BC HYDRO
WAC BENNETT DAM
EXPERT ENGINEERING PANEL
REPORT

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Robin Fell

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EXECUTIVE SUMMARY

1. EXPERT ENGINEERING PANEL (EEP)

The Expert Engineering Panel was first appointed in 2011 to make an independent interpretation of the seepage control functions of the WAC Bennett Dam.

The members of the Expert Engineering Panel are Dr Kaare Hoeg, Norwegian Geotechnical Institute, Emeritus Professor Robin Fell, University of New South Wales, Australia, and Mr Rodney Bridle, Dam Safety Ltd, United Kingdom. The Panel visited BC Hydro in Vancouver on three occasions, in February and June 2011, and in February-March 2012. Both the 2011 visits included visits to the WAC Bennett Dam site to inspect the dam and meet the dam safety staff on site and become familiar with the dam and the monitoring systems.

The Panel reported in August 2012 on the satisfactory condition of the dam and the actions taken by BC Hydro to maintain and improve it, and made several recommendations on further investigations that should be taken to confirm its condition. An Executive Summary of the report was issued in February 2013.

The Panel was re-appointed in 2015 to give a further independent assessment of the seepage flow control functions of the Bennett Dam after receiving reports and presentations from BC Hydro on the studies and investigations recommended by the Panel in 2012.

2. INDEPENDENT ASSESSMENT

The extensive work performed by BC Hydro during the past 3 years has verified the conclusions drawn by the Panel in the EEP Report of August 2012. The further tests have confirmed that even in circumstances where fines have been locally washed out of the filtering layers or there was material segregation at the time of construction, internal erosion will be arrested. A possible exception to the capability of the fill materials to ensure filter protection occurs at the crest of the dam, as discussed below.

The stability of the dam during earthquakes has been examined using 2D dynamic analyses based on an earthquake with annual probability of occurrence of 1/10,000. This Safety Evaluation Earthquake (SEE) generates a horizontal Peak Ground Acceleration of 0.16g and is assumed to have a very long duration. The dam will be stable during and after this seismic loading. The alluvium in the foundation and in the fill in the cofferdam at the upstream toe will tend to liquefy, but this will not affect the stability of the upstream dam slope. Moderate differential settlements are expected above the canyon walls and around the Sinkholes. This may result in some transverse cracking, especially in the vicinity of Sinkhole 1 due to its location at the edge of the canyon and the relatively low density of the materials around the sinkhole. Consideration should be given to assessing the feasibility of densifying or replacing the disturbed Core and Transition materials at the sinkholes to reduce the settlements and make these areas behave more consistently with the remainder of the dam under seismic loads.

The Filter and Drain are omitted in the top part of the dam, and the Transition alone must provide filter protection and adequate drainage capacity at these upper levels. As mentioned above, cracks may form across the crest as a result of differential settlement at Sinkhole 1 during severe earthquakes. Such cracks may remain open on wetting for long enough to allow the flow of water to erode the walls of the cracks. Further tests are recommended to examine the rapidity with which the cracks close on wetting. If closure is too slow, it may be necessary to provide mitigating measures. The risk may be substantially lowered by raising the Filter and the Drain to top of Core level.

The upper part of the spillway wall is significantly steeper than the lower part, and a crack may open when the embankment fill settles during an earthquake. BC Hydro should study the conditions at the interface between the spillway and the embankment in more detail and evaluate the likelihood of cracking and the potential need for improving the downstream filtering action at this location.

The project to restore rip-rap protection against wave erosion on the upstream face of the dam is in hand. To ensure high standards, the new rip-rap will be placed ‘in the dry’ when reservoir water levels are low. Depending on water levels, it is planned to complete the work over three seasons 2017-2019.

The Observation Wells, which were previously of some concern, have been successfully grouted up. Piezometer tubes and optic fibres are being installed in some to provide further facilities for monitoring. The high-quality cross-hole tomography monitoring continues and shows no changes in the measured shear wave velocities. Advice has been taken on further improvements in the analysis and interpretation of the results, and work is in hand on improving the network of monitoring boreholes as recommended in 2012.

Further seepage analyses are recommended to provide a basis for monitoring now that close to steady state seepage conditions appear to have been reached. This follows the expulsion from the fill of the large quantities of air that probably led to the high initial pore water pressures that were measured in the dam. Further analyses are recommended to identify areas where hydraulic fracture may have occurred soon after impoundment.

1 INTRODUCTION

1.1 EXECUTIVE SUMMARY OF EXPERT ENGINEERING PANEL'S 2012 REPORT (ISSUED IN FEBRUARY 2013)

The situation following the EEP's work in 2011 and 2012 was summarised in the 2013 Executive Summary, as follows:

1. The dam was well designed for the time it was constructed, and the extensive construction control testing indicates it was well constructed.

2. The standard of monitoring of the dam is extremely high, and those involved clearly understand the dam and its performance.

3. The dam has a good seepage and internal erosion control system consisting of the Transition, the Filter and Drain to maintain the integrity of the dam. It may allow a small amount of future erosion at the Core/Transition interface, but from the available information, will prevent on-going erosion. There are no situations where erosion after initiation could continue unchecked. The EEP has made some suggestions for future investigations by BC Hydro to further confirm this assessment.

4. The Drain has a large capacity to discharge leaks resulting from any future internal erosion and to prevent instability of the dam. However, near the crest of the dam, the drainage capacity is less as there is no Drain, and there is no Filter in the upper part, only the Transition zone.

5. Sinkhole 1 and Sinkhole 2 (remediated in 1996) are directly related to the benchmarks, the lightly compacted Core fill around the benchmark tubes, their proximity to the canyon walls, and the irregular bedrock topography. Sinkhole 1 may also be related to its proximity to Splitter Dyke 2. Those conditions do not exist elsewhere in the dam. The instrument Risers constructed on Instrument Planes 1 and 2 for construction year 1966 have similar lightly compacted Core fill surrounding them, but there is no evidence of cavity formation or sinkhole development or other changes that give reasons for concern.

Recommendations:

1. The Observation Wells (OWs) are important for cross-hole seismic monitoring to identify any further loosening of the lightly compacted Core fill around the Observation Wells and the Risers. The EEP recommends that the Observation Wells be sealed on the inside by grouting, but first a small diameter casing should be installed inside the well which would allow cross-hole seismic measurements to continue. The EEP concludes that it is not warranted to try to further compact the soil surrounding the OWs.

2. The EEP concludes that it is not warranted to attempt to densify the Core surrounding the Risers, and that it is sufficient to continue to monitor by cross-hole seismic measurements at annual intervals. The ability to do cross-hole seismic measurements between Cross Arm 1 and OW2 should be restored, or a new hole drilled in the Core if the blockage of Cross Arm 1 cannot be removed.

3. The cross-hole monitoring at Instrument Plane 2 (IP2) involves a long distance between source and receiver. Because of this the velocities are dominated by Core which is not affected by the lower velocity material around the Riser. It is recommended that a new casing be installed close

to the Riser area to allow readings to be obtained which give the required degree of confidence that any tendency to the development of a cavity will be detected.

4. Seismic stability investigations using the new seismic hazard assessment are needed. The upper part of the dam may be vulnerable to damage during seismic events, and there is liquefaction potential of a 50-ft deep scour hole filled with sands, gravels and boulders over the upstream third of the canyon floor.

5. The EEP notes that preparations are being made to repair the rip-rap on the upper upstream slope of the dam. This upper part of the dam may also be vulnerable to internal erosion through concentrated leaks in cracks when the reservoir level is high. The EEP recommends that the three issues, rip-rap repair, seismic resistance, and vulnerability to cracking and internal erosion be considered simultaneously to produce a solution to works at the upper part of the dam that addresses all risks there, including the risks that will arise during the construction phase.

6. The EEP recommends that the following investigations be carried out to further confirm that the filter and drain system is effective:
    a. Carry out laboratory experiments on representative samples of the Core and Transition to confirm that the Transition will in all situations arrest erosion.
    b. Carry out investigations to determine whether the Transition in the upper part of the dam may hold a crack due to high fines content and possible cementation caused by a significant content of carbonate rock.
    c. Assess the drainage capacity in the upper part of the dam to assess whether there is sufficient capacity to cope with any foreseeable concentrated leak or other internal erosion scenario.

7. The following investigations and analyses are desirable to better understand the behaviour of the dam:
    a. Complete the characterization of the dam materials and the foundation and use these data as the base properties in improved seepage analyses.
    b. Further improvement in cross-hole monitoring methodology and interpretation of measurements.
    c. Further investigate the air occlusion and ex-solution theory to explain the pore pressure development in the dam since first impoundment.
    d. Complete the 2D numerical analysis of stresses and strains in the dam, especially in the vicinity of the canyon walls and shoulders.

1.2 TERMS OF REFERENCE 2015

The 2015 Terms of Reference reflect BC Hydro’s adoption of a “risk-informed” approach to the management of the safety of its dams and for dam safety decision-making. The Terms of Reference state that the success of the risk-informed approach is critically dependent on the integrity of the information that informs the decision and risk management process. Ultimately the success of the decision and risk management process depends on:

- The robustness of BC Hydro’s understanding of the performance characteristics of the dam as influenced by the characteristics of the core, filter and transition, and drains (including the influence of monitoring arrangements), and,
- How BC Hydro senior management uses this understanding to inform on-going and future risk management decisions and activities.

Within the context of this risk-informed decision-making process, the EEP will provide advice to the Director of Dam Safety, Stephen Rigbey, as follows:

- Provide an independent assessment of the seepage flow control functions of the Bennett and Mica embankment dams.

- Comment on the completeness and reasonableness of the understanding of the dam performance, after receiving information on the past and current studies, engineering analysis and interpretation activities.

- Provide an independent opinion on the state of the scientific knowledge of the analysis of the mechanics of the internal erosion process and the appropriateness of the use of any or all of this knowledge in informing risk management decisions.

- Provide an independent opinion on the localised stability around features of interest that have been identified either from records or which can be inferred from data

- Recommend additional investigations and analysis in the event of scientific weaknesses or inadequacies in the current state of understanding of the performance of the dam.

- Recommend on future instrumentation/surveillance requirements for the dam and highlight when certain proactive and/or reactive response approaches could be implemented.

Some aspects of the Terms of Reference were covered in the 2012 EEP Report and will not be repeated here, but all items will be addressed in the future review of Mica Dam.

In this report, the EEP will be dealing with BC Hydro's responses to recommendations in their 2012 report on Bennett Dam. BC Hydro provided numerous reports (listed in Appendix A) and made many presentations to the EEP during its visit to BC Hydro's offices in Burnaby from 16 to 20 November 2015.

2 ASSESSMENT OF SEEPAGE CONTROL FUNCTIONS OF BENNETT EMBANKMENT DAM

2.1 INTRODUCTION

In the EEP 2012 report there were a number of issues which the EEP recommended be further investigated. These included:

- Analyses of the particle size distribution tests taken during construction and carry out laboratory experiments on representative samples of the Core and Transition to confirm that the Transition will in all situations arrest erosion.
- Investigations to determine whether the Transition in the upper part of the dam may hold a crack due to high fines content and possible cementation caused by a significant content of carbonate rock.
- Assessing the drainage capacity in the upper part of the dam to assess whether there is sufficient capacity to cope with any foreseeable concentrated leak or other internal erosion scenario.

The EEP also recommended that in designing the planned rip-rap repair, seismic resistance and vulnerability to cracking and internal erosion be considered simultaneously to produce a solution to works at the upper part of the dam that addresses all risks there, including the risks that will arise during the construction phase.

Since 2012 BC Hydro has investigated these matters and the results were available for review as part of this meeting. The following are the EEP's comments on what has been done.

2.2 RIP-RAP REPAIR

In 2012, the Panel was concerned that progressive failure through the crest may be initiated by wave erosion on the area on the upstream slope where the rip-rap had been damaged and removed in part by wave erosion during a high wind event. A project to replace rip-rap is now progressing. The rip-rap along much of the upper part of the upstream slope including a 9-ft wave run-up zone will be replaced as it was found that the original rip-rap was undersized. A quarry has been selected and design is complete. Documents were recently issued which will lead to the appointment of a Contractor to complete the work 'in the dry' (when water level is low) over three seasons from 2017-2019. It is a strict requirement of the specification that rip-rap must be placed on all prepared surfaces at the end of each construction period to prevent erosion of unprotected surfaces.

The Panel have been shown an extract of the specification and drawings of the rip-rap contract. The EEP was concerned that all loose existing material be removed and replaced before the new bedding and rip-rap is placed, and that any remaining loose Zone 5 is compacted before the Bedding Layer is placed so that Zone 5 will not be contractive and potentially lose strength under seismic loads. Also, that bedding should be placed at the interface with dam fill at the key trench above the toe berm (see Drawing 1006-C02-01287). If bedding is omitted, the underlying dam fill may be drawn up through the rip-rap by wave action. New rip-rap will be placed in the immediate vicinity of Sinkhole 1, but not at Sinkhole 2, which was repaired in 1996/97. Great care will be taken to conserve, and provide access to, the instrumentation in those areas throughout the rip-rap construction period.

There is a possibility that stripping of the existing rip-rap might destabilize the upper slopes and measures will need to be in place to manage such a situation.

2.3 REVIEW OF DAM FEATURES AND PROPERTIES IN RELATION TO SEEPAGE CONTROL AND INTERNAL EROSION

2.3.1 Properties of fill materials related to seepage and internal erosion

BC Hydro has completed the characterisation of the materials in the embankment. This is presented in Report E 1138, WAC Bennett Dam, Site Characterization Report, September 2014.

The report presents a summary of the available information and the BC Hydro assessment of the properties of the embankment materials. The following comments relate to the permeability of the fill materials.

2.3.2 Permeability of the core

Information relating to the permeability of the core is summarized in the Site Characterisation Report:

- Section 3.1.4, In-situ Permeability Tests.
- Section 3.2.5, (Laboratory) Permeability Testing.
- Section 5.5.1, Zone 1 Core.
- Table 5-18 presents the assessed “Current State Dam Fill Parameters.

.The permeabilities in the Site Characterization Report are as summarized in Table 2.1. The Site Characterization report indicates that the adopted permeabilities are the geometric mean of the laboratory data.

Also shown for comparison are the permeabilities assessed by the EEP in the 2012 report.

It can be seen from Table 2.1 that the assessed Site Characterization Report horizontal permeabilities (K_h) are much lower than estimated by the EEP, as are some of the K_h/K_v ratios. The reasons for this seem to be:

(a) The Site Characterization Report assessment seems to not take account of the rapid response of the piezometers in the core to rising reservoir levels on first filling. These were analysed in 2012 by BC Hydro at the request of the EEP and resulted in a permeability of $K_h = 5x10^{-6}$ m/sec. A more simplified analysis by the EEP gave $K_h > 1x10^{-6}$ m/sec.

(b) The marked dependence of saturated permeability on the degree of saturation of the Core when compacted shown in the Laval University testing (Watabe et al, 2000) and the construction data on degree of saturation as shown in Appendix E of the EEP Report. This was taken into account by the EEP and has not been accounted for in the Site Characterization Report. As a result, the Site Characterization Report possibly underestimates the effects of the layering with different degrees of saturation and hence permeability of the Core as it was placed and hence the estimated K_h/K_v ratio in the early years of construction.

It is noted that the vertical permeabilities (K_v) in the Site Characterization Report are similar to the EEP Report values.

The EEP suggests that BC Hydro review the permeabilities and K_h/K_v ratios taking into account the comments above.

Table 2.1. Permeability of the Core as assessed in the Site Characterization Report, and EEP 2012 Report.

Material	Year	Site Characterization Report		EEP Report, 2012	
		K_h, m/sec	K_h/K_v	K_h, m/sec	K_h/K_v
Core (Zone 1)	1964	$5x10^{-8}$	5	$2x10^{-6}$	100
	1965	$1x10^{-7}$	5	$2x10^{-6}$	100
	1966	$5x10^{-7}$	5	$2x10^{-6}$ (a) $4x10^{-6}$ (b)	10 2
	1967	$9x10^{-7}$	5	$4x10^{-6}$	2
Abutment Contact		$1x10^{-6}$		Not estimated	
Winter Horizon	1965	$1x10^{-6}$	5	$2x10^{-6}$	100
Piezo. Lead Trench		$1x10^{-6}$	5		
BM/SH 1		$1x10^{-6}$	5	$2x10^{-5}$	Not estimated
BM/SH 2		$1x10^{-6}$	5	$2x10^{-5}$	Not estimated
OW's		No change	5		
IP1		No change	5		
IP2		No change	5		

Note. (a) For first half of season, up to end of July. (b) Second half of season after 1st August.

2.3.3 Permeability of other zones

The assessed permeabilities of the other zones are summarized in Table 5-18 of the Site Characterization Report. Note 3 in Table 5-18 says the permeabilities are the geo-mean of the laboratory tests and consistent with Morgan and Harris (1967). The Zone 2, 3 and 6 fills had two values, those in parentheses being post winter 1964/1965.

A check on the permeability of the Transition was carried out using the range of particle size distributions from the construction records, and using the Sherard et al (1984) method. Sherard et al (1984) carried out laboratory permeability tests on 15 different filter sands and sandy gravels with D_{15} in the range from 0.1 to 10 mm. The materials were mostly broadly graded and with little or no fines. From these tests they concluded that the permeability could be estimated from:

$$k = C (D_{15})^2$$

Where: k = permeability in metres / sec, D_{15} = particle size for which 15% is finer in mm, C = 0.0035 with a range of median test values of C = 0.002 to C = 0.006 for densely compacted filters (70% relative density). Sherard et al. (1984) found that the permeability of the same materials placed very loose were higher by a ratio in the range 1.1 to 3.0 but up to 13.7 for a very broadly graded silt sand gravel soil.

Another source of data on the permeability of the Transition is the multistage permeameter tests carried out by UBC.

Table 2.2 summarizes the estimated permeabilities.

Table 2.2 Horizontal permeability (Kh) of the Transition

Site Characterization Report m/sec	UBC Multistage permeameter Tests m/sec	Estimates using Sherard et al (1984) m/sec	Suggested values
$1x10^{-6}$ 1964 Kh/Kv =3	$1.3x10^{-5}$ and $1.6x10^{-6}$ at start of test, generally reduced with time	$1.4x10^{-4}$ to $5.6x10^{-4}$ All years	$1x10^{-5}$, Kh/Kv =10
$1x10^{-5}$ post 1964/1965 winter Kh/Kv =3			$5x10^{-5}$, Kh/Kv =5

The EEP's experience is that the Sherard et al (1984) method generally gives reasonable values, so the Site Characterization Report values seem low. Suggested values for consideration by BC Hydro are shown. If the permeability of the Transition above EL 2190-ft becomes critical in the seepage analyses, it is suggested some in-situ tests be carried out.

Further insights into the permeability of the fills during first filling and later may be gained from the 'air theory' investigations recommended in the 2012 EEP Report and in 3.2.2 below.

2.4 PERFORMANCE OF THE FILTER AND DRAINAGE SYSTEM TO CONTROL INTERNAL EROSION

2.4.1 The Bennett Dam filter and drainage system

Figures 2 and 3 (in the EEP 2012 Report) show the zoning at Instrument Planes 1 and 2 (IP1, IP2). From these it can be seen that the filter system as constructed for Bennett Dam consists of:

Transition (Zone 2) which is a broadly graded gravelly sand and sandy gravel, with generally between 2% and 8% fines passing 0.075 mm.

Filter (Zone 3) which is sandy gravel with generally less than 5% fines passing 0.075 mm.

Drain (Zone 4) which is gravel with some to a trace of sand and less than 2% fines passing 0.075 mm.

The Transition may be considered as a fine filter, and the Filter as a coarse filter. The Transition, Filter and Drain all contribute to form the drainage system.

The critical elevations for internal erosion are:

Dam Crest	2230-ft to 2234-ft
Top of Core	2220-ft
Top of Filter	2190-ft
Top of Drain/change in core section	2160-ft
Normal Maximum reservoir level	2205-ft
Normal Minimum reservoir level	2147-ft
PMF Flood Level, current operating rules	2207-ft spring flood; 2208-ft summer flood.

Above EL 2190-ft, the Transition is the single line of filter defence. From EL 2190-ft to EL 2160-ft the Filter provides the bulk of the drainage capacity. Below that the Drain is the primary drainage zone.

The filter system also consists of a horizontal Drain layer placed on the rock foundation (the blanket Drain), and a Transition layer on top of the Drain to control erosion of the Random Shell Zone 6 Fill into it.

The function of the filter system is to control erosion of the Core, and to provide drainage for seepage or any foreseeable leaks through the Core and the foundation.

As recognised by BC Hydro the ability of the Transition and Filter to arrest erosion in the Core is affected not only by the gradation as placed, but also by the potential for suffusion to occur under leakage flows with selective removal of part of, or the entire, finer fraction resulting in a coarser gradation. It is also affected by segregation which may occur during placement of the fill in the embankment.

2.4.2 Laboratory testing carried out to assess suffusion of the transition.

Laboratory tests to assess the loss of material from the Transition by suffusion have been carried out by University of British Columbia (UBC) under the direction of Professor Jonathan Fannin. These are reported in:

> WAC Bennett Dam, Embankment Dam Project, Assessment of the filter capability of the filter system, Appendix C, WAC Bennett Dam, Multi-stage Permeameter (MSP) Tests on Transition Materials. March 2014.

Two soils representative of the coarser Transition materials in the dam were tested. The gradations were modified between 25 mm and the maximum size of 37.5 mm by replacing the plus 25 mm size with an equal proportion of 25 mm to 37.5 mm particles. The soil was supported on a 2.76 mm square mesh.

The samples were subject to gradually increasing gradients in the permeameter beginning at a gradient of 1.3 and increasing to a gradient of 22. The particle size distributions of the layers of soil in the permeameter were determined after the test. The erosion of the soil was observed but the eroded soil was not collected for weighing or particle size distribution.

The finer of the samples (E2-1065) showed turbid outflow at gradient 1.3, and again at gradient up to 15. The particle size distributions (PSD's) of the test sample showed only minor changes during the test and it seems only a small amount of erosion occurred.

The coarser sample (E2-1059) showed turbid outflow at gradient 1.0, but no further turbidity up to gradients of 17. Further turbid outflow was observed when the gradient was increased to 30. The PSD's showed an accumulation of finer particles on the middle layer, and some erosion of the bottom layer. The D_{15} of the material in the bottom layer was about 0.5 mm after the test, compared to the original soil D_{15} of 0.4 mm; that is only a minor change.

The following are some comments on the tests:

(a) Given that turbid water flowed from the samples at gradients of only 1.0 and 1.3 indicates that the erosion process was suffusion.

(b) The small amount of erosion experienced in these tests is likely related to the size mesh used at the base of the sample. A mesh size of 2.76 mm is equivalent to a D_{15} size of 25 mm based upon the relationship developed by Sherard Dunnigan and Talbot (1984) and confirmed by Foster and Fell (2001) that the equivalent opening size of a filtering material can be approximated by D_{15} /9. The D_{15} of the continuing erosion filter for sample E2-1065 is 32 mm and for sample E2-1059 17 mm, so the mesh is a continuing erosion size for sample E2-1059, but finer than a continuing erosion filter for sample E2-1065.

(c) Recent testing at UNSW has shown that the erosion behaviour of soils similar to the soils tested by UBC is very dependent on the base mesh size. Some soils which did not erode significantly with a base mesh 1.2 to 1.6 times the CE (continuing erosion) size eroded a large amount and very rapidly for a mesh size 3 times the CE size.

(d) Above EL 2190-ft the Transition does not have the Filter downstream of it to limit the transport of the coarser particles under seepage flows so the behaviour may be better represented by MSP tests using coarser base meshes. In view of this, it is recommended that further tests be carried out on these soils with base mesh sizes of 4.75 mm and 9.5 mm. For these tests the eroded material should be collected, dried and weighed, and PSD's carried out on the eroded soil, as well as post-test PSD's on the layers of soil in the permeameter.

(e) From the UNSW tests the Transition soils most susceptible to internal erosion will not necessarily be the coarsest and more tests should be carried out on soils representing the finer, straighter line gradations. These should also be done on 2.76 mm, 4.76 mm and 9.5 mm base meshes.

2.4.3 Laboratory testing carried out to assess the performance of the Transition as a filter to the Core.

Laboratory tests to assess the performance of the Transition as a filter have been carried out by University of British Columbia (UBC) under the direction of Professor Jonathan Fannin. These are reported in:

WAC Bennett Dam, Embankment Dam Project, Assessment of the filter capability of the filter system, Appendix D, WAC Bennett Dam, Continuing Erosion Filter (CEF) Tests on Core and Transition Materials. Tests on Transition Materials. March 2014.

WAC Bennett Dam, Continuing Erosion (CEF) Tests, 26th October 2015.

The tests carried out are continuing erosion tests (CEF) with test procedures similar to those developed by Foster and Fell (2001), and refined in Foster (2007). Dr Foster advised UBC on the test procedures as detailed in Appendix E of the March 2014 report.

In the first series of tests a material representing the finer side of the gradations in the Core was tested against base filters with gradations described as “Washed out Transition” and “Segregated Transition”. The “Washed out transition” PSD was determined by taking a grading representative of the coarser gradings of the Transition in the dam, and removing all the minus 1.0 mm fraction. The “Segregated Transition” PSD was obtained by removing the minus 4.75 mm fraction.

The “Washed out Transition” had a D_{15} = 3.5 mm about in the middle of the some erosion range, and the “Segregated Transition” a D_{15} = 9 mm, which is close to the excessive erosion size.

The test on “washed out Transition” sealed after “some erosion” in the Foster and Fell (2001) classification, and the test on the “Segregated Transition” sealed after “excessive erosion”.

In the second series of tests soils with a range of fines content from 22% to 35% were tested against filters which ranged in gradation to be in the “some” or “excessive” erosion range. Table 1 in the UBC report summarizes the results.

All but one of the tests sealed on the filter after “some” or “excessive” erosion. The filter for the test which failed to seal has a D_{15} = 12.6 mm, which is coarser than the excessive erosion filter size. It seems there was insufficient soil in the 100 mm layer to seal the filter.

UBC found the description of what constitutes “excessive erosion” for Group 4 soils somewhat contradictory. Given the soils have significant fines content it is considered that the better way to classify the erosion is using the 1.0 g/cm^2 limit for Group 2 soils rather than the 100 g total erosion for Group 4 soils. Also the test cylinder size used by UBC was larger than Foster and Fell (2001) used so the 100 gram requirement might be scaled up. If this is done tests CT23/T8.4W; C25/T3.5W; C26/T3.5W and C28/T3.5 (FF) WP would be “some erosion”, not “excessive erosion”.

It should be noted that as pointed out in the paper, Foster and Fell (2001) do not advocate the use of the “no-erosion” criteria in Table 4 of their paper for design purposes.

2.4.4 Site investigations and laboratory tests to assess the likelihood the Transition will hold a crack

BC Hydro has carried out investigations to assess the likelihood for the Transition to hold a crack because of cementation resulting from the high carbonate content of the materials in the Transition, and the presence of fines, and as a result potentially not performing its filtering function for the Core. This is particularly important in the upper part of the dam because there is no Filter to control erosion from the Transition above EL 2190-ft.

The cementing studies (GEOTECH-392-CWA Appendix B4) found that carbonates are abundant in the fill materials and there was potential for calcite precipitation from carbonates by three processes:

a) De-dolomitisation of carbonate: this would lead to precipitation at very slow rates at depth in the dam, below about EL 2030-ft.
b) Gas mixing: mixing induced calcite precipitation is a minor source and could occur only where seepage flow paths converge at depth
c) CO_2 out-gassing from the seeping reservoir water: out-gassing occurs at the phreatic surface and calcite would accumulate along it. There has been limited potential for significant cementation over the last 45 years. The amount of newly precipitated calcite did not exceed 1% in the samples analysed. However, if calcite cementing did take place, and erosion occurred through cracks in it, in all situations there would be a 'filter' downstream of the crack. In the upper part of the dam, flownets show that the phreatic surface drops into the downstream part of the Core, emerging at the Core-Transition interface at depth, below about EL 2160-ft. Consequently, the Transition downstream of the Core in the upper part of the dam will not be cemented.

From this it is concluded by BC Hydro and by the EEP that carbonate cementation above EL 2160-ft is not currently an issue nor is it likely to become one.

The potential for the Transition to hold cracks was further examined by sand castle tests:

(a) The PSD's of the Transition placed downstream and in the chimney area during construction showed that the mean % fines above EL 2160-ft was 4%, with approximately 38% of tests in the range 4% to 5% fines, and 11% in the range 5% to 6%.
(b) Sand castle tests were carried out in the field on samples trimmed from the Transition above EL 2160-ft. The tests were partial immersion tests replicating the Vaughan and Soares (1982) method. The time to initial failure ranged from 4 minutes 45 seconds to 18 minutes 42 seconds, with times to failure ranging from 5 minutes 10 seconds to 29 minutes 53 seconds. The soils tested had fines contents between 3.1% and 4.1% so were not representative of the higher fines content Transition.
(c) Sand Castle tests were also carried out by UBC. The fully immersed test BT 02 collapsed in 1 minute 40 seconds. In comparison a test on a soil representing Bennett Core collapsed in 9 minutes 38 seconds. No tests were carried out according to the Vaughan (1978) method which requires partial immersion.
(d) Tests carried out by UBC using the method of Soroush et al (2012) did not collapse even after 72 hours in one test and 23 hours in a second test. This type of performance only occurred in the Soroush et al (2012) testing with soils with fines content as high as 10% which would be highly likely to hold a crack. Soroush et al (2012) indicate that if the collapse time is > 60 minutes it is almost certain the soil will hold a crack.
(e) To investigate the reasons for the difference in the laboratory test results, UBC replicated one of the Soroush et al (2012) tests which had collapsed in 1 minute 40 seconds. This failed to collapse in 24 hours. The UBC tests used a smaller height to width ratio which may have influenced the test but it seems there are experimental problems with that test method.
(f) Recent testing by USACE and USBR reported in Howard et al (2015) indicates that even filters with zero fines content can hold a crack. A broadly graded sample with zero fines content and somewhat coarser than the Bennett Dam Transition did hold a crack but did not

erode substantially in their test set up. Erosion was restricted by the larger particles not being transported within the crack.

The EEP considers the sand castle tests as indicating that cracks in the Transition will probably not stay open as water flows through the crack; that is the sides of the crack will probably collapse. The EEP would expect however that finer particles in the Transition would be eroded from the crack and surrounds as the sides of the crack collapse resulting in a gradation which might be similar to the gradations of the Transition after 50% or 100% finer fraction is removed, or even the segregated gradation.

However, the sand castle tests are not totally conclusive and there remains some chance that the crack may stay open in a continuing erosion condition. This is because the field tests took some time to totally collapse, the Soroush et al (2012) type tests did not collapse and the Howard et al (2015) tests stayed open.

Given how important it is to the filter function of the dam above EL 2190-ft where the Transition is both filter and drain, the EEP recommends a more extensive series of sand castle tests to include Transition gradation soils with 4%, 5%, 6% and 7% fines content, compacted to a density and degree of saturation representative of what the Transition was compacted to in the dam above EL 2190-ft, and tested according to the Vaughan and Soares (1982) partially immersed procedure. These tests are considered to better model the condition in a crack than the full immersion tests. At least two tests should be carried out for each of the fines content to check repeatability.

BC Hydro might also consider having some crack-box tests carried out by USBR on the Transition material using the procedures reported in Howard et al (2015).

These tests are empirical and will not result in a definitive yes-no answer, but will give more information upon which to make judgements.

2.4.5 BC Hydro assessment of the filter system

BC Hydro presents an assessment of the filter capability of the filter system in Report MER No. 2014-033 and accompanying Appendices A and B. Since that report was written the second series of continuing erosion tests (CEF) have been carried out. The results of these tests have not yet been assessed in a report by BC Hydro.

The overall findings are listed below with comments from the EEP in parentheses.

Relating to the Core-Transition interface of the dam:

(a) Gradation shape analysis indicates that few, if any, of the Core gradations are potentially internally unstable. *(EEP agrees)*

(b) Gradations shape analysis indicates that as much as 25% to 50% of the Transition gradations may be potentially internally unstable. *(EEP agrees but likely percentage is probably lower)*

(c) Empirical assessment suggests the as-placed Transition gradations provide a 'no-erosion filter' to the Core gradations; *(EEP agrees and notes these "empirical assessments" use well established and widely accepted design criteria).*

(d) Empirical assessment for an assumed scenario of segregation of the Transition gradations during construction, or an assumed scenario of seepage-induced wash-out over the service life of the structure, suggests there may be 'some to excessive erosion' of the Core – however, there does not appear to be any likelihood of 'continuing erosion' of Core material; *(EEP agrees. The more likely scenario is for some erosion for the wash out cases and some to excessive erosion in the segregated scenario. In all cases the erosion should seal on the Core-Transition interface).*

(e) Laboratory investigation has confirmed a susceptibility of two coarser gradations of Transition material to seepage-induced internal instability, which takes the form of suffusion; *(EEP agrees and points out that very little erosion occurred and the Transition remained a no-erosion filter gradation for the tests carried out. The amount of erosion will depend on the gradation of the Filter after suffusion, and allowing for segregation. More tests are required to confirm the results).*

(f) Laboratory investigation has confirmed the assumed segregation of Transition material renders it a 'some to excessive erosion' filter to the Core, and the assumed washed out Transition renders it a 'some erosion' filter to the Core - however, there was no evidence of 'continuing erosion' in the laboratory test specimens; and, *(EEP agrees. The second series of tests which covered the plausible range of Filter gradations allowing for wash-out and segregation confirm this).*

(g) The Splitter Dyke material is expected to behave in a manner very much like the Transition material of the dam.*(EEP agrees)*

Relating to the Transition-Filter and the Filter-Drain interfaces of the dam:

(h) Gradation shape analysis indicates that as much as 75% or more of the Filter gradations may be potentially internally unstable *(EEP agrees).*

(i) Gradation shape analysis indicates that less than 10% of the Drain gradations may be potentially internally unstable *(EEP disagrees. The Kenney and Lau (1985) method is most applicable and nearly all gradations are internally unstable using this method. It is understood that BC Hydro only considered the gradations of the Drain ignoring the finer fraction "tail". The EEP feels this is potentially misleading.).*

(j) Empirical assessment indicates the as-placed, and the assumed washed-out scenario, both result in the Filter gradations providing a 'no-erosion' filter to the Transition gradations – 'some erosion' may occur for the assumed scenario of segregation in the Filter, and experience of problems with excessive segregation was reported at the time of construction – however, there does not appear to be any likelihood of 'excessive erosion' of Transition material; and,*(EEP agrees)*

(k) Empirical assessment indicates the as-placed Drain gradations provides a 'no erosion' filter to the Filter gradations - the assumed segregation of Drain gradations renders it a 'some to excessive erosion' filter to the Filter – however, there does not appear to be any likelihood of 'continuing erosion' of Filter material. *(EEP notes that as-placed the most likely scenario is no-erosion, but there are some combinations of gradations which give some erosion using USBR (2011) and Foster and Fell (2001) design rules. BC*

Hydro need to review the findings allowing for wash-out of fines in the drain. The small percentage of soil which will wash out means that the outcome will not be significantly changed.)

The following are some comments on the details of BC Hydro assessment:

Appendix A, Report MER No 2014—33 Potential for seepage induced instability

- The assessment carried out has some value in getting an overall picture, but given the importance of whether internal instability occurs and more particularly how much erosion occurs more emphasis should be placed on laboratory tests on a wider range of the materials from the Transition and Filter, modelling the actual gradings, not truncated to fit a test cylinder size, and a range of plausible base mesh sizes to represent the filtering zone.
- The EEP is not convinced the “recommended” variations and refinements of the methods used are appropriate. Given the first dot point above the EEP has not tried to understand the implications of the modifications.

Appendix B, Report MER No 2014—33 Preliminary Assessment of compatibility of the filter system

- Overall the results are similar to what the EEP reported in the 2012 report but with smaller ranges of results. This is because BC Hydro has used only gradations representative of the finer and coarser materials in each zone, rather than a larger sample. As a result they have not obtained the same range of required D_{15} sizes that the EEP obtained in Appendix G of the 2012 report. BC Hydro need to check for the finer gradations after re-grading on the 4.75 mm sieve because that is not necessarily the finest plot on the full gradation.
- For the Transition wash-out scenarios, it has been assumed that 36% is finer fraction. This is correct for the coarse side material but not for the finer. The finer fraction should be determined from the point of inflection of the plots, in this case nearly always on the 4.75 mm size.

2.4.6 Overall assessment of the filter and drainage system

(a) Below EL 2190-ft

The filter system to the Core below EL 2190-ft consists of Transition and Filter; and between El 2160-ft and El 2190-ft of Transition, Filter and Drain. If a concentrated leak occurs in the Core the Transition is required to act as the primary filter. However the gradation of the Transition is dependent on segregation during construction and how much suffusion occurs in the Transition as the leakage flow passes through the Transition. This in turn is dependent on how much suffusion occurs in the Filter, and the resulting capability of the Filter to limit erosion of the Transition.

Transition to Filter compatibility

From Table F9 of the EEP 2012 report the coarsest D_{15} after 100% of the finer fraction of the Filter is eroded by suffusion is between 0.6 mm and 12 mm, and allowing for segregation, 13 mm and

14 mm. This is finer or close to the excessive erosion D_{15} requirement for the Transition and is the worst credible situation.

There is no credible scenario which has the Transition to Filter being a continuing erosion filter.

Core to Transition compatibility

The UBC MSP tests on the Transition used a base mesh equivalent to 12.5 mm D_{15} for the drain, so was modelling close to the worst credible drain situation, and resulted in very little erosion with D_{15} after erosion < 0.5 mm, in the no-erosion range.

From Table F 8 of the EEP 2012 report the Transition D_{15} allowing for 100% erosion of the finer fraction is in the range 1.4 mm to 8 mm, and allowing for complete segregation of the finer fraction from the coarser fraction, 8 mm to 12 mm.

The UBC continuing erosion tests all sealed with some or excessive erosion except for one test using a 12 mm D_{15} filter, i.e. as coarse as is possible. That test would probably seal if a thicker layer of base soil was available.

It is concluded that the UBC analyses of data and the laboratory testing supports the conclusion reached in the EEP 2012 report that the dam below EL 2190-ft has a good filter system consisting of the Transition, Filter and Drain, which may allow a small amount of erosion at the Core/Transition interface but from the available information will prevent on-going erosion. There are no situations where erosion after initiation could continue unchecked.

(b) Above EL 2190-ft.

The situation is not so clear in the upper part of the dam above EL 2190-ft where the filter system consists only of the Transition.

The most likely scenarios leading to a concentrated leak in the core are likely to also result in a crack in the Transition. In view of this for the Transition to perform its function the crack needs to collapse as water flows through it. While this is the most likely outcome the EEP believes there is a small but some chance that the crack will stay open and a continuing erosion condition could result.

Even if the Transition does not hold a crack there is no Filter to control erosion from the Transition and it is possible that under large leakage flows much more erosion could occur than indicated by the MSP tests carried out so far. Predicting the gradation and permeability of the remaining material is difficult, as is predicting whether the resulting leakage might be sufficient to initiate breach of the dam by slope instability or unravelling.

The additional testing suggested above will assist in getting a feel for the likely outcomes.

One way to get a higher degree of confidence that the filter system will control erosion from the Core under all possible static and dynamic loading scenarios would be to extend the Filter and possibly Drain protection from EL 2190-ft to the crest of the dam at least in the areas most vulnerable to cracking and concentrated leaks.

The areas considered to be the most vulnerable to formation of a concentrated leak above EL 2190-ft are at the two sinkholes; and at the interface of the embankment and the spillway

2.5 REVIEW OF FOUNDATION CHARACTERIZATION

2.5.1 Introduction

The characterization of the foundation of the dam is described in Chapter 4 of Report No E1138, “Site Characterization” and related Appendices.

BC Hydro has carried out a thorough data collection and analysis of the foundation of the dam so far as these relate to seepage and internal erosion control. The outputs of the analyses are the compression modulus of the foundation, and hydraulic conductivities to be used in seepage analyses. The hydraulic conductivities are in the form of geometric mean estimates for:

- Untreated uncompressed bedrock
- Untreated compressed bedrock
- Grouted uncompressed bedrock
- Grouted compressed bedrock.

“Compression” relates to compression of the foundation by the weight of the dam which is assumed to significantly reduce the foundation hydraulic conductivity compared to test values determined prior to dam construction by site investigations and testing of the primary grout holes.

2.5.2 General purpose of the characterization and limitations for other uses

The characterization of the foundation has been specifically directed towards:

- The bedrock surface geometry.
- Bedrock stratigraphy.
- Intact rock modulus of elasticity.
- Hydraulic conductivity of the foundation bedrock including the effects of stress relief, stratigraphy, compression of the bedrock by the weight of the dam, and grouting.

It should be made clear in the report that the characterization is not adequate to allow consideration of the stability of the dam, stability of canyon slopes, or in relation to the appurtenant structures. Important factors relating to these such as the presence of mylonite seam in the upper right abutment terrace are not covered.

2.5.3 Bedrock stratigraphy and data on defects

The bedrock stratigraphy has been reviewed and the rock units amended from what was adopted during construction and had been used for quite detailed studies of the dam performance since construction.

There may be some advantages in the new interpretation but it means that it will always be difficult to relate the earlier studies to the revised stratigraphy. There also seems to have been important detail lost in the new model in regards to varying thicknesses and lensing of the finer grained rocks as shown in cross sections and longitudinal sections with the new units being shown as uniform thicknesses across the site. To assume the beds are of uniform thickness across the whole site seems to be a simplification of the real situation.

There is not sufficient data in the report on joints or other geological features of the site. There should be a discussion of the joint spacing, orientation and persistence within the stratigraphic units and in the various parts of the foundation. There are also details of observed valley stress relief features in earlier reports which should be summarized and referenced where more details can be found because they and the joints will be important pathways for seepage in the foundation.

2.5.4 Bedrock modulus of elasticity

This section has not been reviewed by the EEP.

2.5.5 Bedrock hydraulic conductivity

The following are comments on this study:

(a) Geometric mean estimates of the hydraulic conductivity of each stratigraphic unit are given in Table 3-7 for untreated uncompressed rock. Frequency plots of hydraulic conductivity for each stratigraphic unit are given in Appendix F. It would be better to include in Table 4-7 the mean plus one and two standard deviation values because it is the larger values which will represent wider open joints which are important to potential erosion into and in the foundation.

(b) Section 4.5.3.5 describes how the grout take data was used to estimate the hydraulic conductivity. The EEP cautions against doing this as the methods and properties used are very approximate. Published data on the correlation between grout take and water pressure test Lugeon values show the correlation is very poor to almost non-existent so to use the grout takes is likely to be misleading.
As noted in the EEP 2012 report the grout pressures used were very high and the very large grout takes in some grout holes means that this almost certainly caused jacking and hydraulic fracture opening of the bedding partings.
It is recommended that the foundation hydraulic conductivity prior to grouting only be estimated from water pressure tests in site investigation boreholes carried out prior to construction of the dam and the water pressure tests in the primary grout holes before any grouting is carried out;

(c) The report discusses the effects of valley stress relief and foundation location and these are accounted for in Figures 4-9 and 4-10 but then are all lumped together for the compressed foundation case and the values used in the seepage analyses. The limitations of doing this are clearly evident when quite high pore pressures are modelled in seepage analyses downstream of the grout curtain for Instrument Plane 1, whereas piezometers show the pore pressures are at tailwater level due to the underdrainage effect of high permeability strata in Units N5 and N6 which have not been modelled in the seepage analyses.

(d) The use of lower hydraulic conductivities allowing for the effect of the weight of the dam is based upon data from water pressure tests done during grouting from the grout gallery when the embankment was already under construction, drilling of one near horizontal hole in un-grouted rock for the 1986-1987 deficiency investigation; and drill holes in the right abutment during the 1996 sinkhole investigation. The report refers to Figures F6 showing hydraulic conductivities in the valley area, with Figures F5 and F7 in the left abutment and right abutments.

What appears to be missing in these figures is that the valley section had already been grouted by the blanket grouting and treated with cleaning out of the open defects and covering with shotcrete. Hence the lower hydraulic conductivities apparent in the figures may be partly because the most stress relieved rock had already been grouted.

It is also questionable whether any reduction should be deducted from data in the valley section. This section was particularly affected by valley stress relief giving opening of the bedding partings to areas like the canyon gorge sides where the dominant open features are likely to be near vertical due to valley stress relief. In any case as pointed out in (c) the assumed compressed conductivities seem to be too low because too-high pore pressures are modelled in the foundation downstream of the grout curtain.

The EEP members have not seen evidence elsewhere in their experience that the weight of the dam significantly reduces the hydraulic conductivity of dam foundations. It seems reasonable in principle to assume some reduction in conductivity where horizontal defects are the dominant contributor to the conductivity, but probably not as much as has been assumed in the Foundation Characterisation Report.

It is suggested that this matter be re-considered taking account of these comments.

It is the EEP view that to reduce the whole dam foundation to effectively uniform conditions except for allowing for grouting and the compression of open defects by the weight of the dam is an over-simplification and potentially misleading. Each area should be considered separately in the characterization and seepage analyses.

2.6 3D CAD AND GIS MODELS

BC Hydro has responded to the Panel’s recommendations by assembling all the dam data in new and updated CAD and GIS models.

The data set of information from the construction period, the sinkhole investigations and the 50-years of operation is large, and the new models now store and provide easy access to it. All the foundation bedrock levels, the fill levels, instrument positions, instrument records, the winter horizons, lead trenches, and the locations of fill samples have been loaded in to GIS.

New surveys by Lidar of the crest and downstream surfaces of the dam, and underwater surveys of the upstream slopes by Multi-beam Sonar from a Remotely Operated Vehicle have been undertaken and stored on the GIS model. The upstream survey achieves repeatable definition of 0.06 m vertically and 0.6 m horizontally. The survey data can be added to and GIS includes capabilities to identify and display changes of the dam surfaces.

The GIS model also stores the new bedrock characterisation, updated and improved from construction records, aerial photographs and other records from the construction period, see Section 2.5 above.

The Sonar survey has identified two interesting features, one which looks like a shallow landslide on the upstream slope of the embankment below the rip rap; and another localised depression near where the embankment abuts the canyon wall. The reasons for these are being investigated by BC Hydro.

3 UNDERSTANDING OF DAM PERFORMANCE

3.1 INTRODUCTION

Since 2012 further seepage analyses have been completed and response times of piezometers and trends over the lifetime of the dam examined. A steady state situation has now been reached with almost consistent responses in piezometer and weir readings to the annual water level cycle (Section 3.2). Further investigations of ‘air theory’ or other causes of the very high early pore pressures have not yet been made.

Numerical analyses have been performed of stresses and deformations during dam construction and at end of construction. Sensitivity studies have been included by varying different material parameters and to compare computed results to field measurements. The effects of reducing the shear modulus in Sinkholes 1 and 2 and Transition Zone 2 have been studied. The reduction in shear modulus is compatible with the observed reduction in cross-hole shear wave velocities (Section 3.3).

A comprehensive set of seismic analyses has been performed for eight different acceleration-time histories to estimate the permanent displacements caused by earthquake shaking. Likewise, the build-up in pore water pressures in the upstream shell of the dam is estimated and the liquefaction potential is evaluated. Special attention is given to the response of the vulnerable top of the dam (Section 3.4).

3.2 SEEPAGE STUDIES

3.2.1 Performance assessment of fill piezometers and weirs

Since 2012 BC Hydro has carried out an instrument performance assessment examining by regression analysis the changes in response of the piezometers in the Core and the multi-port piezometers in the Transition, and the flow measured at Weirs R1 + R1S in the canyon between Splitter Dikes 1 and 2 above the right wall of the canyon and Splitter Dike 4 above the left wall of the canyon; and Weir 6 +3 recording seepage from the entire right flank above Splitter Dikes 1 and 2. Weirs R1 + R1S were estimated by trials in 2007 to capture about 50% of the seepage from the canyon section of the dam. Weir 6 + 3 captures all the seepage from the right flank of the dam. Records from Weir T5 above the canyon wall near Splitter Dike 4 may provide another record of seepage flows through the dam.

The regression analyses demonstrated that a ‘steady state’ had now been reached with rapid responses to reservoir water level in the piezometers and at the Weirs. This indicates that the large quantities of air present in the fill after construction, augmented by ex-solution of dissolved air from the reservoir water as it seeps through the dam, have probably been driven out progressively to ‘atmosphere’ at the phreatic surface by the annual cycle of reservoir water level changes.

3.2.2 Understanding the air occlusion and ex-solution process

Table 6.2 in the EEP’s 2012 Report records measured seepage flows over time, and Figure 10 shows reservoir level against measured flow. The earliest reading was 1575 L/min in 1974 when reservoir level was EL 2205-ft. Figure 10 probably shows the influence of the large volumes of air in the fill at that time. Seepage is low and the piezometers showed pore water pressures to be high. Ten years later when water level was again EL 2205-ft, the seepage flow was 2103 L/min, probably showing the effect of driving out air.

The Panel recommended in the 2012 report that BC Hydro investigates the air occlusion and ex-solution theory (e.g. Sobkowicz et al, 2000) to possibly explain the pore pressure development in the dam since first impoundment. The Panel confirm this recommendation as it is important to fully understand the pore pressure response over time as a component of dam behaviour. The regression analyses suggest that the 'steady state' was reached from about 2006-2008 and a good understanding of the air situation may confirm this, and indicate whether there are any continuing effects from 'air'.

3.2.3 Routine seepage monitoring using piezometric records

Seepage measurement is one of the fundamental means of monitoring a dam's performance. Seepage measurement provides information about a large section of the dam, piezometers give only very local information. Irregularities in seepage flow may indicate some malfunction and other monitoring devices, such as piezometers, may assist in identifying its location and cause. At Bennett Dam, the record at Weirs R1 + R1S is based on an estimate of the total seepage captured. This may vary with seepage quantity, and extra flow from a leak may by-pass the weirs, consequently Weirs R1 + R1S do not provide an entirely reliable means of monitoring leakage in the canyon section. Routine seepage measurement of all seepage is limited to the right flank, unless Weir T5 on the left flank can also be used, and the relationship between reservoir water level and the seepage at Weir 6 + 3 should be well understood so that the weir record there can be used to identify anomalies should any occur.

3.2.4 Reliance on flownets and piezometer records for monitoring

In the absence of reliable routine weir seepage measurements, as in the canyon section, comparing piezometer records with flownets provides a means of routine monitoring. The flownets presented to the Panel showed good agreement with piezometer readings. The 2012 EEP Report, Section 6.1.5 concluded that the seepage patterns 'seem to be what would be expected through a core of broadly uniform but anisotropic fill in which any local variations in permeability are masked'.

As comparison to flownet predictions should be used routinely to augment seepage measurement in Weirs R1 + R1S for monitoring performance in the canyon section, pore pressures downstream of the axis should be monitored, if possible. OW2, now provided with a piezometer tip, some of the foundation piezometers and dipping at the weir chamber may provide useful information.

3.3 ANALYSIS OF STRESSES AND DEFORMATIONS DURING DAM CONSTRUCTION AND AT END OF CONSTRUCTION

As recommended by EEP in the 2012 report, BC Hydro has performed two-dimensional stress-deformation analyses for three cross-sections of the dam (the Canyon cross-section, section with Sinkhole 1, section with Sinkhole 2) and one longitudinal section along the dam axis. The computer program FLAC was used with a modified Mohr-Coulomb material model, supplemented with linear-elastic computations by the program Sigma/W. In the numerical analyses the dam was constructed in 16 layers. The bedrock contours were modeled in the longitudinal section across the valley as well as in the three cross-sections. However, some of the rock surface slopes had to be made more gentle and gradual than the bedrock mapping actually shows to avoid numerical difficulties in the computations.

The results presented show the vertical and horizontal stresses and displacements in the three cross-sections and the longitudinal section at the end of construction. The corresponding values at intermediate stages of construction, that the Panel had recommended, were not presented, as the study is not completed and the data not yet fully interpreted.

The sensitivity of the results to the assumed material parameters has been investigated, but the studies including interpretation of results are not yet completed. The results for the dam cross-sections are most sensitive to the relative stiffnesses between the different zones (assigned modulus values), while the results for the longitudinal section are most sensitive to the assumed Poisson's ratio. Based on the analyses performed and a comparison between computed and measured vertical settlements, BC Hydro has concluded that the confined compression modulus of the compacted Core material is high, about 400 MPa.

The computed results for the longitudinal section show that the horizontal compressive stresses are low in the vicinity of Sinkholes 1 and 2. There is a tendency to extensional strains due to the proximity of the canyon shoulders and walls. The reduction in stress level is most pronounced in the vicinity of Sinkhole 1 because there is a steep bedrock slope under the sinkhole not only in the longitudinal section but also in the cross-section. The 2D computations at end of construction do not show any tensile stresses at these locations, but tensile stresses may have existed temporarily during construction and this will be investigated. In the first computations, a Poisson's ratio of 0.35 was used. An analysis with a lower Poisson's ratio = 0.20 was performed during the present EEP meeting and shows a very significant reduction in the compressive lateral stresses compared to the previous case with the higher Poisson's ratio. The lateral stresses against the canyon walls have decreased by a factor of almost 3. These findings support the EEP's opinion expressed in the 2012 EEP report that the special locations of Benchmarks 1 and 2 close to the canyon walls and the topography of the bedrock in the vicinity of the benchmarks may have played an important role in the sinkhole formations.

Low local compressive stresses may lead to local hydraulic fracturing during impounding. In the numerical studies performed so far, steady state seepage pore pressures for full reservoir level have been put into the model to compute effective stress levels. The EEP recommends that the pore pressures measured shortly after first impoundment should be used to compute the effective stresses (minor principal stresses) at the end of construction. That will define local areas where hydraulic fracturing may have occurred before the pore pressures gradually reduced.

Post-construction settlements and permanent displacements that occur during and after earthquake shaking cause redistribution of stresses in the dam body and possible cracking. These effects may be estimated by imposing the computed displacements on the completed structure. Alternatively, the situation may be simulated by adjusting/reducing the compression modulus in various zones, turn on gravity, and try to match the post-construction and post-earthquake dam displacements. The EEP recommends that this be attempted as analyses of stresses in other dams have shown that post-construction settlements are important in forming low stress zones.

Corresponding results for stresses and deformations have also been presented for the situation where the stiffness (shear modulus) of the material in Sinkholes 1 and 2 has been decreased significantly compared to the shear modulus values in undisturbed zones of the core. The shear modulus in these loosened zones has been estimated based on the reduced shear wave velocities that were measured after the sinkholes occurred and were repaired by compaction grouting. Likewise, the shear wave velocities in Zone 2 (Transition) downstream of the Core, were found to be much lower than in the originally compacted soil, and this led to a reduction in stiffness (shear modulus) also in that zone. The results show a very significant change in the distribution and magnitude of stresses and deformations inside the dam, compared to that computed for the situation when the soil parameters were based on the results from the original laboratory and field tests at the time of compaction in the dam. As discussed in Section 3.4, the effects on the computed seismic stresses and deformations are also very significant.

It should be kept in mind that only 2D analyses have been performed, which means that the locally weakened zones at the sinkhole columns are assumed to stretch for a long distance along the dam axis. Thus, the effects of the local sinkholes are exaggerated in the static as well as in the dynamic analyses.

3.4 SEISMIC STABILITY AND DEFORMATION ANALYSES

In its report dated August 2012, EEP recommended that seismic stability and deformation analyses should be performed using the updated seismic hazard assessment for the site. There is particular concern about the seismic performance of the top part of the dam where amplified accelerations occur, and above EL 2190-ft there is only a Transition zone downstream of the Core, no Filter and Drain zones as discussed in Section 2.4.

Bennett Dam is located in a region of moderate seismicity, and by means of a probabilistic seismic hazard assessment the Safety Evaluation Earthquake (SEE) has been determined to have a Richter magnitude of about 6.5 and horizontal peak ground horizontal acceleration, PGA=0.16g with an annual exceedance frequency (AEF) = 1/10,000. The corresponding design spectrum was specified, and eight acceleration-time histories were selected and linearly scaled to satisfy the specified design spectrum. Seven of the earthquake records are from earthquakes experienced in Western North America and one, the Shawnee earthquake, is taken from the data for Central and Eastern North America. A complete set of dynamic analyses was performed for each of the 8 acceleration-time histories. Emphasis in the reporting has been on the results from the Shawnee earthquake. This is the first time an extensive earthquake response analysis study has been performed for the Bennett Dam.

The results are presented with focus on:

- Build-up in pore pressures in the upstream dam shell;
- Build-up in pore pressures in the alluvium deposit under the upstream cofferdam;
- Settlement and permanent horizontal displacement of the dam crest;
- Settlement and horizontal displacement at Sinkholes 1 and 2;
- Amplification of ground acceleration at the crest of the dam.

Three cross-sections (Canyon, Sinkhole 1 and Sinkhole 2) and one longitudinal section along the dam axis were analysed. Before the dynamic analysis was performed, the initial static stresses and steady state pore water pressures were computed and used as input to the earthquake response analysis (see Section 3.3). For the dynamic analysis a total stress version of FLAC was used including the capabilities of UBCTOT (Beaty and Byrne, 2008) to model the non-linear stress-strain and liquefaction behaviour of the soils. Details of the analyses and soil properties in the different zones of the dam are described in BC Hydro Report, MER No. 2013-110, December 2013, and were summarized in the presentation at the present meeting. Professor Peter Byrne has served as an adviser and reviewer during the performance of the analyses.

In addition to FLAC UBCTOT, the finite element computer code VERSAT (Wutec, 2012) was used for independent computations and comparisons with the FLAC results. Computed accumulated displacements were also compared with results from Newmark's (1965) simplified analyses and empirical predictions based on field records for dams subjected to earthquakes (e.g. Swaisgood, 2013).

After the earthquake has passed, and excess pore pressures have been built up in the upstream shell, post-earthquake stability analyses were performed for the upstream slope by computing safety factors against instability. With time, these pore pressures will dissipate and the safety factors gradually increase.

A very extensive series of analyses has been performed by BC Hydro, and the results were presented and systematically compared using the computed response from the 8 input acceleration-time histories. The comparisons clearly show that the characteristics of the earthquake shaking, e.g. frequency content and duration, have very significant effects on the computed results of pore pressure build-up, accumulated displacements and acceleration amplification towards the dam crest. As expected, the Shawnee earthquake causes significantly larger computed permanent vertical and horizontal displacements and pore pressures than the other earthquakes.

The cross-sections with Sinkholes 1 and 2 are each analysed for two different cases. In one case the shear modulus values for the properties in the Sinkhole and Transition are as constructed and computed based on laboratory and field tests. In the other case the shear modulus values in the Sinkhole and Transition are made to correspond with the measured, reduced shear wave velocities. The reduction in measured shear wave velocity leads to a considerable reduction in shear modulus compared to the as-constructed value. In the 2D analyses performed, the local reductions in shear modulus lead to very significant increases in the computed crest settlements and in the horizontal displacements.

3.4.1 Summary of results from seismic analyses

- Based on the FLAC UBCTOT results, portions of the alluvium underlying the upstream cofferdam will liquefy. The waste fill in the cofferdam is also expected to liquefy and undergo large upstream displacements. However, this local liquefaction and loss of stiffness and strength will not affect the stability of the upstream dam shell.
- For the deep Canyon cross-section, the maximum computed permanent horizontal crest displacement is about 1-ft in the upstream direction, and the crest settlement is about 1.5-ft.
- Based on the analyses of the cross-sections with Sinkholes 1 and 2 and the reduced shear modulus also in the Transition zone, the maximum permanent horizontal crest displacement in the upstream direction is about 3.3-ft. The corresponding crest settlement is estimated to be about 4.4-ft. This may lead to transverse cracking in the top part of the core, but it leaves a freeboard down to full reservoir water level of about 20-ft.
- The pore water pressure build-up in the upstream shell corresponds, on average, to a value of r_u = 0.2. The computed post-earthquake factor of safety for the upstream slope is about 1.2 and will gradually increase as the excess pore pressures dissipate.
- The post-earthquake crest settlement at the location of Sinkhole 1, due to excess pore-pressure dissipation, may amount to about 2-ft.

3.4.2 Evaluation of analyses and computed results

2D plane strain analyses have been performed, and this leads to significant overestimates of the effects of the reduced shear wave velocities in the cross-sections with Sinkholes 1 and 2. 3D analyses would be much more appropriate, as the effects of Sinkholes 1 and 2 are local. Thus, both the displacements and excess pore pressures are overestimated. They may be overestimated by a factor of 1.5 – 2.0.

On the other hand, vertical ground accelerations are not included in the earthquake input. The additional response caused by including a vertical acceleration – time history depends on the phase difference between the horizontal and vertical acceleration input. Based on experience with other dam analyses, the computed displacements may increase by 20 %.

The analyses using the finite element computer code VERSAT gave consistently lower estimates of displacements than the results from FLAC UBCTOT.

The displacements and excess pore pressures resulting from the Shawnee earthquake are much larger than the corresponding results from the other 7 earthquake acceleration – time records. This is mainly caused by the unusually long duration of the Shawnee earthquake. Having computed the response from as many as eight seismologically relevant acceleration-time inputs, it may be justifiable to use average response values (e.g. displacements and excess pore pressures) rather than the maximum computed values. On the other hand, it may be argued that the 84 percentile should be used rather than the mean peak ground acceleration. The mean horizontal peak ground acceleration is 0.16g while the 84 percentile is 0.22g and would give higher response values.

For the Safety Evaluation Earthquake, the ICOLD design requirements accept that the dam may sustain significant damage, but should still be able to prevent an uncontrolled release of the reservoir.

3.4.3 Conclusions from seismic analyses

Even with the conservative 2D analyses, the computed crest settlements and permanent horizontal displacements caused by the earthquake loading are moderate. The likelihood of overtopping is extremely small. The freeboard between the crest level of the dam at Sinkhole 1 and full supply level is about 20 ft after the maximum computed settlement caused by seismic loading has occurred. According to the computations, the height difference between the top of the core and full supply level is about 9-ft after the post-earthquake consolidation settlement has taken place.

The computed build-up of excess pore pressures is moderate and will not cause instability in the upstream shell.

Uneven horizontal and vertical crest displacements around the loosened fill at Sinkhole No 1 will combine with the effects of cross valley differential settlements due to the earthquake which have not been modelled. These combined will probably lead to differential settlements, shear distortions and longitudinal and transverse cracking in the top part of the dam around Sinkhole No 1. There is a low likelihood that the earthquake induced cracks at crest level will penetrate as deep down as to full supply level where the width of the dam is about 120 ft. However the deformations and cracking are difficult to model because of the complex 3D geometry and the difficulty in modelling the pore pressure build up and loss of stiffness in the loosened fill around Sinkhole No 1 under seismic loading. The 84 percentile PGA is 0.22g and this is only three times less likely to occur but might result in significantly larger deformations.

The behaviour of the dam around Sinkhole No 1 could be significantly improved if the loosened fill could be densified either by removal and replacement by compacted fill and / or in-situ densification. BC Hydro may wish to investigate the feasibility of doing this and improving the filter drain system in the upper part of the dam around Sinkhole No 1 by including Filter and Drain zones up to crest level in this area.

Earthquake shaking may cause reduced compressive stresses and even tensile strains at some locations in the interior of the dam, at interfaces between different zones, at the canyon walls, or at the interface between the embankment and spillway. This may lead to local tensile cracks and hydraulic fracturing in the Core. Protective Transition and Filter zones are designed to arrest any crack erosion. However, the Transition and Filter zones may themselves be subject to cracking caused by the earthquake shaking and differential distortions. The properties of the Transition and other filter materials in Bennett Dam are such that cracks that have been opened will most likely be closed and self-heal and prevent further erosion. This is discussed and evaluated in detail in Chapter 2.

In general, the interface between a concrete spillway and an embankment core is a particularly vulnerable location during an earthquake, partly because the dynamic response of the embankment is different from that of the concrete spillway. The upper part of the spillway wall at Bennett Dam is much steeper than the lower part, and this may cause a gap/crack to open when the embankment settles during earthquake shaking. BC Hydro should study the conditions and contact stresses at this interface in more detail and evaluate the likelihood of cracking and possible erosion, see discussion in Chapter 2.

4 POTENTIAL FAILURE MODES ANALYSIS WITH RESPECT TO INTERNAL EROSION AND PIPING

4.1 INTRODUCTION

BC Hydro has carried out a detailed Potential Failure Modes Analysis for internal erosion and piping modes. This is reported in a Draft Report WAC Bennett Dam, Large Embankment dams project, Potential Failure Modes Analysis, November 2015.

The outcomes of the PFMA are categorised using the system used by Federal Energy Regulatory Commission, Engineering Guidelines for the Evaluation of Hydropower Projects" (FERC, 2015), with a further categorization allowing for what is termed the PFM duration, to allow for the assessed duration of each step in the failure mode flow diagrams. This approach is unique to BC Hydro Bennett Dam.

The following comments are not meant to be a detailed review of the Draft Report. The EEP has not had sufficient time to do that.

4.2. OVERALL COMMENTS ON THE PFMA DRAFT REPORT

The analysis has been very thorough and has considered the plausible potential internal erosion and piping failure modes under normal, flood and seismic loading. The standard of documentation to justify the process is excellent.

The overall outcomes and rankings are considered to be generally reasonable subject to the detailed comments below. However the assessed PFM durations are in a number of cases too long, as concentrated leak processes in the core of Bennett Dam can be expected to occur rapidly, in hours or days, not months or years as has been assumed. This has important implications for the ability to detect and intervene if internal erosion initiates.

The analysis is unusual for a PFMA in that it is semi-quantitative, with verbal descriptors of likelihood assigned to the likelihood of initiation of erosion given a flaw exists, the likelihood the filter system will provide no-erosion, some erosion, excessive erosion, or continuing erosion; the likelihood the erosion in the core will progress; and the likelihood for each branch of filtering, that the dam will breach.

The analysis does not consider the likelihood of the load condition and the likelihood given that load condition, a flaw will exist or be formed. This means that the fact that a seismic event sufficient to form a flaw below reservoir level may be a 1 in 10,000 annual probability, but a normal operating load has a 1 in 1 annual probability is potentially lost in the outcomes.

The ability to detect or intervene is also not allowed for other than in an overall assessment of response options.

It is suggested that BC Hydro consider these factors further for the more important PFM as that should allow better discrimination of those PFM which are most likely to potentially lead to failure. This would help in better focussing efforts in monitoring and surveillance, and might identify PFM which warrant remedial works.

4.3 SOME DETAILED COMMENTS ON THE PFMA DRAFT REPORT

There are some comments which apply to most of the PFM:

(a) *Use of mapping schemes to estimate probabilities*: The report uses the USBR mapping scheme to link verbal descriptors to conditional probabilities. The problem with this is that terms such as “likely” mean different probabilities to different people. This is discussed in AGS (2007) from which Table 4.1 is taken. It can be seen that a wide range of values apply, and even the USBR and Vick schemes (from which the USBR scheme was developed) are 10 times different in some cases. AGS (2007) and ANCOLD (2003) recommend such schemes are not used. The experience of EEP member Robin Fell is that more consistent and reasonable estimates can be made using the Barneich et al (1996) approach which is shown in Table 4.2. Within the Table the “scenarios” approach has been found to be the most useful. However it must be accepted that all these schemes are limited and it is better to actually calculate probabilities where the data exists such as for the probabilities of the four classes of filter capability.

Table 4.1 Some published relationships between verbal descriptors and conditional probabilities (AGS 2007)

Verbal Descriptor	Conditional Probability				Annual Probability		
	USBR (2003)	Vick (1992)	Bowden *et al.* (2003)	Reagan *et al.* (1989)	AGS (2000) Appendix G	De Ambrosis & Mostyn (2004)	Moon & Wilson (2004)
Virtually certain	0.999	0.99	0.999	0.9	Approx 0.1 *	>=0.1*	>0.2*
Very likely	0.99	0.9		0.85			0.2 to 0.02
Likely	0.9			0.7	Approx 0.01	>=0.01	0.02 to 0..002
Neutral (even chance)	0.5	0.5		0.5			
Unlikely	0.1		0.001	0.15	Approx 0.0001	>=0.0001	<0.0002
Very unlikely	0.01	0.1	0.0001	0.1			<<0.0002
Virtually impossible	0.001	0.01	0.000001	0.02	<0.000001*	<0.000001*	

Note: * Verbal descriptor similar

Table 4.2 Mapping scheme linking description of likelihood to quantitative probability (Barneich et al, 1996)

Description of Condition or Event	**Order of Magnitude of Probability Assigned**
Occurrence is virtually certain	1
Occurrences of the condition or event are observed in the available database	10^{-1}
The occurrence of the condition or event is not observed, or is observed in one isolated instance, in the available database; several potential failure scenarios can be identified.	10^{-2}
The occurrence of the condition or event is not observed in the available database. It is difficult to think about any plausible failure scenario; however, a single scenario could be identified after considerable effort.	10^{-3}
The condition or event has not been observed, and no plausible scenario could be identified, even after considerable effort.	10^{-4}

(b) *Estimating probabilities of filter capability*: The sum of the probabilities of no, some, excessive and continuing erosion should equal 1. In many of the event trees in the report the sum exceeds 1 if the mapping scheme is applied.

Given the importance of the filter capability to controlling internal erosion, and hence the safety of the dam; the almost unequalled amount of data about the particle size distributions of all the materials in the dam, and the availability of laboratory continuing erosion tests to back up the application of the Foster and Fell (2001) method for assessing the effectiveness of the filters, it is suggested that BC Hydro formally estimate the probabilities of no, some, excessive and continuing erosion for the upper and lower parts of the dam, and the other more specific PFM. This needs to allow for segregation and suffusion effects which will require some judgement. The advantage of this is that it will likely result in lower probabilities for continuing erosion than have been estimated so far in the PFMA report and represent hard data to support the conclusion that the filter system at Bennett will control internal erosion and piping.

(c) *Recognising that the probabilities of progression and breach are conditional depending on which branch of continuation is being considered*: In Figures B-1, B-2, B-4 and B-10, the probabilities of progression and breach for the continuing erosion branch should be higher, not lower than for excessive erosion. These probabilities are conditional on the erosion being in the continuing category and are not meant to reflect that the likelihood of continuing erosion may be very small.

(d) *Re-assessing the PFM durations and response times*: As discussed above, the PFM durations for the concentrated leak PFM are too long, because the rate of erosion of the non-plastic silt sand gravel soils will be very rapid. Many of the assessed PFM Categories need to be changed from a "C" to "B". This includes but is not limited to PFM N.01 B3; N.01B 4; N.02 A2, N.02 A3; N 03 A2; N.03 A3. Some need to be changed from "C" to "A" including PFM N.04 B2; N.04 B3; N.04 B4; N.04 C 3 and N.04 C4 for the spillway wall cases where the whole erosion process to breach is likely to occur in hours once initiated.

(e) Upstream flow limitation and crack filling potential of Zone 6: The Zone 6 fill upstream of the Core has the potential to limit flows into a concentrated leak.

(f) As BC Hydro does not wish to use quantitative numbers in the PFMA, they could use the estimated probabilities described in (a), and (b), and replace them with the qualitative terms from the USBR mapping scheme in the PFMA results.

The following are some specific comments:

(g) For N.02, progression is probably "less likely" because the upstream Random Fill Zone 6 may limit flow.

(h) For N.04, concentrated leak along the outside of the spillway wall: The "wrap around" detail has some reasonably good design features including at the eastern (embankment) end the contact being rotated so the force of the water will push the fill against the concrete and the concrete slopes being battered at 1H to 10V. However the downstream detail is not so good with the potential for the fill to move away from the concrete because the lower part is at a batter of about 0.6H to 1V. In any case there will be a tendency for a gap to form between the fill and the concrete as the fill settles, particularly under seismic loads. There is also a potential for transverse cracking due to differing seismic responses of the embankment and concrete sections. As elsewhere the Filter is only taken to EL 2190-ft so the Transition has to perform the function of filter and drain. If a concentrated leak was to initiate erosion progression and breach is likely to be rapid because of the sloping surface of the fill.

(i) For N.05, the probability of initiation of erosion by global backward erosion (GBE) is probably lower than "neutral" given the GBE would require quite high gradients in the core.

(j) For N.06, the probabilities of continuation and progression are not "very unlikely" given suffusion initiates. The same applies to N.07.

(k) For N.08, see the comments in Page D-9 in the EEP 2012 report. It seems unlikely wide open joints were left in the foundation of the Transition.

(l) For N.10, these PFM would probably require erosion of soil through the core surrounding the casing through the filter system to give a potential breach. This is what happened in Sinkhole

No 1, and that did not give sufficient settlement of the crest to result in a breach. To assign a classification of I-C to N.10 B, erosion along the OW's is probably conservative given they were installed in backhoe pits, where backfill compaction should have been reasonably good. To assign I-C to N.10 C hydraulic fracture resulting from leakage out of the OW casings is also probably too conservative.

(m) For N.11, the amount of erosion of core into the grouting culvert will be limited by the width of the joints in the culvert concrete. It is understood these are small, so the continuation probability should be lower than estimated.

(n) For N.12, the PFM would seem most likely to be erosion of core into the adit through opening of joints in the rock overlying the concrete plug.

5 MONITORING BY CROSS-HOLE MEASUREMENTS

5.1 INTRODUCTION

Seismic shear wave velocity measurements are essential in monitoring the on-going condition of the dam at the Sinkholes and the vertical Instrument Island Risers at Instrument Planes 1 and 2. In its 2012 report the EEP recommended several improvements, most of which have been carried out, as described below.

5.2 OBSERVATION WELL AND PIEZOMETER IMPROVEMENTS

In response to concerns about creating hydraulic fracturing and collapse settlement in the soil around them, six observation wells (OW2, OW4, OW5, OW7, OW9, OW10) have been sealed by grouting. The OWs are 3-inch diameter inclinometer casings. They were installed in pits in stages from the foundation to the crest as the fill was raised. Standpipe piezometer tubes 32 mm in diameter were carefully grouted in, with sand pockets about 30-ft deep at the base of the OWs. The 32 mm tubes can be used for cross-hole seismic monitoring. The work seems to have been done very systematically and with high quality.

Two optic fibre cables, one including a heating wire, were attached to the piezometer tubes. These are potentially multi-functional, providing temperature measurement to estimate flow by the passive or active method by heating the cable and estimating flow from the rate of cooling. The fibres monitor dynamic strain along the optical fibre, and could be used in future for cross-hole S-wave measurement, vertical seismic profiling and passive acoustic monitoring.

Several piezometers, installed as part of the 1996-97 sinkhole investigations, and identified as needing early attention, have also been grouted up. Work is on-going to grout and install a single optic fibre cable in DH96-34 P1 and DH96-37 P1 to provide additional monitoring capabilities. DH96-34 P2, DH96-37 P2, and DH97-01 P2 have been grouted up with no installation of fibre optic cables.

5.3 MONITORING MEASUREMENTS AT SINKHOLE 1

There is a large number of monitoring installations and instruments in Sinkhole 1 and in the immediate vicinity of the sinkhole. The diameter of the heavily disturbed column of soil down to about 100-ft is estimated to be about 10-ft, and there is more or less disturbed material in a circular column 26-ft in diameter. The sinkhole goes deeper than that, but the estimated diameter is significantly smaller. Not all of the local instrumentation is essential or very useful, and BC Hydro is reviewing the situation.

A review is especially important at this time when plans are being made for the required construction work related to new layer of riprap to be placed on the upstream side of the dam. Heavy equipment, transportation and construction work will affect the monitoring activities at Sinkhole 1, and the instruments considered valuable for future monitoring must be protected to remain functional.

A draft of an Inter-Office Memo (dated 18 November 2015) was made available to the EEP during the present meeting. The EEP has not had the time to study the Memo, but it seems to give a detailed review and analysis of the situation and the possible actions to be taken. Especially the capability to do cross-hole shear wave velocity measurements must be maintained. EEP will not attempt to evaluate the overall situation at this meeting, but based on the Memo and what the EEP learned during their previous site visit, it seems reasonable to decommission the extensometer arrangement at the site. It does not seem to give information of much value to the overall monitoring at this location.

5.4 MONITORING BY CROSS-HOLE MEASUREMENTS OF SHEAR WAVE PROPAGATION

Shear wave velocity propagation measurements are relied upon in the monitoring of the dam's health and behaviour. The measurements are primarily used to observe any changes in the interior of the dam body with time. Reduction in local shear wave velocities from one year to the next may indicate that the shear modulus of the soil is decreasing due to soil loosening and/or effective stress reduction. The EEP recommended in their previous report (August 2012) that BC Hydro explore the possibilities for further improvements in the cross-hole methodology and in the interpretation of measurements. BC Hydro contracted researchers at the University of Texas to explore the possibilities and carry out the development work required. That work is underway, and the researchers seem confident that the improved methodology will increase the value and the information gained from the cross-hole monitoring. To date, November 2015, the basic methodology is developed and is ready to be applied on data from actual measurements in the Bennett Dam.

BC Hydro's attempts to remove the hammer blocking the Cross Arm tube continue. If this proves to be unsuccessful, the Panel recommended that it be replaced by another borehole, located to provide more precise results from cross-hole monitoring.

5.5 ADDITIONAL BOREHOLES

In Section 8.2.1.4 in its 2012 report, the EEP recommended that the ability to do cross-hole seismic measurements between Cross Arm 1 and OW 2 should be restored, or a new hole drilled in the Core if the blockage of Cross Arm 1 cannot be removed.

Furthermore, the cross-hole monitoring at Instrument Plane 2 involves a long distance between source and receiver. Because of this the velocities are dominated by Core which is not affected by the lower velocity material around the Riser. The Panel recommended that a new casing be installed close to the Riser area to allow readings to be obtained which give the required degree of confidence that any tendency degradation will be detected.

The Panel maintains these recommendations.

6 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

6.1 ASSESSMENT OF THE SEEPAGE FLOW CONTROL FUNCTIONS OF BENNETT DAM

Bennett dam is a zoned dam with a wide Core of silt-sand-gravel material, a wide Pervious Shell upstream of the Core and Transition and Filter and Drain downstream of the Core, providing a comprehensive filtering and drainage system to protect the Core against internal erosion.

The Panel previously reported, subject to further tests and investigations, that the filtering system would be effective and was compatible and would prevent erosion in normal seepage flows. The further tests confirm this finding, and have demonstrated that even in extreme circumstances where fines have been washed out of the filtering layers or there was severe segregation during construction, erosion will be arrested, after leakage and erosion in some cases. Below EL 2190-ft there are no situations where erosion after initiation could continue unchecked.

In the 2012 report the Panel expressed concern about the effectiveness of the upper part of the dam in resisting internal erosion. The Filter and Drain are omitted above EL 2190-ft and EL 2160-ft respectively, well below crest level of EL 2234-ft to EL 2230-ft, and the normal maximum reservoir level of EL 2205-ft, and the Transition alone must provide filter protection and adequate drainage capacity at these upper levels.

Based upon the further investigations, laboratory tests and analyses carried out since 2012 the EEP concludes that:

(a) The upper part of the dam is vulnerable to deformation and transverse cracking under seismic loads, in particular in the areas affected by the two sinkholes, and is also vulnerable to seismic loads at the Spillway-Embankment interface.

    In the sinkhole areas the Transition would be required to control internal erosion in cracks in the Core formed under seismic loads. This is particularly an issue at Sinkhole No 1 where the core is damaged by the sinkhole. These cracks might also penetrate through the Transition. While it is likely the Transition would perform its required function, there is a chance that it might hold a crack, or be subject to significant wash-out of the finer fraction. That means the Transition might not perform its function and if this occurred, it could conceivably lead to a breach condition. However, this is unlikely because the analyses have been performed for an earthquake with a low annual exceedance frequency of 1:10,000, and there is a low likelihood that given the earthquake occurs the induced cracks in the top of the dam will penetrate as deep as to full supply level where the dam is very wide. However as discussed in Section 3.4.3 the deformations and cracking are difficult to model because of the complex 3D geometry and the difficulty in modelling the pore pressure build up and loss of stiffness in the loosened fill around Sinkhole No 1 under seismic loading.

(b) At the Spillway–Embankment interface a gap could open up or transverse cracking could occur under seismic loads. Again the Transition would be required to control erosion. As for the sinkhole areas, it is most likely the Transition would perform its required function, but there is a chance that it might hold a crack, or be subject to significant wash-out of the finer fraction. If this occurred, it could conceivably lead to a breach condition. Although it is very unlikely, BC Hydro should study the conditions and contact stresses at this interface in more detail than the Panel has had the time to do, to evaluate the possibility of local cracking and erosion.

Further laboratory testing has been suggested which would assist in assessing the likelihood of the Transition performing the required filter function. There will however remain some uncertainty in the performance. More certainty could be provided in these areas by providing a Filter downstream of the Transition in the two sinkhole locations and at the Spillway–Embankment interface from EL 2190-ft to the crest of the dam. Consideration might also be given to assessing the feasibility of densifying or replacing the disturbed Core and Transition materials at the sinkholes so these areas behave more consistently with the remainder of the dam under seismic loads.

BC Hydro can then decide whether any upgrade works are warranted consistent with their risk informed decision processes.

In the 2012 Report, the Panel recommended that the potential for cementing of filters be investigated. Cementing of the filter materials does occur. Three processes are involved. In the major one, calcite is formed by the reaction of calcium carbonate in the limestone fragments in the glacial till fill materials and carbon dioxide released by ex-solution from the reservoir water as it seeps to ‘atmosphere’ in the partially saturated pore spaces at the phreatic surface. The calcite will build up over a long period along the phreatic surface. It is not expected to have any detrimental effects on the performance of the dam because if the Transition cracks and allows fill upstream of it to erode, the filters downstream of it will arrest the erosion. In the upper part of the dam above EL 2160-ft the studies show that cementation will not occur.

6.2 UNDERSTANDING OF DAM PERFORMANCE

Further seepage analyses have been completed and response times of piezometers and trends over the lifetime of the dam examined. A steady state situation has now been reached and consistent responses have been observed to the annual water level cycle. Use of piezometer records is recommended to augment monitoring at the canyon section where Weirs R1 + R1S do not capture all the seepage. Further investigations of ‘air theory’ or other causes of the very high early pore pressures have not yet been made.

Bennett Dam is located in a region of moderate seismicity, and the Safety Evaluation Earthquake (SEE) was determined to have a horizontal peak ground horizontal acceleration, PGA = 0.16g with an annual exceedance frequency (AEF) = 1/10,000. Dynamic 2-D analyses have been carried out. The assessment was based on the results from the Shawnee earthquake (Oklahoma) which has a very long duration and causes the largest computed build-up in pore pressures and accumulated horizontal and vertical dam displacements.

Three cross-sections (Canyon, Sinkhole 1 and Sinkhole 2) and one longitudinal section along the dam axis were analysed. At the Sinkholes, the analyses included for the upper 100-ft of less dense fill, loosened by the collapse of the lightly compacted fill around the benchmarks.

The computed crest settlements and permanent horizontal displacements caused by the earthquake loading are moderate. For the deep Canyon cross-section, the maximum computed permanent crest settlement is about 1.5-ft. Based on the analyses of the cross-sections with Sinkholes 1 and 2 and the reduced shear modulus also in the Transition zone, the maximum permanent crest settlement at Sinkhole 1 is estimated to be about 4.4-ft. The post-earthquake crest settlement at the location of Sinkhole 1, due to excess pore-pressure dissipation, may amount to 2-ft, making 6.4-ft in all. This may lead to transverse cracking in the top part of the core, but it leaves a freeboard down to full reservoir water level of about 20-ft.

The pore water pressure build-up in the upstream shell corresponds, on average, to a value of r_u = 0.2. The computed post-earthquake factor of safety for the upstream slope is about 1.2 and will gradually increase as the excess pore pressures dissipate.

Uneven horizontal and vertical crest displacements will probably lead to differential settlements, shear distortions and longitudinal and transverse cracking in the top part of the dam. It is very unlikely that the induced cracks at crest level will penetrate as deep down as to full supply level where the width of the dam is about 120 ft.

The earthquake shaking may lead to local tensile cracks and hydraulic fracturing in the Core. Protective Transition and Filter zones are designed to arrest any crack erosion. However, the Transition and Filter zones may themselves be subject to cracking caused by the earthquake shaking and differential distortions. The properties of the Transition and Filter materials in Bennett Dam are such that cracks that have been opened will most likely be closed and self-heal and prevent further erosion, as evaluated in detail in Chapter 2.

The interface between a concrete spillway and an embankment core is a particularly vulnerable location during an earthquake, partly because the dynamic response of the embankment is different to that of the concrete spillway. The upper part of the spillway wall is significantly steeper than the lower part, and a crack may open up when the embankment settles. BC Hydro should study the conditions and contact stresses at this interface at Bennett Dam in more detail and evaluate the likelihood of cracking and possible erosion, as discussed in Chapter 2.

6.3 POTENTIAL FAILURE MODES ANALYSIS WITH RESPECT TO INTERNAL EROSION AND PIPING

The Panel welcomes BC Hydro's decision to adopt a "risk-informed" approach to the management of the safety of Bennett Dam in relation to internal erosion.

BC Hydro has carried out a detailed Potential Failure Modes Analysis (PFMA) of internal erosion and piping failure modes. They have followed the form of event tree modelling of the internal erosion process from initiation to breach which is detailed in the "Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping" (Fell et al 2008), which is used by organisations such as USBR, USACE, and Australian Dam organisations.

In doing this study BC Hydro has taken account of the mechanics of internal erosion applicable to Bennett Dam which include concentrated leak erosion, suffusion and global backward erosion, as detailed in the ICOLD Bulletin 164 on Internal Erosion in Existing Dams Dikes and Levees and their Foundations (ICOLD, 2014). This represents the state of practice at this time.

BC Hydro have estimated in qualitative terms the likelihood of parts of the event tree from the likelihood of initiation of erosion given a flaw in the Core, the ability of the Transition and remainder of the filter system to control erosion, whether erosion will progress, and potentially lead to dam breach.

This provides useful information which has been used to categorize the PFM into classes according to the FERC (2015) approach.

The EEP believes that these classifications might be better differentiated if the frequency of the load condition likely to result in a flaw in which internal erosion might initiate was included.

The EEP is concerned that the time for development of internal erosion from initiation to breach has been overestimated in a number of PFM, and that in reality, concentrated leak erosion may develop in hours or days, not months and years as assumed. This may have important implications for how the risks are managed.

Methods for assessing these times are given in ICOLD (2015) and Fell et al (2008).

6.4 MONITORING BY CROSSHOLE MEASUREMENTS

Improvements have been made or are being considered to monitoring arrangements at Sinkholes 1 and 2, the Observation Wells and to the cross-hole shear wave monitoring of the Sinkholes and the Instrument Islands at Instrument Planes 1 and 2. Detailed proposals for improvements at Sinkhole 1 have been put forward for completion before the rip-rap construction commences. Advice is being taken on improved cross-hole interpretation. The Observation Wells have been grouted up, with piezometer tubes and optic fibres installed to provide continuing and new monitoring systems in future. Attempts are being made to remove the obstruction in the Cross Arm instrument at Instrument Plane 1. If unsuccessful, another borehole should be installed. An additional borehole was recommended by the EEP in 2012 to improve cross-hole sights on the Riser at Instrument Plane 2

6.5 RECOMMENDATIONS ON INVESTIGATIONS AND ANALYSIS

The Panel have made a number of recommendations for further work and given reminders on earlier recommendations not yet completed, including the following:

- Additional sand castle, continuing erosion and other tests to reduce the remaining uncertainty regarding potential of the Transition to hold cracks in the upper part of dam. Consider running some crack-box tests in the apparatus at USBR
- Additional laboratory tests on a wider range of the materials from the Transition and Filter, modelling the actual gradings, not truncated to fit a test cylinder size, and a range of plausible base mesh sizes to represent the filtering zone.
- Consider raising the Filter and possibly the Drain at vulnerable locations (such as downstream of Sinkholes 1 and 2) in the upper part of dam
- Consider the feasibility of densifying or replacing less dense fill in Sinkhole 1 to limit uncertainty about performance of upper part on dam near the sinkhole
- Examine filtering or other improvements at Spillway-Dam interface where a crack may open
- Review bedrock characterisation to retain local information

- Investigate the use of "air theory" to explain and understand the high early pore pressures and their dissipation
- Continue numerical analyses and analyse effect of post-construction settlements on distribution of stresses, particularly to identify any low stress zones. Apply pore pressures measured soon after impoundment to compute effective stresses and minor principal effective stresses at end of construction and define areas where hydraulic fracture may have occurred
- Clear blockage from Cross-Arm or install new borehole. Install additional borehole for improved cross-hole monitoring of soil around riser at Instrument Plane 2

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Appendix A: Information provided to EEP

	REPORTS
1	GEOTECH-392-CWA (FINAL) (no appendices): Evaluation of Potential Failure Scenarios in Upper Part of Dam
	GEOTECH-392-CWA Appendices includes:
	B1 Field work plan,
	B2 Test pits and field sand castle tests,
	B3 Laboratory sand castle tests (Powerpoint summary only),
	B4 Cementing potential studies,
	C Filter compatibility, preliminary, superceded by MER-2014-033 ML_appxB: Preliminary Assessment of Compatibility of the Filter System and GEOTECH-518-ML GMS Filter Assessment Update and 1. CEF Report_text_26Oct15, 2. CEF Report_tables_figures_26Oct15, 3. CEF Report_Appendix A_26Oct15, 4. CEF Report_Appendix B_26Oct15 (see below).
	GEOTECH-392-CWA Figures
	GEOTECH-392-CWA Tables
2	MER-2014-033 ML: Assessment of Filter Capability of the Filter System
	MER-2014-033 ML_appxA: Assessment of Potential for Seepage Induced Internal Instability of Construction Materials
	MER-2014-033 ML_appxB: Preliminary Assessment of Compatibility of the Filter System
	MER-2014-033 ML_appxC: Multi-stage Permeameter Tests on Transition Materials
	MER-2014-033 ML_appxD: Continuing Erosion Filter Tests (CEF) on Core and Transition Materials
	MER-2014-033 ML_appxE: Review of Proposed CEF Testing (by Dr Mark Foster)
3	CEF Report 1_text_26Oct15
	CEF Report 2_tables_figures_26Oct15
	CEF Report 3_Appendix A_26Oct15
	CEF Report 4_Appendix B_26Oct15
4	GEOTECH-518-ML GMS Filter Asssessment Update
5	MER2013-110_GMS11DSD_Seismic_Stability_Assessment_Dec_2013_Main: Seismic Stability and Deformation Assessment
	MER2013-110_GMS11DSD_Seismic_Stability_Assessment_Dec_2013_Appx: Seismic Stability Assessment Appendix A Seismic Parameters
6	Report E1138-Vols.1 and 2 Final: WAC Bennett Dam Site Characterisation Report – Volumes 1 & 2
7	UTexas - Xhole Phase I report Crosshole Shear Wave Velocity Interpretation

	UTexas - Xhole Phase II report(Jul 2015) Crosshole Shear Wave Velocity Interpretation: Numerical Implementation and Validation
8	Report E1213 Surface monitoring report
9	Report E1233 OW Casing Remediation Design Report
10	Report E1234 Hazard Assessment and Remediation Options for 1996/97 Drill Holes
11	Report E1331_Draft (12Nov15) Potential Failure Modes Analysis
12	Memo - Draft Performance Analyses + 8 appendices, figures, etc, as follows:
	Memo - Draft Performance Analyses Appendix A
	Memo - Draft Performance Analyses App A Fig 1 Series
	Memo - Draft Performance Analyses App A Fig 2 Series
	Memo - Draft Performance Analyses App A Tables
	Memo - Draft Performance Analyses App B Tab and Fig
	Memo - Draft Performance Analyses App B Text
	Memo - Draft Performance Analyses Figs. 1 to 23
	Memo - Draft Performance Analyses Figs. 24 to 35
	PRESENTATIONS
	EEP 2015 Meeting Agenda
	EEP 2015 Stress Analyses
	EEP 2015 OW Grouting
	EEP 2015 Upper Dam Cracking Potential
	EEP 2015 Potential Failure Modes
	EEP 2015 Technologies for Dam Safety Monitoring
	EEP 2015 Dam Fill Characterisation
	EEP 2015 Filter Assessment
	EEP 2015 Foundation Characteristics
	EEP 2015 Seismic Analysis
	EEP 2015 Trends and Seepage Analysis
	MEMORANDA
	Rip-rap Update
	Instrumentation Requirements at Sinkhole 1
	WAC Bennett Dam: Modified Sand-Castle Tests by J Fannin, 30 April 2013