 16. Damage to the filter zone will result in loss or reduction in the degree of protection against internal erosion from seepage that is anticipated will be initiated upon cracking of the concrete wall.

Due to the presence of soft unconsolidated alluvial deposits at the valley bottom, there may be more deformations experienced during the actual seismic event at the upstream and downstream toe of the dam than modelled. For the downstream shell, the extent of filter layer, 1918 fill and original rock fill distortion will be more severe near the toe. The influence of the soft alluvial materials will be limited as the relatively competent materials on the valley walls and associated three dimensional effects associated with the v-shape valley at the dam site will have a greater influence on the seismic response of the downstream shell and concrete wall.

The increased seepage through the concrete wall will result in an increase of the water levels in the downstream shell of Lower Chase Dam. Damage to the foundation materials could increase seepage through the bedrock due to dilation of joints. The relatively free draining nature of the original downstream rock fill will tend to keep water levels low within the downstream shell, although finer zones would be likely washed out with corresponding settlement of the dam crest. The presence of the relatively fine grained 1918 fill on the downstream face, even after deformation during the seismic event, will tend to cause water levels to build up within the generally intact downstream rock fill shell. This would tend to destabilize the downstream shell material (i.e. risk of Failure Mode # 3 discussed in Section 6.1). However, it is EBA's opinion that this will result in eventual erosion of the 1918 fill and the damaged filter layer at the base of the valley which would cause subsequent drainage of the downstream shell rock fill with associated improvement in the stability of the deformed and failed downstream slope.

It is important to keep in mind the interaction between the two subject dams after the design seismic event (or a large seismic event) in their current condition. The flood wave from failure of Middle Chase Dam would overtop Lower Chase Dam and likely cause an overtopping failure, erosion of the downstream shell and loss of support for the concrete wall followed by toppling and subsequent uncontrolled discharge from Lower Chase Dam (i.e., Failure Mode #2 discussed in Section 6.1). This failure mode would occur irrespective of the influences of elevated seepage rates and water levels on the stability of the of Lower Chase Dam downstream shell.

10.5 ALARP ASSESSMENT OF LIKELY SEISMIC RESPONSE

The analysis and modelling conducted by EBA indicates that a 1:3,000 year (1.66 percent chance in 50 years) seismic event will likely cause uncontrolled discharge from both reservoirs during or shortly after the seismic event. Assuming ten people die due to the resulting flood wave and inundation, this results in the potential for loss of life in the event of the design seismic event being in the Unacceptable Risk Zone suggested by the CDA as shown in Figure 17. Circumstances that exacerbate this situation are the likelihood that a smaller return seismic event could cause a similar failure and inundation or that the 2010 inundation study concludes that more than ten people could die. Based on this assessment, it becomes apparent that the City will be obliged to reduce the risk associated with the seismic hazards posed by the subject dams. This ALARP assessment needs to be reviewed by the City to ensure it meets with their expectations and tolerances for risk of loss of life.

11.0 CONCEPTUAL DESIGN OF RISK MANAGEMENT OPTIONS

11.1 GENERAL

The City's post seismic performance expectations, the budget for such work and the social and environmental value of the Colliery Dams Park will to a large part determine what measures are appropriate for addressing the seismic hazards posed by the existing dams. In general, there are three general options that the City has to address the seismic hazard risk posed by the subject dams:

- Option 1 - Eliminate the seismic hazards by removing the dams;
- Option 2 - Conduct seismic upgrades to the existing dams that bring the dams to a state where they safely impound their reservoirs during and shortly after the design seismic event but will need an engineering inspection immediately thereafter to assess the damage that has occurred, possibly followed by major maintenance or removal and, if necessary, evacuation of the potential inundation area; or
- Option 3 – Bring the impoundments into a state where not only do the dams safely impound the reservoirs during and after the design seismic event, but also require minimal maintenance after the design seismic event. This will require construction of new dams or extensive improvement of the fill in the existing dams with jet grouting or other in-situ treatment.

Evacuation measures are not viewed to be stand alone options to address the loss-of-life risks associated with the seismic hazards posed by the subject dams. Evacuation is discussed in Section 11.2.

EBA also considered the feasibility of the City purchasing residences at risk and relocating the school and daycare. However, this option was not pursued further as, the cost of relocation and school reconstruction aside, it would be prohibitively expensive to sterilize an significant parcel of land within City limits to address the risks associated with dam failure. This would be a more practical and feasible option if the degree of development was much lower and if the site was outside of the City limits. This option has not been developed further.

Major maintenance is defined as the repairs or reconstruction necessary for the dams to safely impound their reservoirs in the long term and meet all required design criteria, such as withstanding another design seismic event. Minimal maintenance is the repair work necessary to restore elements of the dam to operable condition that are not critical for the continued safe impoundment of the reservoirs.

The design seismic event for Options 2 and 3 may not be the 1:3,000 year event approved by BCMoE for the existing structures as per the 2003 Dam Safety Review report and the 2008 Interim Consequence Classification document presented in Appendix D. Depending on the findings of the flood inundation study, the consequence classification may change with a possible increase in the design seismic event for ***rehabilitation*** works to the 1:5,000 year event or greater. The design of any of the options discussed herein will require completion of the inundation study to be initiated by the City in 2010 and subsequent re-assessment of the consequence classifications.

More importantly, should the City be favourable to Option 2, for both dams the City should consider the impact of having three structures (Westwood, Middle and Lower Chase Dams) with toe berms designed to allow dams to survive a seismic event but sustaining enough damage where major maintenance or removal is necessary. Responding to the damages experienced by multiple structures requiring immediate attention could overtax the ability of the City to safely manage the aftermath of a major seismic event.

Each risk management option is discussed further in Sections 11.3, 11.4 and 11.5. Initial discussions with the City in early December 2009 indicated that significant investment in new dams or extensive and expensive in-situ treatment of the fill within the dams to maintain a public park may not be considered to be a wise use of tax payer funds.

It is important to note that the scope of work for this assignment did not include designing the various options, but to prepare conceptual sketches and cost estimates for the following:

- Engineering design required to bring the option selected by the City to a state where "Issued for Tender" drawings could be released to procure the services of a contractor;
- Approximate construction costs based on historical bid averages for similar work conducted for the City; and

- Approximate construction monitoring costs.

The concepts proposed in this report will need to be the subject of detailed analysis similar to what has been conducted as part of the seismic hazard assessment described herein. The degree of wall deformation that is modelled during the detailed design phase may necessitate additional measures or considerations that can not be predicted at this time. An additional consideration is the contractual arrangements under which any future rehabilitation design would be conducted. To date, EBA is satisfied that our approach to this assessment, which did not include drilling boreholes and conducting additional in-situ testing, was sufficient.

These costs are discussed further in Section 11.8.

11.2 USE OF EVACUATION TO MITIGATE RISKS

The proximity of the subject dams to a downstream urban area combined with the findings of this seismic hazard assessment means that relying solely on evacuation of the inundation zone will be insufficient to prevent loss of life. EBA recommends that the City commission the upcoming 2010 flood inundation study to consider cascade dam breach as described herein with the purpose of providing sufficient information for the City to decide if evacuation should be included as an Emergency Preparedness Plan/Emergency Response Plan action in the event of a large seismic event in the time between the present and the time when any of the aforementioned options are implemented. The upcoming flood inundation study should also be commissioned to refine the extent of inundation from a seismic event so that any evacuation notices, trials or other related actions are as focussed as reasonably practical. In the interim, with respect to completion of the flood inundation study, the City should consider what measures are necessary to address the potential inundation associated with seismically induced dam failure as described herein to address the potential for loss of life downstream of the dams.

EBA recognizes the roll-out of any short-term or long-term recommendation to include evacuation is a sensitive matter for the City to carefully consider in terms of their overall dam safety management systems and public communications protocols as well as impact on the general public.

11.3 OPTION 1 – DAM REMOVAL

Dam removal is an option for addressing the seismic hazards posed by the dams. From a long-term risk management perspective, removing the risk is the most appealing option. However, given that the dams are part of a popular public park and constitute fish habitat as well as support a sport fishery, any decision to remove the dams will need to consider the financial, social and environmental aspects of dam removal. The financial side of this assessment will have to consider the likely continued increase in design seismic loading as advances are made in understanding the potential magnitude of earthquakes near Vancouver Island. The triple bottom line (TBL), a popular sustainability model, can be used to illustrate the difference between a traditional financial bottom line and a balanced approach that seeks to find more appropriate solutions by including environmental and social aspects. Although this work is not part of EBA's scope of work for the seismic hazard assessment, conducting this assessment should be part of any detailed future assessments of the feasibility and cost of dam removal.

11.4 OPTION 2 – SEISMIC REHABILITATION OF DAMS (MAJOR MAINTENANCE OR DAM REMOVAL REQUIRED AFTER DESIGN SEISMIC EVENT)

In this option, the dams would be seismically rehabilitated to survive the design seismic event and safely impound the reservoir for a short period of time to allow the City to respond to the damage caused by the seismic event. However, the rehabilitation works conducted would be such that major maintenance or removal, with associated dewatering of the reservoir, may be required upon post-seismic event inspection. The Westwood Lake Dam seismic stabilization conducted in 2008 would fall into this class of rehabilitation in that the downstream toe berm was for stabilizing the dam until the repairs could be made to the dam or the dam removed given that the upstream slope of the dam was expected to fail. That repair is similar to the Option 2 repairs described herein.

The Option 2 type repairs will minimize, but not eliminate, deflection of the concrete walls during the design seismic event. Deflection and cracking of the concrete walls in either dam, most likely due to opening of cold joints or along weak zones due to honeycombing or poor quality concrete, will cause leakage through the concrete wall. The purpose of the Option 2 type repair is to provide the City with time to either draw down the impoundments or effectively evacuate the downstream inundation zone. The extent of post seismic event leakage will need to be assessed by an engineer and the decision made then to conduct additional repairs, remove the dams or implement other measures such as dewatering of the impoundments or evacuation of the inundation area.

Option 2 type repairs should include installation of performance monitoring instruments within the dams such as piezometers to record groundwater levels in the downstream shell before and after the seismic event and inclinometers, and/or fibre optics sensors either within or attached to the concrete walls to record their deflection and points of inflection, as well as seepage monitoring. These latter instruments may only sense the uppermost depth of damage but they would provide some insight into how the dam deformed during the design seismic event. Finally, survey monitoring points should be established at various points of the upstream and downstream crest to be initially surveyed upon installation and then again after any seismic event to provide an indication on lateral movement. A "straight line of sight" array of survey monitoring points should be used on the upstream and downstream crests.

In the case of Middle Chase Dam, Option 2 rehabilitation measures include:

- Excavation of the upstream rock fill berm and replacing it with a compacted rock fill buttress (this will require sequential excavation and backfilling techniques)(Figures 18 and 19); and
- From the upstream side, decommission the low level conduit where it passes through the concrete wall and cast a concrete bulkhead over the existing wall.

The relatively thin concrete wall at Middle Chase Dam combined with its more apparent deterioration is such that EBA does not believe installing steel rebar in holes drilled through the existing wall is as feasible as it is at Lower Chase Dam with its thicker wall and apparently more durable concrete. EBA considered installation of steel columns on the upstream face of the Middle Chase Dam wall to add additional stiffness but the poor durability and condition of the concrete wall combined with its relatively thin section did not appear to be compatible with this kind of solution. However, a new reinforced concrete wall constructed immediately upstream of the existing concrete wall could be used in conjunction with a new dense upstream rock fill berm. This new concrete wall would have to be constructed in sections to avoid instability of the existing concrete wall due to the full height of fill downstream of the concrete wall.

In the case of Lower Chase Dam, Option 2 rehabilitation would include constructing a rock fill buttress upstream of the concrete wall to minimize the deformations and associated cracking experienced by the wall during the design seismic event. Although it seems intuitive that the proposed upstream rock fill buttress should be compacted to a dense state throughout its full extent, the detailed design of this option should consider the imbalance of material stiffness on either side of the upper portion of the wall. The hammering action caused by the denser sand and gravel fill in the downstream shell of Middle Chase Dam could be replicated at Lower Chase Dam if a uniformly dense upstream rock fill berm was constructed. It may be necessary to include an undensified, compact zone of rock fill adjacent to the concrete wall to act as a dampening layer to absorb any imbalance of shaking energy created by a dense rock fill buttress on the upstream side of the wall.

Although significant deformations of the downstream shell of the Lower Chase Dam have been modelled, these deformations are mostly within the loose, end-dumped 1918 fill and do not appear to impact the seismic response of the wall. The small deformations of the concrete wall in Lower Chase Dam lead to the initial conclusion that a downstream buttress is not required. However, the presence of a school, day care and residences downstream of the subject dams is an important consideration. Without any rehabilitative work on the downstream side of the dam, Lower Chase Dam could be viewed to be a latent threat of failure (i.e., Failure Mode #4 discussed in Section 6.1). Additionally, the piping risk assessment discussed in Sections 9.2.3 and 10.3 indicates that the risk of piping failure developing after a significant seismic event would be in the Unacceptable Risk Zone as shown in Figure 17. Therefore, a downstream berm should be included in Option 2 type repairs to Lower Chase Dam (Figures 20 and 21).

The final design of any Option 2 rehabilitation works for the Lower Chase Dam should include an assessment of the following:

- Impact of a downstream berm used in conjunction with an upstream berm on the performance of the concrete wall;
- Impact of settlement of the loose, 1918 fill upon loading with a downstream toe berm should be considered with regards to maintaining the continuity of the toe filter placed in 1980; and

- Limiting the lateral spreading capacity of any alluvial sediments under the downstream toe of the dam through a rock filled key trench excavated to bedrock near the downstream toe of the buttress, similar to what was constructed at Westwood Lake Dam.

Additional considerations relating the potential for modifying the downstream slope of Lower Chase Dam to improve flood discharge capacity are discussed in Section 11.6.

EBA had initially considered drilling boreholes through the 1.2 m wide section of the concrete wall at Lower Chase Dam and grouting reinforcing steel into the holes to ensure the wall remains as one structural unit, though most likely severely cracked and leaking after the design seismic event. However, the relatively small deformations modelled, as discussed in Section 9.3, are such that this measure does not appear to be required for an Option 2 type repair.

All options relating to construction on the upstream side of either dam will require dewatering of the reservoir(s).

It is important to recognize that future changes to the design seismic events will likely result in increased ground shaking severity associated with the design seismic event. Therefore, as a function of future increases in knowledge of seismicity in the Vancouver Island area, the design seismic loading may increase in the future, requiring additional future assessments and upgrading work. Additionally, as public risk tolerance decreases, future seismic design codes or guidelines may result in a higher return period design earthquake being adopted, similar to the increased conservatism inherent in the 2007 Canadian Dam Association Guidelines.

11.5 OPTION 3 – REPLACEMENT OF DAMS (MINIMAL MAINTENANCE REQUIRED AFTER DESIGN SEISMIC EVENT)

In this option the City would select a new concrete dam structure to act as the primary water retention element. The majority, if not all, of the existing dams would be left in place to preserve existing aesthetics and to minimize costs associated with removing them. EBA had initially considered the option of conducting significant in-situ improvement of the dams such as jet grouting, foam injection but will not carry these options forward for the following reasons:

- The City has advised EBA that the state of the dams and expected budget for rehabilitation of dams to maintain the current condition of a park and upgrade safety of the permanent and temporary inhabitants of the inundation area will be such that high cost options, while innovative, would not be a wise use of the tax payers money given the other priorities the City has within their dam safety management program; and
- Rehabilitation of the subject dams with in-situ treatment such as jet grouting, Uretek foam injection or other means could improve their performance during the design seismic event. However, it is EBA's opinion that it will not remove the potential that

the treatment will not improve the dams sufficiently to provide reliability in that it will provide a minimal maintenance solution.

In the case of Middle Chase Dam, the most feasible and reliable Option 3 repair consists of removing the upstream rock fill buttress and constructing a concrete gravity dam upstream of the existing concrete wall. The low level conduit would be decommissioned from the upstream side of the dam similar to what was proposed for an Option 2 type repair.

In the case of Lower Chase Dam, the most feasible and reliable Option 3 repair consists of constructing a new concrete gravity dam downstream of the existing dam. The void between the two structures could be backfilled with rock fill if a cushioning layer of finer material was placed against the concrete structure to protect it from damage.

Post tensioned anchors are an option to reduce the mass of the new concrete gravity dam section. However, anchor service life and the potential for corrosion will necessitate a monitoring program and possibly retrofits in the future. Considering the cost of the City's monitoring, maintenance and future retrofits, it may be more economical in the long term to use a larger concrete section and develop resistance due to mass as opposed to tension in anchors.

Concrete gravity dams are not immune to damage or failure due to seismic loading, but they are among the most stable dam structures when constructed on a competent foundation. Future increases in the severity of the design seismic event would not have as much of an impact on a properly designed and constructed concrete gravity dam as they would on the aforementioned Option 2 repairs.

11.6 OPTIONS FOR IMPROVING FLOOD DISCHARGE CAPACITY

The 2003 Dam Safety Review conducted on the subject dams recommended that the spillways be enlarged to accommodate a 1:3,000 flood event (Golder, 2004a, 2004b). The estimate flood flow rate associated with a 1:3,000 flood event was provided in the 2003 Dam Safety Review reports to be 85 m^3/s. A flood inundation study using the PMF was conducted in 2002 which used the Probable Maximum Flood (PMF) as flood discharge plus flood waters released by breach of all dams on the Chase River system. One of the findings of the 2003 Dam Safety Review was that the spillways need to be enlarged to pass the 1:3,000 year flows, not the PMF which was judged to be unnecessarily conservative for a design flood in 2003 (Golder, 2004a, 2004b).

The content of the BCMoE Interim Consequence Classification document (BCMoE, 2008) applies in the case of the flood event to be used in Dam Safety Assessment and in design of rehabilitation measures. The design flood event used for design of spillway improvements will depend on the results of the flood inundation assessment and confirmation of consequence classification. A higher consequence classification than the interim one presented herein would result in a higher magnitude of design flood event.

Discussions with the City during execution of the seismic hazard assessment study described herein indicated that the City was looking for an opportunity to find a solution that addressed some or all of the seismic hazard risks as well as some or all of the flooding risks. If this option was considered, a concrete gravity dam with integral overflow spillways could be considered to address flood discharge capacity concerns with the existing spillways.

Depending on the results of the 2010 flood inundation study and subsequent re-assessment of consequence classification, the City may have the following options:

- Spillway Option A - Construction of a heavy rock rip rap, armoured channel over the crest and downstream slope of each dam; and
- Spillway Option B - Construction of a concrete spillway over the crest and downstream slope of each dam.

In both cases, it is recommended that the additional spillway capacity be only used for high return period flood events (e.g., 1:500 or larger). Each option is described briefly in the following paragraphs.

Spillway Option A will require construction of rock fill buttress on the downstream slope of the subject dams to permit placement of a heavy rock rip rap armoured channel on the downstream slope of each dam. The 1995 CDA Guidelines (CDA, 1995) provided preliminary guidance on the design slope of a flow over rock fill dam. Based on the 1995 CDA Guidelines, a maximum slope of 5H:1V would need to be provided. Given the magnitude of flows and the height of the subject dams, the spillway slope may need to be flatter, and incorporate very large rip rap as well as some form of steel mesh designed to minimize displacement of rip rap particles by tractive forces associated with overflow, a sacrificial layer or some combination of all of these measures. The crest of each dam would have to be modified with armouring and a lowered crest to confine overtopping flow and direct it into the armoured channel. Erosion of the natural channel downstream of the dam would have to be assessed for the channel reach below Lower Chase Dam. In the case of Lower Chase Dam, the rock fill buttress would have to be designed to stabilize the downstream slope to minimize seismic damage that impairs the serviceability of the overflow spillway. This may result in the toe of the rock fill berm and channel reaching the bend in the Chase River channel that occurs where the Harewood Creek joins the Chase River Valley. This may prove to be a hydraulic control section which may require physical modelling in the design process.

Spillway Option B would require construction of a concrete spillway channel over the crest and downstream slopes of the subject dams. Middle Chase Dam, with its compacted downstream shell would offer the best spillway foundation but would still experience some degree of deformation. Typical earthworks compaction standards are usually less than those used for structural fill supporting a rigid structure like a concrete spillway channel. In the case of Lower Chase Dam, the loose nature of the 1918 fill and underlying original rock fill is such that significant settlements would occur causing extensive cracking and deflection of the concrete spillway channel. The downstream shell would have to be improved with in-situ treatment or require excavation and replacement with compacted materials to provide an acceptable base for a concrete spillway channel. Extensive concrete reinforcement may permit a structural slab approach to be used if a maintenance program was considered to be acceptable by the City to address cracking due to settlements.

11.7 ADDITIONAL DESIGN WORK REQUIRED

The Option 2 and 3 rehabilitative measures will require detailed design effort to be expended to produce the engineering report, design drawings and specifications required to procure the services of a contractor through a competitive bid process.

The modelling conducted to date as part of the seismic hazard assessment has focused only on the current condition of the dams. The seismic response of the modified dam geometry for the Option 2 measures (i.e. upstream toe berms) will need to be modelled using analysis methods similar to what has been conducted as part of this assessment. Note that this modelling was outside the scope of this study. It is possible, but not probable that the addition of upstream rock fill berms will cause the dams and, in particular, the concrete walls to behave adversely in the computer model created to support the detailed design process. However, based on EBA's judgement and experience, this is judged to be unlikely.

In the case of Option 3, the selection of a concrete gravity dam will require a subsurface drilling program to assess the properties and permeability of the bedrock foundation. This is due to the much higher stresses that are applied to the foundation and the much shorter seepage path associated with a concrete gravity dam. With regard to an extensive fill improvement program, any specialty contractor involved in such works would likely require additional boreholes drilled through the body of the dam with potential for specialized in-situ testing upon which to base their tendered cost estimates. However, it is understood the probability of Option 3 being selected by the City is remote.

11.8 ROUGH ORDER OF MAGNITUDE COST ESTIMATION FOR VARIOUS OPTIONS

Part of the scope of work for the seismic hazard assessment presented herein was to provide rough, order of magnitude cost estimates for the various options described herein for addressing the seismic hazards posed by the subject dams. It is understood the City will need a sense for what the general cost will be to implement Options 1, 2 or 3 for each of the subject dams.

Conceptual level plans and sections have been discussed in the previous sections and presented in the figures; however, the detailed design work necessary to prepare design quantities and specifications upon which an engineering cost estimate could be based has not been completed. As such, the cost estimates presented herein are rough order of magnitude. The cost associated with each option has been presented to the nearest $0.5 million. Without any design work it is not possible to assign a confidence level to these costs (e.g., +/- 20%, 50% or higher). Completion of an engineering study into each option would permit assignment of such confidence levels. The costs as presented should be used comparatively, recognizing that future study and refinement may change how the costs associated with the various options compare to one another.

Discussions with the City have indicated that constructing new dams as part of Option 3 is not viewed to be a wise use of the tax payer's money. New concrete gravity dams to replace the Middle and Lower Chase Dams with integral overflow spillways to aid in addressing the flood discharge capacity of the spillways will cost in the order of $10 million, or more, combined. This cost could be refined with some preliminary design work. However, for the purposes of this report, Option 3 will not be considered further.

EBA has reviewed the Westwood Lake Dam rehabilitation tenders submitted to the City in 2008 to get a sense for what the unit costs were for the tasks necessary to do that work. That project has some relevance to the berms that could be implemented as part of Option 2, but less so for the Option 1 – Dam Removal concept. The scale of berming at Middle Chase Dam and on the upstream side of Lower Chase Dam is within the general magnitude of the Westwood Lake Dam work, but, should a berm be required on the downstream side of Lower Chase Dam (depending on the results of the 2010 inundation study), the scale of that work would exceed that of the Westwood Lake Dam work which introduces a level of reliability into using these costs.

In review of the Westwood Lake Dam costs, EBA noted the following bid items had relevance:

- Mob/demob;
- Public access management;
- Survey and layout of the proposed works, including quantity surveys;
- Unit costs for fill materials, clearing/grubbing, excavation, low level conduit removal; and
- New v-notch weir structure.

Although there was a dewatering cost, the extent of dewatering was not defined in the bid items. As such, it was judged that the maximum cost bid for the Westwood Lake Dam works would be insufficient for dewatering one of the reservoirs and maintaining that water level throughout construction. The cost of dewatering would apply to each impoundment. Additionally, the extent of dewatering and discharge of pumped water could also become environmentally sensitive depending on the turbidity of the discharged water. Depending on the nature of work conducted (Option 1 versus Option 2), management of public interaction could become a significant undertaking with corresponding increase of costs well beyond what was incurred at Westwood Lake Dam.

It is important to recognize that the following costs are not included in the rough, order of magnitude costs presented herein:

- Public consultation associated with reservoir drawdown and/or dam removal. It is expected that dam removal, or even prolonged drawdown of the reservoir associated with upstream works associated with seismic rehabilitation, could be a contentious issue;

- Environmental baseline studies to establish what environmental measures are necessary for temporary dewatering or full removal. This should include internal and disbursement costs of environmental studies, meetings with DFO and any other regulators having or claiming jurisdiction, legal matters (if necessary) and other interactions with regulators, engineers and the public to get the approvals necessary to temporarily lower the reservoirs or permanently dewater them in a dam removal scenario;
- In the case of dam removal, any costs associated with any regulator mandated fish habitat compensation or rehabilitation of pre-impoundment habitat, any third party costs or claims associated with dam removal such as reduced property value, whether justified or not, along with any legal and internal costs associated with them and the costs associated with re-establishing public access across the river channel to restore the functionality of the park, if preserved after dam removal;
- Cost premiums associated with working in or adjacent to the lowered impoundments that was not as pronounced with the Westwood Lake work; and
- Costs of any environmentally related work stoppages, work scope revisions and other delays which appear to be more likely than what was possible during the Westwood Lake Dam works.

The environmental and public interaction/communication/consultation costs could well approach or surpass the engineering costs as dam removal is an environmentally, and in this case, socially, contentious issue. The cost of City involvement is also unknown at this time and similarly has not been included in the costs presented in this section.

The rough, order of magnitude costs presented herein should not be used for establishing budgets but rather only to gain insight into the relative magnitude of cost associated with each option, subject to the limitations discussed herein. The following rough, order of magnitude engineering and construction costs, with all aforementioned limitations, caveats and exclusions, have been estimated:

<u>Option 1 – Dam Removal</u>

- Middle Chase Dam (approximately 4,700 m^3, including concrete) - $0.5 million, not including dewatering, environmental or public interaction costs; and
- Lower Chase Dam (approximately 18,800 m^3, including concrete) - $1.5 million, not including dewatering, environmental or public interaction costs.

Middle Chase Dam is about ¼ of the volume of Lower Chase Dam. Although the cost of removal appears to be 1/3 of that required for Lower Chase Dam, this reflects the rounding to the nearest $0.5 million. The precise, but inaccurate (as discussed), unrounded figures calculated by EBA indicate that the cost of removal of Middle Chase Dam is approximately ¼ of that of Lower Chase Dam which proportionally makes sense. An important consideration is that the unaccounted aforementioned environmental, dewatering and public interaction costs will likely be the same for each dam.

Option 2 – Seismic Rehabilitation (Major Maintenance of Dam Removal Required After Design Seismic Event)

- Middle Chase Dam (upstream berm, 2,800 m^3) - \$0.5 million, not including dewatering, environmental or public interaction costs; and
- Lower Chase Dam (upstream berm, 1,100 m^3) - \$0.5 million, not including dewatering, environmental or public interaction costs.

The rounding to the nearest \$0.5 million obscures the difference between the Middle and Lower Chase Dams berms. The costs calculated by EBA indicate that the cost of the upstream Lower Chase Dam upstream berm will be less than the Middle Chase Dam upstream berm, but not proportionally so due to the increased difficulties anticipated with construction access at Lower Chase Dam. The unaccounted for dewatering, environmental or public interaction costs associated with both berms is anticipated to be approximately the same.

The need for a downstream berm will depend on the results of the 2010 flood inundation study. Due to the valley geometry and height of the dam as measured from its downstream toe), the volume of the conceptual downstream berm will be 25,000 m^3, a greater volume than the existing dam. It should be recognized that detailed design will be needed to refine this conceptual design, but the volume of a downstream berm, if required, will still end up being quite high compared to the volume of the existing Lower Chase Dam.

The cost of the downstream berm at Lower Chase Dam, if required, will be in the order of \$1.5 million, not including environmental and public consultation costs.

The total cost of berming the Lower Chase Dam does not appear to justify constructing a new concrete dam; however, it is viewed as justification for installing steel reinforcement in boreholes drilled through the concrete wall to limit wall deflection during the design seismic event and afterwards due to increased seepage and possible post seismic instability of the deformed downstream shell. The upstream berm may still be required, but additional design would be needed to verify this. The rough, order of magnitude cost of installing steel reinforcement in the wall is expected to be in the order of \$1.0 million, depending on the diameter, length and spacing of the bars and difficulty in drilling, insertion of bars and grouting. Corrosion considerations could also increase the cost of this option through the potential need for corrosion protection. This option is one that would require dewatering of the reservoir due to the unknown condition of the concrete wall and anticipated zones of honeycombing, cold joints and zones of poor durability concrete.

12.0 CONCLUSIONS

The following conclusions can be drawn based on the findings of this study:

Background Information

- The dams were constructed by Western Fuel Corporation around 1911;

- The subjects dams appear to have constructed using a combination of poor concrete construction practices (by today's standards) and either end dumping or labour intensive fill placement. The fill materials placed during initial construction and in 1918 are relatively deformable compared to modern dam fills due to the expected lack of fill compaction by any recognizable engineering standard;
- The concrete walls in both dams should be expected to be of poor quality without steel reinforcement. Construction cold joints, honeycombing and zones of poor durability concrete should be expected to be prevalent from the top to the bottom of the original walls in both dams;
- The foundation beneath the Middle Chase Dam fill is bedrock whereas the foundation of Lower Chase Dam appears to be either till or bedrock;
- The original rock fill in both dams is generally in a compact state but material variability associated with excavation from the Harewood Mine and subsequent placement has likely caused extensive segregation with varying rock fill density;
- The rock fill is most likely sedimentary rock associated with excavation of adits in the Harewood Mine;
- The cinders, slag and ash placed in the Lower Chase Dam were most likely an end dumped sliver fill placed as part of the Wakesiah Mine railway crossing constructed over the Lower Chase Dam in 1918. This fill is generally loose;
- The 1980 fill placed in the downstream shell of the Middle Chase Dam and in the filter zone at Lower Chase Dam was compacted during placement and is most likely in a dense state;
- Although the Reservoir #1 Dam and Upper Chase Dam have been assessed in previous seismic hazard assessments, the increase in the understanding of magnitude of possible seismic events has likely increased the seismic loading on these structures for their design seismic events. The seismic hazard assessment for Upper Chase Dam is considered to be adequate given its small size provided the water line issues identified in 2004 have been addressed; and
- Failure of Harewood Dam appears to contribute to the inundation area associated with the subject dams.

<u>Field Work</u>

- The results of the reinforcing steel survey are that it can be concluded that steel reinforcement is not present in the Middle Chase Dam and similarly not in the Lower Chase Dam;
- The compressive strength of the exposed concrete wall above reservoir level in both dams is quite low compared to current standards for similar structures;
- The concrete used in the concrete walls has low strength and poor durability, especially in Middle Chase Dam;

- A zone of original rock fill was left in place in Middle Chase Dam which has been shown to be the likely location of the low level conduit located by diver inspection as part of this study;
- The seepage observed over the years from the downstream toe of the Middle Chase Dam is most likely from the original low level conduit which appears to still be in place;
- At Lower Chase Dam, bedrock is the foundation material for the majority of the abutments with a thin layer of colluvium of the lower quarter of the valley walls and alluvial soils at the base of the valley. The lack of cracking the concrete wall and low seepage rates suggests that the concrete wall is founded on bedrock;
- The downstream slope of the dam has had historical shallow slope inability indicated by the presence of shallow slope deformations at the crest of the dam. This could be related to run off from rainfall events;
- Historical concrete structures present elsewhere in Collier Dam Park indicate poor concrete construction practices in the construction of bridge piers;
- The low seepage rates at both dams combined with the relatively free draining nature of the downstream shells of both dams permits the conclusion that the water levels in the downstream shell of each dam are low; and
- Liquefaction of the fill material in both dams does not appear to be a concern due to the expected low water tables within the downstream shell of each dam. Some saturation of a thin zone of 1918, fine grained fill in the Lower Chase Dam may be possible but has been judged to be of minor influence. Additionally, the presence of saturated, fine grained alluvium at the downstream toe of Lower Chase Dam may be the source of some liquefaction mobility, but the extent of this material beneath the downstream shell is expected to be minor due to the steepness of the channel beneath the dam inferred from the topographic and bathymetric survey data.

<u>Consequence Classification</u>

- Until the 2010 inundation assessment is complete, it is considered reasonable to assume that 10 people could die as a result of dam failure during or after the design seismic event;
- Both dams are classified as being on the border between a BCMoE High-Low and High-High (2007 CDA Guidelines High and Very High) consequence classification on the basis of the potential for loss of life downstream of the dams;
- As this seismic hazard assessment is being conducted on the current state of the dams, the 1:3,000 seismic event recommended in the 2003 Dam Safety Review will be accepted by BCMoE; and
- Rehabilitation of the subject dams to address seismic or flood hazards may require reclassification of the dams based on the results of the 2010 inundation study. This

could result in higher return flood and seismic periods being used for design instead the design events recommended in the 2003 Dam Safety Review.

Analysis

- Both dams, under current conditions (e.g., static), appear to be within the ALARP zone suggested by the CDA;
- An increase in the rate or turbidity of seepage at Middle Chase Dam due to continued deterioration of the low level conduit will result in the risk of piping approaching or entering the Unacceptable Risk Zone as suggested by CDA;
- The occurrence of a seismic event, even one smaller than the design seismic event will bring the risk of piping failure at both dams into the Unacceptable Risk Zone suggested by the CDA;
- EBA can not decide for the City or their stakeholders what an acceptable risk of loss of life is;
- The modelling indicates that design seismic event will likely cause the concrete wall at Middle Chase Dam to topple into the reservoir early in the design seismic event. The depth of cracking will be well below the normal reservoir operating level;
- Middle Chase Dam will start to develop an uncontrolled discharge either during or shortly after the design seismic event. Due to the presence of sand and gravel in the downstream shell, the breach will erode towards the valley bottom quickly and the floodwater could cause an overtopping failure of Lower Chase Dam;
- The modelling indicates that Lower Chase Dam will experience a much smaller degree of wall deflection with cracking occurring later in the modelled design seismic event. Increased seepage into the downstream shell will occur due to cracking. The downstream shell, in particular the 1918 fill, will be displaced significantly downstream, disrupting the filter zone constructed in the early 1980s. The free draining nature of the downstream shell will eventually permit drainage of the increased seepage. However, overtopping due to failure of Middle Chase Dam could cause uncontrolled discharge through Lower Chase Dam;
- The rate of failure at Middle Chase Dam is unknown. The potential for uncontrolled discharge from Middle Chase Dam causing an overtopping failure is unknown but is considered to be likely; and
- The presence of unconsolidated alluvial sediments at the base of the Chase River Valley below the downstream toe of Lower Chase Dam could result in the toe deformations being greater than what is predicted by the modelling conducted by EBA. However, the three-dimensional effects associated with the valley shape and rock fill valley wall contact will impart additional stability to the downstream shell that is not considered in our analysis.

Risk Management Options

- Due to the expected failure of Middle Chase Dam during or shortly after the design seismic event and possible overtopping failure and uncontrolled discharge from Lower Chase Dam, it is EBA's opinion that a post- seismic event evacuation immediately after the design seismic event will be ineffective as a risk management method on its own; and
- Evacuation notices will be a useful part any Emergency Preparedness/Response Plan between the present time and when seismic hazard risk reduction measures are implemented.

13.0 RECOMMENDATIONS

The following recommendations can be made based on the findings of this study and conclusions presented in Section 12.0.

Background Information

- Any rehabilitative solution selected by the City should have minimal reliance on the structural integrity of the concrete walls due to their anticipated poor condition;
- Any rehabilitative solution selected by the City should consider the inherent variability of the original rock fill placed within the dams. This variability is expected to be such that it will detrimentally affect any in-situ treatment measures in that they will produce a highly heterogenous material that may not be adequate for seismic rehabilitation;
- The 1980 fill placed in the downstream shell of the Middle Chase Dam and in the filter zone at Lower Chase Dam does not need retrofitting to improve it's performance during the design seismic event;
- The seismic retrofit conducted on Reservoir #1 Dam should be reviewed to ensure that it provides adequate stability during the design seismic event appropriate for this structure; and
- The seismic stability of Harewood Dam should be reviewed for a design seismic event appropriate for its consequence classification.

Field Work

- The concrete in the original walls, especially at Middle Chase Dam is sufficiently weak and appears to be of sufficiently poor construction quality that it should not be relied upon for any rehabilitative works; and
- The low level conduit passing through Middle Chase Dam should be decommissioned as soon as possible.

Consequence Classification

- The 2010 inundation assessment should include the following tasks:

1. Assess consequences of failure associated with loss of the Chase River reservoirs at normal operating levels due to a sunny day failure event and during the design flood event (not the PMF). Reservoir loss should assess the consequences of cascade type failures involving Lower and Middle Chase Dam as well as combinations of failure of Reservoir #1, Upper Chase Dam and Harewood Dam.
2. Assess the time required to breach the dams and cause an uncontrolled discharge of the reservoir for Middle and Lower Chase Dams.
3. Provide insight to the extent of inundation as well as peak water depths and velocities near the school, daycare, various residential areas, as well as the Bruce and Howard Avenues, E&N railway and the Trans Canada Highway crossing of the Chase River.
4. Provide an estimate of the number of lives lost associated with the various combinations and permutations of dam failure under sunny day failure conditions. The number of lives potentially lost in the school and daycare area should be reported.
5. Revise the consequence classification which will indirectly establish the design seismic and flood events for any rehabilitation measures. Additionally, consideration will have to be given to societal tolerances for loss of life in the event of a dam failure, especially given the proximity of a school, daycare and residences within what appears to be the inundation area(s).

Analysis

- Decommission the Middle Chase Dam low level conduit as soon as possible or depending on the schedule for future works, as part of Option 2 rehabilitation works;
- The City should review the ALARP assessment presented herein and confirm what their tolerance for loss of life is and advise EBA if the ALARP limits presented in Figure 17 need to be revised or not;
- The City should review the EPP/ERP for the Chase Dams in light of the findings of this assessment and report and, if necessary, prepare a subsection of the EPP/ERP that deals with the failure mechanisms described herein for the dams in their current condition until completion of the 2010 inundation study and, later, upon implementation of the chosen measure(s) of seismic hazard risk reduction. Updates to the City EPP/ERP documents for the subject dams may include post seismic event evacuation measures in the short term or long term. This would apply to any large earthquake event where noticeable ground motions have occurred and deflection/damage to infrastructure has occurred. The observations needed to trigger the EPP/ERP will need to be developed further to minimize the occurrence of false alarms;

- Any seismic hazard risk reduction measures selected by the City that involve maintaining the dams in their current locations through modifications (i.e. Option 2 repairs) should focus on limiting deformations of the concrete walls as much as practical combined with post seismic event measures such as engineering inspection, reservoir lowering, major maintenance or removal and, if necessary, evacuation measures;
- The 2010 inundation study should assess the rate of failure at Middle Chase Dam and assess if it could cause an overtopping and uncontrolled discharge from Lower Chase Dam. The inundation area from failure of Lower and Middle Chase Dams should be quantified for the purposes of evacuation plans for areas downstream of the subject dams; and
- The extent of inundation associated with failure of one or both of the subject dams, with consideration given to the likelihood of failure from other dams on the Chase River system should be considered when deciding if a downstream toe berm is required as part of any selected Option 2 repairs.

<u>Risk Management Options</u>

- The City should review the EPP/ERP for the Middle and Lower Chase Dams in light of the findings of this assessment and report and, if necessary, prepare a subsection of the EPP/ERP that deals with the failure mechanisms described herein for the dams in their current condition until completion of the 2010 inundation study and, later, the chosen measure(s) of seismic hazard risk reduction. This may include post seismic event evacuation measures in the short term or long term. This would apply to any large earthquake event where noticeable ground motions have occurred and deflection/damage to infrastructure has occurred. The observations needed to trigger the EPP/ERP will need to be developed further to minimize the occurrence of false alarms;
- The City should install remotely operate seepage monitoring instrumentation downstream of both Middle Chase Dam and Lower Chase Dam. The historical data should be reviewed to permit establishment of alarm levels that trigger inspection or other actions by City staff;
- In EBA's opinion, based on the results of the study presented herein, the City should consider three options for seismic hazard risk reduction at Middle and Lower Chase Dams:
    1. Dam removal;
    2. Seismic Rehabilitation (Major Maintenance of Removal Required After the Design Seismic Event); and
    3. Replacement of dams (Minimal Maintenance Required After Design Seismic Event).
- From a risk management perspective and upon consideration of the presence of a school, residences and a daycare within the inundation zone, the most practical and socially palatable option for addressing the seismic hazard risks posed by the subject

dams is Option 1 – Dam Removal. Depending on the influence of other social and environmental factors and the risk tolerance of stakeholders (e.g. affected residents, school board, general public), the City may wish to accept some future risk and select the Option #2;

- If Option #2 seismic rehabilitation works is selected, consideration should be given to the following:
  1. The potential that the dams may need major maintenance or removal after the design seismic event and the City's resources and abilities may be allocated to address damages to three structures (Westwood, Middle and Lower Chase Dams) that have been designed to survive the design seismic event with subsequent major maintenance;
  2. Implementing repairs at Middle Chase Dam first consisting of reconstruction of the upstream rock fill buttress and possibly construction of a new concrete wall at Middle Chase Dam, in addition to decommissioning of the low level conduit; and
  3. Construction of an upstream and downstream rock fill buttress at Lower Chase Dam or installing reinforcing steel in boreholes drilled through the concrete wall, evaluation of a cushion zone upstream of the concrete wall to limit loading on the wall, consideration of existing filter integrity in response to settlement of the 1918 fill under loading from the downstream toe berm and inclusion of a rock fill key trench excavated to bedrock beneath the downstream buttress.
- Finally, and most importantly, the 2010 flood inundation study should be completed to address the various unknowns with regard to the extent and variability in the severity of flooding throughout the inundated area so that the consequences of seismically induced or other "sunny day" uncontrolled discharges associated with the dams in their current condition can be better quantified; and
- The results of the 2010 flood inundation study should be incorporated into the design of rehabilitation works associated with Options 2 and 3 and into any interim EPP/ERP sections addressing "sunny day" failure mechanisms.

14.0 OWNERSHIP OF REPORT

As per the conditions included in the Request for Proposal, this report is the property of the City of Nanaimo.

15.0 LIMITATIONS OF REPORT

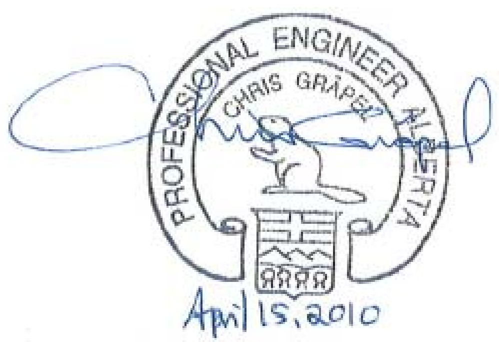
This report and its contents are intended for the sole use of The City of Nanaimo and their agents. EBA does not accept any responsibility for the accuracy of any of the data, the analysis or the recommendations contained or referenced in the report when the report is used or relied upon by any Party other than The City of Nanaimo, or for any purpose other than the subject site. Any such unauthorized use of this report is at the sole risk of the user.

16.0 CLOSURE

We trust this report meets your present requirements. Should you have any questions or comments, please contact the undersigned at your convenience.

Respectfully Submitted;

EBA Engineering Consultants Ltd.



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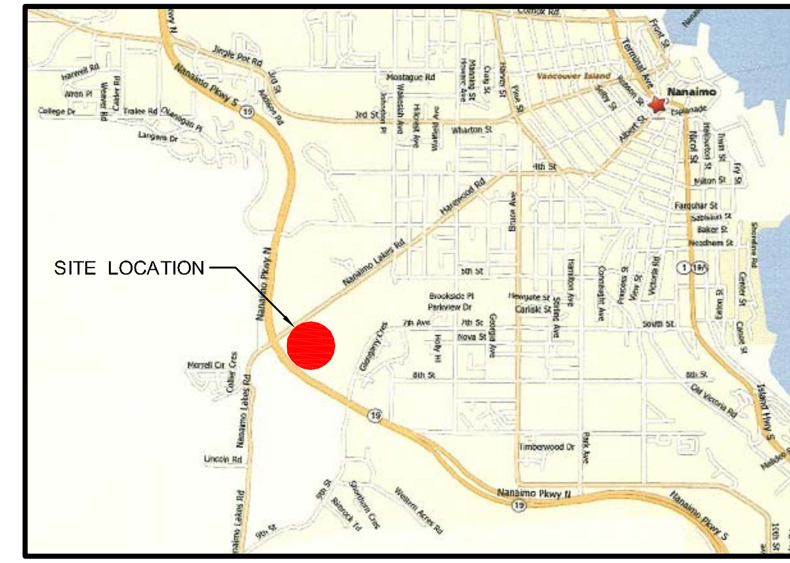
FIGURES



KEY PLAN

REFERENCES:

Key Plan Image: Map Quest

Base Plan Image: Google, 2009 Tele Atlas

0 100

Scale: 1: 2 500 (metres)

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THE HARBOUR CITY

EBA Engineering Consultants Ltd.

Seismic Hazard Assessment
Middle and Lower Chase River Dams

Site Location Plan

PROJECT NO.	DWN	CKD	REV
N13101249	BR	CG	0
OFFICE	DATE		
EDM	April 2010		

Figure 1

ISSUED FOR USE

0 50

Scale: 1: 1000 (metres)

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Seismic Hazard Assessment
Middle and Lower Chase River Dams

Aerial View of Middle Chase River Dam

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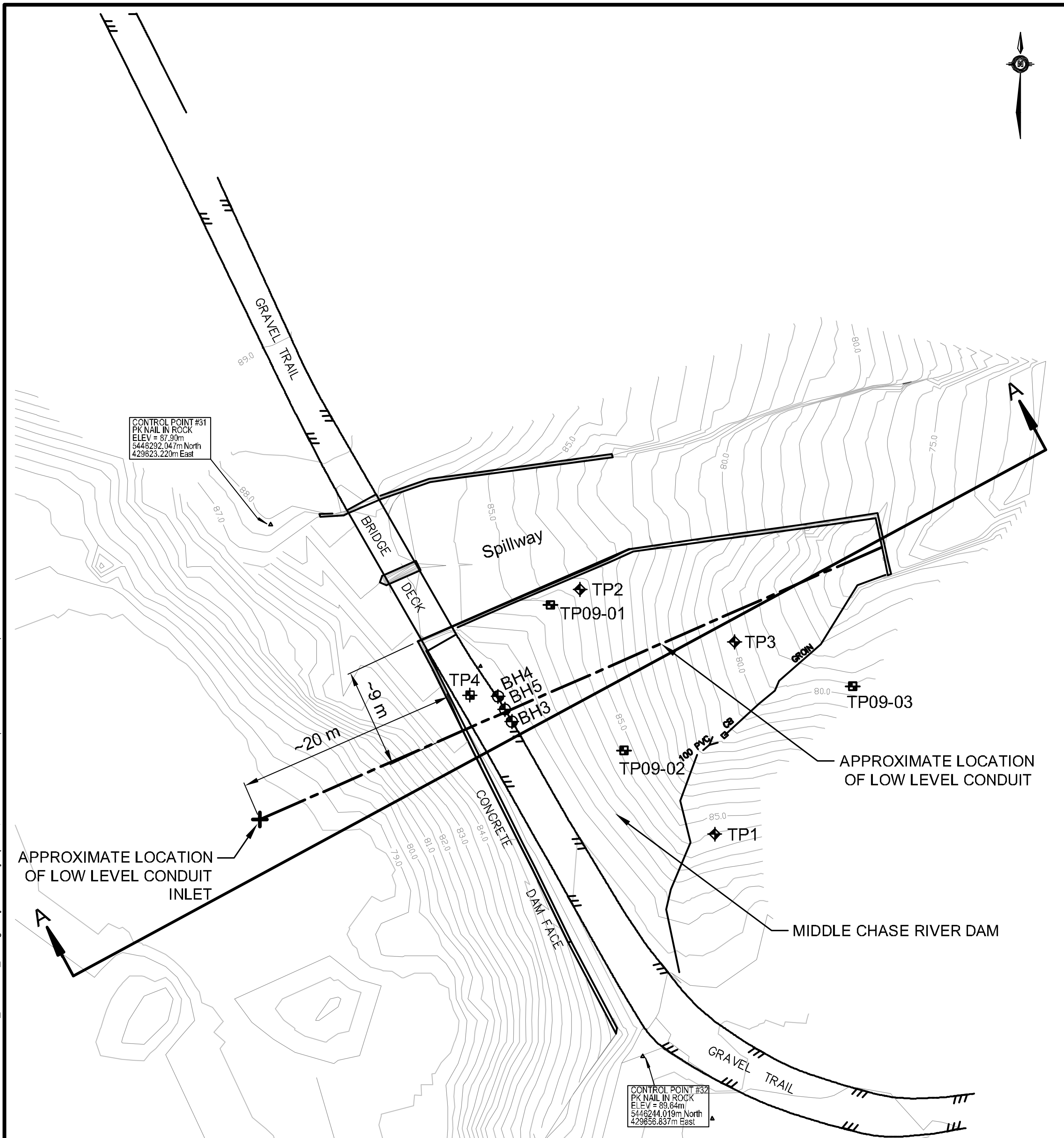
Figure 2



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LEGEND:

- 1978 BOREHOLE
- 1978 TEST PIT DUG BY HAND
- 1978 TEST PIT DUG BY BACKHOE
- TP09-## - 2009 TEST PIT

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Scale: 1: 500 (metres)

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Seismic Hazard Assessment
Middle and Lower Chase River Dams

Topographic / Bathymetric Data and Borehole Locations for Middle Chase Dam

PROJECT NO.	DWN	CKD	REV
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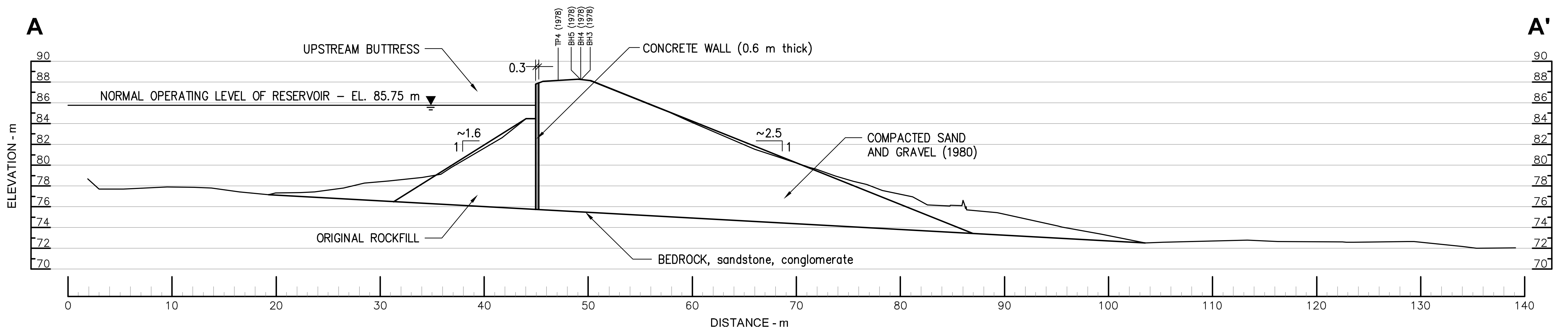
EDM	April 2010

Figure 3

Q:\Edmonton\Drafting\CIVIL3D\Other Offices\N131\N13101249 - Lower and Middle Chase Dams\N13101249_FIGURE 02_R0.dwg [FIGURE 3] April 14, 2010 - 10:21:36 am (BY: BOB RICHMOND)



NOTE: Normal operating level of reservoir based on survey data of spillway

0 20

Scale: 1: 400 (metres)

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Seismic Hazard Assessment
Middle and Lower Chase River Dams

Middle Dam
Cross-Section A-A'

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Figure 4

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0 50
Scale: 1: 1000 (metres)

Seismic Hazard Assessment
Middle and Lower Chase River Dams

Aerial View of Lower Chase River Dam

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N13101249	BR	CG	0
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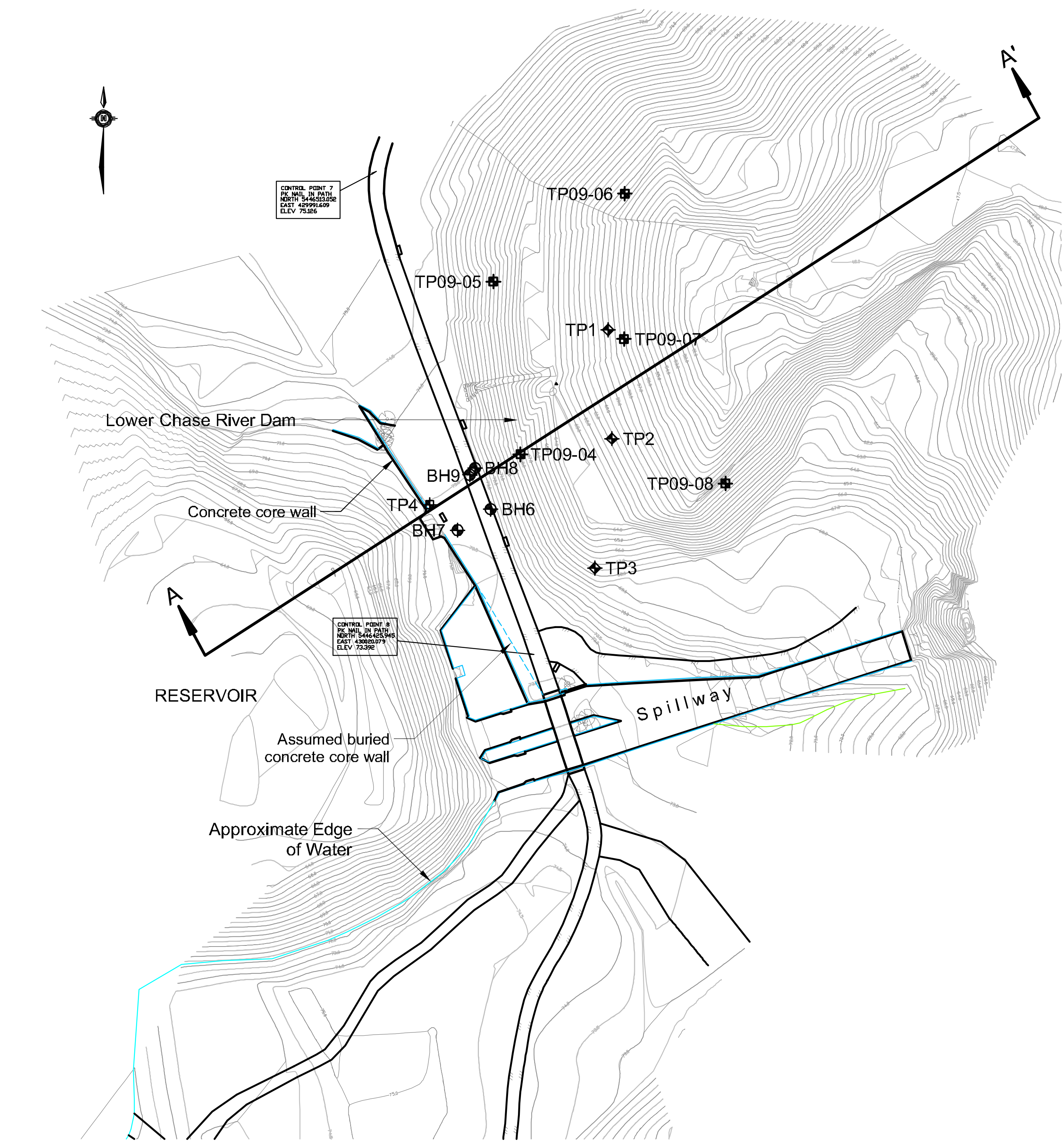
EDM	April 2010		

Figure 5

Q:\Edmonton\Drafting\CIVIL3D\Other Offices\N131\N13101249 - Lower and Middle Chase Dams\N13101249_FIGURE 05_R0.dwg [FIGURE 5] April 14, 2010 - 1:05:19 pm (BY: DEBASHIS DAS)



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LEGEND:

- 1978 BOREHOLE
- 1978 TEST PIT DUG BY HAND
- 1978 TEST PIT DUG BY BACKHOE
- TP09-## - 2009 TEST PIT

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Scale: 1: 1000 (metres)

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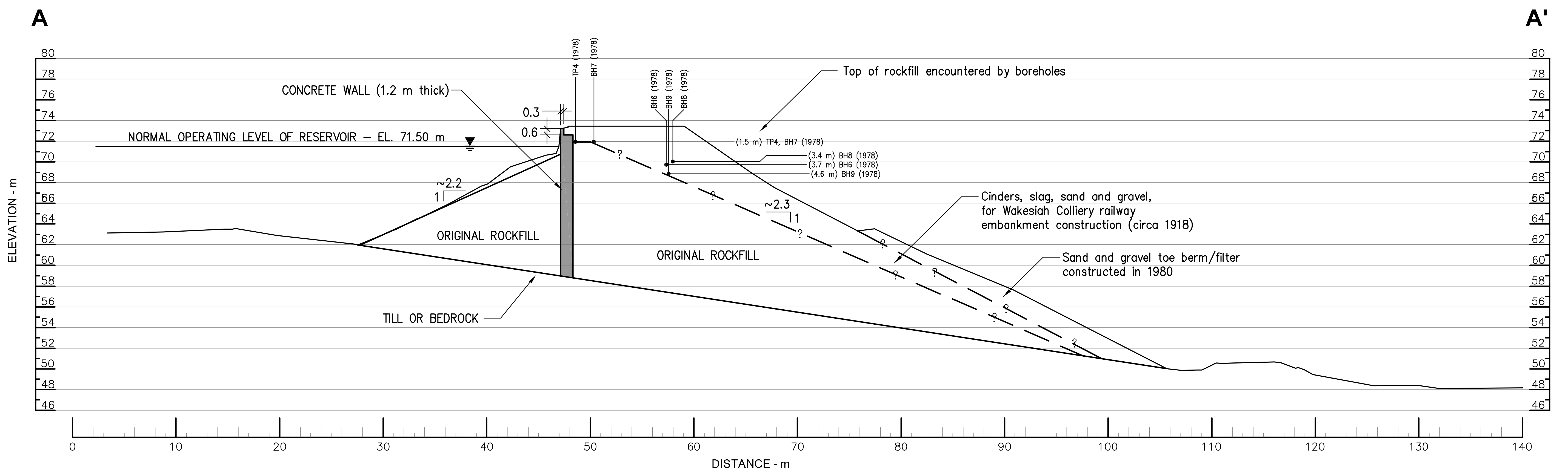
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Topographic / Bathymetric Data and Borehole Locations for Lower Chase Dam

PROJECT NO.	DWN	CKD	REV
N13101249	BR	CG	0
OFFICE	DATE		
EDM	April 2010		

Figure 6



NOTES:

1. Normal operating level of reservoir based on survey data of spillway.
2. Top of rockfill beneath cinders/slag defined by three boreholes drilled at downstream crest of dam in 1978. EBA has selected the lowest top of rockfill elevation for defining the cross–section details which also provides a similar slope on the downstream rockfill as to upstream rockfill.

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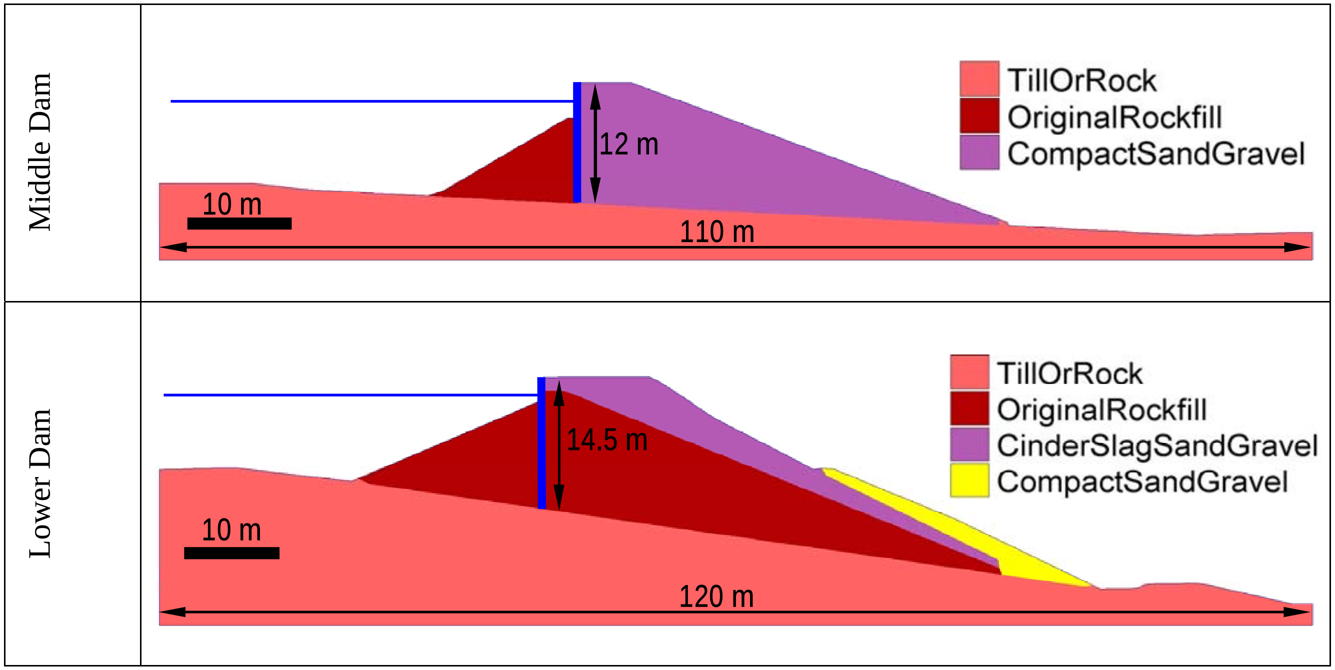
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Lower Dam
Cross-Section A-A'

PROJECT NO.	DWN	CKD	REV	
N13101249	BR	CG	0	Figure 7
OFFICE: EDM	DATE: April 2010			



LEGEND

NOTES

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Seismic Hazard Assessment
Middle and Lower Chase River Dams

Model Layout & Material Zones

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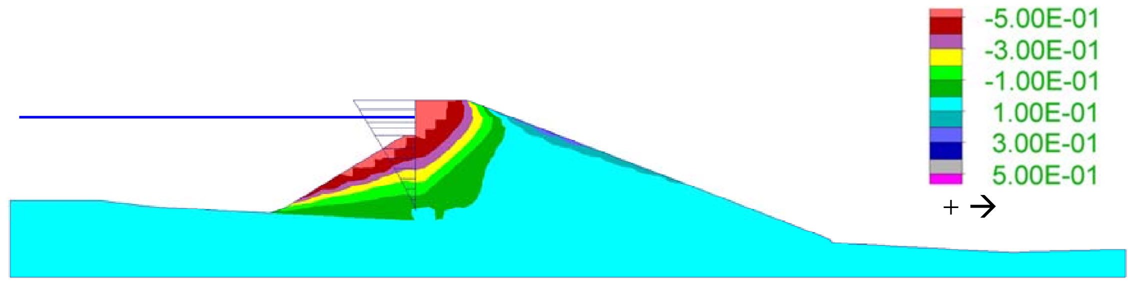
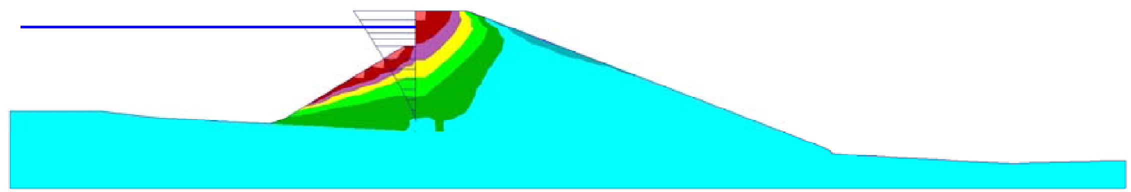
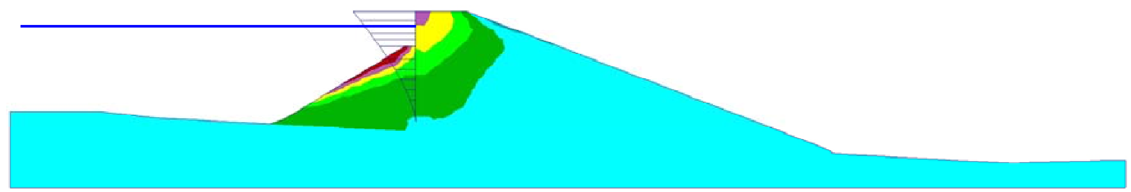
N13101249	AA	CG	0
OFFICE	DATE		
EBA-VANC	April 2010		

Figure 8

Case	Middle Chase Dam
Reasonable Worst	
Most Likely	
Best	

Contours of horizontal displacement (limited to +/- 0.5 m), deformed shape of the concrete wall and its top deflection (m).

LEGEND

NOTES

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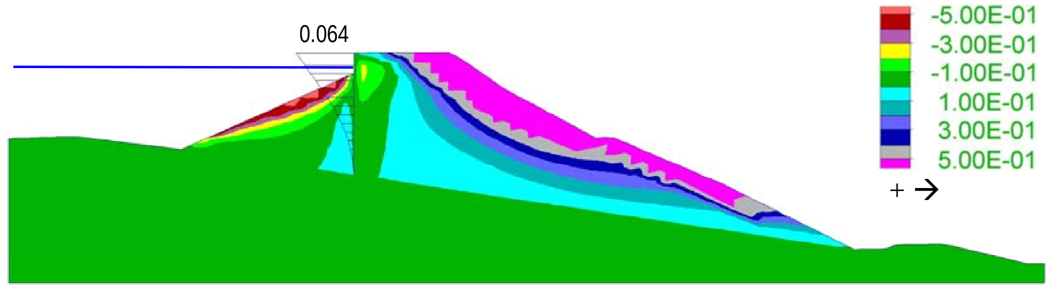
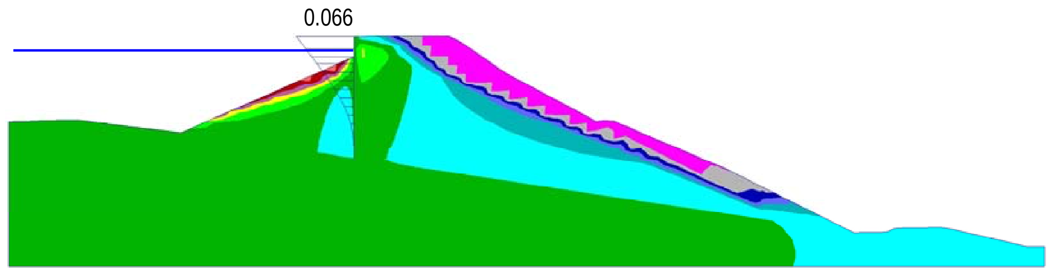
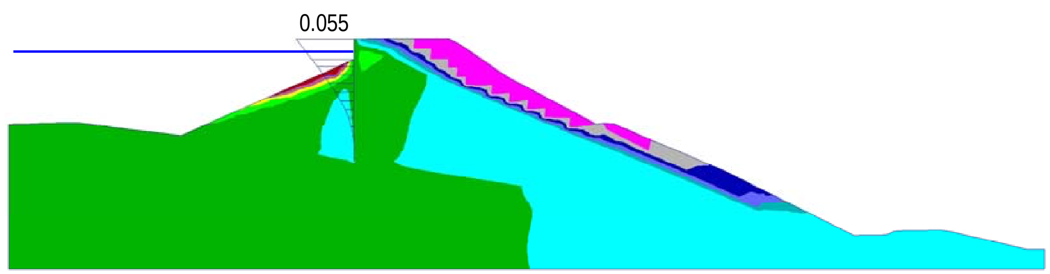
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Middle Dam – Horizontal Displacements

PROJECT NO.	DWN	CKD	REV
N13101249	AA	CG	0
OFFICE	DATE		
EBA-VANC	April 2010		

Figure 9

Case	Lower Chase Dam
Reasonable Worst	
Most Likely	
Best	
Contours of horizontal displacement (limited to +/- 0.5 m), deformed shape of the concrete wall and its top deflection (m).	

LEGEND

NOTES

CLIENT

CITY OF NANAIMO
THE HARBOUR CITY

EBA Engineering Consultants Ltd.

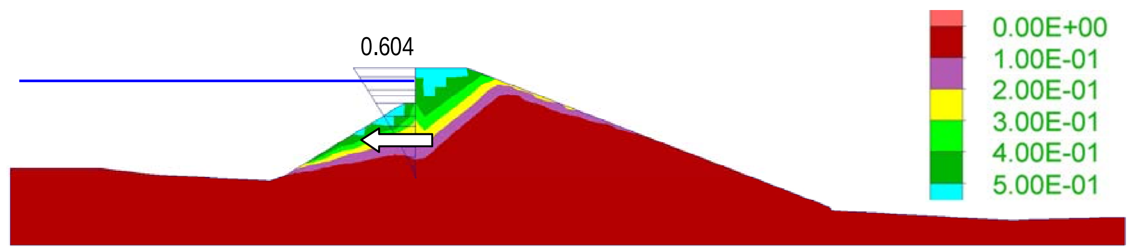
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Lower Dam – Horizontal Displacements

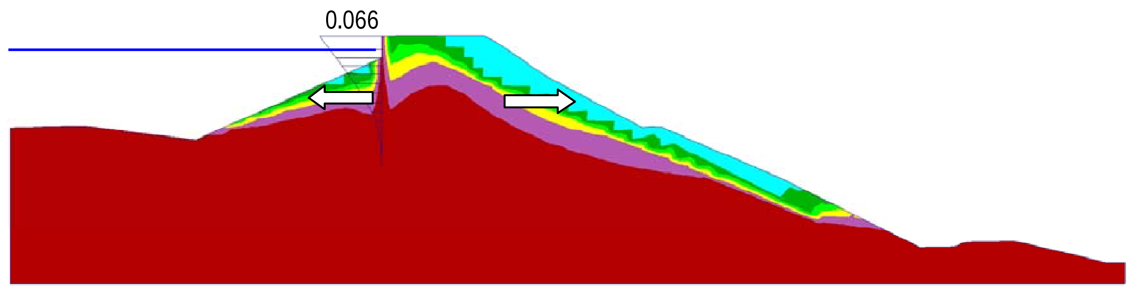
PROJECT NO.	DWN	CKD	REV
N13101249	AA	CG	0
OFFICE	DATE		
EBA-VANC	April 2010		

Figure 10

Middle Dam



Lower Dam



Contours of total displacement (i.e., resultant of horizontal and vertical components, limited to + 0.5 m), deformed shape of the concrete wall and its top deflection (m) for the Most Likely case. (Arrows indicate the direction of movement.)

LEGEND

NOTES

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THE HARBOUR CITY

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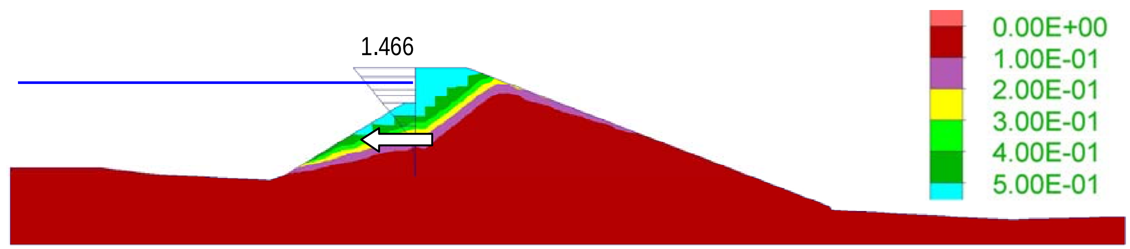
Seismic Hazard Assessment
Middle and Lower Chase River Dams

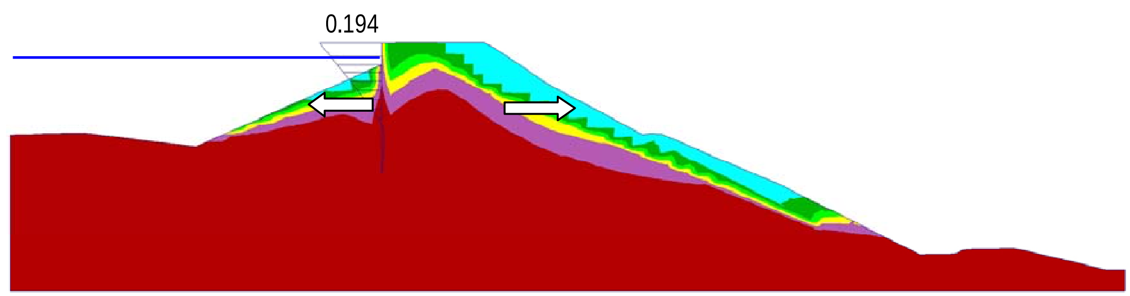
Middle & Lower Dam – Total Displacements

PROJECT NO.	DWN	CKD	REV	
N13101249	AA	CG	0	Figure 11
OFFICE EBA-VANC	DATE April 2010			

Middle Dam



Lower Dam



Contours of total displacement (i.e., resultant of horizontal and vertical components, limited to + 0.5 m), deformed shape of the concrete wall and its top deflection (m) for the Most Likely cases, illustrating the effect of inelastic behavior of concrete walls on deflections. Moment capacities of 150 and 600 kN.m were used for the 0.6 and 1.2 m wide walls of the Middle and Lower dams, respectively. (Arrows indicate the direction of movement.)

LEGEND

NOTES

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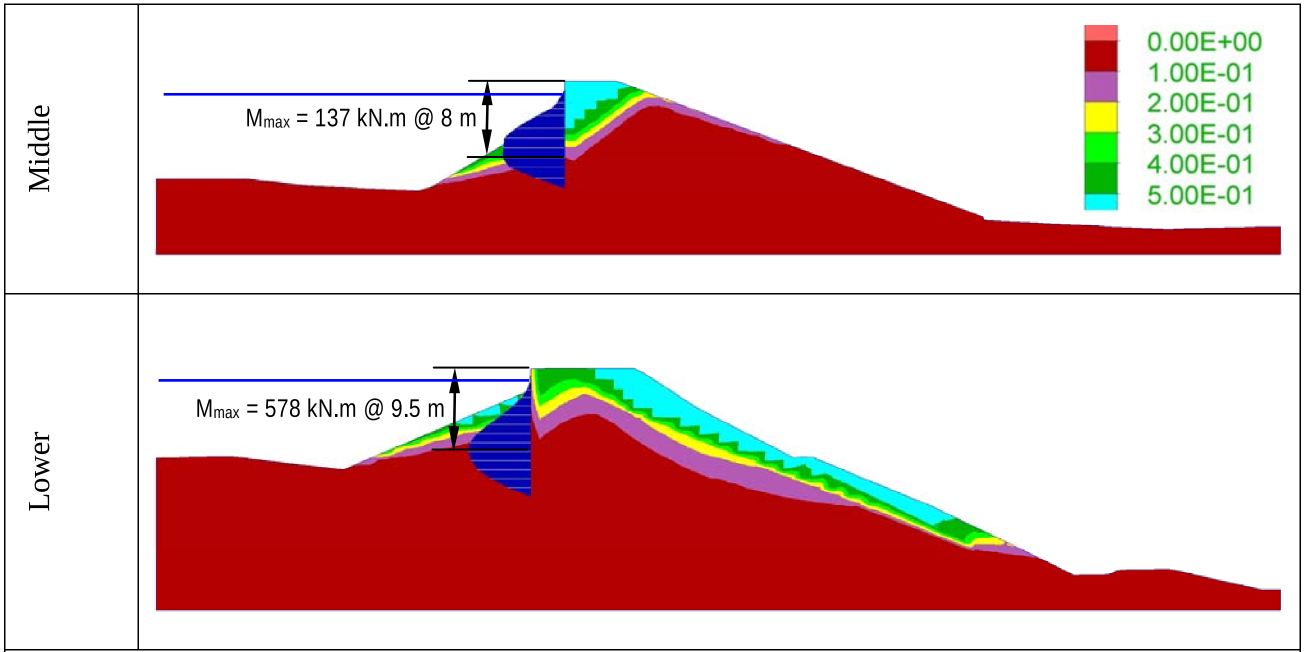
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Middle & Lower Dam – Moment Capacity Effect

PROJECT NO.	DWN	CKD	REV
N13101249	AA	CG	0

OFFICE	DATE
EBA-VANC	April 2010

Figure 12



Bending moment diagrams for the walls at the end of shaking for the Most Likely cases with inelastic wall behavior (moment capacity); and, contours of total displacement (limited to + 0.5 m).

LEGEND

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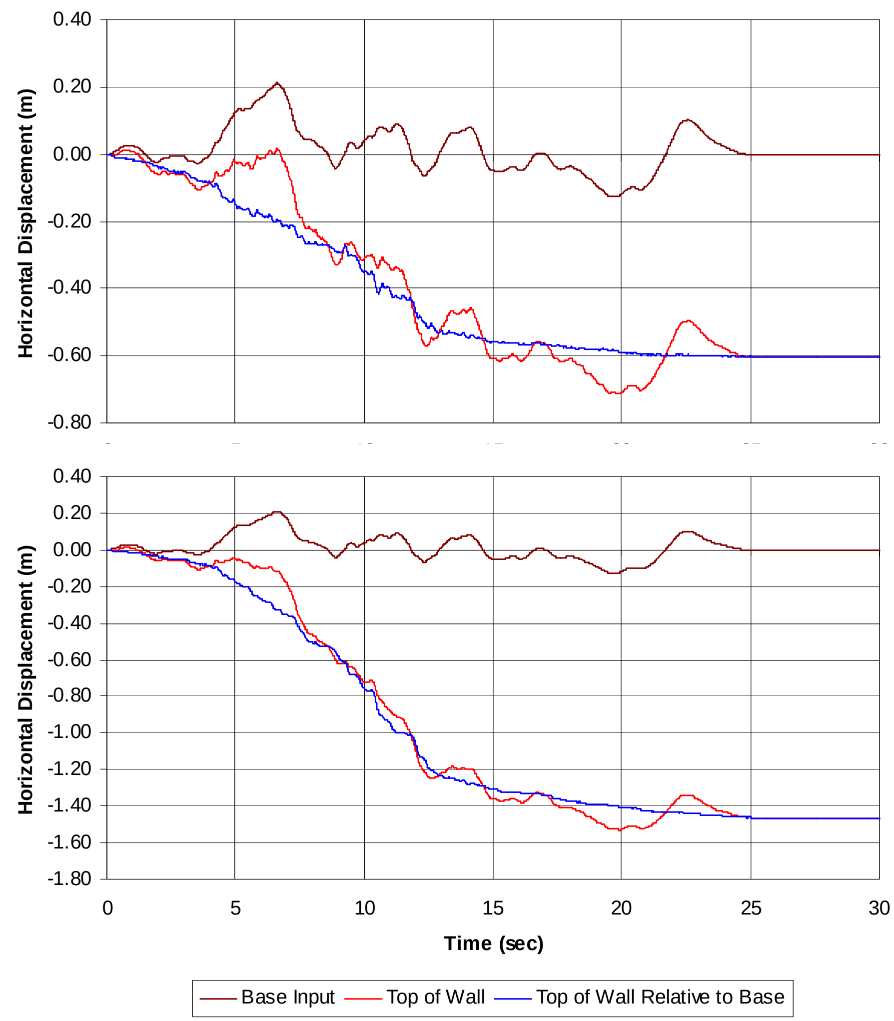
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Middle & Lower Dam – Bending Moment Diagrams

PROJECT NO.	DWN	CKD	REV	
N13101249	AA	CG	0	Figure 13
OFFICE	DATE			
EBA-VANC	April 2010			



Time histories of horizontal displacement of the top of the wall for Middle dam. Top/Bottom plots: cases for elastic/inelastic wall behavior, respectively.

LEGEND

NOTES

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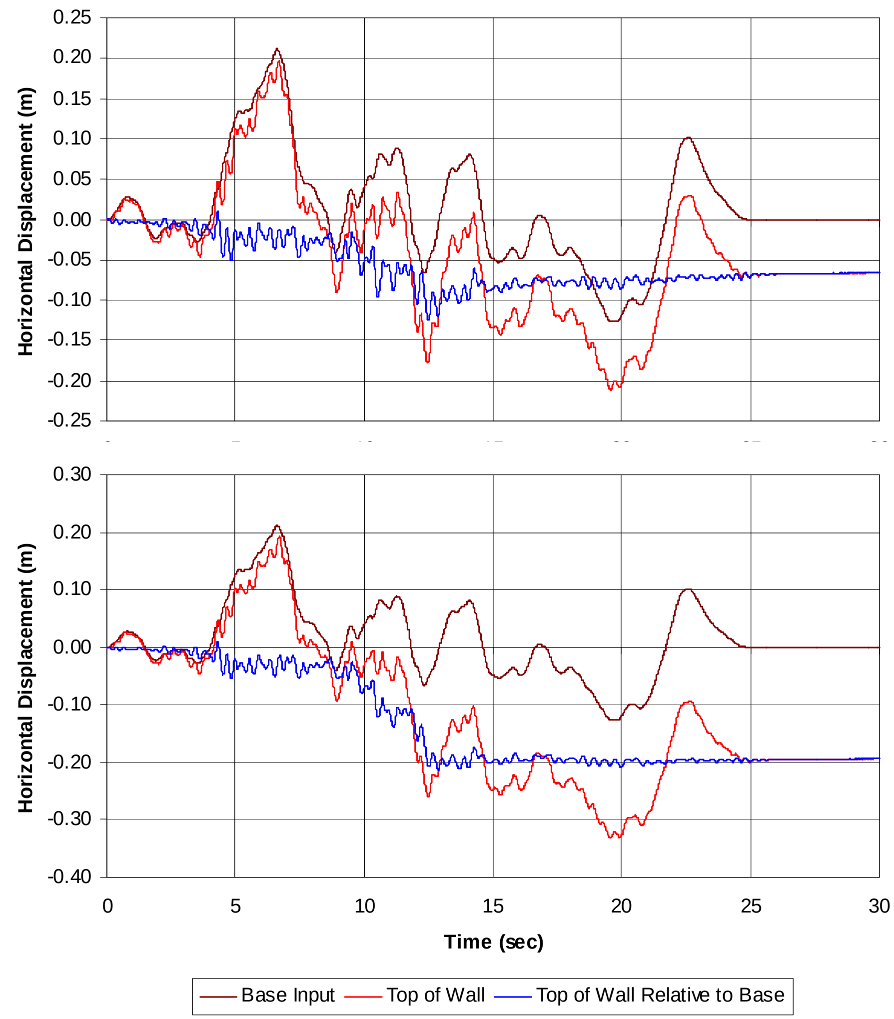
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Middle Dam – Top of Wall Horizontal Displacement Time Histories

PROJECT NO.	DWN	CKD	REV
N13101249	AA	CG	0
OFFICE	DATE		
EBA-VANC	April 2010		

Figure 14



Time histories of horizontal displacement of the top of the wall for Lower dam.
Top/Bottom plots: cases for elastic/inelastic wall behavior, respectively.

LEGEND

NOTES

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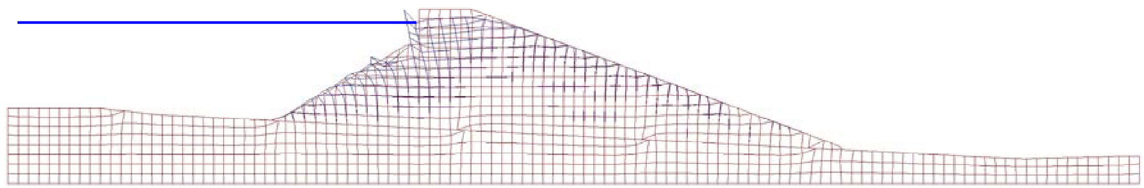
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Lower Dam – Top of Wall Horizontal Displacement Time Histories

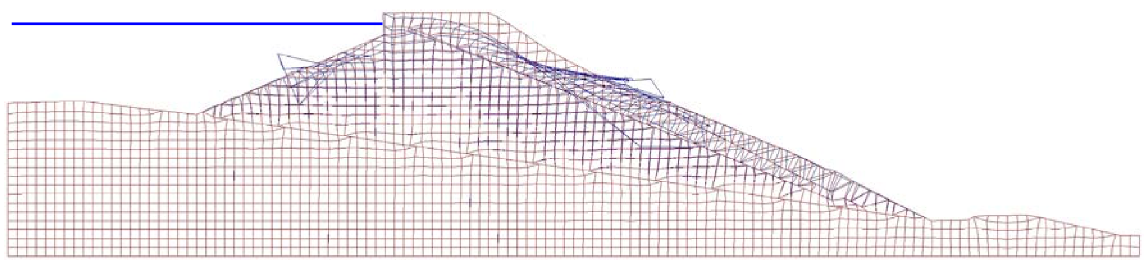
PROJECT NO.	DWN	CKD	REV
N13101249	AA	CG	0
OFFICE	DATE		
EBA-VANC	April 2010		

Figure 15

Middle Dam



Lower Dam



Deformed grids (no exaggeration) at the end of shaking.

LEGEND

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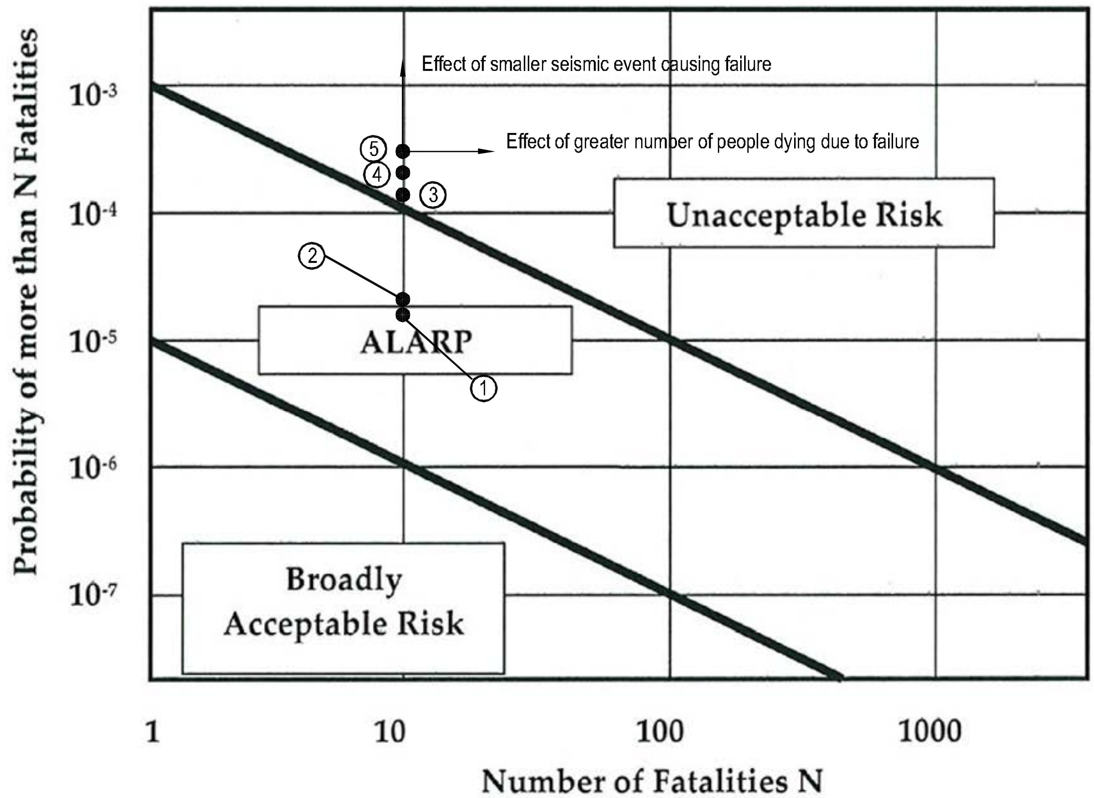
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Deformed Grids

PROJECT NO.	DWN	CKD	REV
N13101249	AA	CG	0
OFFICE	DATE		
EBA-VANC	April 2010		

Figure 16



NOTES

1 CURRENT ESTIMATE OF PROBABILITY OF PIPING POTENTIAL FOR MIDDLE CHASE DAM - 1.85 E-05 PER ANNUM

2 CURRENT ESTIMATE OF PROBABILITY OF PIPING POTENTIAL FOR LOWER CHASE DAM - 3.08 E-05 PER ANNUM

3 POST SEISMIC EVENT PROBABILITY OF PIPING POTENTIAL FOR MIDDLE CHASE DAM - 1.59 E-04 PER ANNUM

4 POST SEISMIC EVENT PROBABILITY OF PIPING POTENTIAL FOR LOWER CHASE DAM - 2.78 E-04 PER ANNUM

5 PROBABILITY OF 1:3000 YEAR SEISMIC EVENT - 3.30 E-04 PER ANNUM

ISSUED FOR USE

NOTE: TAKEN FROM CDA TECHNICAL BULLETIN - DAM SAFETY ANALYSIS AND ASSESSMENT (2007)

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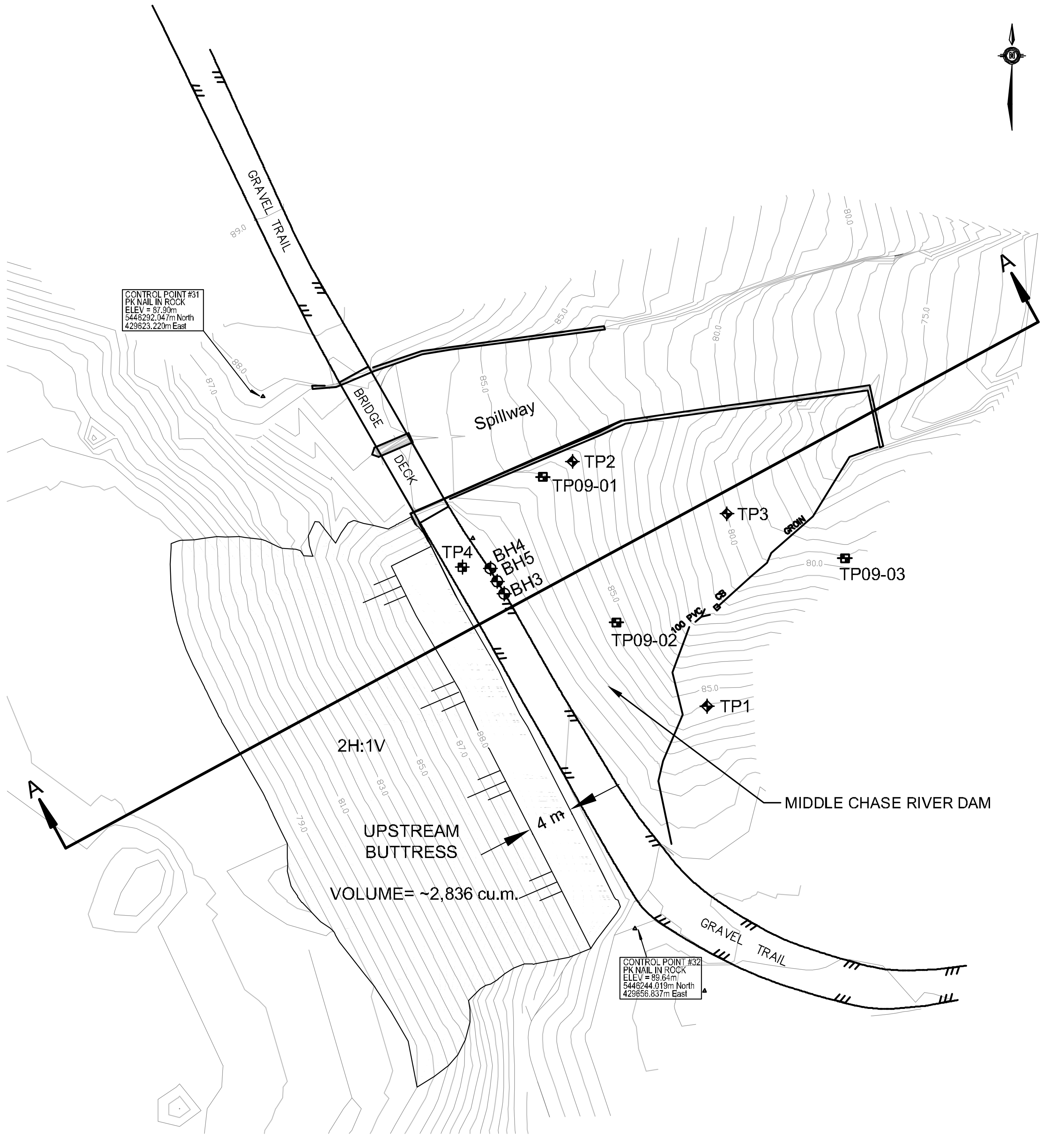
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Societal Risk Criteria for Dam Safety

PROJECT NO.	DWN	CKD	REV
N13101249	BR	CG	0
OFFICE	DATE		
EDM	April 2010		

Figure 17

Q:\Edmonton\Drafting\CIVIL3D\Other Offices\N131\N13101249 - Lower and Middle Chase Dams\N13101249_FIGURE 17_R0.dwg [FIGURE 17] April 14, 2010 - 11:17:49 am (BY: BOB RICHMOND)



ISSUED FOR USE

LEGEND:

- 1978 BOREHOLE
- 1978 TEST PIT DUG BY HAND
- 1978 TEST PIT DUG BY BACKHOE
- TP09-## - 2009 TEST PIT

0 25

Scale: 1: 500 (metres)

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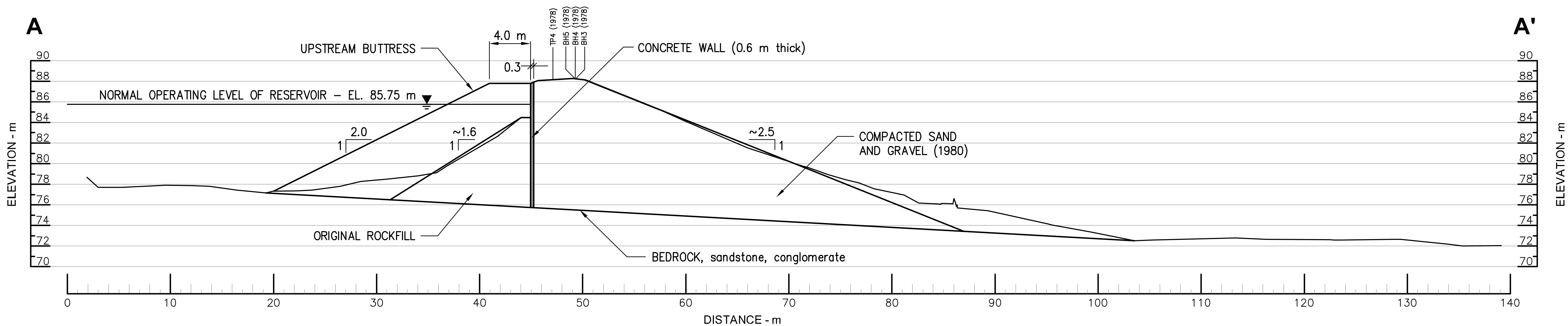
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Conceptual Upstream Buttressing
Middle Chase Dam

PROJECT NO.	DWN	CKD	REV
N13101249	DRG	CG	0
OFFICE	DATE		
EDM	April 2010		

Figure 18

Q:\Edmonton\Drafting\CIVIL3D\Other Offices\N131\N13101249 - Lower and Middle Chase Dams\N13101249_FIGURE 18_R0.dwg [FIGURE 18] April 14, 2010 - 11:20:15 am (BY: BOB RICHMOND)



NOTE: Normal operating level of reservoir based on survey data of spillway

ISSUED FOR USE

0 20
Scale: 1: 400 (metres)

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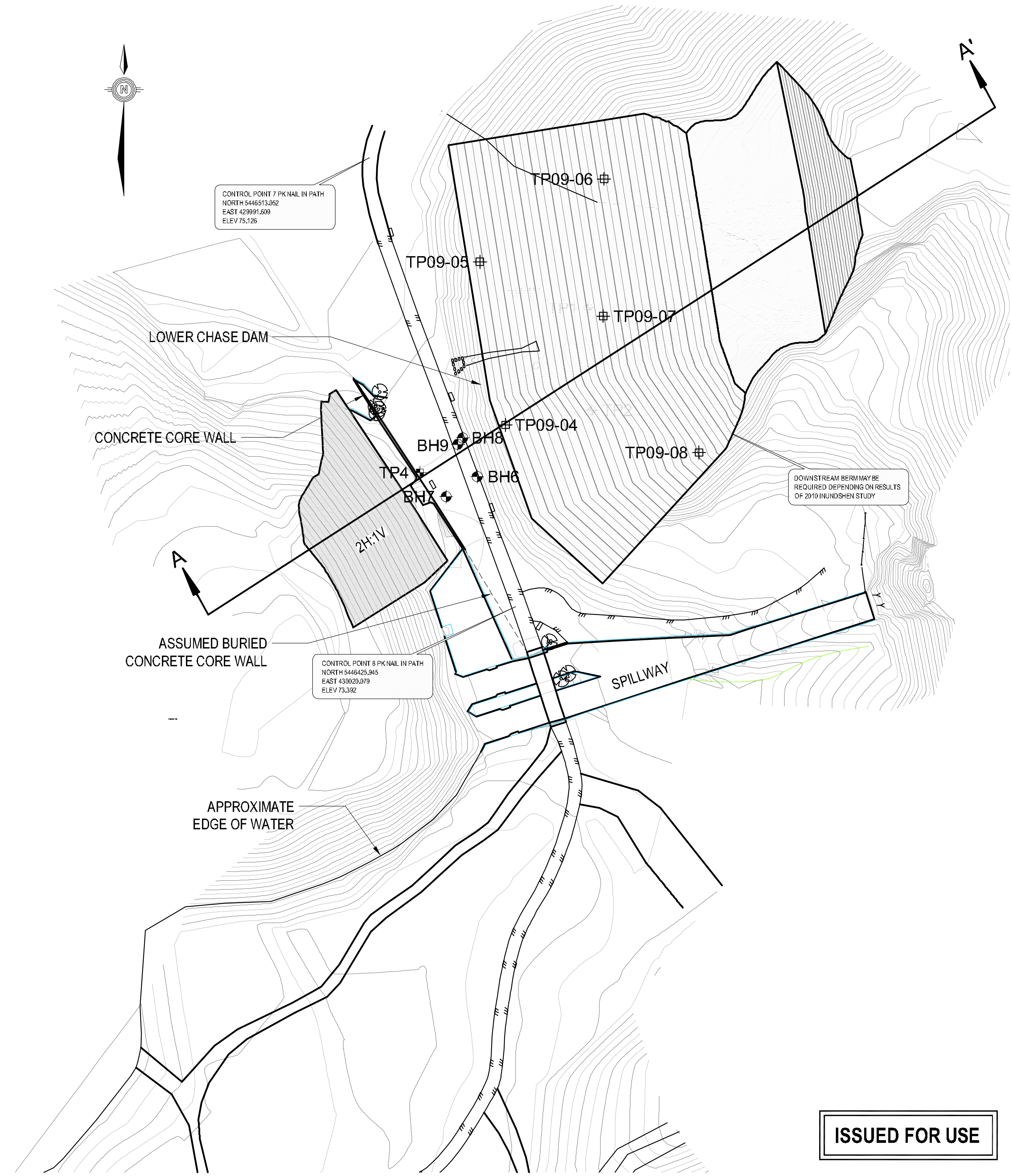
Seismic Hazard Assessment
Middle and Lower Chase River Dams

Conceptual Upstream Buttressing
Middle Dam
Cross-Section A-A'

PROJECT NO.	DWN	CKD	REV
N13101249	DRG	CG	0
OFFICE	DATE		
EDM	April 2010		

Figure 19



Q:\Edmonton\Drafting\CIVIL3D\Other Offices\N131\N13101249 - Lower and Middle Chase Dams\N13101249_FIGURE 20_R0.dwg [FIGURE 20] April 14, 2010 - 11:29:56 am (BY: BOB RICHMOND)

LEGEND

- 1978 BOREHOLE
- 1978 TEST PIT DUG BY HAND
- 1978 TEST PIT DUG BY BACKHOE

TP09-## - 2009 TEST PIT

0 — 50 m

Scale: 1: 1000 at 8.5"x11"

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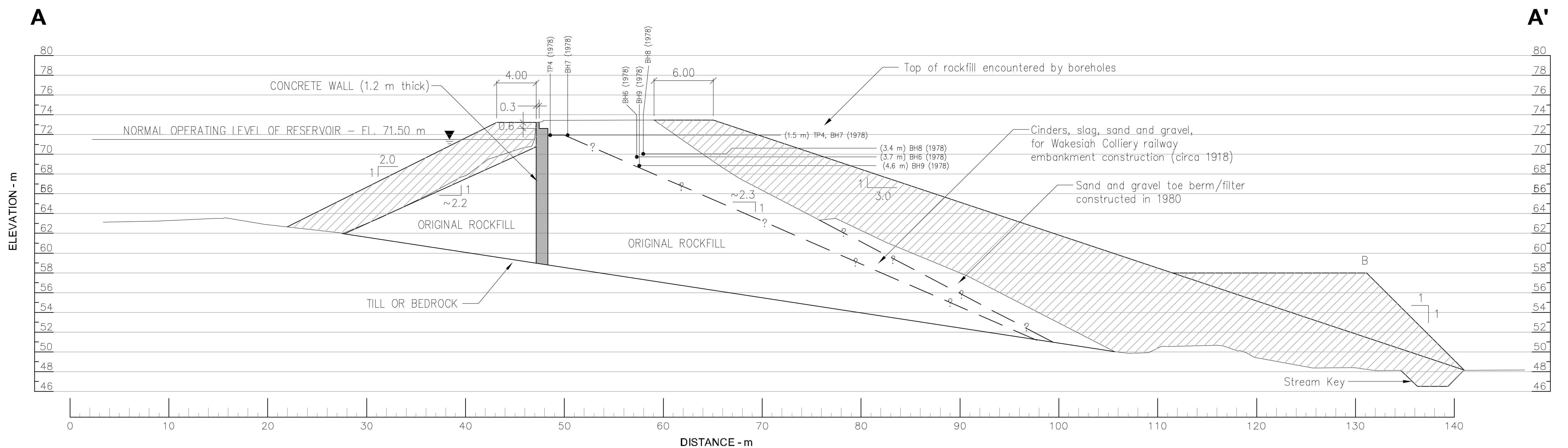
SEISMIC HAZARD ASSESSMENT
MIDDLE AND LOWER CHASE RIVER DAM

CONCEPTUAL UPSTREAM BUTTRESSING
LOWER CHASE DAM

PROJECT NO.	DWN	CKD	REV
N13101249	EL	CG	0
OFFICE	DATE		
EBA-EDM	April 2010		

Figure 20

Q:\Edmonton\Drafting\CIVIL3D\Other Offices\N131\N13101249 - Lower and Middle Chase Dams\N13101249_FIGURE 21_R0.dwg [FIGURE 21] April 14, 2010 - 11:32:32 am (BY: BOB RICHMOND)



NOTES:

1. Normal operating level of reservoir based on survey data of spillway.
2. Top of rockfill beneath cinders/slag defined by three boreholes drilled at downstream crest of dam in 1978. EBA has selected the lowest top of rockfill elevation for defining the cross-section details which also provides a similar slope on the downstream rockfill as to upstream rockfill.

ISSUED FOR USE

Scale: 1: 400 (metres)

CLIENT: CITY OF NANAIMO — THE HARBOUR CITY

EBA Engineering Consultants Ltd.

SEISMIC HAZARD ASSESSMENT
MIDDLE AND LOWER CHASE RIVER DAMNS

CONCEPTUAL UPSTREAM BUTTRESSING
LOWER CHASE DAM CROSS-SECTION A-A'

PROJECT NO.	DWN	CKD	REV	
N13101249	BR/EL	CG	0	**Figure 21**
OFFICE: EBA-EDM	DATE: April 2010			

PHOTOGRAPHS

Photo 1
Upstream face of Middle Chase Dam in February 2009

Photo 2
Concrete deterioration on upstream face of Middle Chase Dam

Photo 3
Upstream face of Lower Chase Dam from left abutment in February 2009.
Zodiak is being used for rebar survey and Schmidt Hammer testing of concrete strength

Photo 4
Deteriorated construction cold joints on upstream face of Lower Chase Dam near left abutment

Photo 5
Deteriorated construction cold joints on entrance to spillway at Lower Chase Dam

Photo 6
Crest and downstream slope of Middle Chase Dam and bridge over spillway, viewed from left abutment in May 2009

Photo 7
Upstream face of Middle Chase Dam viewed from left abutment in May 2009

Photo 8
Concrete placed beneath a rock overhang on the right abutment near the crest of the concrete wall

Photo 9
Upstream face of Middle Chase Dam viewed from right abutment.

Photo 10
Crest of Middle Chase Dam viewed from right abutment.

Photo 11
Downstream crest and slope of Middle Chase Dam viewed from crest of dam near right abutment.

Photo 12
Downstream face of Middle Chase Dam

Photo 13
Mound of fill material adjacent to Middle Chase Dam spillway wall is believed to be the original rock fill left in place in 1980 when the downstream shell was substantially excavated and replaced.

Photo 14
Middle Chase Dam spillway

Photo 15
Bedrock is visible at the bottom of the Chase River valley downstream of Middle Chase Dam

Photo 16
Upstream face of Lower Chase Dam viewed from confluence of Chase River with Lower Chase reservoir

Photo 17
Lower Chase Dam spillway, facing towards left abutment

Photo 18
Honey combing and construction cold joints in left spillway wall at Lower Chase Dam.

Photo 19

Upstream face and crest of Lower Chase Dam. The photograph is lined up with the original concrete wall. The concrete wall that angles off from the original wall appears to have been constructed at a later date for a different purpose.

Photo 20

Original concrete wall with subsequent concrete wall cast on top of it.
Note beveled edge of original wall and deterioration at cold joints or zones of poor quality concrete.

Photo 21
Looking towards right abutment and spillway along the original concrete wall from valve chamber. The concrete wall that angles off from the original wall appears to have been constructed at a later date for a different purpose.

Photo 22
The lower original concrete wall continues to extend in a straight line towards the right abutment. The concrete wall that angles off from the original wall appears to have been constructed at a later date for a different purpose

Photo 23
Looking along the upstream face of Lower Chase Dam towards left abutment along the original concrete wall from valve chamber.

Photo 24
Crest of dam viewed from left abutment

Photo 25
Upper portion of downstream slope and top of toe berm/filter layer viewed from right abutment

Photo 26
Settlement and cracking of asphalt walkway near downstream crest of dam where some shallow instability of the upper downstream face of the dam has occurred in the past.

Photo 27
Downstream face of dam viewed from downstream toe.
Note backscarps of small slope movements
(less than 1.0 m high) at toe of dam.

Photo 28
Proximity of Chase River channel crest to Barsby High School buildings

Photo 29
Little Ferns Daycare and Barsby High Scool Buildings located near crest of Chase River channel

Photo 30
Bruce Avenue crossing of Chase River

APPENDIX A

APPENDIX A EBA'S 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 and a specific scope of work. It is not applicable to any other sites nor should it be relied upon for purposes other than that to which it refers. Any variation from the site or purpose 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. Any unauthorized use of the report is at the sole risk of the user.

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 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.

APPENDIX B

APPENDIX B HISTORICAL SUBSURFACE INFORMATION

Chase River Dams Seismic Hazard Assessment
Summary of SPT Blow Counts from 1978 Investigation

BH#	Dam	Depth (m)	Material	N
4	Middle	2	1904 rockfill	31
		3	1905 rockfill	13
		4	1906 rockfill	>100
6	Lower	1.1	1918 cinders etc	4
		2	1919 cinders etc	1
		3.5	Loose S,G and Cobbles	9
		4.5	1904 rock fill	17
		8	1904 rock fill	22
		11	1904 rock fill	>100
7	Lower	1.4	1904 rock fill	17
		1.6	1904 rock fill	16

Review of N values for density descriptions

0 to 4	Very loose
4 to 10	Loose
10 to 30	Compact
30 to 50	Dense
>50	Very dense

Cinders, slag, sand and gravel

N values
4
1

Max N	Not enough data	
Average N		3
Mean N		
Min N		

1904 Rock fill

N values
31
13
9
17
22
17
16

Max N	31
Average N	18
Median N	17
Min N	9

*Statistically underpowered

Design value	**N value**
Best Case Scenario, upper limit of "Loose"	10
Most Likely Case Scenario, average of 2 tests	3
Reasonable Worst Case Scenario, lower end of tests	1
Best Case Scenario, upper end of "Compact"	30
Most Likely Case Scenario, middle of "Compact"	15
Reasonable Worst Case Scenario, lowest test result	9

1980 Fill in Downstream Shell of Middle Chase Dam and 1980 Lower Chase Dam Toe Berm (No SPT N values available)

Scenario	**N value**	
Best Case Scenario	35	Upper Practical Limit for S&G compacted to 95% SPD (assumed)
Most Likely Case Scenario	30	Mean between two ranges
Reasonable Worst Case Scenario	25	Upper range of "Compact", judged to be lower limit for S&G that was compacted to 95% SPD (assumed)

SOIL DESCRIPTION TERMS USED ON BOREHOLE LOGS

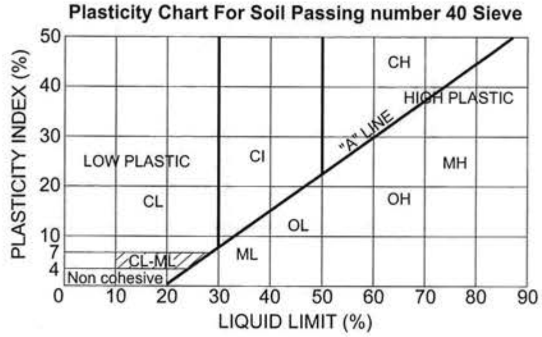
Type	
Gravel:	>#4; Cobbles 75-200mm, Boulders>200mm
Sand:	>#200 <#4 (5mm)
Silt:	.002mm<#200 (80μm)
Clay:	<0.002mm
Organic Matter	

Composition	
"And"	35-50%
"y"	20-35%
"some"	10-20%
"Trace"	<10%

Description: Type, Composition, Consistency/relative density, Plasticity/grain size, Structure/gradation, Moisture, Colour, Inclusions

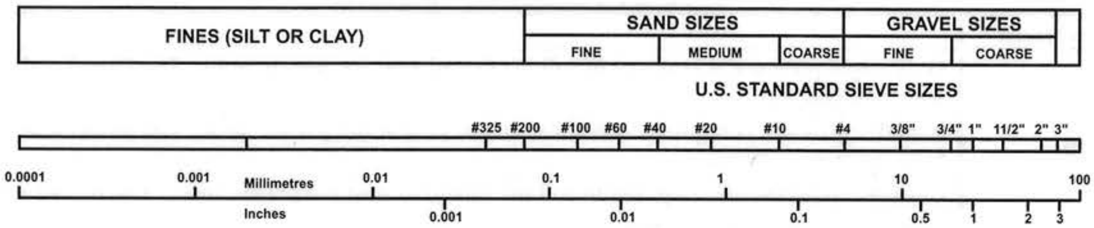
Strength/Relative Density					
COHESIVE	PP	N	CHARACTERISTICS	NON COHESIVE	N
Very soft	<0.25	<2.0	Penetrate with fist	**very loose**	0-4
Soft	0.25-0.5	2-4	indent with fist	**loose**	4-10
Firm	0.5-1.0	4-8	penetrate with thumb	**compact**	10-30
Stiff	1.0-2.0	8-15	indent with thumb	**dense**	30-50
Very stiff	2.0-4.0	15-30	indent with thumbnail	**very dense**	>50
Hard	>4.0	>30	can not indent with thumbnail		

Plasticity Chart For Soil Passing number 40 Sieve



Moisture
Dry
Damp
Moist (optimum)
Wet
Saturated

Grain Size	
Uniformly graded:	**of predominantly one grain size**
Poorly graded:	**having a range of grain sizes with some intermediate size or sizes missing**
Well graded:	**having a wide range of grain sizes with substantial amounts of all intermediate particle sizes**



Definitions	
Blocky:	**has a composition consisting of small cubes and oblong blocks usually not having any dimension greater than 1cm long and often partially cemented together.**
Calcareous:	**containing appreciable quantities of calcium carbonate**
Fissile:	**deposit in which the bedding is less than 2mm thick.**
Fissured:	**contains shrinkage cracks that are usually more or less vertical and are frequently filled with fine sand or silt**
Fractured:	**is broken by interconnecting cracks which are randomly orientated in all three dimensions of space.**
Interbedded:	**composed of alternate layers of different soil types**
Laminated:	**deposit in which the layering or bedding is less than 1cm apart.**
Mottled:	**irregularily marked with spots of different colours, mottling in soils usually indicates poor aeration and lack of good drainage**
Organic:	**containing organic matter; maybe decomposed or fibrous**
Sensitive:	**exhibiting loss of strength on remolding**
Slickensided	**has planes of weakness, usually inclined, that are slick and glossy in appearance**
Stratified:	**containing layers of different soil types**
Striated:	**a surface scratched or indented with minute parallel grooves or lines.**
Varved:	**a sedimentary deposit which shows contrasting laminations that are representative of seasonal sedimentations**

5018.cdr

TABLE II
RECORD OF TEST PITS

B. Middle Chase River Dam

Testpit No.	Depth (meters)	Strata Description
1	0 - 0.5	Sandy SILT, some clay, gravel cobbles, many roots. (Fill)
	0.5 - 0.6	Black TOPSOIL & ORGANICS
	0.6	BEDROCK
2	0 - 0.2	TOPSOIL & ORGANICS
	0.2	BEDROCK
3	0 - 0.3	Loose, fine to coarse GRAVEL, trace sand & silt. (Fill)
4	0 - 2.4	Loose to compact, brown SAND & GRAVEL, some clayey silt, cobbles & boulders. (Fill)

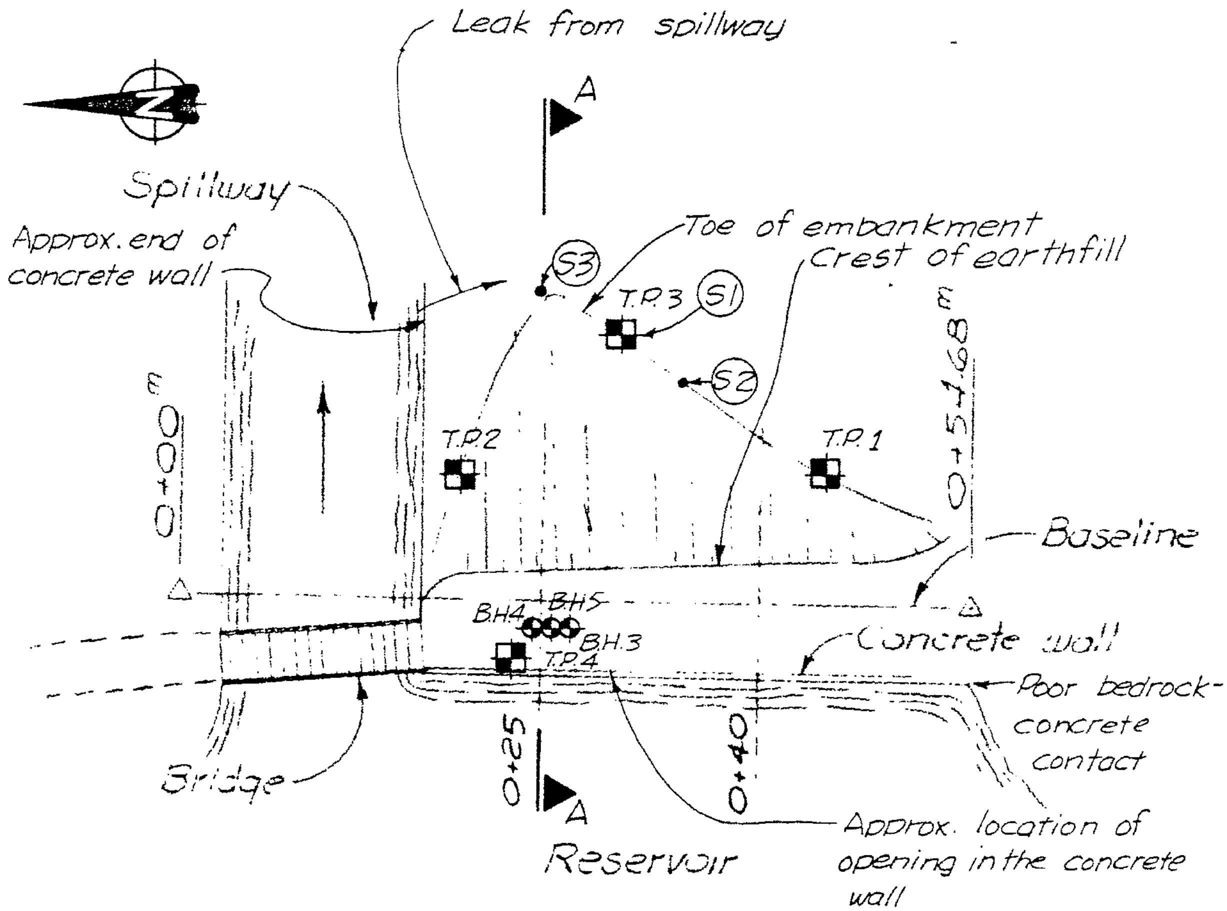
TABLE III
RECORD OF TEST PITS

C. Lower Chase River Dam

Testpit No.	Depth (meters)	Strata Description
1	0 - .3	Loose SLAG, CINDERS, COAL (Fill)
	.3 - .6	TOPSOIL & ORGANICS
	.6 - 1.2	Firm, brown sandy, gravelly SILT roots, occ. cobble
2	0 - 1.2	Loose slag, some SAND & GRAVEL (FILL)
3	0 - .9	Loose SLAG, CINDERS & ROOTS (Fill)
	.9 - 1.2	Dense, grey brown, silty gravelly SAND, some cobbles (Till-like).
4	0 - 1.5	Loose CINDERS, SLAG, sand roots. (Fill)
	1.5	ROCKFILL

SITE PLAN
MIDDLE CHASE RIVER DAM

Figure 3



Scale : 1 : 500

LEGEND

- Hand Dug Test Pits
- Test Pit Excavated by Backhoe
- Borehole
- (S3) Approx. location of seepage

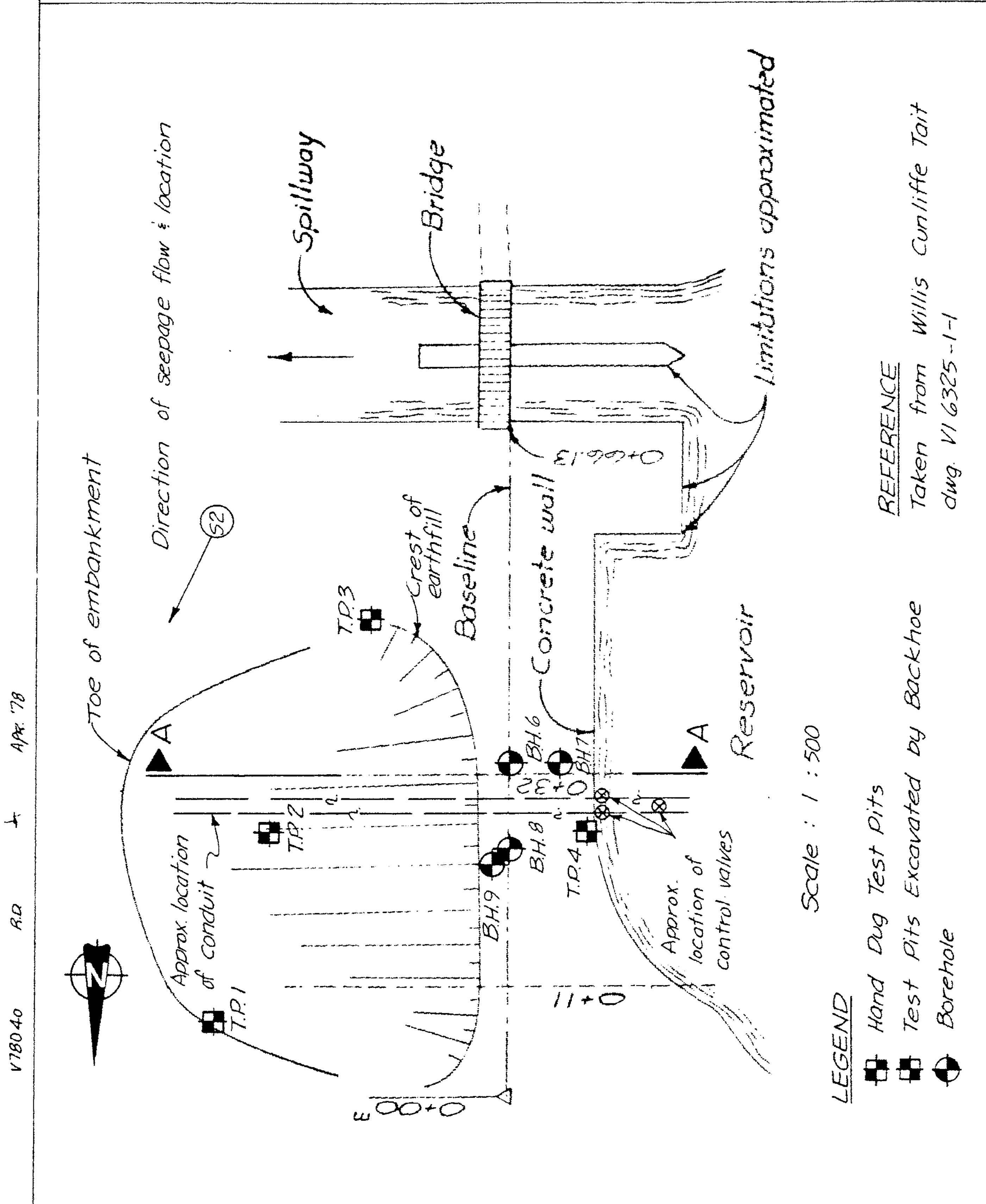
REFERENCE

Taken from Willis Cunliffe Tait dwg. V1 6325-1-1

V78040 R.D. Apr. '78

SITE PLAN
LOWER CHASE RIVER DAM

Figure 4



Project No. 778040

RECORD OF BOREHOLE 3
MIDDLE CHASE RIVER DAM

LOCATION (See Figure 3) BORING DATE March 16, 1978

BOREHOLE TYPE Rotary BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.6 kg. DROP 762mm DATUM WCT Dwg. VI 6325-1-1

SOIL PROFILE								
ELEV. / DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
								- drill with air, no sampling
100.8	Ground Surface							
0.0	Loose to compact brown SAND & GRAVEL some silt. (FILL)							- lost air circulation at 4.6m
94.1 / 6.7	ROCKFILL							- hole open to 7.6m no water encountered
91.7 / 9.1	End of Borehole							

VERTICAL SCALE 1 cm to 1 m

Golder Associates

DRAWN B.D.
CHECKED G.L.

Project No. V78040

RECORD OF BOREHOLE 4
MIDDLE CHASE RIVER DAM

LOCATION (See Figure 3) BORING DATE March 17, 1978

BOREHOLE TYPE Rotary BOREHOLE DIAMETER 114mm

SAMPLER HAMMER WEIGHT 63.6kg. DROP 762 mm DATUM WCT Dwg. VI 6325-1-1

SOIL PROFILE									PIEZOMETER OR STANDPIPE INSTALLATION
ELEV. DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L		ADDITIONAL LAB. TESTING
100.8	Ground Surface								drilling with air
0.0									
			1	Do	31				
	Loose to compact SAND GRAVEL, COBBLES, BOULDERS (FILL)		2	"	13				
			3	"	>100				
90.1									
10.7	End of Borehole								rods stuck in hole

VERTICAL SCALE 1 : 100

Golder Associates

DRAWN R.D.
CHECKED G.T.

Project No. V78040

RECORD OF BOREHOLE 5
MIDDLE CHASE RIVER DAM

LOCATION (See Figure 3) BORING DATE March 28-29, 1978

BOREHOLE TYPE Air Track BOREHOLE DIAMETER 50 mm

SAMPLER HAMMER WEIGHT – LB. DROP – IN. DATUM WCT Dwg. VI 6325-1-1

SOIL PROFILE								
ELEV. / DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P W W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
100.8	Ground Surface							
0.0	Loose to compact sand & gravel (FILL)							
94.7 / 6.1	ROCKFILL							- lost air circ. at 6.1m no return
88.3 / 12.5	TILL-LIKE MATERIAL							- rods stuck in hole
12.8	End of Borehole							

VERTICAL SCALE 1 : 100

Golder Associates

DRAWN R.Q.
CHECKED [illegible]

Project No. K78042

RECORD OF BOREHOLE 6

LOWER CHASE RIVER DAM

LOCATION (See Figure 2) | BORING DATE March 14, 1978

BOREHOLE TYPE Rotary | BOREHOLE DIAMETER 114mm

SAMPLER HAMMER WEIGHT 63.6kg. DROP 762mm | DATUM W,C,T, Dwg. VI 6325-1-1

SOIL PROFILE: ELEV. / DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P — W — W_L)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
100.3	Ground Surface							
0.0	Very loose cinders, slag, etc. (FILL)		1	D.a	4			
			2	"	1/18"			
97.8 / 2.5	Loose to compact SANDS GRAVELS, COBBLES (FILL)		3	"	9	Lost		
96.6 / 3.7	Loose gravels cobbles & boulders (ROCKFILL)							- Lost mud circulation
			4	"	17			
			5	"	22	Lost		
			6	"	>100	Lost		Hole cased to 10.3 - lost mud circ at 10.3m, no return 10.3m - 11.6.m
88.7 / 11.6	End of Borehole							- Stopped because possibility of jamming rods in hole, also casing damaged from driving into rockfill.

VERTICAL SCALE 1 : 100

Golder Associates

DRAWN R.D.
CHECKED G+

Project No. V78042

RECORD OF BOREHOLE 7
LOWER CHASE RIVER DAM

LOCATION (See Figure 2) BORING DATE March 15, 1978

BOREHOLE TYPE Rotary BOREHOLE DIAMETER 114 mm

SAMPLER HAMMER WEIGHT 63.6 kg DROP 762 mm DATUM WCT Dwg. VI 6325-1-1

SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P — W — W_L)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
100.3	Ground Surface							
0.0	Loose, rust brown fine to coarse SAND, some gravel trace silt & cinders (FILL)		1	DO	17			
98.8 1.5	Very coarse gravels with cobbles & boulders (ROCKFILL)		2	"	16	lost		- Lost mud circ. at 1.5m - drill with air to 4.6m - no return - stopped because of possibility of jamming rods in hole.
95.7 4.6	End of Borehole							

VERTICAL SCALE 1 : 100

Golder Associates

DRAWN B.Q.
CHECKED [illegible]

Project No. V78042

RECORD OF BOREHOLE 8
LOWER CHASE RIVER DAM

LOCATION (See Figure 2)
BORING DATE March 16, 1978
BOREHOLE TYPE Rotary
BOREHOLE DIAMETER 114 mm
SAMPLER HAMMER WEIGHT 63.6 kg. DROP 762 mm
DATUM W, C, T Dwg. VI 6325-1-1

SOIL PROFILE: ELEV. / DEPTH	SOIL PROFILE: DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT (W_P, W, W_L)	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
100.3	Ground Surface							Drill with air
	Loose black SAND GRAVEL, SLAG, CINDERS (FILL)							
96.9 3.4	Loose coarse GRAVELS COBBLES & BOULDERS (ROCKFILL)							- hole cased to 6.7 m. No return from 6.7 - 8.1 m
92.2 8.1	End of Borehole							- driving shoe for casing damaged

VERTICAL SCALE 1 : 100

Golder Associates

DRAWN B.D.
CHECKED G.L.

Project No. V78040

RECORD OF BOREHOLE 9
LOWER CHASE RIVER DAM

LOCATION (See Figure 2)
BORING DATE March 28, 1978

BOREHOLE TYPE Air Track
BOREHOLE DIAMETER 50 mm

DATUM WCT Dwg. VI 6325-1-1

SOIL PROFILE								
ELEV. / DEPTH	DESCRIPTION	STRATIGRAPHY PLOT	SAMPLE NUMBER	SAMPLE TYPE	BLOWS / FOOT	ELEVATION SCALE	WATER CONTENT PERCENT W_P — W — W_L	PIEZOMETER OR STANDPIPE INSTALLATION / ADDITIONAL LAB. TESTING
100.3	Ground Surface							
0.0	Loose cinders, sand, gravels							
95.7 / 4.6	Cobbles & boulders (ROCKFILL)							- lost air circ. at 4.6m - no return
85.4 / 14.9	TILL-LIKE MATERIAL							
84.8 / 15.5	End of Borehole							

VERTICAL SCALE 1 : 100

Golder Associates

DRAWN B.D.
CHECKED G.

APPENDIX C

APPENDIX C 2009 TESTPITS AND SCHMIDT HAMMER TEST RESULTS

Project: Chase River Dam Seismic Inspections
Project Number: N13101249
Task: Lower and Middle Chase Dam Testpitting
Excavation Method of Testpits: Spade and Pick Axe
Date: March 6, 2009

Middle ChaseDam:

TABLE 1: TESTPIT - TP09-01
START: 12:45PM, END: 1:15PM

Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.05	Veneer of grass/topsoil			
0.05	1.10	SAND (FILL) – gravely, some cobbles to 200 mm, trace silt, dense, angular to rounded gravel and cobbles, moist, greyish brown, trace organics (roots) to 0.30 m. Groundwater not observed Testpit backfilled on completion.	D	0.60-0.70	SA-01

TABLE 2: TESTPIT - TP09-02
START: 1:25PM, END: 2:00PM

Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.05	Veneer of grass/topsoil			
0.05	1.10	SAND (FILL) – gravely, some cobbles to 200 mm, trace silt, dense, angular to subrounded gravel and cobbles, moist, greyish brown, trace organics (roots) to 0.3 m. Groundwater not observed Testpit backfilled on completion.	D	0.80-0.90	SA-01

TABLE 3: TESTPIT - TP09-03
START: 2:10PM, END: 2:30PM

Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.20	SAND (TOPSOIL) – gravely, trace silt, compact, subrounded gravel, damp, blackish brown, organics (roots)			
0.20	0.80	SAND – gravely, trace cobbles to 250 mm, dense, subangular to subrounded gravel and cobbles, reddish brown, trace roots. Groundwater not observed Testpit backfilled on completion.	D	0.70-0.80	SA-01

Lower ChaseDam:

TABLE 4: TESTPIT - TP09-04 START: 11:40AM, END: 12:00PM					
Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.01	Veneer of mowed brush/topsoil			
0.01	0.50	CLAY (FILL) – some silt, trace gravel, firm, low to medium plastic, rounded gravel, moist, brown, organics (roots).			
0.50	1.30	SLAG AND CINDERS (FILL) – some sand, trace coal, loose to compact, subrounded to rounded slag, cinders, sand and coal, damp, grey/brown, trace organics (roots) to 1.0 m. Groundwater not observed Testpit backfilled on completion.	D	0.80-1.00	SA-01

TABLE 5: TESTPIT - TP09-05 START: 11:10AM, END: 11:30AM					
Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.05	Veneer of grass/topsoil			
0.05	1.30	SLAG AND CINDERS (FILL) – some sand, trace coal, loose to compact, subrounded to rounded slag, cinders, sand and coal, damp, grey/brown, trace organics (roots) to 1.0 m. Sloughing to 1.0 m Groundwater not observed Testpit backfilled on completion.	D	0.60-0.80	SA-01

TABLE 6: TESTPIT - TP09-06 START: 10:30AM, END: 11:00AM					
Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.40	SAND (TOPSOIL) – some silt, some gravel, trace clay, dense, subrounded gravel, damp, black and brown, trace organics (roots)			
0.40	0.80	SAND (GLACIAL TILL) – some gravel, some silt, trace clay, dense to very dense, subrounded gravel, damp, light brown. Groundwater not observed Testpit backfilled on completion.	D	0.60-0.70	SA-01

TABLE 7: TESTPIT - TP09-07 START: 9:00AM, END: 9:40AM					
Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.05	Veneer of grass/topsoil			
0.05	0.15	SAND (TOPSOIL) – trace silt, compact, subrounded medium sand, moist, brown and black, organics.			
0.15	1.10	SAND (FILL) – gravely, trace cobbles to 250 mm, dense, rounded to subrounded gravel and cobbles, moist, brown. At 0.9 m, becomes wet Groundwater not observed Testpit backfilled on completion.	D	0.60-0.80	SA-01

TABLE 8: TESTPIT - TP09-08 START: 9:50AM, END: 10:20AM					
Depth (m)		Soil Description	Sample		
From	To		Type	Depth (m)	N/PP
0.00	0.80	COBBLES & CLAY – gravely, some silt, dense, angular cobbles and gravel, wet, brown, trace organics (roots) to 0.5 m The soil encountered is likely weathered bedrock in a clay/silt matrix Groundwater not observed Testpit backfilled on completion.	D	0.60-0.70	SA-01

APPENDIX D

APPENDIX D CONSEQUENCE CLASSIFICATION FOR DAMS IN BC

DRAFT

Interim Consequence Classification Policy For Dams in British Columbia

July 2008

Background

In 1999 the Canadian Dam Association (CDA) published Dam Safety Guidelines to establish safety requirements for new and existing dams, enable the consistent evaluation of dam safety deficiencies and to provide a basis for dam safety legislation and regulation. The 1999 CDA Guidelines defined 4 dam classifications in Table 1-1, "Classification of Dams in terms of Consequence of Failure". In February 2000, the BC Dam Safety Regulations, under the *Water Act* of BC, were enacted. The BC Dam Safety Regulations also defined 4 dam classifications in Schedule 1, "Downstream Consequence Classification Guide". The two systems are similar; both use the same classification names, but Schedule 1 defines the classifications in greater detail than Table 1-1. An important distinction to note is that Dam Safety Regulation classifications are for **dam owner** requirements and the CDA Guidelines classifications are for **dam design** criteria.

The Water Stewardship Division has assigned consequence classifications to most of the 1,980 dams in BC based on available information and using Schedule 1. Many dam owners or their engineering consultants have undertaken dam break inundation studies which have confirmed the consequence classifications or provided evidence for a revised classification. As of June 2008, the numbers of dams in the 4 consequence classifications is as follows: Very High – 31, High – 257, Low – 498, Very Low or not regulated[1] – 1194.

Canadian Dam Association 2007 Dam Safety Guidelines

The CDA Guidelines were completely rewritten and published in 2007 along with a binder of Technical Bulletins. One important change is the new consequence classification system as described in Table 2-1 "Dam Classification". Table 2-1 describes 5 new consequence classifications that are described in more detail than the 1999 CDA Table 1-1. It is possible to make a reasonably good conversion table between the new CDA Classification table and Schedule 1 in the Dam Safety Regulations. Please see the comparison table attached.

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[1] These dams would be one of the following: too small, removed, not yet constructed or unclassified.

Consequence Classifications	Loss of Life		Persons at Risk (CDA Only)	Economic and Social Losses[2]		Environmental and Cultural Losses		Consequence Classifications
BC Dam Safety Regulation	**BC Reg**	**CDA**		**BC Reg[3]**	**CDA**	**BC Reg**	**CDA**	**CDA 2007**
Very High	>100	>100	*Permanent Residents*	>$100M Very High Infrastructure; Public, Commercial, Residential	Extreme - Critical Infrastructure or Services	Nationally & Provincially Important Habitat & Sites - Restoration Chance Low	Major Loss of Critical Habitat - No Restoration Possible	**Extreme**
High (High[4])	10 – 100	10 – 100		$10M – 100M Substantial Infrastructure; Public, Commercial	Very High - Important Infrastructure or Services	Same as Above but Restoration Chance High	Significant Loss of Critical Habitat - Restoration Possible	**Very High**
High (Low[4])	1 – 10	1 – 10		$1M – $10M Same as Above	High – Infrastructure, Public Trans & Commercial	Same as Above	Significant Loss of Important Habitat - Restoration Possible	**High**
Low	Some Possible	Unspecified[5]	*Temporary Only*	$100K - $1M Limited Infrastructure; Public, Commercial	Temporary & Infrequent	Regionally Important Habitat & Sites - Restoration Chance High	No Significant Loss of Habitat - Restoration Possible	**Significant**
Very Low	Minimal	0	*None*	< $100K Minimal	Low	No Significant Loss of Habitat or Sites	Minimal Short Term Loss	**Low**

[2] CDA name this category 'Infrastructure & Economics'
[3] Dollar values from year 2000
[4] Internal "High" sub-classification used for Dam Safety Program risk-based assessment.
[5] Significant category may not always line up with Low (BC Reg). A temporary population (e.g. in recreation al areas) could be quite large and a "sunny-day" failure could result in multiple fatalities.

Interim Policy for using both classification systems

The Water Stewardship Division (WSD) will incorporate the 2007 CDA Dam Safety Guidelines consequence classification system into the Dam Safety Regulations as soon as possible. This will require a revision of the regulations and that may take a considerable amount of time to achieve. The interim policy on the application of the regulations with respect to the revised 2007 CDA guidelines has 2 parts as follows:

1. For the purpose of undertaking Dam Safety Reviews (by review engineers) and plans review for new and existing dams (by the Dam Safety Officers) the dams should be classified under both the Dam Safety Regulations and the 2007 CDA Dam Safety Guidelines. The attached comparison chart shows how the WSD interprets the two different classifications and where the 2 consequence classification ratings align. Dam review engineers may use their discretion when they assign consequence classifications based on the 2 systems.

2. Until further notice, for the purpose of reviewing dam design criteria only, the 1999 CDA Guidelines may be used for dams constructed before 2008 (see CDA 1999 Tables 5-1 & 6-1). The main reason for this policy is the change in the Inflow Design Flood (IDF) and Maximum Design Earthquake[6] (MDE) recommended for the "High" consequence dams in the 2007 CDA Guidelines (see CDA 2007 Table 6-1). The 2007 CDA Guidelines suggest 3 classes where a permanent population is at risk (High, Very High and Extreme). For dams where less than 10 people are at risk (High), this results in a recommendation for a more conservative IDF and MDE than the 1999 guidelines. Some owners of dams classified as "High" consequence have previously been informed that a minimum IDF and MDE of 1:1000 would be acceptable. It would be inappropriate now to require that the higher 2007 CDA Guidelines IDF and MDE be applied immediately. However, the WSD recommends that the owner make every effort to move toward these new design criteria targets as soon as possible.

To be signed by Glen Davidson, Deputy Comptroller of Water Rights

[6] Now called *earthquake design ground motion* (EDGM) in the 2007 CDA Guidelines

Table 1-1 CDA Dam Safety Guidelines 1999

TABLE 1-1
CLASSIFICATION OF DAMS
IN TERMS OF CONSEQUENCES OF FAILURE

CONSEQUENCE CATEGORY	POTENTIAL INCREMENTAL CONSEQUENCES OF FAILURE [a]	
	LIFE SAFETY [b]	SOCIOECONOMIC FINANCIAL & ENVIRONMENTAL [b] [c]
VERY HIGH	Large number of fatalities	Extreme damages
HIGH	Some fatalities	Large damages
LOW	No fatalities anticipated	Moderate damages
VERY LOW	No fatalities	Minor damages beyond owner's property

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

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

[c] The Owner may wish to establish separate corporate financial criteria which reflect their ability to absorb or otherwise manage the direct financial loss to their business and their liability for damage to others.

Schedule 1 – Dam Safety Regulation, Feb. 2000

Downstream Consequence Classification Guide

Rating	Loss of Life	Economic and Social Loss	Environmental and Cultural Losses
VERY HIGH	Large potential for multiple loss of life involving residents and working, travelling and/or recreating public. Development within inundation area (the area that could be flooded if the dam fails) typically includes communities, extensive commercial and work areas, main highways, railways, and locations of concentrated recreational activity. Estimated fatalities could exceed 100.	Very high economic losses affecting infrastructure, public and commercial facilities in and beyond inundation area. Typically includes destruction of or extensive damage to large residential areas, concentrated commercial land uses, highways, railways, power lines, pipelines and other utilities. Estimated direct and indirect (interruption of service) costs could exceed $100 million.	Loss or significant deterioration of nationally or provincially important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance. Feasibility and/or practicality of restoration and/or compensation is low.
HIGH	Some potential for multiple loss of life involving residents, and working, travelling and/or recreating public. Development within inundation area typically includes highways and railways, commercial and work areas, locations of concentrated recreational activity and scattered residences. Estimated fatalities less than 100.	Substantial economic losses affecting infrastructure, public and commercial facilities in and beyond inundation area. Typically includes destruction of or extensive damage to concentrated commercial land uses, highways, railways, power lines, pipelines and other utilities. Scattered residences may be destroyed or severely damaged. Estimated direct and indirect (interruption of service) costs could exceed $1 million.	Loss or significant deterioration of nationally or provincially important fisheries habitat (including water quality), wildlife habitat, rare and/or endangered species, unique landscapes or sites of cultural significance. Feasibility and practicality of restoration and/or compensation is high.
LOW	Low potential for multiple loss of life. Inundation area is typically undeveloped except for minor roads, temporarily inhabited or non- residential farms and rural activities. There must be a reliable element of natural warning if larger development exists.	Low economic losses to limited infrastructure, public and commercial activities. Estimated direct and indirect (interruption of service) costs could exceed $100 000.	Loss or significant deterioration of regionally important fisheries habitat (including water quality), wildlife habitat, rare and endangered species, unique landscapes or sites of cultural significance. Feasibility and practicality of restoration and/or compensation is high. Includes situations where recovery would occur with time without restoration.
VERY LOW	Minimal potential for any loss of life. The inundation area is typically undeveloped.	Minimal economic losses typically limited to owner's property not to exceed $100 000. Virtually no potential exists for future development of other land uses within the foreseeable future.	No significant loss or deterioration of fisheries habitat, wildlife habitat, rare or endangered species, unique landscapes or sites of cultural significance.

Table 2-1 CDA Dam Safety Guidelines 2007

Table 2-1: Dam Classification

Dam class	Population at risk [note 1]	Incremental losses		
		Loss of life [note 2]	Environmental and cultural values	Infrastructure and economics
Low	None	0	Minimal short-term loss No long-term loss	Low economic losses; area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No significant loss or deterioration of fish or wildlife habitat Loss of marginal habitat only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of *important* fish or wildlife habitat Restoration or compensation in kind highly possible	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Significant loss or deterioration of *critical* fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of *critical* fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)

Note 1. Definitions for population at risk:

None—There is no identifiable population at risk, so there is no possibility of loss of life other than through unforeseeable misadventure.

Temporary—People are only temporarily in the dam-breach inundation zone (e.g., seasonal cottage use, passing through on transportation routes, participating in recreational activities).

Permanent—The population at risk is ordinarily located in the dam-breach inundation zone (e.g., as permanent residents); three consequence classes (high, very high, extreme) are proposed to allow for more detailed estimates of potential loss of life (to assist in decision-making if the appropriate analysis is carried out).

Note 2. Implications for loss of life:

Unspecified—The appropriate level of safety required at a dam where people are temporarily at risk depends on the number of people, the exposure time, the nature of their activity, and other conditions. A higher class could be appropriate, depending on the requirements. However, the design flood requirement, for example, might not be higher if the temporary population is not likely to be present during the flood season.

Table 5-1 CDA Dam Safety Guidelines 1999

TABLE 5-1
USUAL MINIMUM CRITERIA FOR DESIGN EARTHQUAKES

CONSEQUENCE CATEGORY [a]	MAXIMUM DESIGN EARTHQUAKE (MDE)	
	DETERMINISTICALLY DERIVED	PROBABILISTICALLY DERIVED (Annual exceedance probability)
Very High	MCE [b]	1/10,000
High	50% to 100% MCE [c] [d]	1/1000 to 1/10,000 [d]
Low	- [e]	1/100 to 1/1000 [e]

[a] See Section 1.4 for consequence classification.

[b] For a recognised fault or geographically defined tectonic province, the Maximum Credible Earthquake (MCE) is the largest reasonably conceivable earthquake that appears possible. For a dam site, MCE ground motions are the most severe ground motions capable of being produced at the site under the presently known or interpreted tectonic framework.

[c] MDE firm ground accelerations and velocities can be taken as 50% to 100% of MCE values. For design purposes the magnitude should remain the same as the MCE.

[d] In the High Consequence category, the MDE is based on the consequences of failure. For example, if one incremental fatality would result from failure, an AEP of 1/1000 may be acceptable, but for consequences approaching those of a Very High Consequence dam, design earthquakes approaching the MCE would be required.

[e] If a Low Consequence structure cannot withstand the minimum criteria, the level of upgrading may be determined by economic risk analysis, with consideration of environmental and social impacts.

Table 6-1 CDA Dam Safety Guidelines 1999

TABLE 6-1
USUAL MINIMUM CRITERIA FOR INFLOW DESIGN FLOODS

CONSEQUENCE CATEGORY [a]	INFLOW DESIGN FLOOD (IDF)
Very High	Probable Maximum Flood (PMF) [b]
High	Annual Exceedance Probability (AEP) between 1/1000 and the PMF [c]
Low	AEP between 1/100 and 1/1000 [c] [d]

[a] See Section 1.4 for consequence classification

[b] An appropriate level of conservatism shall be applied to loads from this event, to reduce the risks of dam failure to tolerable values. Thus, the probability of dam failure could be much lower than the probability of extreme event loading.

[c] Within the High Consequence category, the IDF is based on the consequences of failure. For example, if one incremental fatality would result from failure, an AEP of 1/1000 could be acceptable, but for consequences approaching those of a Very High Consequence dam, design floods approaching the PMF would be required.

[d] If a Low Consequence structure cannot withstand the minimum criteria, the level of upgrading may be determined by economic risk analysis, with consideration of environmental and social impacts.

Table 6-1 CDA Dam Safety Guidelines 2007

Table 6-1: Suggested Design Flood and Earthquake Levels (for Use in Deterministic Assessments)

Dam class [note 1]	AEP	
	IDF [note 2]	EDGM [note 3]
Low	1/100	1/500
Significant	Between 1/100 and 1/1000 [note 4]	1/1000
High	1/3 between 1/1000 and PMF [note 5]	1/2500 [note 6]
Very high	2/3 between 1/1000 and PMF [note 5]	1/5000 [note 6]
Extreme	PMF [note 5]	1/10,000

Acronyms: AEP, annual exceedance probability; EDGM, earthquake design ground motion; IDF, inflow design flood; PMF, probable maximum flood.

Note 1. As defined in Table 2-1, Dam Classification.

Note 2. Extrapolation of flood statistics beyond 1/1000 year flood (10^{-3} AEP) is discouraged.

Note 3. AEP levels for EDGM are to be used for mean rather than median estimates of the hazard.

Note 4. Selected on the basis of incremental flood analysis, exposure, and consequences of failure.

Note 5. PMF has no associated AEP. The flood defined as "1/3 between 1/1000 year and PMF" or "2/3 between 1/1000 year and PMF" has no defined AEP.

Note 6. The EDGM value must be justified to demonstrate conformance to societal norms of acceptable risk. Justification can be provided with the help of failure modes analysis focused on the particular modes that can contribute to failure initiated by a seismic event. If the justification cannot be provided, the EDGM should be 1/10,000.

APPENDIX E

APPENDIX E CAN ENGINEERING LTD. REPORT "CHASE RIVER – 3000-YEAR RETURN PERIOD SEISMIC HAZARD ASSESSMENT

DRAFT

Chase River – 3000 year return period seismic hazard assessment

Introduction

EBA consultants have requested a suite of ground records to represent the 1/3000 probability per annum seismic hazard for the Chase River dam site. They have provided information from NRC[1] on the site seismic hazard for several probabilities of occurrence.

The site hazard assessment does not consider ground motions from a possible Cascadia subduction earthquake as the spectral acceleration values from such an earthquake are estimated to be considerably smaller than that from the probabilistic estimates of crustal earthquakes. A Cascadia subduction earthquake may be of higher magnitude than crustal earthquakes, and thus have a longer duration of strong motion, and should be considered if duration is expected to have a strong influence on the response.

Methodology

The Chase River site 2005 NBCC seismic hazard values, in terms of an acceleration response spectrum, are available from NRC[1] for the 1/2475 per annum probability (2% in 50 year probability per annum), as well as for the 5%, 10% and 40% in 50 year per annum probabilities.

The shape of the spectra for the different probabilities are almost identical, and so were scaled so that the average spectral value equalled the average spectral value of the 1/2475 spectrum. The scale values were extrapolated, on a log-log plot, to the 1/3000 per annum probability and a scale factor determined enabling the 1/3000 per annum probability spectrum to be calculated.

The 1/3000 spectrum shape was compared to the 1/2475 spectrum for Vancouver and found to be nearly identical, so the 1/3000 per annum hazard can be represented by a scaled version of the 1/2475 per annum Vancouver spectrum, and ground motion records used in Vancouver can be used for the 1/3000 per annum Chase River spectra by applying a single scale factor.

Results

The spectral hazard values for the Chase River site (49.1497 ^{0}N 123.9619 ^{0}W) for different probabilities of exceedance, or return periods, are given in Table 1 and plotted in Figure 1.

Figure 2 shows the spectra for the different return period probabilities scaled so as to give the same average spectral values as the 2475 return period spectrum. There is little difference between the scaled spectra, showing that the shapes of the spectra are essentially independent of the probability of occurrence. The scale values are shown in the figure.

The scale values are plotted against the annual probability in Figure 3 for the three lowest probabilities of occurrence. A polynomial curve is fit to these three values, which nearly fall on a straight line, and is extrapolated out to the 1/3000 per annum probability (1.653%/50 years) giving a scale factor of 0.947, ie, the 3000 year spectrum is 1/0.947 times the 2475 year spectrum.

Figure 4 shows the Chase River 3000 year return period hazard and the Vancouver 2475 year return period hazard scaled by a factor of 1.11. The two match very well over the entire spectrum shown, and so the ground motion records used for Vancouver 2475 need only be multiplied by the factor 1.11 to provide ground motion records for the Chase River 3000 year return period hazard.

Ground motion records

Ten records from crustal earthquakes recorded on firm ground (corresponds to the site condition used by NRC[1] in determining the spectral hazard) have been selected. These are some of the same records chosen by the APEGBC/UBC study for the Performance-based Seismic Retrofit of British Columbia School Buildings[2]. Table 2 gives lists the earthquake names and PGA values.

The records have been altered using two different methods to provide a set of records representative of the seismic hazard at the site. They were initially altered to represent the Vancouver 2475 year hazard but need only be increased by the factor of 1.11 to represent the Chase River 3000 year hazard. The two sets of records representing the Vancouver 2475 spectrum have been transmitted to EBA (c/o Ali Azizian) in earlier emails dated May 6, 2009.

Figure 5 shows the spectra of the original records plus the Chase River 3000 year hazard. There is much scatter in the lower period range of the spectra, and in general most of the records need to be scaled upwards to better represent the site hazard.

Modified records

The original records have been modified using the program SYNTH[3], a frequency based method, to match the Chase River spectrum over the period range of 0.04 to 5 seconds. Figure 6 shows the spectral response of the modified records for the period range of 0.1 seconds to 5 seconds, plus the mean response and the Chase River 3000 year hazard target spectrum. Periods less than 0.1 seconds have much more scatter as the time step of the original ground motion recording in many of the records (0.02 seconds) does not contain good information for periods below about 0.1 seconds. These short period spectra are not shown and generally are unimportant for seismic response.

Scaled records

The original records have been scaled to match the Chase River spectrum over the period range of 0.5 to 1.5 seconds. Figure 7 shows the spectral response of the modified records for the period range of 0.1 seconds to 10 seconds, plus the mean response and the Chase River 3000 year hazard. Figure 8 shows the same plot but only over the period range of 0.5 to 1.5 seconds, the range over which the records were scaled. The records are scaled so that the area under the spectral velocity plot from 0.5 to 1.5 seconds matches the area under the spectral velocity plot for the Chase River 3000 year hazard target spectrum. As seen in Fig. 8 the mean spectra deviates from the target spectrum in the shorter period range, but overall the average of the two spectra agree well.

Summary

It was shown that for the Chase River site the acceleration spectrum for several different probabilities of seismic hazard had essentially the same shape and differed only by a scale factor. The scale factor to extrapolate from the 2475 year to the 3000 year return period spectra was determined using a log-log plot and used to calculate a 3000 year spectrum for the Chase River site. The spectrum shape of the Chase River 3000 year hazard and the Vancouver 2475 year hazard are essentially identical, and so records used to model the Vancouver hazard were scaled to produce a set of records for use in representing the Chase River 3000 year spectrum.

A suite of ten records was in one case modified so that each record matched the Chase River 3000 year hazard target spectrum over the entire spectrum range. The same records were also scaled so that the average spectral value of each record, over the period range of 0.5 to 1.5 seconds, matched the average of the target spectrum.

Experience with modified or scaled suites of records shows that there is little difference in the mean response of the results when the response of the problem being analyzed is primarily in the period range of the scaling procedure, and when about ten records are used, but the scatter of the results is much less for the modified suite of records compared to the scaled suite.

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Donald L. Anderson, Ph.D, P.Eng.

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1. NRC, Geological Survey of Canada, Open File 4459, 2003. Fourth generation seismic hazard maps of Canada: Values for over 650 localities intended for the 2005 National Building Code of Canada. Available on the web at: www.seismo.nrcan.gc.ca.

2. APEGBC/UBC, Commentary to the Bridging Guidelines for the Performance-based Seismic Retrofit of British Columbia School Buildings, Second Edition, BG2-Commentary Draft#1B-C, October 2007.

3. SYNTH (1985), Naumoski, N., Generation of Artificial Acceleration Time History Compatible with a Target Spectrum. McMaster Earthquake Engineering Software Library, Dept. of Civil Engin. and Eng. Mech., McMaster Univ., Hamilton, Canada.

Tables

Table 1. Chase River seismic hazard spectral values, g. (earthquakescanada.nrcan.ca)

Probability	Return period, years	PGA	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)
2%/50 years	2475	0.499	1.012	0.691	0.351	0.178
5%/50 years	975	0.360	0.725	0.489	0.248	0.123
10%/50 years	475	0.267	0.531	0.357	0.181	0.089
40%/50 years	98	0.125	0.244	0.162	0.082	0.040

Table 2. Ground motion suite

Name	Station	PGA - g
SO90	Sherman Oaks 105 deg	0.214
WW235	Wadsworth – 235 deg	0.303
WW325	Wadsworth – 325 deg	0.389
CCO	Canyon County – 0 deg	0.396
SARA0	Saratoga – 0 deg	0.505
CP196	Canoga Park – 196 deg	0.388
C[106	Canoga Park – 106 deg	0.350
PK90	Pacoima Kagel – 90 deg	0.301
MD35	12521 Mulholland Dr. – 35 deg	0.588
GIL67	Gilroy Gavilon College – 67 deg	0.356

Figures

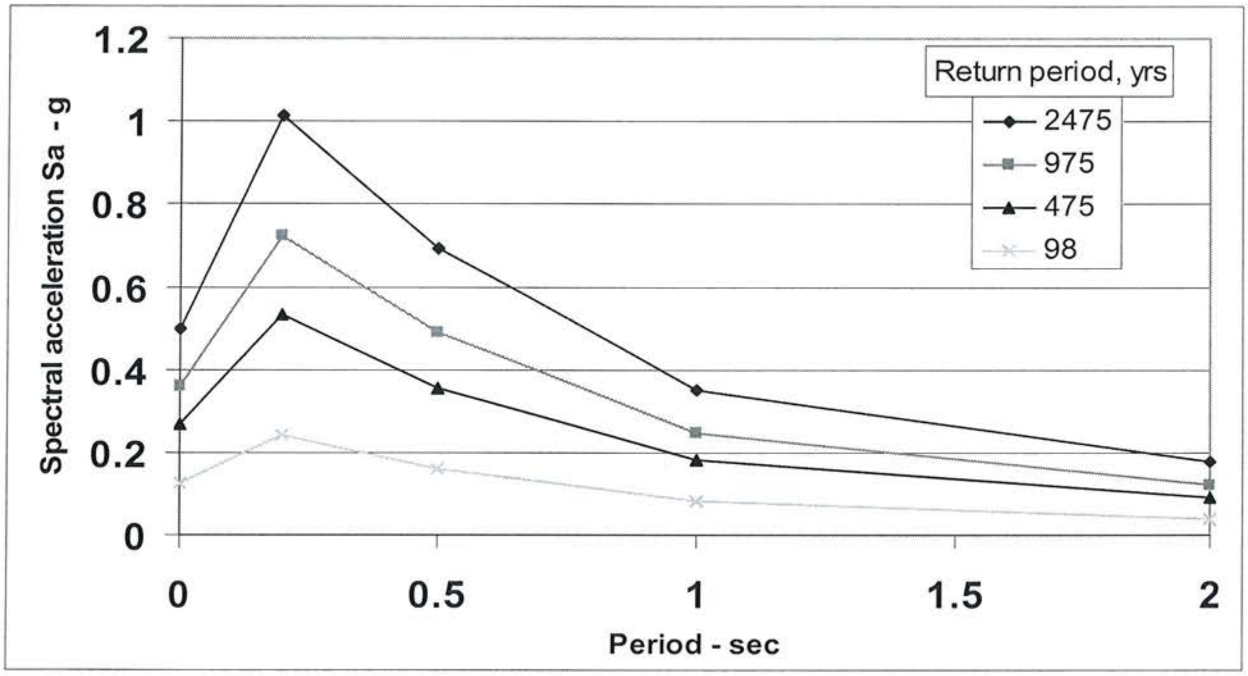

Figure 1. Chase River seismic hazard for different return period probabilities.

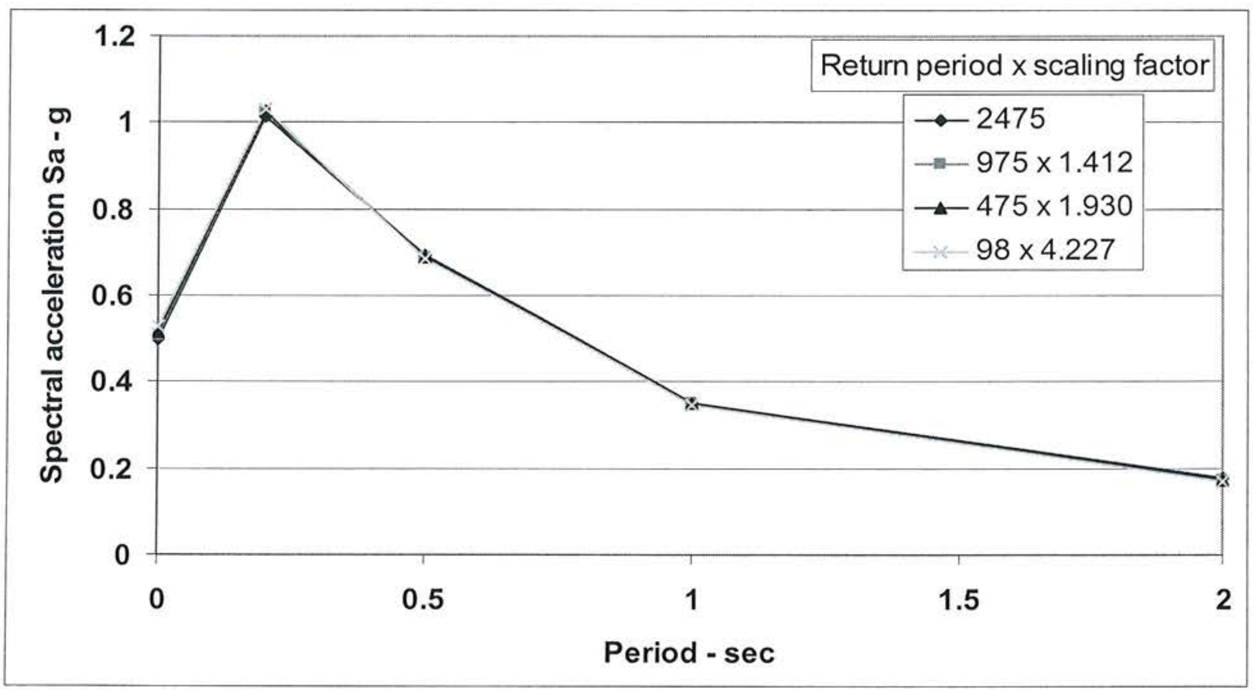

Figure 2. Chase River seismic hazard spectra for different return period probabilities scaled to give the same average spectral values as the 2475 year hazard.

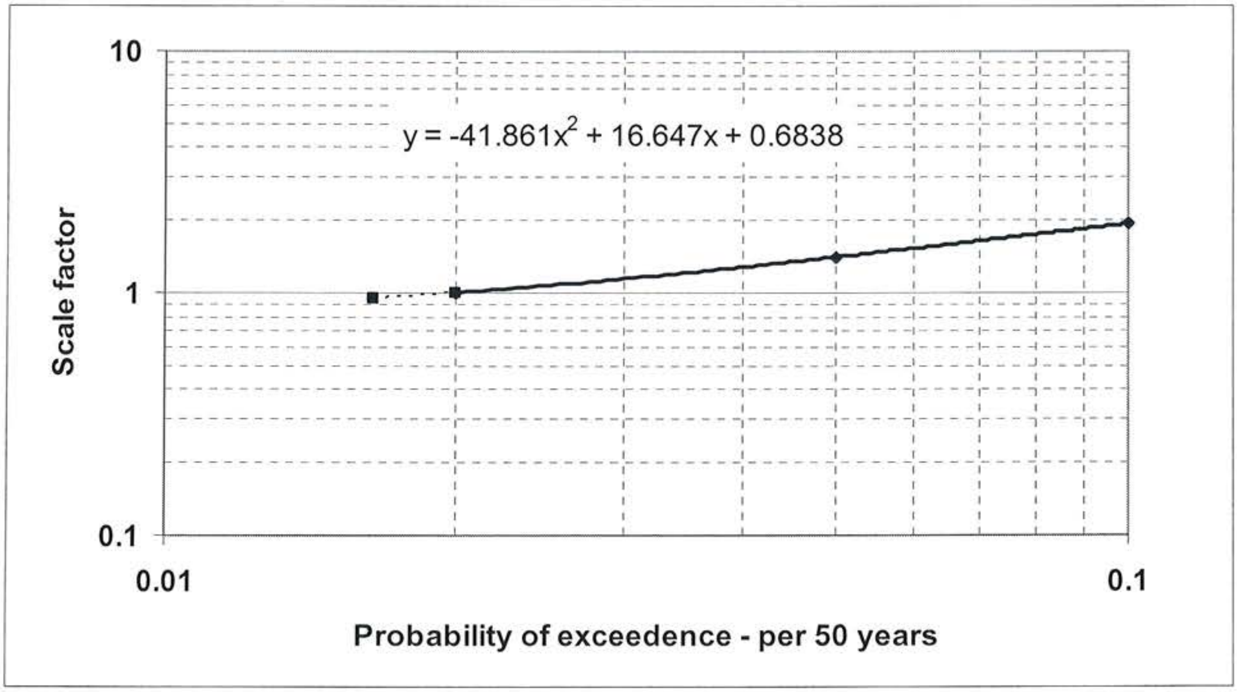

Figure 3. Extrapolation of scale factor to 1.653%/50 year probability for Chase River seismic hazard.

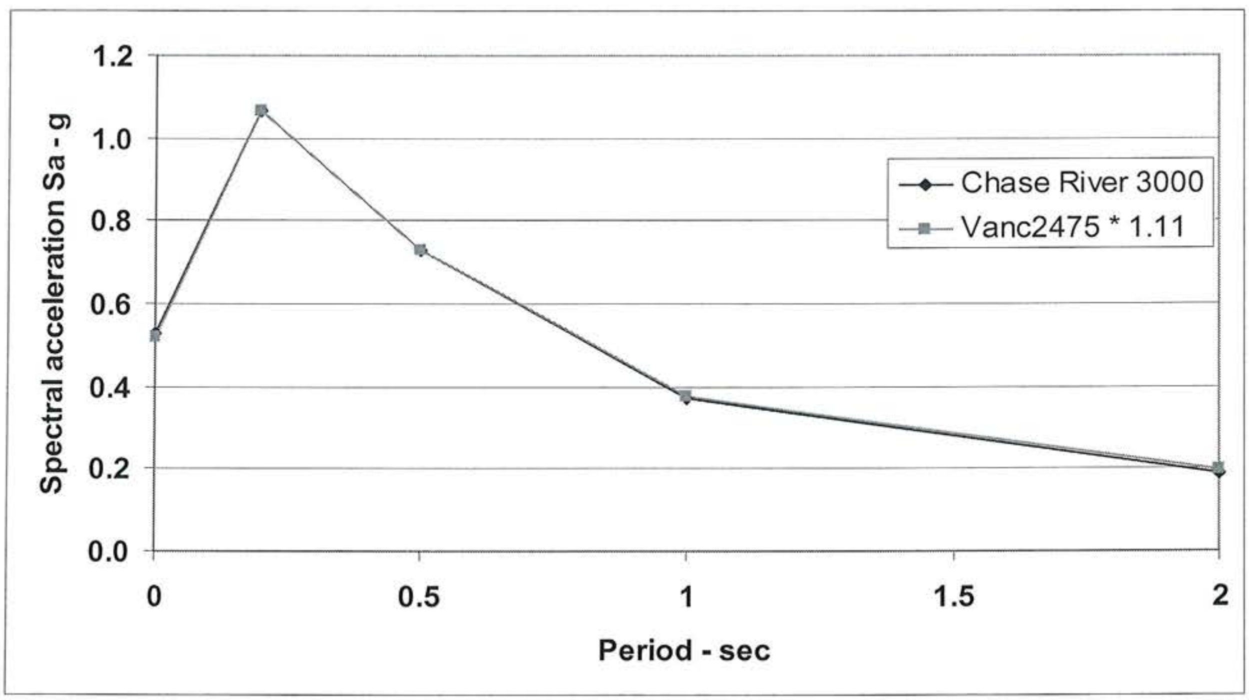

Figure 4. Chase River 3000 year return period hazard compared to the Vancouver 2475 year hazard scaled by the factor 1.11.

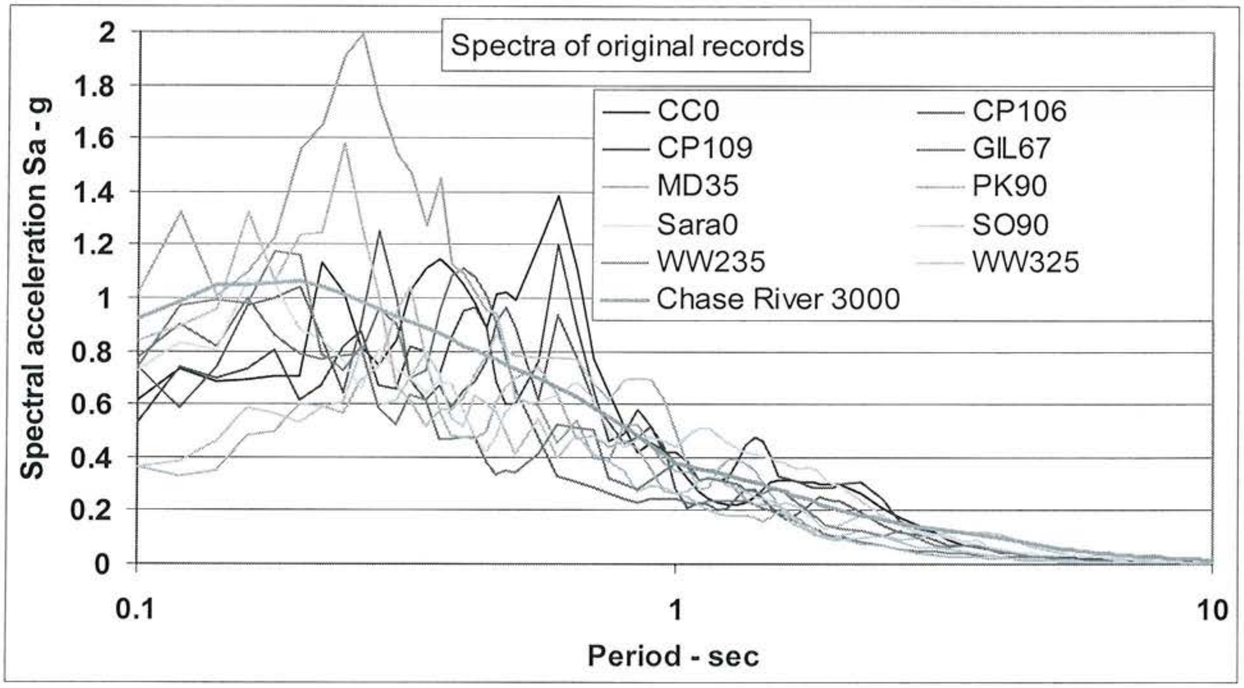

Figure 5. Spectra of the original records and the Chase River 3000 year hazard curve.

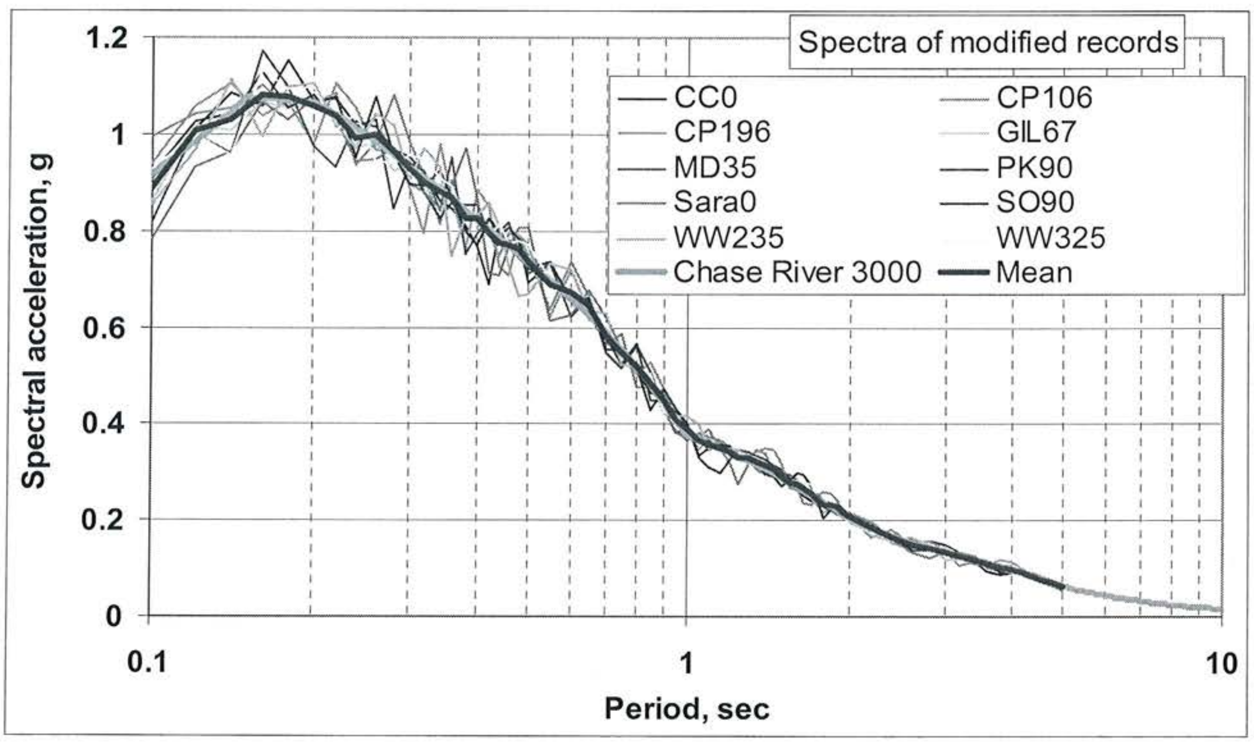

Figure 6. Response spectra of records modified using SYNTH plus their mean value, compared with the Chase River 3000 hazard.

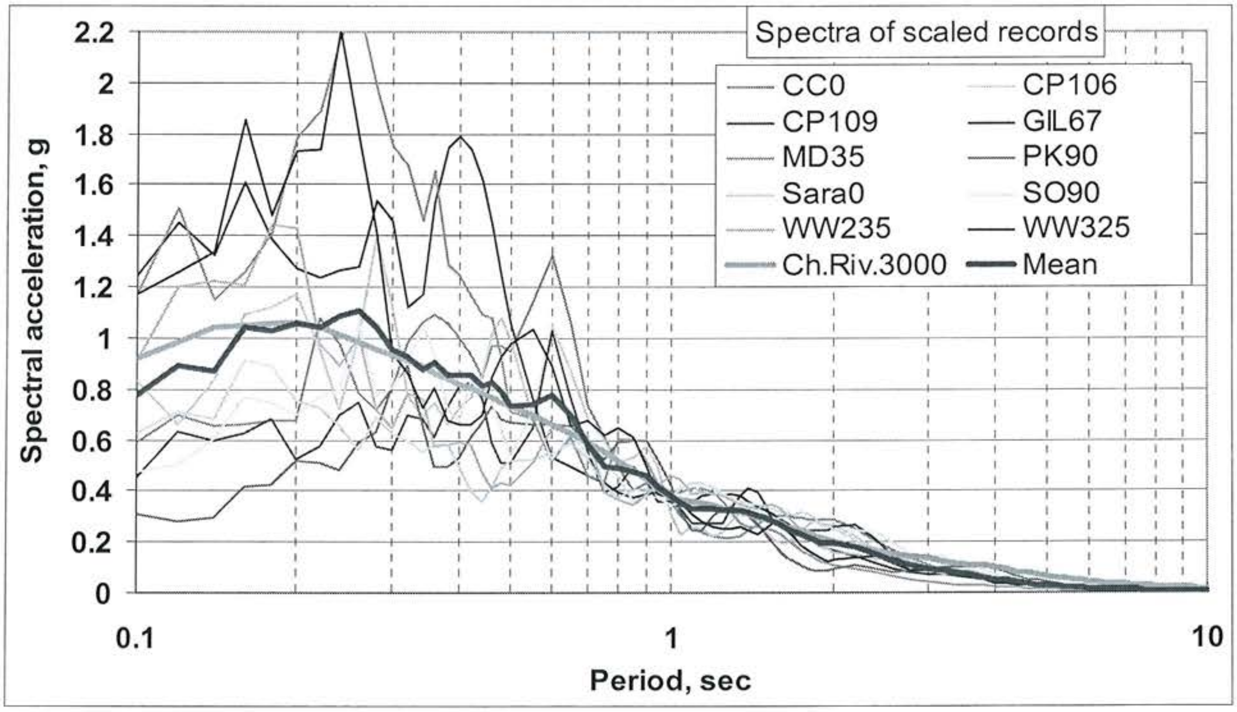

Figure 7. Acceleration response spectra of records scaled to have the same average velocity response spectra as the Chase River 3000 hazard over the period range of 0.5 to 1.5 seconds, plus their mean value and the Chase River 3000 hazard.

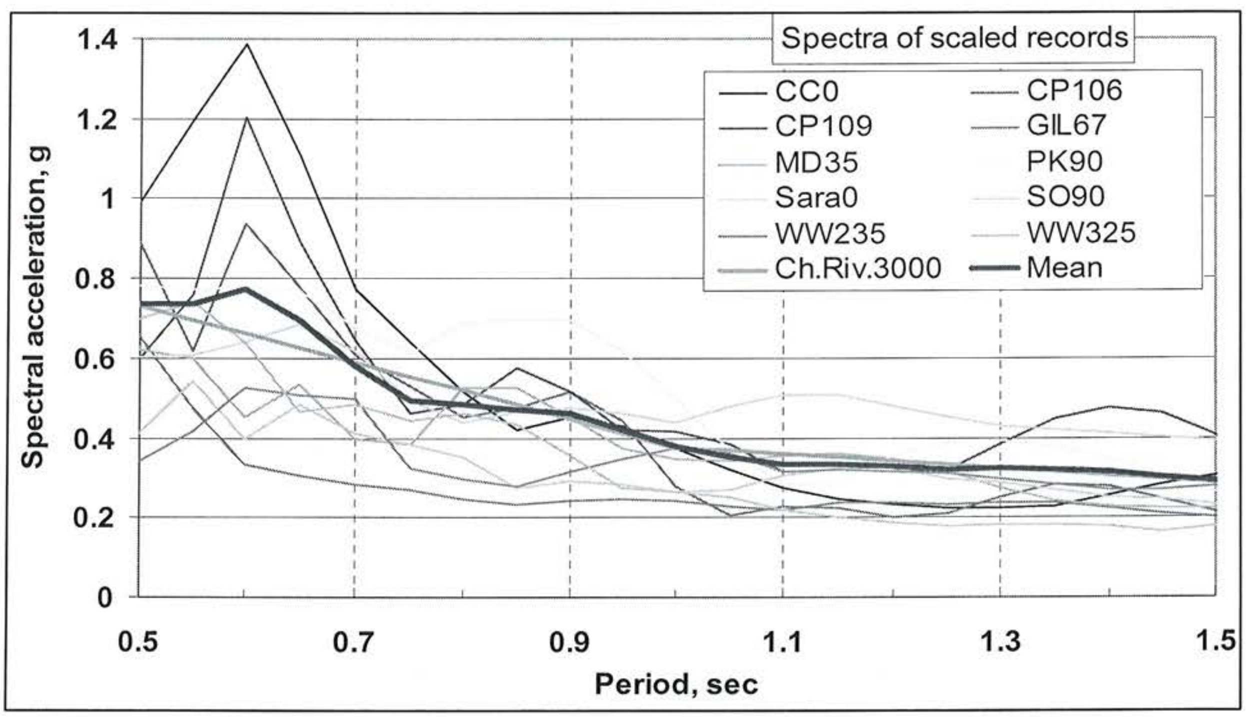

Figure 8. Same plot as Fig. 7 except plotted only for the period range of 0.5 to 1.5 seconds over which the records were scaled.

APPENDIX F

APPENDIX F UNSW PIPING FAILURE RISK ASSESSMENT

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 (Pe), foundation piping (Pf) and piping of the embankment into the foundation (Pef). This includes consideration of whether or not the dam is greater or less than 5 years in age as approximately 2/3 of piping failures (Foster et al, 2000) have been found to occur in the first five years following first filling:
- Calculate weighting factors for each of the aforementioned piping failure modes (we, wf, wef) which take into account dam characteristics such as core materials/properties, compaction and foundation geology as well as 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; and
- Calculate the annual likelihood of piping failure (Pp) using the following formula\
- Pp = Pe x we + Pf x wf + Pef x wef

A drawback of the UNSW method is that is based on a retrospective study that tends to lump together the factors that influence the initiation and progression of piping and breach formation for historical 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 on 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 Middle and Lower Chase Dams will have enough similar characteristics to the population of dams within the database that the findings of the database review are statistically relevant for the purposes of this assessment.

EBA has assumed that the Middle and Lower Chase Dams qualified as an "Rockfill Dam with Core Wall" as presented in Table 1 of the Foster et al, 2000 paper. One key consideration is that there appears to have been no dam within the database with this description that has failed. This may not be due superiority in design concept, but rather due to the limitations of the database. The paper prepared by Foster et al, 2000 presents an assumed average probability of failure for this type of dam. Looking at the Chase River Dams, the following piping failure mechanisms are considered to be credible:

- Middle Chase Dam – continued deterioration or seismic damage of the wooden low level conduit causes an increase in seepage through the downstream shell which accelerates and erodes a large part of the downstream shell. The original rock fill downstream shell has been replaced with a sand and gravel downstream shell in 1980. Loose rock fill has a high degree of erosion resistance compared to sand and gravel. This results in loss of support for the concrete wall which could topple and cause an uncontrolled discharge. The nature and characteristics of the exposed bedrock abutments suggests that piping within the foundation or piping of the embankment into the foundation is much less likely. However, no records are available to confirm this; and

- Lower Chase Dam – the low level conduits have been sealed with concrete and the entrance chamber filled with concrete. Seepage rate increases through or around the wooden low level conduit pipes are not credible given the attention given to these features in 1980. The overburden cover and results of drilling in 1978 suggest that either the concrete wall is founded on rock or dense till. The original rock fill is still in place in the downstream shell which means cracks that form in the wall are less likely to result in leakage that causes initiation of piping. There are limited bedrock exposures near Lower Chase Dam which does not support any comment on the relative likelihood of piping within the foundation or piping from the embankment into the foundation.

Based on the aforementioned discussion, the UNSW method is viewed to be more applicable to Middle Chase Dam than Upper Chase Dam. Therefore, the results for Lower Chase Dam may not be reliable but are presented in Figure 8 and discussed in the body of this report for comparative purposes.

The average annual probabilities presented in table F1 were selected from the Foster et al 2000 study and the weighting factors were calculated using the descriptors presented in the same paper. The tabulated weighting factors are presented as follows.

TABLE F1: MIDDLE CHASE DAM CALCULATION OF ANNUAL LIKLIHOOD OF PIPING FAILURE					
Piping Failure Mode	Average Annual Probability of Failure (before seismic event i.e. "age" > 5 yrs)	Average Annual Probability of Failure (after seismic event i.e. "age"< 5 yrs)	Overall Weighting Factor	Weighted Likelihood of Piping Failure Before Seismic Event	Weighted Likelihood of Piping Failure After Seismic Event
Piping through embankment	1.30E-05	1.30E-04	0.72	9.36E-06	9.36E-05
Piping through foundation	1.90E-05	2.55E-04	0.13	2.55E-06	3.43E-05
Piping from embankment into foundation	4.00E-06	1.90E-05	1.65	6.59E-06	3.13E-05
			SUM	**1.85E-05**	**1.59E-04**

CALCULATION OF WE		
Embankment filter	0.2	Downstream shell fill unknown self filtering capacity
Core geologic origin	1	Not applicable, factor of 1.0 does not modify result
Core soil type	1	Not applicable, factor of 1.0 does not modify result
Compaction	0.5	Downstream shell compacted and tested, no test results available
Conduits	5	Conduit through core wall, anticipated to be in very poor shape
Foundation treatment	1.2	Irregularities in foundation or abutment, steep faces (untreated vertical faces or overhangs gives factor of 2)
Observations of seepage	1	Leakage clear and steady, has increased and decreased in early 1990s
Monitoring and surveillance	1.2	Monthly inspections conducted by City

CALCULATION OF WF		
Filters	0.8	No foundation filter, not likely one required, sand and gravel will provide some filtering
Foundation below Cut Off	0.2	Expect good rock foundation
Cutoff (soil foundation)	1	Not applicable, factor of 1.0 does not modify result
Cutoff (rock foundation)	1	Assume average or not required
Soil geology (below cutoff)	1	Not applicable, factor of 1.0 does not modify result
Rock geology (below cutoff)	0.7	Sandstone, conglomerate foundation
Observations of seepage	1	Leakage clear and steady, has increased and decreased in early 1990s
Observations or pore pressures	1	None available, no modification of rating
Monitoring and surveillance	1.2	Monthly inspections conducted by City

CALCULATION OF WEF		
Filters	1	Core or shell won't pipe into foundation
Foundation cutoff	0.8	Shallow or no cut off trench
Foundation	1.5	Founded on rock
Erosion control measures in fndn	1	Core foundation not an issue for concrete
Grouting?	1.3	No grouting
Soil geology	1	No soil beneath core
Rock geology	0.8	Sandstone, conglomerate
Core geological origin	1	Core is concrete, factor of 1 does not modify result
Core soil type	1	Core is concrete, factor of 1 does not modify result
Core compaction	1	Core is concrete, factor of 1 does not modify result
Foundation treatment	1.1	Irregularities in foundation or abutment, steep abutments
Observations of seepage	1	Leakage clear and steady, has increased and decreased in early 1990s
Monitoring and surveillance	1.2	Monthly inspections conducted by City

TABLE F2: LOWER CHASE DAM CALCULATION OF ANNUAL LIKLIHOOD OF PIPING FAILURE					
Piping Failure Mode	Average Annual Probability of Failure (before seismic event i.e. "age" > 5 yrs)	Average Annual Probability of Failure (after seismic event i.e. "age"< 5 yrs)	Overall Weighting Factor	Weighted Likelihood of Piping Failure Before Seismic Event	Weighted Likelihood of Piping Failure After Seismic Event
Piping through embankment	1.30E-05	1.30E-04	1.44	1.87E-05	1.87E-04
Piping through foundation	1.90E-05	2.55E-04	0.20	3.83E-06	5.14E-05
Piping from embankment into foundation	4.00E-06	1.90E-05	2.06	8.24E-06	3.91E-05
			SUM	**3.08E-05**	**2.78E-04**

CALCULATION OF WE		
Embankment filter	1	If core wall on soil, need a filter which isn't likely present
Core geologic origin	1	Not applicable, factor of 1.0 does not modify result
Core soil type	1	Not applicable, factor of 1.0 does not modify result
Compaction	0.5	No compaction in majority of shells
Conduits	2	Conduit through embankment and core wall, some poor details
Foundation treatment	1.2	Assume irregularities in foundation or abutment, steep faces (untreated vertical faces or overhangs gives factor of 2)
Observations of seepage	1	Leakage clear and steady
Monitoring and surveillance	1.2	Monthly inspections conducted by City

CALCULATION OF WF		
Filters	1.2	No foundation filter
Foundation below Cut Off	0.2	Could be soil
Cutoff (soil foundation)	1	Shallow or no cut off trench
Cutoff (rock foundation)	1	Assume soil foundation
Soil geology (below cutoff)	1	Glacial till possibly beneath core wall
Rock geology (below cutoff)\	0.7	Assume soil foundation
Observations of seepage	1	Leakage clear and steady, has increased and decreased in early 1990s
Observations or pore pressures	1	None available, no modification of rating
Monitoring and surveillance	1.2	Monthly inspections conducted by City

CALCULATION OF WEF		
Filters	1	Core or shell won't pipe into foundation
Foundation cutoff	1	Shallow or no cut off trench
Foundation	1.5	Founded on or partly on rock (abutments)
Erosion control measures in fndn	1	Core foundation not an issue for concrete
Grouting?	1.3	No grouting
Soil geology	1	No soil beneath core
Rock geology	0.8	Sandstone, conglomerate
Core geological origin	1	Core is concrete, factor of 1 does not modify result
Core soil type	1	Core is concrete, factor of 1 does not modify result
Core compaction	1	Core is concrete, factor of 1 does not modify result
Foundation treatment	1.1	Irregularities in foundation or abutment, steep abutments
Observations of seepage	1	Leakage clear and steady, has increased and decreased in early 1990s
Monitoring and surveillance	1.2	Monthly inspections conducted by City

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

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

	Embankment			Foundation			Embankment into foundation		
		Average annual P_e ($\times 10^{-6}$)			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.

Factor*	General factors influencing likelihood of failure: 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.

Factor*	General factors influencing likelihood of failure				
	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.

Factor*	General factors influencing likelihood of initiation of piping: 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 $w_{E(cc)} = 1.0$, as their compaction method has already been taken into account by the zoning.

Conduits $w_{E(con)}$

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

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

Foundation treatment $w_{E(ft)}$

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

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

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

Piping through the foundation (Table 3)

Foundation filters $w_{F(filt)}$

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

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

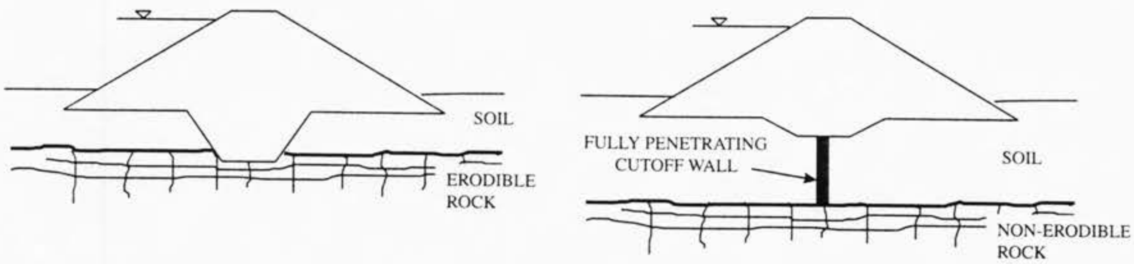
The three categories of foundation below the "cutoff" of the dam are soil foundations; erodible rock foundations, with erodible materials present such as clay-filled joints or infilled karstic channels; and non-erodible rock foundations. The cutoff is either a cutoff trench or a sheetpile or slurry trench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

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

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

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

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



FOUNDATION TYPE (below cutoff) = ERODIBLE ROCK

FOUNDATION TYPE (below cutoff) = NON-ERODIBLE ROCK

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

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

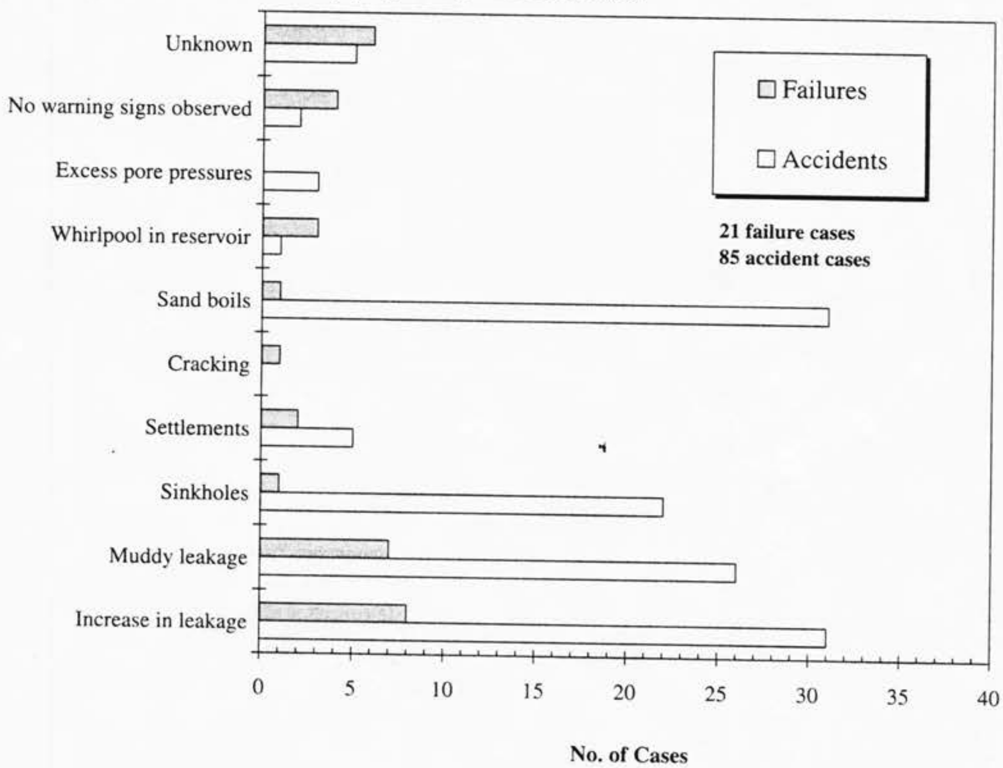
Figure 3 summarizes the observations during incidents of piping through the foundation. Increases in leakage and muddy leakage were commonly observed during both failure and accident foundation piping cases. Sinkholes and sand boils were frequently observed in the accident cases, but rarely in the failure 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.

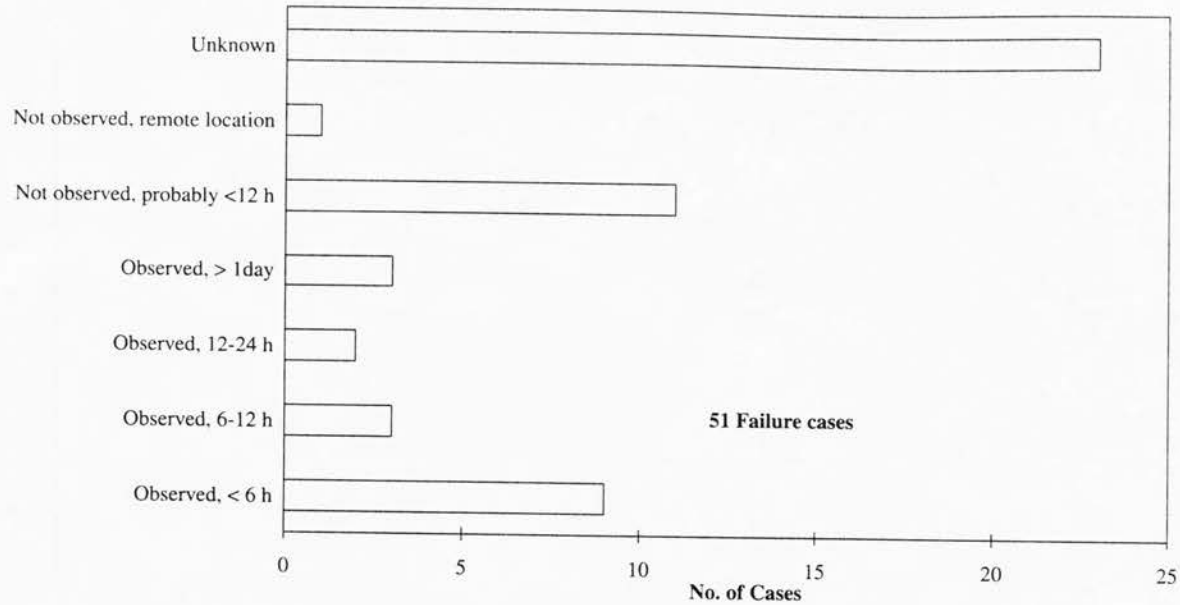


ping from embankment to foundation

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.

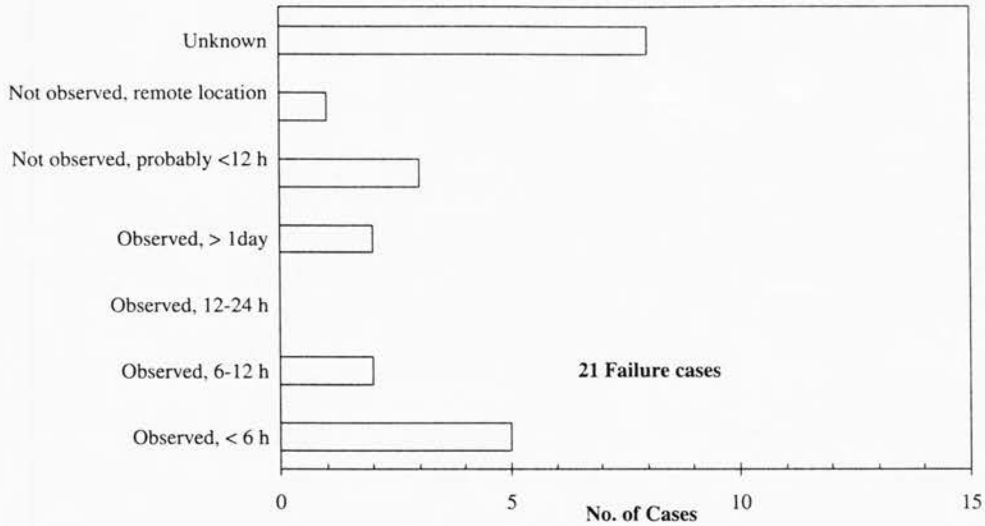
Descriptions of the observations leading up to and during the piping incidents for all of the failure cases and for a select group of accident cases are given in Appendix 1. It is evident that in a few of the failure cases the dams were poorly maintained and remedial work was not carried out despite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

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Appendix 1. Descriptions of warnings of piping failures and selected accidents.

This appendix is made up of six tables outlining the descriptions of warnings of piping failures and selected accidents.

Table A1. *Descriptions of warnings of failures resulting from piping through the embankment.*

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	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	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	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 A. [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	Warning: 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	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	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	Warning: 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