BC HYDRO
WAC BENNETT DAM
EXPERT ENGINEERING PANEL
REPORT - VOLUME 1

Kaare Hoeg

Robin Fell

Rodney Bridle

13 August 2012

Report No. N3405

BC HYDRO

WAC BENNETT DAM

EXPERT ENGINEERING PANEL

REPORT - VOLUME 1

AUGUST 2012

Dr Kaare Hoeg
Norwegian Geotechnical Institute
PO Box 3930 Ullevaal Stadion
NO-0806 Oslo, Norway

Emeritus Professor Robin Fell
UNSW Global Consulting Pty Ltd
University of New South Wales
Sydney Australia 2052

Rodney Bridle
Dam Safety Ltd
Amersham HP7 0DT
United Kingdom

BC HYDRO

WAC BENNETT DAM

EXPERT ENGINEERING PANEL

REPORT - VOLUME 1

AUGUST 2012

Prepared by:

Dr Kaare Hoeg
Norwegian Geotechnical Institute
PO Box 3930 Ullevaal Stadion
NO-0806 Oslo, Norway

Prepared by:

Emeritus Professor Robin Fell
UNSW Global Consulting Pty Ltd
University of New South Wales
Sydney Australia 2052

Prepared by:

Rodney Bridle
Dam Safety Ltd
Amersham HP7 0DT
United Kingdom

Report No. N3405
August 2012

BC HYDRO

WAC BENNETT DAM

EXPERT ENGINEERING PANEL March 2012

REPORT - VOLUME 1

CONTENTS

TABLES

FIGURES

APPENDICES IN VOLUME 2

1 INTRODUCTION AND BACKGROUND

1.1 EXPERT ENGINEERING PANEL (EEP) AND TERMS OF REFERENCE

The Expert Engineering Panel was appointed to examine the information available on the history and performance of WAC Bennett Dam and make an independent interpretation of its seepage control functions. Also to determine whether further work is required to decide if a reactive or proactive approach should be taken to remedial work at the dam.

The Terms of Reference are included in Appendix A (in Volume 2). In addition to making the independent interpretation, the EEP were to provide advice on the need for investigations, risk mitigation and developing technologies to maintain the flow control and filtration capacity of the dam, and to deal with known distinct defects such as casings and 'instrument islands' and prepare for any future malfunction including sinkholes.

The members of the Expert Engineering Panel were 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.

BC Hydro provided an extensive collection of information including copies (electronic and paper) of almost all the reports and papers published on WAC Bennett Dam, copies of instrumentation records and copies of many drawings. Lists of most of the documents provided are included in Appendix B. The information covers the entire history of the dam from construction to the present.

Much additional information was passed on in formal and informal presentations and meetings and discussions with engineers working in many roles on the dam. The Panel thanks all these engineers for their conscientious, patient and professional attention to providing the Panel with the information needed for the independent review of the performance of the dam.

The Panel submitted the draft report and appendices for formatting in March 2012, and the final report was completed on 13 August 2012, including improvements and changes in response to comments received.

1.2 DEVELOPMENT, DESIGN AND CONSTRUCTION OF WAC BENNETT DAM 1964-69

An account of the development of the design and the construction of WAC Bennett Dam taken from papers by the engineers and contractors involved is included in Appendix C. Brief details are included here. WAC Bennett Dam, formerly known as Portage Mountain Dam, is a 600-ft high earthfill structure, 6,700-ft long. The dam crosses the Peace River Canyon and the terraces to the right and left of it. The length of the dam across the canyon is about 1,000-ft on the dam axis, about 3,500-ft on the right terrace which has a maximum height at the canyon edge of about 400-ft, and about 2,000-ft on the left terrace with a maximum height at the canyon edge of about 200-ft. The intake penstocks pass under the right terrace and it houses the underground 2,270 MW installed capacity power station. The crest level varies from 2230-ft at the abutments to 2234-ft above the canyon. The fill volume is 57.5 million yds^3. The reservoir with top water level elevation of 2,200-ft is 225 miles long and provides 41 x 10^9 m^3 (35.5 million acre-feet) of active storage. Maximum flood level is 2215-ft, normal operating maximum is now 2205-ft.

The dam is sited in the Peace River Canyon (see plans on Figures 1 and 8 in Figures at end of report and in Appendix C), which is at the end of a broad east-west valley passing through the generally northeast-south west Rocky Mountain ridges. Most of the reservoir is situated in the Rocky Mountain Trench upstream of the valley through the ridge. Downstream of the dam the valley widens and the Peace River which discharges into Lake Athabasca about 520 miles downstream of the dam. Lake Athabasca discharges along the Slave River to the Great Slave Lake and from there to the Arctic Ocean.

The dam section (see Figures 2, 3, 4 and 5) was developed with three factors receiving particular attention:

- Stability of slopes
- Differential movements within the fill under its own weight and the full reservoir load
- Pattern of seepage through foundation and fill as a unit

The dam was thoughtfully designed to the most modern standards of the time. It was painstakingly constructed from 1964 to 1967. It has been carefully monitored since. Filling commenced in November 1967 and reached full storage capacity (Elevation 2,205-ft, 672 m) in the summer of 1970.

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

2.1 ZONING OF EMBANKMENT

Figures 1 to 8 show the features of the dam most relevant to seepage and internal erosion.

Figure 1	Plan layout of the dam, spillway, showing the location of the typical sections, and the location of Instrument Planes 1 and 2; Benchmarks 1 and 2, the Observation Wells, survey station numbers, splitter dykes
Figures 2 and 3	Cross Sections at Instrument Planes 1 and 2 showing the zoning of the dam and the hydraulic piezometers in the dam, instrument risers, Observation Wells.
Figures 4 and 5	Cross Sections at Instrument Planes 1 and 2 showing the zoning of the dam, stratigraphy of the foundation rock, and the hydraulic piezometers in the foundations, and the grout curtain
Figure 6	Longitudinal section along the dam centreline, showing the location of the Instrument Planes, Benchmarks, Observation Wells
Figure 7	Longitudinal Section along the dam centreline showing the foundation, crest and the surfaces for each year of construction
Figure 8	Contour plan of the dam foundation with outline of the Core from the updated contours.
Figure 9	Moisture content, degree of saturation and fines content of Zone 1 Core fill from construction records
Figure 10	Measured seepage v Reservoir level at Instrument Plane 2, Weir 6

The Zoning of the Dam is summarized in Table 2.1.

Table 2.1: Zoning of the dam and description of the function of each zone

Zone	Name	Function, placing methods and standards
1	Core	Low permeability zone to contain the reservoir Placed in maximum 10-inch compacted layers (reduced to 6-inch late-August 1965) Surfaces scarified to about 2-inches before new layer placed Compacted by four coverages 90-ton rubber tyred compactor, 100 lb/sq in tyre pressure, increased to 130 lb/sq in,mid-1965 Density minimum desirable: 98% standard Proctor density (128 lb/cu ft) (98.2%-100% achieved) Moisture content: +1% to -2% of optimum
2	Transition	Fine filter to the Core Placed in maximum 20-inch compacted layers Surfaces scarified to about 2-inches before new layer placed Compacted by four coverages 12,0 00-lb vibratory roller Moisture content: +2% to -2% of optimum Density minimum desirable: 95% vibrated density (130 lb/cu ft)
3	Filter	Coarse filter to the Core Placed in maximum 20-inch compacted layers Compacted by two coverages vibratory roller Moisture content: 5% minimum Density minimum desirable: 96% vibrated density (133 lb/cu ft)

Zone	Name	Function, placing methods and standards
4	Drain	Drain for discharging seepage through the dam and from the foundation Placed in maximum 20-inch compacted layers Compacted by two coverages vibratory roller
5	Pervious Shell	Bedding layer for Rip Rap slope protection
6	Random Shell	Provides the weight for the stability of the dam Placed in 15-inch compacted layers (reduced to 6-inch late-August 1965) Placed perpendicular to dam axis, coarser 'streaks' permissible and provide drainage as a precaution against high pore pressure Compacted by two coverages vibratory roller Density minimum desirable: 95% vibrated density (108-139 lb/cu ft) 90% achieved (125 lb/cu ft) Moisture content: +2% to -2% of optimum
	Rip Rap	Upstream Slope protection against action of waves in the reservoir

2.2 REVIEW OF FOUNDATION CONDITIONS AND PROPERTIES RELATED TO SEEPAGE AND INTERNAL EROSION

2.2.1 Introduction

Three issues for consideration in explaining the behaviour of Bennett Dam which relate to the foundation are:

1. The effects of the foundation permeability on the seepage flow patterns and how these have varied with time since first filling.
2. Whether there has been erosion or deterioration of the grout blanket and curtain, and other foundation treatment which could explain these variations.
3. Whether there has been erosion of the embankment materials into open defects in the foundation of the dam.

This section of the report discusses the first of these issues. The second and third issues are discussed in Section 6.2. More details on all issues are included in Appendix D.

The discussion centres on the conditions at Instrument Planes 1 and 2. Figures 4 and 5 show the location of foundation piezometers.

2.2.2 Geotechnical models at Instrument Planes 1 and 2

2.2.2.1 Stratigraphy and structure

Figures 4 and 5 show the stratigraphy at IP1and IP2. From these and the information in the reports it can be seen that the foundation consists of interbedded shale and sandstone with minor coal. The bedding dips downstream at between 5 and 10 degrees.

The Design Report H 1756 indicates that the sandstones are sparsely jointed. Steeply dipping joints are present, one set striking northwest, and the other northeast. A weaker set strikes north.

2.2.2.2 Permeability of the rock foundation including the effects of grouting

The Design Report indicates that the thick sandstone and shale beds are impermeable. Groundwater flow is mostly along top bedding planes of the shale layers, and within mixed shale and coal layer. The Report indicates that the upper 40 ft weathered surface had an average permeability of 10^{-6} m/sec and the rock at greater depths a permeability of 10^{-7} m/sec.

However there is considerably more data on the permeability of the foundation from the water pressure testing carried out as part of the grouting program during the dam construction. As part of the EEP review this data has been reviewed and a preliminary re-assessment made of the foundation permeability. Details are given in Appendix D.

From these data the following permeability model can be interpreted. A lugeon is a water inflow of 1 litre/minute/metre of borehole water pressure tested corrected to a pressure of 1000 kPa. It is equivalent to a rock mass permeability of about 1.3 x 10^{-7} m/sec.

Instrument Plane 1

N5 sandstone after blanket grouting and washing of alluvium in stress	3 lugeon vertical 30 lugeon horizontal relief defects
N5 sandstone after blanket grouting and limited washing of alluvium	5 lugeon (??, limited data)
N5 sandstone no grouting outside stress relief area	At least 5 lugeon (limited data)
N6 shale and 10 ft into the N6 sandstone,	25 lugeon
N6 sandstone excluding top and bottom 10ft	0.1 lugeon
Bottom 10 ft of N6 sandstone and top 20 ft of N7 shale	15 lugeon
Central 10 ft to 15 ft of N7 shale	0.5 lugeon

Lower 10ft of N7 shale and top 20 ft of N7 sandstone	10 lugeon
Below top 20 ft of N7 sandstone to RL 1380 ft at least	0.5 lugeon

The grouting appears to have been effective in reducing the permeability in the N6 shale and the N6 shale N6 sandstone interface to about 2 lugeon with a likely effective width of 30 ft.

The permeable strata below the N7 shale may not have been as well grouted because the central vertical line does not penetrate that deep. The grout takes in the outer holes do reduce from the primary takes. The permeable strata below N7 may be 5 lugeon with an effective width of 30 ft but more permeable effectively non-grouted sections may exist. This seems to be the case because two piezometers in the N7 shale show near reservoir level pressures indicating the grout curtain is ineffective.

Instrument Plane 2

Upper 40 ft of N3 sandstone	2 lugeon
40 ft to 60 ft in N3 sandstone	40 lugeon
From 60 ft to within 10 ft of the base of the N4 upper sandstone including the N4 upper shale	2 lugeon
From 10 ft above the base of the N4 upper sandstone to the top of The N4 lower shale	0.5 lugeon
N4 lower shale	35 lugeon
N5 sandstone	1 lugeon

The high permeability at 40 to 60 ft below the surface seems likely to be a stress relief feature and not necessarily related to stratigraphy.

The grouting seems to have closed down to about 0.5 lugeon with an effective width about 30 ft.

The Grout Blanket Construction Report by IPEC has drawings showing the expected permeability of the blanket grout based on the closure achieved. These are between 1 and 3 lugeon, so it would be reasonable to adopt 3 lugeon as the permeability of the blanket.

2.2.2.3 Comment on permeability model

It can be seen that the suggested models are quite different to what is recorded in the Design Report.

It is apparent that the rock mass permeability is highest in the margins of shale and sandstone, but that in the Canyon and right bank terrace, at least, stress relief effects are also having a significant influence.

Data from the left abutment indicates valley stress relief has affected the rock mass permeability down to the Canyon floor level. The effects are not as severe or deep on the right bank probably because the bedding is dipping into that abutment. Hence different models may be applicable there. The model for IP2 is also not necessarily applicable to other parts of the right abutment.

The EEP recommends that as part of the current work on the characterization of the dam and its foundation all the available data be assembled and used to develop permeability models. The suggested values above should not be adopted as they are based on a limited assessment of the data. They are included only to emphasise that it is possible to refine the permeability model. This is important because the foundation is one of the boundaries for the dam seepage flows and has a significant effect on piezometric pressures in the dam.

2.2.3 Foundation treatment beneath Core, Transition and Filter

See Section 6.3.3

2.2.4 Foundation deformation properties

See Chapter 4

2.2.5 Alluvium remaining in foundation beneath upstream shell

The Design Report indicates that there is a deep scour hole in the rock beneath the upstream shell. This is filled with alluvium consisting of sand and sand / gravel mixes. Only some of this was removed during construction.

It will be necessary to assemble the data relating to this alluvium because under earthquake loading it may have a build-up of pore pressure and reduction of stiffness, resulting in larger deformations than if the dam was founded upon rock. In the worst case scenario it may liquefy and suffer large loss of strength and stiffness.

This will affect the deformations of the dam in the upstream-downstream direction. This in turn can lead to settlement of the dam in the canyon area and potentially development of transverse cracks in the upper part of the dam.

2.3 PROPERTIES OF FILL MATERIALS

2.3.1 Introduction

The EEP has considered two aspects of the properties of the dam fill materials as part of the Review. These are:

(a) The as constructed particle size distributions of the Core, Transition, Filter, Drain and Random Shell Zones as these affect the filter and drainage capability of the dam. This is discussed in Section 6.5.3 and in Appendix F.
(b) The data which is available relating to the permeability of the Core. This is summarized below and discussed in more detail in Appendix E.

2.3.2 Sources of data on the permeability of the Core.

There are several sources of information available to assess the permeability of the Core:

- Permeameter tests carried out during construction.
- Consolidation tests carried out during construction.
- Well permeability tests carried out during construction.
- Constant Head permeability tests carried out in test pits and Observation Wells during construction.
- Falling head tests in Sonic Drill holes during the 1996/1997 investigations.
- Laboratory permeability tests carried out at Laval University and other laboratories as part of the gas ex-solution theory investigations.
- Simplified analysis of the velocity of the wetting front on first filling from the hydraulic piezometer records.
- Transient seepage analyses of first filling carried out by BC Hydro at the request of the EEP, using SEEP/W, a finite element model.

Table 2 .2 summarizes the results from these sources.

Table 2.2: Summary of permeability data for undisturbed core

Source of Information	Type of Testing	Permeability data		
		Median	Maximum	Minimum
Construction Records: Permeameter	Laboratory	$1.5x10^{-6}$ m/sec	$4x10^{-6}$ m/sec	$3x10^{-8}$ m/sec
Construction Records: Consolidation	Laboratory	$1x10^{-7}$ m/sec RL 1675 ft to RL 1825 ft $2\ x10^{-8}$ m/sec. RL 1825 ft to RL 1900 ft	$1x10^{-6}$ m/sec $4.5x10^{-8}$ m/sec	$1.4x10^{-8}$ m/sec. $1.4x10^{-8}$ m/sec
Construction Records: Well permeability	In-situ	$1.8x10^{-7}$ m/sec	$5.3x10^{-7}$ m/sec	$4.4x10^{-8}$ m/sec
Construction Records: Chasi Tests	In-situ	$2.7x10^{-7}$ m/sec	$8.8x10^{-7}$ m/sec	$7x10^{-8}$ m/sec

Source of Information	Type of Testing	Permeability data		
		Median	Maximum	Minimum
Falling Head in Sonic Core Holes	In-situ	$5x10^{-7}$ m/sec	5×10^{-6} m/sec	1×10^{-7} m/sec
Laboratory permeability from 2003 investigations.	Laboratory Radial (Kh) Axial (Kv)	1×10^{-6} m/sec 3×10^{-7} m/sec	1.2×10^{-6} m/sec 1×10^{-6} m/sec	2×10^{-8} m/sec 7×10^{-9} m/sec
Back-analysis of First Filling Simplified analysis	Based on "In-situ" Data	$5x\ 10^{-6}$ m/sec.		
Transient Analysis of First Filling		$Kh > 10^{-6}$ m/sec.		

The most reliable information for assessing the horizontal permeability is considered to be the permeameter tests carried out during construction, the Laval University tests, and in particular the analyses of the first filling.

The Laval University tests are very important because they show that the saturated permeability of the Core is very dependent on whether the Core was placed dry or wet of optimum, as measured by the degree of saturation of the soil. The ratio of the saturated permeabilities at 45% and 85% degree of saturation at placement is between 60 and 140.

As can be seen in Figure 9 (see Figures), the degree of saturation of the placed fill varied from very low (around 40%) to quite high (85%) almost on a daily basis in 1964, 1965 and the first half of 1966. For the second half of 1966 and in 1967 the upper limit of the degree of saturation was about 70% which means the fill in those periods was placed dry of optimum which has a degree of saturation of about 75%.

The effect of this is that the fill placed in the first two and a half years has low permeability layers throughout which give a much lower vertical permeability than the horizontal permeability.

This anisotropy is very important when modelling seepage pore pressures.

2.3.3 Assessed permeability of the Core and comparison with values used in seepage analyses

From the available data the EEP assess that the following permeabilities are reasonable for the Core in its undisturbed condition:

1964 $Kh = 2 \times 10^{-6}$ m/sec; Kh/Kv = 100

1965 $Kh = 2 \times 10^{-6}$ m/sec; Kh/Kv = 100

1966 $Kh = 2 \times 10^{-6}$ m/sec; Kh/Kv = 10 (First half of season, to end of July)

$Kh = 4 \times 10^{-6}$ m/sec; Kh/Kv = 2 (Second half of season from 1st August)

1967 $Kh = 4 \times 10^{-6}$ m/sec; Kh/Kv = 2

These are significantly higher horizontal permeabilities than used by BC Hydro in seepage analyses and are more anisotropic in the early years of construction.

The EEP recommends that as part of the current characterization project the permeability data be reviewed in a manner similar to this approach. This should include the effects of the gradation of the Core which, as can be seen in Figure 9, varies throughout the fill.

The figures above should not be adopted as they are based on a limited study of the data.

2.3.4 Permeability of the winter horizons

Studies were carried out during construction to assess the effects of frost action on the embankment fill. These are reported in:

> Frost penetration in Portage Mountain Dam, Winter 1964-65.
> Memorandum on Effect of Frost Action on Embankment Fill, February 1965.

From the first report it is apparent that frozen ground extended to a depth of 5 or 6 ft in unprotected zone 1 Core fill. The heave was only 0.05ft or 15mm. This is equivalent to about 1% if taken over the full 5ft to 6ft of frozen soil. Density tests carried out in test pits showed a reduction in density of 2%.

Laboratory tests carried out at University of Alberta are reported upon in the second report. They include plots of % volume change versus moisture content and degree of saturation. They show that the volume change is less than 3% provided the degree of saturation was less than about 80%, and less than 2% for degree of saturation less than 65%.

Based on these data the constructors concluded that the changes in density due to frost were negligible and they allowed placement of fill early in the following construction season on frozen fill provided the upper 1 foot of the fill had thawed and could be scarified and mixed with dry new fill.

Figure 9 shows the records of moisture content and degree of saturation for the Core (Zone 1) for the construction period. From this it can be seen that the degree of saturation varied widely but was below 75% and generally below 70% at the end of the 1964 season; below 70% at the end of the 1965 season and below 75% and generally below 65% at the end of 1966.

From this, using the University of Alberta tests data, the average % volume change would be around 1.5% for 1964 and 1965, and 1% for 1966. This is consistent with the measured value.

It is concluded that the effect of frost on density and void ratio are likely to be small, but that freezing may create a more open or cracked soil structure. It is likely therefore that the winter shutdown surfaces are more permeable than core constructed at the same moisture content and degree of saturation. The effects are likely to be greatest for Core compacted with high moisture contents, and therefore the lowest saturated permeability in the undisturbed state.
Given this, and the attempts by the Constructors to mix dry soil with the wet upper 1 ft at the beginning of the next season, and the highly anisotropic nature of the Core as discussed above, the construction horizons are unlikely to be of significantly higher permeability than the Core generally. If

this becomes an important factor in modelling the piezometric pressures, laboratory tests to assess the permeability after freezing would be required.

2.3.5 Permeability of the other Zones

The EEP has not carried out a review of the permeability of the other Zones. It is noted however that the Random Shell (Zone 6) must have high permeability compared to the Core because the piezometers installed in the Upstream Shell show pore pressures nearly equal to reservoir level during first filling.

This is a guide to the permeability of the Transition as placed because the gradations of the Transition and Random Shell are similar. The permeability of the Transition with higher fines content as a result of suffusion will be lower. However, as the multiport piezometers in the Transition show low or zero pressures, the Transition must be more permeable than the Core.

3 SUMMARY OF DAM PERFORMANCE

WAC Bennett Dam is well instrumented for performance monitoring, and the monitoring has been significantly improved over the years by a dedicated surveillance team and by installing an automatic data acquisition system. Much of the instrumentation inside the embankment itself is concentrated in two instrumentation planes (IP1 and IP2) located in the Canyon Section and the Terrace Section, respectively. The heights of the dam at these locations are about 600-ft (183 m) and 400-ft (122 m).

3.1 DAM DEFORMATIONS DURING AND AFTER CONSTRUCTION

Eight inclinometer casings (later named observation wells, OWs) were installed in the core in various locations along the dam, all approximately 18 ft downstream of the dam axis. One inclinometer was installed in the upstream shell and one in the downstream shell in Instrumentation plane 1(IP1). In IP1 there are also two cross-arm settlement devices (units 1 and 2), one installed in the crest through the core and the other in the downstream shell. These were read regularly during construction and on the same timescale as settlement measurements in the many inclinometers. However, the latch cone device used to read the cross-arm units became jammed in Cross-arm 1 in 1969, and the cross-arm units have not been read since then. Unfortunately, the data from the cross-arm units have not been recovered, and the vertical settlements during construction have had to be based on only the inclinometer measurements.

Based on a re-evaluation of the inclinometer settlements, it is concluded that the maximum settlement during construction in the Canyon Section was about 35 cm and occurred at about half-height of the embankment. This does not include the settlement of the rock foundation which is reported to have been 25 cm during construction. All embankment settlement data have been re-analyzed to derive compression modulus values for the core. The back-analyzed modulus (one-dimensional compression modulus) is found to be high as around 400 MPa, indicating a very well compacted and stiff core.

However, there are some uncertainties associated with the magnitude of settlements obtained by the inclinometers due to possible slip between the casing and the surrounding core material which was not so well compacted (see Chapter 5). Measurements of post-construction crest settlements obtained by surveying of the crest bolts, give settlements which are about twice those obtained from the inclinometer readings. Thus, the maximum construction settlement may actually have been larger than 35 cm, possibly closer to 60 cm. This would give a back-calculated compression modulus in the range of 200 MPa, which still indicates that the core was well compacted and is stiff. Surface bolts are also installed on the downstream berms for measuring vertical and horizontal displacements.

The horizontal displacements along the dam axis were measured by the inclinometers above the steep canyon walls. The maximum horizontal displacement during construction above the right canyon wall was about 5 cm and above the left canyon wall about 6 cm, both inwards towards the centre of the canyon. However, the measurements obtained for horizontal displacements using the early generation inclinometer equipment available at the time are reported not to be very reliable.

The deformations measured during construction should be compared with the results from the numerical analyses described in Chapter 4.

The crest settlement in the canyon section due to impounding and post-construction deformations during the first 3 years after full pool level was reached, amounted to about 8 cm. As this settlement was recorded on the downstream side of the crest, the upstream side of the crest probably settled a little more, but, in any case, these are small values confirming the stiff behaviour. The horizontal downstream crest displacement during the same period was recorded to be about 12 cm.

Post-construction crest settlements have been recorded by the crest bolts since end of construction in 1967. The shape of the post-construction settlement-time curve is very similar to those for most embankment dams, with a fairly high settlement rate during the first years and gradually a much lower rate, often showing an approximately linear relationship when settlements are plotted against the logarithm of time. The post-construction settlements are primarily caused by reservoir impoundment (unless the reservoir is filled during construction), creep deformations, and by the cyclic lowering and rising of the reservoir level. In addition there are settlements caused by any earthquake shaking in seismic regions.

As the crest settlements at Bennett Dam have been obtained by conventional surveying, the measurements at any time will be within about +/- 5 mm (personal communication, Scott Gillis). Therefore, post – construction settlement curves usually appear somewhat irregular (jagged). The Panel does not believe that the irregularities indicate any “soil structure collapses” inside Bennett Dam, as is postulated in BCH Report E301 (March 2005) where the mathematical model for fines migration is presented. The trend of the post-construction settlement curve is clear, and the maximum crest settlement in the deep Canyon Section is now (2012) about 17 cm, increasing about 2 mm per year. This amounts to only 0.1 % of the dam height and confirms the very well compacted and stiff behaviour of the core.

A few years after construction deformation measurements in the inclinometer casings ceased and they were transformed into what are now called observation wells (OWs).

3.2 PORE PRESSURES IN EMBANKMENT AND FOUNDATION

There are 22 twin tube hydraulic piezometers in IP1 and 12 in IP2. In addition, there are 8 vibrating wire piezometers in the upstream shell of IP1 and 4 hydraulic piezometers in the downstream shell. After the sinkhole occurrences in 1996 multiport piezometers were installed in the Transition (Zone 2) in 4 locations. Many hydraulic piezometers were installed at different depths in the foundation to better define the flow regime in the dam-foundation system, to monitor the magnitude of water pressures under the weak mylonite seam which during design was thought could create dam instability problems, and to check the efficiency of the grout curtain and grout blanket. In general, the piezometer installations have proved to be successful and have played a central role in the analysis of the performance of the dam and foundation with time.

Unexpected pore pressure behaviour has been observed in the core. The pore pressures in the downstream part of the core were found to be much higher than those expected, and they were continually rising during the first years after full impoundment. This caused very high gradients on the interface between the core and the Transition zone (Zone2). Then after a few more years the pressures started to decline (in 1974 for IP2 and in 1984 for IP1). Several hypotheses have been put forward to explain this unexpected behaviour. Since about 2005 there have been only small changes in the pore pressure regime as a steady state situation seems to be approached with strongly reduced hydraulic gradients in the downstream part of the core. Plausible explanations for this temporary rise and decline in pore pressures and hydraulic gradients are discussed in Chapter 6.

3.3 SEEPAGE MEASUREMENTS

The designers attempted to measure seepage through different sections of the dam by building four special splitter dykes on the dam foundation to divide the blanket drain into four discrete sections. Downstream of each section, i.e. the Right and Left Bank Sections and the Terrace and Canyon Sections, there are weirs over which the seepage is monitored. Unfortunately, this important part of the monitoring program has not fully achieved its goals in spite of big investments by BC Hydro. The measurements from the Terrace Section seem successful, both with respect to rate of seepage compared to analytical predictions and with respect to measurements of any turbidity. The amount of turbidity is negligible from the Terrace Section. However, according to the results from the 2007 inflow tests (BCH Report E631, 2008) BC Hydro estimates that only about 50% of the total seepage through the Canyon Section is going over the measuring weirs (Weirs R1 and R1S). The rest must be disappearing through joints in the rock foundation. The 2007 tests showed that the piezometers and weirs in the Canyon Section would identify changes in leakage greater than about 300 L/min, very much less than the 100 L/sec that might occur (temporarily and then seal) if a ‘some erosion’ or more serious event occurred (see Section 6.5.2.1 and Terminology). Although the seepage quantity measurements should, therefore, indicate if internal erosion occurs, the turbidity measurements in the Canyon Section cannot give reliable indications of any ongoing fines migration. This is an unfortunate situation which has been the subject of much work but seems difficult to improve further.

3.4 SINKHOLE OCCURRENCES IN 1996

A small pothole was discovered 14 June 1996 in the asphalt pavement on the dam crest in the Terrace Section not far from the right canyon wall. A benchmark casing was uncovered approximately in the centre of the sinkhole, and this was actually how it was discovered that such a benchmark had been put in during construction. Extensive investigations started, and about 21-ft (6.4 m) settlement occurred in a single collapse during initial drilling around the pothole. Subsequent investigations beneath and around the surface expression of the sinkhole revealed an 8-10 ft (2.5-3 m) wide column of highly disturbed core surrounded by a moderately disturbed zone 20-26 ft (6 to 8 m) in diameter. This geometry extended to about 80 m (about 260-ft) depth with evidence of core degradation found below 100 m (330-ft).

During the intensive investigation programme another sinkhole was discovered in September 1996 on the upstream side of the crest close to the steep left canyon wall. This sinkhole was found to be much smaller, but was also centred on a benchmark casing. BC Hydro believes that these are the only two benchmarks that have been installed. They were not in the design drawings, but were put in on the initiative of the contractor to facilitate surveying during construction of the dam.

After rigorous investigations and safety evaluations, both sinkholes were repaired by compaction grouting. The drilling and grouting process was at the same time used to further explore the characteristics of the sinkholes with depth. A discussion of how and why the sinkholes occurred is presented in Section 6.4.

3.5 FIELD TEST TO CONTROL DRAINAGE CAPACITY

During the sinkhole investigation program, boreholes were drilled into the blanket drain under the downstream shell. Samples collected indicated that the blanket drain had a higher fines content than assumed in design and thus, by implication, that the drainage capacity might have been reduced during the operation of the dam between the time of construction and 1996. This led to testing of the blanket drain capacity by injection of water at a high inflow rate at the upstream end of the blanket and by flooding the blanket drain by temporarily damming the discharge and subsequently releasing the flood water. The drain capacity was investigated directly by the large scale injection tests, and the drain capacity was also determined by 3-D, transient seepage modelling of the injection and flood tests using the computer program MODFLOW. The conclusions from these unique tests are that the drainage capacity of the Terrace Section drain and the Canyon Section drain have been shown to be at least 1000 l/s. Based on numerical modelling it is predicted that the capacity is in excess of 3100 l/s, and further estimates indicate that the drainage capacity of the blanket drain in the canyon is in the range of 20 m^3/s.

Since the inflow tests were performed, the downstream toe of the dam has been strengthened with an inverted filter with bigger rocks to avoid any unravelling at the toe for any conceivable leakage flow through the blanket drain.

3.6 CROSS-HOLE SHEAR WAVE VELOCITY MEASUREMENTS

After the sinkhole occurrences the safety monitoring of the dam has been further improved. Eight observation wells (OWs) are systematically being used to perform cross-hole shear wave velocity measurements in various parts of the dam (2 wells have been grouted on the inside and cannot be used for that purpose, but piezometers have been installed in them). The shear wave velocity measurements provide information about the in-situ density (void ratio) and stress condition, and any changes in the measured velocities from year to year are important performance indicators. The measurements have been found to be useful and reliable in detecting any loose regions and/or regions with low effective stresses. They are being performed annually across different planes of various lengths, focusing on the regions around the repaired sinkholes, instrument islands and risers in Instrument planes 1 and 2. The Panel encourages further development of this technique for the performance monitoring of Bennett Dam.

3.7 DAMAGE TO UPSTREAM SLOPE AND REQUIRED REPAIR TO TOP PART OF DAM

Wind generated waves have eroded Zone 5 material through gaps in the damaged riprap. The erosion has been severe between Stations 20+00 and 60+00. The damage to the riprap and the erosion of gravel in Zone 5 have seriously undercut the upper part of the upstream slope as shallow sliding has occurred, and Zone 5 is now exposed in the slide head scarps. Continued wave action, floods and relatively minor earthquake loads may trigger deeper sliding that could endanger the integrity of the crest and even cause overtopping and eventual breaching of the dam.

If instability of the upstream slope were to occur it would remove support for the core, longitudinal cracking of the remaining core would be likely, and transverse cracks in the core might occur, forming a potential pathway for concentrated leak erosion under high reservoir levels.

The Panel considers this to be a serious deficiency that should be remedied as soon as possible. The Panel is pleased to learn that a ”Riprap Upgrade Project” is scheduled to start in 2012 and is planned to be completed within 3 years. It should not be delayed. If an earthquake were to happen before the upstream slope and the top of the dam are repaired, serious damage may occur.

The Panel recommends that when designing the new top part of the dam, BC Hydro considers the effect of earthquake loading (acceleration amplification) and the potential development of cracks in the core that may initiate damaging erosion. In the top part of the dam there is no Filter and Drain on the downstream side of the Transition zone. Thus, there is no protection of the Transition zone, and the drainage capacity in the top part of the dam is much lower than deeper in the dam. The Panel recommends that the scope of the current upgrading project also considers these aspects (see Section 7.2.11).

4 STRESS-STRAIN STATES INSIDE EMBANKMENT

4.1 NUMERICAL ANALYSES PERFORMED AND IN PROGRESS

The main purpose of the numerical analyses is to investigate the stress-strain conditions above and adjacent to the canyon walls and in the vicinity of Sinkholes 1 and 2. Tensile stresses and tensile strains have most likely developed in these regions during construction and during subsequent differential settlements. There may have been larger regions with higher tensile stresses during construction than after end of construction, and this should be investigated. The stress-strain analyses will assist in evaluating the likelihood of crack formation adjacent to the canyon walls and in the vicinity of the sinkholes. Tension regions around the sinkholes contribute to explaining why the sinkholes occurred, how fines may have migrated out of the sinkholes, and how the sinkhole settlements could become so large (see Chapter 6.4).

Previously, approximate 2-D analyses have been performed by BC Hydro (1990, 2005), but now (February 2012) more accurate modelling of bedrock topography and more detailed analyses are in progress.

Dam material deformation properties should be derived from the field deformation measurements presented in Section 3.1.This work is underway, and measured shear wave velocities are also being used to estimate the low-strain modulus (G_{max}) for the materials in the different dam zones. In this connection one should note that the modulus that is derived from the maximum settlements in the Canyon Section during construction, is a one-dimensional compression modulus, not a Young's modulus. Average deformation properties of the bedrock may also be derived from the field deformation measurements.

It is essential that the bedrock topography be mapped fairly accurately because this will have significant effects on local stress-strain magnitudes and distributions. The BC Hydro Engineering Project Team is in the process of doing this detailed mapping (February 2012). The 3-D mapping shows significant bedrock irregularities in the vicinity of both sinkholes. 3-D stress-strain analyses may not be achieved in the near future, but it is essential that the bedrock irregularities are reflected in both the longitudinal and transverse directions in the simplified 2-D analyses.

4.2 ANALYSIS OF EFFECTS OF EARTHQUAKE LOADING

The Panel has been informed that an updated seismic hazard study for the Bennett Dam region is about to be completed. Preliminary results indicate that the seismicity is significantly higher than assumed during dam design and in subsequent safety evaluation analyses. The Panel has been informed that the design earthquake may be of Richter magnitude 6.0 – 6.5, the release of energy is some 30 – 50 km away from the dam site at a relatively shallow depth of 20 – 30 km, and the horizontal peak ground acceleration for the design earthquake may be in the range 0.25 – 0.30 g. With this information, the Panel recommends that a dynamic stress-strain analysis (as opposed to a simplified pseudo-static analysis) be performed. Special attention should be paid to stress-strain conditions adjacent to the canyon walls, potential build-up of pore pressures in the saturated upstream zones of the dam, potential build-up of pore pressures in the alluvium deposit left under the upstream shell/toe of the dam in the Canyon Section, and conditions at the crest of the dam where the ground accelerations are amplified.

5 EFFECTS OF INSTRUMENT INSTALLATIONS ON DAM PERFORMANCE

5.1 INSTRUMENTS AND THEIR LOCATION IN THE EMBANKMENT

There are a number of instruments within the embankment of Bennett Dam which may, because of the way they were installed may affect seepage and seepage flow nets, and / or be potential locations for internal erosion. These include:

Ten Observation Wells (OWs)
Two Cross Arm Settlement Devices (CA)
34 Twin tube hydraulic piezometers in the dam Core including near horizontal tube trenches and two vertical riser pipes.
8 Telemac electric piezometers installed in the Upstream Shell and 4 hydraulic piezometers in the Downstream Shell.

Figures 2 and 3 and Figures G1, G2 and G3 in Appendix G show the location of these instruments.

BC Hydro has been concerned that the Observation Wells and the soil in the immediate vicinity of them may represent a hazard and have considered backfilling them.

BC Hydro have recognized that the trenches in which the piezometers were installed and the vertical risers and surrounding for the tubes and OW2 from EL 1925 to EL 2045 on Instrument Plane 1 and for the piezometer tubes and OW 4 from EL 1955 to EL 2030 on Instrument Plane 2 may also affect pore pressures and may pose a hazard for internal erosion.

At the request of the EEP, BC Hydro has prepared a memo “WAC Bennett Dam-Core instrumentation installation details” which summarizes the installation methods, and includes photographs taken during the instrument installation. This discussion relies largely on that data.

5.2 INSTALLATION METHODS, CONSTRUCTION CONTROL DATA, AND INFORMATION AVAILABLE FROM SITE INVESTIGATIONS

5.2.1 Hydraulic piezometer trenches

The hydraulic piezometers were installed with the tubes in “trunk” trenches excavated into the compacted fill by backhoes. The trenches were 2 ft wide and specified to be no more than 1.5 ft to 3 ft deep to avoid cracking forming in the sides of the trench due to instability of the cut. The trenches in the core were backfilled with a “few inches” of minus 4.75mm Zone 1, the tubes laid, then covered with 2 to 3 inches of the backfill. In the Transition the trenches were backfilled with a maximum of 5% fines.

When trenches were deeper than 3-4-ft, cracks formed about 1-2-ft from side of trench. This was corrected by excavating a 2-ft deep wide cut at sides of trenches, backfilled in 10-inch layers compacted by the 100-ton tyred Core roller.

A review by BC Hydro of the construction photographs and notes indicated that many of the trenches exceeded the 3 ft depth limit as the fill surface was sloped. In some instances the trenches were up to 9 ft deep (for EP 08). These trenches were shored to prevent collapse. Other trenches from 4 to 6 ft deep were not shored.

“Bentonite” cut-offs consisting of 1 part bentonite by volume with 20 parts minus ¾ inch Zone 1 were placed across the trenches at 50 ft intervals through the core. They were at least 12 inches wide and extended 12 inches into the sides of the trench.

The backfill was compacted by hand operated vibratory plates in 4 inch lifts. Compaction may have been made more difficult by the use of wooden guides inside the trench in which the piezometer tubes were run. These appear to have been left in place. Compaction would also have been difficult adjacent the sides of the vertical trenches, and particularly in the trenches which were shored.

A total of 25 density tests were carried out during construction. These resulted in:

- Trench backfill average density ratio 93.9%, range 87.6% to 99.5%, 6 / 13 tests below 92.5%
- Fill adjacent the trenches, average 95.4%, range 91.5% to 100.4% for 7 tests.
- Around piezometers tips, average 94.4%, range 86.9% to 100.2% in 7 tests.

5.2.2 Observation Wells

The observation wells (OWs) were generally installed by excavating pits about 7 ft long by 2.5 ft to 3 ft wide and 7ft to 9ft deep through previously compacted fill. After the OW casing was installed and connected to the casing below, the pits were backfilled with minus ¾ inch Zone 1 material compacted by pneumatic jumping jacks in about 9 inch loose layers reducing to about 6 inch after compaction.

BC Hydro has searched the compaction records for tests carried out in the backfill within 2 ft in the upstream / downstream direction and 5 ft in the longitudinal direction. There are 28 tests with an average of 99.4% density ratio, and only one test lower than 95%. Seven of these tests are tagged as being “Observation Well” so are probably within the pit backfill.

5.2.3 Cross Arm Deformation Devices

The two Cross Arm settlement units are on Instrument Plane 1. Unit 1 is at the downstream side of the crest close to OW2 and the piezometer tube riser. Unit 2 is through the Downstream Shell just above the berm at EL 2010. They were installed by excavating pits in the same manner as the OWs, and Unit 1 was included in the riser from EL 1925 to EL 2045.

5.2.4 Instrument Risers

These islands were oriented parallel to the dam axis, and were of oval shape approximately 8ft to 10 ft wide and 30 ft to 35 ft long. Photographs from during construction indicate the main 90 ton roller was able to get within 2ft to 3ft of the instruments. Between the instruments and presumably within the whole of the area not able to be rolled by the main roller compaction was by hand-operated compaction equipment including plate tampers and a 2 ton Bomag roller. There are photographs showing this equipment.

As pointed out in the BC Hydro Memo this would likely have resulted in lower densities over a larger area than the pit method. BC Hydro has searched the compaction records for tests carried out in the backfill within 2 ft in the upstream / downstream direction and 4 ft in the longitudinal direction for OW2 and 4 ft in the upstream / downstream direction and 12 ft in the longitudinal direction for OW4. There are 6 tests with density ratios all greater than 97%, average 99%.

5.2.5 Telemac electric piezometers installed in the upstream shell

The EEP has not been presented with any information about the installation of these instruments. It seems likely they were installed in trenches in the same way as the hydraulic piezometers.

5.3 EFFECT OF THE INSTRUMENTS ON PERMEABILITY, EFFECTIVE STRESSES, SETTLEMENT, AND THE LIKELIHOOD OF INTERNAL EROSION

5.3.1 Hydraulic Piezometer Trenches

Backfill materials which were compacted to less than 95% density ratio, and particularly lower than 92%, and as low as 88% are likely to be subject to wetting compaction on first filling. This may lead to open pathways in the trenches in which concentrated leak erosion may initiate. The problems are exacerbated by the unusual approach of installing the piezometer tubes in locally deep trenches so the depth of lightly compacted soil is substantial.

The piezometer data on first filling showed very similar rates of advance of the wetting front in all lines of piezometers. The piezometers also trend relatively slowly with time rather than showing sudden rises and drops as would be expected if erosion was occurring along the trenches.

Recent seepage analyses by BC Hydro have assumed that the piezometer tube trenches may, in conjunction with hydraulic fracture of the core upstream of the piezometers, be forming higher permeability surfaces. There is no direct evidence to show this is occurring.

It is concluded that despite the rather poor compaction in the piezometer trenches the piezometer performance indicates that the trenches are not significantly affecting the pore pressures and have not been a location for internal erosion.

5.3.2 Observation Wells

The results of the density tests indicate that the backfill in the pits in which the OWs were installed was compacted to between 95% and 99% density ratio. This would result at most 1% of collapse settlement under high applied stresses. However it is likely that some poorly compacted soil is in the immediate proximity of the OW casing. Arching of the collapsing soil in the narrow trench would limit the effects.

It is assessed that there would be little effect on seepage flows and little implication for internal erosion. These issues are discussed further in Chapter 8.

5.3.3 Cross Arm Deformation Devices

As the Cross Arm units were installed in the same manner as the OWs, including part of Unit 1 being in the riser at Instrument Plane 1, the same discussion and conclusions apply.

5.3.4 Instrument Risers

The cross-hole seismic surveys between OW2 and DH96-35 (Plane 10), and OW4 and DH 96-14 (Plane 19) give some guide to the extent of the lower density in these areas, Figures G4, G5 and G6 show this data.

(a) Instrument Riser on Instrument Plane 1

It can be seen that the riser area in Plane 10 has significantly lower seismic velocity, about 250 ft / sec below the expected velocity from EL 1925 to 1970, and 450 ft/sec lower from EL 1995 to 2045. The 2009 report indicates that the upper boundary of low velocity moved upwards 5 ft in the year. As can be seen in Figure G5 the single measurement taken in Plane 31 which is wholly within the Riser shows a shear wave velocity as low as 700 ft / sec averaging about 900 ft / sec. This indicates a lower degree of compaction and / or stress condition in the riser area than for the Core generally.

The piezometers in this area are EP 18 which is upstream of OW2 and has been relatively stable since 1980 at about EL 2060, and EP14 which is at the same relative location as OW2 and which peaked at EL 2060 in 1980 and has since stabilized to around EL 1990.

This can be interpreted to indicate that from EL 1995 to EL 2045 the relatively less compacted Zone 1 around OW2 has been subject to wetting compaction on saturation, and that there may be some slow extension of the Core fill surrounding the instruments.

(b) Instrument Riser on Instrument Plane 2

In Plane 19 from DH96-14 to OW4 the effect of the riser is quite small, about 100 ft / sec seismic velocity, so it seems that the area around OW4 was better compacted. However the distance from DH96-14 to OW4 is large, about 300 ft, so the effect of lower velocity zones around OW4 are lost by the averaging effect of the well compacted soil beyond the riser.

The piezometers in this area are EP 66 which shows EL between 2150 and 2160, and EP 71 which fluctuates annually between EL 2170 and 2180. The 2009 report indicates that the upper lower velocity zone between EL 2130 and 2170 extended up by 5ft.

The data from construction and the crosshole shear wave velocity testing show that the Core surrounding the risers in Instrument Planes 1 and 2 has been less well compacted than the surrounding Core. It is likely that wetting compaction of the Core around the risers has occurred.

The evidence is that this has not resulted in a cavity or forming a sinkhole above the risers. The soil surrounding the Risers is likely to be higher permeability than the Core generally so could affect the seepage flow. However the piezometer data do not seem to indicate this is occurring.

5.3.5 Telemac electric piezometers installed in the upstream shell

The only data for these piezometers is for first filling. The piezometers were somewhat slow to react to the reservoir, but then rose rapidly to very close to reservoir levels, indicating that the Upstream Shell has a very much higher permeability than the Core. The trenches in which the piezometers are installed are unlikely to have any effect on seepage flow or the likelihood of internal erosion.

6 INTERPRETATION OF DAM PERFORMANCE

6.1 SEEPAGE PATTERNS

Seepage patterns have been much examined because:

- early pore pressures were higher than had been predicted
- some recorded pore pressures may have indicated that internal erosion had been initiated
- most of the seepage analyses seeking to replicate the observed pore pressures have had to invoke fines migration to do so

Other than Sinkhole 1, there have been no obvious indications that internal erosion has occurred, observed seepage waters have never carried sediment, for example. (However, as stated in Section 3.3, in the Canyon Section only about 50% of the total seepage may be going over the measuring weir, so the turbidity measurements there cannot be relied upon).In these circumstances, it would be expected that seepage patterns would not show conditions in which erosion would occur or that models would require erosion to have occurred in order to match measured pore pressures.

6.1.1 Uniform Core or high-permeability layers

Debate has centred on whether the actual pore pressures are best modelled by assuming uniform properties for all the Core fill, or by including a few high-permeability zones (such as the winter shutdown construction horizons) within the less permeable general Core fill and / or temporary “blockages” at the Core /Transition interface. It should be noted that modelling the pore pressures as measured in recent years on either approach will yield similar results. This is because the pore pressures in the bulk of the fill will have reached equilibrium either in the case of the uniform Core as a result of seepage across the Core from the upstream Zone 6 Shell to the downstream Zone 2 Transition; or in the case of the high-permeability layers as a result of seepage up and down from them into the general Core fill.

The challenge is to successfully model the high early pore pressures and the subsequent gradual reduction in pore pressures. If high permeability layers are included from the outset, water seeps through them rapidly and seepage quantities and measured pore pressures should rise rapidly and respond to seasonal reservoir level changes. If uniform less permeable fill is assumed seepage must occur across the whole Core before seepage discharges into the seepage measurement chambers.

Examination of seepage quantities entering the measurement chambers over time, particularly during first filling, may show whether or not high permeability layers are present. If the high- permeability layers are layers, and not temporary openings formed by hydraulic fracture, or concentrated leaks resulting from settlement of fill in instrument trenches for example, seepage would continue to flow in them up to the present. The multiport piezometers installed in 1996, long after first filling, show evidence of flows into the Transition at several levels.

The piezometers are at four levels at Instrument Plane 1 in the canyon at elevations 1680-ft, 1795-ft, 1925-ft and 2045-ft. The three construction horizons are at similar elevations to three of the piezometer lines at 1690-ft, 1900-ft and 2010-ft, only the piezometers at 1795-ft are distant (about 100-ft above and below) from construction surfaces. If the construction horizons are high permeability layers, the piezometers on the construction horizons would respond rapidly and the piezometers at 1795-ft would lag. The piezometer records show no obvious lag between the 1795-ft piezometers and the others and, unless the piezometer trenches are also more permeable zones connected to the upstream Shell, it appears that preferential seepage flow does not occur along the construction horizons.

There are three levels of piezometers in Instrument Plane 2 on the right terrace, at 1865-ft, 1955-ft and 2030-ft. The two construction horizons are at about 1900-ft and 2100-ft, close to only one of the piezometer lines. The piezometers at 1955-ft are about 55-ft above and 145-ft below construction horizons and piezometers at 2030-ft are 130-ft above and 70-ft below construction horizons. The piezometer records show that all the piezometers respond at a similar rate to changes in reservoir level, and at Instrument Plane 2 it also appears that preferential seepage flow does not occur along the construction horizons.

6.1.2 Seepage patterns at Instrument Plane 1

Pore pressures in Instrument Plane 1 in the canyon, other than those at low level (1680-ft) which peaked in 1975, rose to a peak in 1984 and dropped afterwards. Canyon seepage drains towards Weirs R1 and R1S, which have operated satisfactorily only since 2000. However, much seepage bypasses the weirs and it has been found by inflow tests in 2007 that only about 50 % of the seepage passes through the weirs. As there is data only since 2000, the records provide little information about the early high pore pressures, but results from four analyses are recorded in the table below.

A number of seepage analyses have been carried out to examine the effects on pore pressures and seepage quantities of various combinations of fill and foundation permeabilities (both horizontal and vertical). The analyses were carried out at Instrument Planes 1 and 2 in the as-built dam sections and in sections including 'defects' (e.g. winter horizons) and the effects and extent of changed permeabilities resulting from fines migration during assumed local episodes of internal erosion. The analyses were:

- Progressive seepage modelling carried out to 2011. In this modelling, the observed pore pressures were matched by making many changes in properties assumed to have resulted from erosion. These changes were summarised in documents given to the Panel in 2012. They have matched current pore pressures without any changes in properties in recent years, showing that steady state conditions now exist.

- A similar model, the 'concentrated seepage hypothesis', assuming intermittent hydraulic fracture on 'defects' (winter horizons and instrument trenches), was presented to the Panel at the meeting in February and March 2012.

- Transient modelling carried out in 2011 to examine the effect of gradual saturation on flownets, particularly on first filling. The marked difference in permeability of the Core depending on degree of saturation on placement was included in these analyses. The permeabilities assumed in the 'as designed' model and the 'dry of optimum' were too low to cause predicted pore pressures to rise to match those measured. The model including defects achieved a better match.

- An additional 'Dry of optimum' transient analysis was carried out in 2012 at the Panel's request assuming a more permeable uniform (anisotropic) Core (without defects).

The results of these analyses have been used to examine seepage patterns, as summarised in the Tables 6.1 and 6.2:

Table 6.1: Calculated seepage through the embankment compared to measured seepage for Instrument Plane 1

Instrument Plane 1						
Year	Measured seepage L/min (all R1 except 1984)	Calculated seepage L/min	Reservoir Level ft	Changes in properties, dimensions	Calculated /Measured Seepage	Measuring Weirs
Progressive Seepage Modelling to 2011, summarised in 2012						
Ideal		207	2205			
1970		1980	2170			
1974		2172	2205			
1978		1874	2185			
1984	615	2400	2205			Weir R1
1988	190	2370	2190			Weir R1 peak
1992	71	2561	2197			Weir R1
1996	73	2622	2195			Weir R1
2000	638	2654	2189		416%	Weirs R1 + R1S
2004	638	2720	2190		426%	Weirs R1 + R1S
2006	703	2794	2200	Same	397%	Weirs R1 + R1S
2010	564	2604	2183	Small changes	462%	Weirs R1 + R1S
Transient Seepage Modelling Sept 2011 (seepages not recorded)						
As designed (wet of optimum, k_h =1E-06 cm/sec k_v=2E-07 cm/sec)						Pore pressures very low
Dry of optimum k_h=1E-04 cm/sec k_v=2E-05 cm/sec						Pore pressures low
With construction defects Core as wet of optimum, defects in core as dry of optimum						
						Pore pressures similar to measured
Dry of optimum modelling 2012 k_h=5E-04 cm/sec k_v=5E-06 cm/sec						
1990		6530	2194			Seepage high
Concentrated seepage hypothesis 27 Feb 2012						Seepage not recorded

The 2011 Progressive Seepage Modelling gives seepage quantities similar to those measured (on the assumption that only 25% of the seepage was measured at the weirs, although investigations in 2007, BCH Report E631 of October 2008, showed that about 50% of the total seepage passes over the weirs). This model invokes migration of fines to achieve compatibility with measured pore pressures.

The 2012 'dry of optimum' modelling assumes uniform Core without preferential seepage zones. The Transition is flooded to high level and seepage drains down the Filter. The model over-estimates seepage quantities, but selection of lower permeabilities would produce closer agreement. Pore pressure predictions would resemble measured values, but the model has been tested only after the peak of pore pressures, and would need further tests to examine if it could match the high early pressures.

6.1.3 Seepage patterns at Instrument Plane 2

Pore pressures in Instrument Plane 2 on right flank terrace rose to a peak in 1974-75 and dropped afterwards and now seem more or less steady, fluctuating with reservoir level. Seepage from the right flank terrace drains to Weir 6. Good records are available and provide a means of checking analyses, as shown in the Table below:

Table 6.2: Calculated seepage through the embankment compared to measured seepage for Instrument Plane 2

Instrument Plane 2					
Progressive Seepage Modelling (S Garner variations in dimensions and properties)					
Year	Measured seepage L/min	Calculated seepage L/m	Reservoir Level ft	Changes in properties, dimensions	Calculated /Measured Seepage
Ideal		193	2205		
1968		480	2090		
1970		853	2170		
1974	1575	981	2205		62%
1978	1521	845	2185		56%
1984	2103	1407	2205		67%
1988	1732	1308	2190		76%
1992	1885	1418	2197		75%
1996	2027	1407	2195		69%
2000	1775	1396	2189		79%
2004	1758	1460	2190		83%
2006	1834	1593	2200	Same	87%
2010	1537	1368	2183	Same	89%

It can be seen that the modelling, which invokes migration to adjust modelled pore pressures to match measured pore pressures, has predicted seepage quantities below those measured. The assumed properties have remained the same since 2004, and calculated seepage quantities have been about 85% of those measured.

6.1.4 Seepage quantities at Instrument Plane 2

Seepage responds to high reservoir water levels as shown on Figure 10.

The apparently linear relationship between water level and seepage indicates that seepage would cease at a reservoir level of about 2135-ft suggesting that the profile of the dam results in a non-linear relationship and/or that not all the seepage reaches the weir but flows into the rock foundation.

Referring to Table 6.2 and Figure 10, the 'low' 1575 L/min record at 2205-ft water level was in 1974 when pore pressures were at a peak, supporting the proposition that seepage through the dam was restricted at that time. The restriction is variously thought to be because of a skin of fines on the Transition, or because of resistance caused by occlusion and ex-solution of air driven out of the gradually saturating fill but not yet driven into the coarser Transition and Filter downstream (see 6.3.2 below). On examination, it was found that the records earlier than the 1575 L/min reading in 1974 were taken at different weirs and no comparable earlier records were available to provide further evidence on restriction to seepage flows.

6.1.5 Seepage patterns conclusions

The seepage patterns do not support the concept that there are high-permeability layers in the Core through which most seepage occurs. Pore pressures throughout the fill respond to changes in water level more or less simultaneously, there does not seem to be any lag in response between high permeability and low permeability fill.

The seepage quantities were not well modelled in the progressive analyses which invoke migration and high permeability zones to match measured to modelled pore pressures. Adjustments in permeability in the 'dry of optimum' uniform model may achieve a better match.

The present seepage patterns, setting aside disruption that the piezometer trenches and risers may cause, 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. Other mechanisms must be invoked to explain the high early pore pressures, a matter which is discussed in Section 6.3.

6.2 EFFECT OF FOUNDATION ON SEEPAGE

6.2.1 Introduction

This Section discusses the effects of the foundation on the seepage flow nets, and summarizes some observations from a review of the data from the foundation piezometers.

The stratigraphy and permeability model of the foundation is discussed in Chapter 2, Section 2.2. The foundation treatment and grouted blanket and the likely permeability are discussed in Section 6.3.3. Appendix D has a more detailed discussion of these.

6.2.2 Instrument Plane 1

(a) There are generally relatively low pressures in the N5 sandstone below the grout blanket and in the N6 shale. These are lower than the embankment pressures and lower than the pressures in the grout blanket. It is likely that this is a result of relatively high horizontal permeability in the N5 sandstone and N5 shale which is affected by stress relief and the presence of sand seams. The N6 shale and its margins with the N6 sandstone are also relatively high permeability. These strata are acting as a drain to the higher pressures above, below and upstream.

(b) The pressures in the N5 shale, N5 sandstone and N6 shale downstream of the centreline are at tail water level.

(c) There are high pressures within the zone of sand treatment and grouting to 60ft downstream of the grouting culvert. This seems to indicate that grouting was successful as the pressures are contained. There are some gaps noted between the concrete of the culvert and the surrounding rock.
(d) The pressures in piezometers FP40, 41A, and 43 reduced significantly from 1972 to 1987. This coincides with the major drop in pressures in embankment piezometers D1 to D4 indicating the coupling of the flow from the Core into the foundation.
(e) The pressures in the other horizontal boreholes drilled through the grouted blanket in 1987 are much higher than in DH86-1. From this it appears that the under drain effect of the stress relief features in the N5 sandstone is less in these areas. This is consistent with the extent of stress relief features mapped in Figure 4-2 of Report H1973. It means that at Instrument Plane 1 pore pressures in the foundation and, because the embankment and foundation act together, pore pressures in the embankment may not be typical.
(f) There are high pressures in three piezometers in or on the margins of the N7 shale unit. The piezometers fluctuate with reservoir level. There is not a great drop in pressure through the grout curtain. A piezometer at the toe of the dam also shows high pressures in this rock unit. From this it is apparent that the grouting in these strata was not successful in forming a low permeability zone despite achieving closure during the staging of the grout.

The general behaviour of the piezometers is consistent with the permeability model discussed in Section 2.2.

6.2.3 Instrument Plane 2

There is much less data on this Instrument Plane than for Instrument Plane 1, so the conditions are not so well known.

(a) The piezometric pressures in the upper foundation strata are low on this section and Sections A and B. This may be due to the grout curtain and / or the drainage system installed from the drainage tunnel being effective.
(b) It is likely that the 40 lugeon strata from 40 to 60 ft in the N3 sandstone and the underlying strata are acting as a drain to the embankment.
(c) Piezometer FP 15B has high pressures. It is located in the N6 shale but possibly overlaps the N5 sandstone / N6 shale contact. The pressure in FP 15A is lower indicating the lower strata are confined. The pressures vary with the reservoir level. This is because the grout curtain does not extend this deep.

There is a need to better develop the detailed geotechnical models throughout the dam from the stratigraphy, water pressure testing during the grouting and site investigations, and grout takes, and to relate these to the observed piezometer data. This is necessary to get better modelling of the foundation permeability in seepage modelling.

6.3 REASONS FOR TEMPORARY HIGH PORE PRESSURE PATTERN

6.3.1 Fines migration

6.3.1.1 Evidence of fines migration.

Fines migration has almost certainly occurred as evidenced by:

- The presence of additional fines in the Transition throughout the height of the Transition.
- The apparent erosion of Core at Benchmark 1 (see 6.4 below).
- The apparent blocking / unblocking interpreted from seepage analyses to explain the local variations in piezometer readings.

The fines migration generally seen in the Transition is probably the result of suffusion of the Transition but may also include internal erosion by hydraulic fracture in low stress zones, and fines migration resulting from gas ex-solution on first filling more generally throughout the dam. Internal erosion from the Core has most likely contributed to the fines content observed at Benchmark 1, and where the very high fines contents are detected on IP1.

About 20% of the gradations of the as-placed Transition are internally unstable by the Wan and Fell (2008) method. Mobilization of the finer fraction in internally unstable soils occurs at gradients less than 1, so it can be expected that some suffusion will occur in the internally unstable gradation layers in the Transition. This will coarsen the upstream parts of the Transition, and possibly throughout the layers gradation resulting in it not being a no-erosion filter, and therefore subject to “some erosion” and possibly even “excessive erosion” conditions for the Core (refer to Terminology for explanations of terms used in internal erosion).

Given these factors it is not unexpected that some migration of fines has been observed. The relatively steady piezometric conditions now seem to indicate an equilibrium condition of filtering has been reached.

6.3.1.2 The type of internal erosion of the Core

The types of internal erosion of the Core which may have occurred include suffusion, backward erosion, and concentrated leak erosion. Considering each of these:

(a) Suffusion

(i) The Core is all internally stable using the Wan and Fell (2008) method, and all except for about 2 % of the gradations by the Kenny and Lau (1985, 1986) method.

(ii) The UBC testing confirms that it is stable except under very high gradients (>25). The actual gradients measured by piezometers inside the Bennett Core were at no time greater than about 3.

(iii) Higher gradients may have occurred at the Core / Transition interface but the gradients from the downstream piezometers and the Core / Transition interface do not indicate it. In any case higher gradients there are expected if the Transition is performing its role as a filter.

(b) Global Backward Erosion

(i) The UBC experiments are set up as for backward erosion experiments, so the conclusions above apply.

(ii) Global backward erosion tests on broadly graded glacial soil in Australia (the finer of the two samples tested as shown in Appendix F which is similar to the Bennett Core) showed only very minor erosion at the interface of the soil and the (too coarse) filter even under average gradients of 9.

(iii) Given these the likelihood is that global backward erosion may have occurred at the Core / Transition interface but the amount of erosion is likely to be small.

(c) Concentrated Leak Erosion

(i) The geometry of the foundation is such that it is highly likely that there are low stress zones in the dam which would have been subject to hydraulic fracture on first filling. These are low in the dam at the base of steep slopes, over the large changes in height of the dam on both sides of the canyon, and near benches which cross the Core contact.

(ii) The effects of wetting compaction settlement of the relatively lightly compacted core surrounding the Benchmarks would have lead to very low stresses in these areas. This is confirmed by the crosshole shear wave testing. These are quite close to the downstream side of the Core so would have made hydraulic fracture more likely in these areas.

(iii) The laboratory tests for gas ex-solution showed mobilization of particles within the sample. This is a form of hydraulic fracture. It is quite likely to have occurred throughout the core on first filling under the high gradients which would have been present at the wetting front.

(iv) The Core is non-plastic silt sand gravel and would have a very low critical shear stress for concentrated leak erosion. Even cracks / openings 1mm or 2mm open formed by hydraulic fracture would be likely to erode.

(v) These mechanisms should not be on-going as they require first filling conditions. However they may occur high in the dam under high reservoir levels.

(vi) The same effect could have occurred at the IP1 and IP2 Risers but is less likely. The Core surrounding the Risers appears to be relatively better compacted than in the Benchmarks, the Risers are only affecting part of the height of the dam, and most importantly they are in area where the cross valley differential settlements are not causing extension as they are at the Benchmarks. The high fines content in parts of the Transition at IP1 are partly explained by breakdown of coarse particles by the drilling, but may be a result of the mobilization of fines by gas ex-solution as described in (iii). They may be due only to suffusion of the Transition.

6.3.1.3 The likely effect of "sealing" of the Core-Transition interface as the result of internal erosion and the Transition being an effective filter.

In laboratory experiments to assess the effectiveness of filters to arrest erosion high pressures build up at the soil / filter interface when the filter becomes effective. For no-erosion filters this may occur as the hydraulic load is applied, or after a small amount of erosion mobilizes the coarser particles in the soil to allow self filtering to occur. For some and excessive erosion conditions more erosion of the base soil occurs before the particles eroding from the soil forms a filter on the soil / filter interface. In laboratory experiments pressures on the interface after stable sealing of the filter were about 300kPa in Foster and Fell (1999) experiments and 240 to 300 kPa in USSCS (Sherard and Dunnigan, 1985) experiments. That is equivalent to 80 to 100 ft head.

6.3.1.4 Can fines migration explain the temporary high pore pressures?

Inspection of the gradients along the lines of piezometers at IP1 and IP2, at the time of maximum piezometric pressures, and in 2011 after the high pore pressures had dissipated and an apparently near steady state reached showed that:

(a) At the time of maximum pressures (1982-84 on IP1, 1974-75 on IP2), all the upstream piezometers and the second of three piezometers at the 1793ft level on IP1 had pressures close to reservoir level. The downstream piezometers all had pressures significantly lower than the piezometer upstream with gradients of between 0.3 to 1.0 between them.

(b) The 2011 pressures which are shown in Figures 2 and 3 have all reduced considerably from the high values, and the gradient between the downstream piezometer and the piezometer on the same EL upstream are all 0.6 or greater.

So rather than the highest pressure being at the downstream piezometers as would be expected if erosion of the Core with sealing of the eroded soil on the Transition had occurred, they are within the Core. This is a somewhat simplified view of the pore pressures in the full 2-dimensional flow net and it does not rule out that there is some significant head build-up at the Core / Transition interface, but the conclusion remains.

It is also difficult to explain why the piezometric pressures would have reduced so much over time. It would require a breakdown of the filtering capacity at the Core/Filter interface, and in a very gradual manner. It is possible that the suffusion process in the Transition is slow, and may take years. Even so it would require a varying head loss with time over the interface which is not what is seen in laboratory tests or what would be expected from the likely gradation of the filter “seal”.

It is concluded that on balance this mechanism cannot explain the overall behaviour of the piezometers but might be a secondary contributory factor.

6.3.2 Air occlusion and ex-solution at the Core-Transition interface

Many different mechanisms can explain why the pore pressures towards the downstream side of the core may be much higher than anticipated during design. However, only a few mechanisms can at the same time explain why these high pore pressures should decrease with time and approach values corresponding to a steady state situation. The air occlusion concept provides such a plausible mechanism (St. Arnaud, 1995; Sobkowicz et al, 2000):

(a) Water in the reservoir is saturated with dissolved air at atmospheric pressure;
(b) During impounding this water moves through the upstream shell and core, displacing some air, and trapping and driving into solution other air;
(c) As water with dissolved air approaches the downstream face of the core, the pressures drop and air ex-solves forming bubbles;
(d) Thus, the decreasing saturation of the core near the downstream face of the core causes a reduction in permeability and a raising of pore pressures throughout the core;
(e) When the saturation in the soil in the core drops, the air phase starts to become continuous and flows independently of the water;

(f) Over time the amount of air in the dam starts to decrease (all air has been moved through the shell and core), saturation and permeability near the downstream face of the core increase, and pore pressures approach long term values in a saturated steady state flow.

The validity of the concept is documented by for instance the large scale flow tests and numerical models presented by Sobkowicz et al. (2000). The concept has also been used to explain the pore pressure behaviour in some of Hydro Quebec's dams. The migration of air bubbles also seems to facilitate the transportation (migration) of fines as was shown in the large scale experiments by Sobkowicz et al. (2000). Combined with the migration of air, they document some migration of fines.

The Panel is of the opinion that air occlusion and ex-solution is an important component in explaining the high pore pressures and the subsequent reduction in pore pressures and hydraulic gradients on the downstream side of the core in Bennett Dam. During construction the degree of saturation of the Core was fairly low (down to 45% in some locations), and significant amounts of air were therefore available for the air transportation and occlusion process. This led to a very large reduction in Core permeability and a permeability contrast towards the Core-Transition interface.

6.3.3 Deterioration of the Foundation Treatment and Grouting

6.3.3.1 Foundation treatment beneath the Core, Transition and Filter

The foundation treatment beneath the Core consisted of:

- Slush grouting as required to seal any open cracks.
- Application of gunite (also called pneumatically applied mortar, or PAM). This was extensive and only areas of massive sandstone free of joints appear to have not been treated. The drawings say this is to be 2 inches (50mm) thick unless otherwise stated. From the drawings the majority of the shale areas were covered by gunite.
- Smoothing of steps in the foundation by concrete.
- Anchoring of concrete as required to withstand the uplift from grouting.
- A special program of washing out the alluvium which had filled stress relief joints and bedding partings was carried out for a width of 60 ft upstream and downstream of the grouting culvert centreline. This was to a depth of 60 ft or less if there was no alluvium infill. This did not include beneath the culvert. Details are shown in Figure D6 in Appendix D. This treatment was planned to be extended to a further four lines downstream as shown in Figure D6. This was not completed as described in that figure as it was considered unnecessary.

This treatment was based on detailed geological mapping in 100ft x 100 ft areas. Each area has a map and a corresponding treatment drawing.

The foundation for the Transition and Filter were mapped but not to the same detail as the Core foundation. A check of the mapping for the core in adjacent areas in the canyon area showed that there were no areas of open jointing. This is consistent with Figure D6.

The drawings show that steps in the foundation profile for Zones 2 and 3 were treated by concrete as they were for the Core.

In general gunite treatment did not extend beneath Zones 2 and 3 stopping at the downstream boundary of Zone 1. However, for example, there is description of open defects in the foundation of Zones 2 and 3 in the geology map for Area 303 in the upper left abutment and the corresponding treatment drawing shows that this area was covered in gunite. This gives some confidence that the Constructors had a policy of inspecting the foundation for the Transition and Filter, and if there were open defects, treating them as for the Core foundation.

It is reasonable to conclude therefore that there were no areas of wide open joints beneath the Transition and Filter.

The drawings of the upper left abutment show that the excavation of weathered rock was carried out for Zones 1, 2 and 3. That is Zones 2 and 3 were founded on the same surface as Zone 1, not on a steep batter slope.

6.3.3.2 Foundation blanket grouting

The area beneath the Core was blanket grouted. The blanket grout holes were vertical, generally 20 ft deep at spacing 10 to 17ft in most of the valley section. In some areas towards the downstream side of the Core / Transition contact the holes were generally 15ft deep. The 1987 investigations involved drilling several boreholes through the grouted blanket with water pressure testing to determine the permeability in sections of the boreholes. Based on the results of the water pressure tests in those holes it is assessed that the vertical permeability of the grout blanket is 3 lugeons or about 4 $x10^{-7}$ m/sec. This is consistent with a well grouted foundation and indicates that up to 1987 at least the grout blanket had not deteriorated as the report on grouting indicates that the target for closure was between 1 and 3 lugeons.

That means that the grouted blanket is of higher permeability than the vertical permeability of the Core and therefore the foundation, as well as the Core / Transition interface, acts as a drain to seepage through the Core.

The pressures in the rock below the blanket are generally very low, near to tail water level, so any seepage is able to exit without affecting the boundary conditions at the base of the blanket.

The grouting was carried out at initial water/cement ratios of 5:1, and progressively thickened to 4:1, 3:1, 2:1, 1:1 if there was grout take. Grouts with water/cement ratio of 5:1 are potentially not durable but the thicker mixes used where there were takes would be durable.

There is nothing in the foundation seepage or piezometer readings to indicate that there has been significant deterioration of the foundation treatment or blanket grouting since 1987, and therefore since construction.

6.3.3.3 Potential for erosion into the foundation

As discussed in Section D6 of Appendix D there is a very low likelihood of erosion of Zone 1 Core into the foundation. If any occurred it would be of very limited extent (“some erosion” in filter terms) because the core would filter against the defects.

Even though the Zone 2 Transition, and Zone 3 Filter foundations were generally not treated with slush concrete or gunite the lugeon values in the foundations are such that the maximum defect openings would be less than 0.5mm, at most 1mm.

It can be concluded therefore that there is a very low likelihood of erosion of the Transition and Filter into the foundation. If any occurred it would be of very limited extent (“some erosion” in filter terms) because Zones 2 and 3 would filter against the defects.

6.3.3.4 Can deterioration of the foundation treatment and grouting explain the temporary high pore pressures?

Given that:

- The very extensive treatment of the Core foundation including the use of 2 inches of gunite over all shale areas and areas with open joints or bedding partings.
- The foundation was thoroughly grouted with a blanket using procedures which would give a durable grouted rock.

- It was demonstrated in 1987 that the blanket grouting had not deteriorated, and
- The lack of any significant changes in seepage or foundation piezometer pressures since then,

It is concluded that deterioration of the foundation treatment cannot explain the temporary high pressures.

It is also concluded that where there were open defects in the foundation of the Transition and Filter it is most likely that they were treated as for the Core.

If there were any open defects left in the foundation of the Core, Transition or Filter they would be narrow and at worst there would be a “some erosion” condition, so only a small amount of material would erode into the foundation before reaching a stable filtering condition.

6.4 SINKHOLE FORMATION

Immediately after the discovery of the pothole on 14 June 1996 and the benchmark casing in what is now called Sinkhole 1, extensive investigations started. About 21-ft settlement occurred in a single collapse during initial drilling around the pothole. Subsequent investigations beneath and around the surface expression of the sinkhole revealed a 8 to 10-ft wide column of highly disturbed Core surrounded by a moderately disturbed zone 20 to 26-ft in diameter. This geometry extended to about 260-ft deep with evidence of Core degradation found below 330-ft. In-situ stresses in the disturbed zone have been reported to be anomalously low although disturbed and undisturbed core registered similar piezometric pressures.

Two main mechanisms may explain the formation of the sinkholes:

A. Compaction due to wetting of lightly compacted Core material around the benchmark casings (see Chapter 5);
B. Seepage causing internal erosion and migration of fines away from the zone around the benchmark casings.

The conditions in and around the sinkholes are discussed in Appendix H that focuses on whether Mechanism A can explain the large mass volume reductions that occurred in connection with the formation of Sinkholes 1 and 2. Estimates have been made of these volume reductions based on the recorded depths of the sinkholes. There was an increase in the accumulated settlement during the site investigations. For instance, after Sinkhole 1 was first filled an additional settlement of 6 to 8-ft occurred during the cone penetration testing (CPT) and drilling. Total settlement for Sinkhole 1 was estimated to be approximately 33-ft and the reduction in volume approximately 48 cu yds (40 m^3). This amounts to a volumetric strain around 13%.

The Advisory Board, consisting of Drs. Peck and Morgenstern, established immediately after Sinkhole 1 was discovered, maintained their reasoning and conclusion through several meetings in 1996-97 that wetting compaction was the main (only?) reason for the sinkhole formation. Boncompain et al. (1989) refer to very similar situations for some Hydro Quebec dams and conclude that the crest sinkholes may be explained by the settlement upon wetting of poorly compacted material in instrument islands in those dams.

However, based on best estimates of the density (void ratio) of the lightly compacted Core material around the Bennett benchmark casing, results from special laboratory tests on wetting compaction for the Bennett Core material, and results from published results on other soils, the Panel concludes that volumetric strains caused by wetting compaction cannot alone explain the large settlements in Sinkhole 1. For Sinkhole 1 the sinkhole region must also have experienced some loss of Core material by some form of internal erosion and fines migration caused by seepage forces.

However, based on volume reduction estimates, the Panel finds that the settlements and volume changes associated with Sinkhole 2 may be explained by wetting compaction alone.

The challenge with accepting Mechanism B is that there must be an explanation for how the fine material can get out of the sinkhole region. The Panel has considered four mechanisms that in a major or minor way may have contributed to this transportation of fines:

- through cracks caused by hydraulic fracturing in the Core mainly during first reservoir filling;
- internal erosion and loss of material through the bottom of Sinkhole 1 due to the proximity of Splitter Dyke 2 and possible lack of proper filtering;
- through looser and more pervious layers (e.g. construction horizons) in the Core which may lead to fines transport mainly by backward erosion;
- general suffusion of fines through the downstream part of the Core into the Transition zone (Zone 2);

There are many ways in which small cracks can develop in the core even when the dam is well designed and built, e.g. differential settlements leading to stress transfer and local hydraulic fracturing. This is discussed and documented by Sherard (1984 and 1985) and by Peck (1990). Concentrated leak erosion can occur especially during a period of high pore pressures in the core causing fines to migrate through these cracks. Furthermore, the sinkholes in Bennett Dam are in locations where one might expect local tension stresses and tensile strains to develop due to the proximity to the canyon walls and the topography of the bedrock foundation in close vicinity to the sinkholes. This is discussed in Chapter 4.

As discussed in Section 6.5 the Transition which is the filter to the Core is, throughout the dam, partly internally unstable and subject to suffusion. If this occurs some erosion of the Core can be expected before the Transition arrests erosion. As discussed in Appendix H Cone Penetration Test data indicates that this has occurred at Sinkhole 1 at a number of levels.

Splitter Dyke 2 is located on bedrock downstream of Sinkhole 1. It is made of Shell (Zone 6) material with a gradation similar to the Transition. The material in the Splitter Dyke is potentially internally unstable and the finer fraction may erode into the Drain by seepage from the Core and by seepage flow along the rock surface. This erosion process is suffusion. During the period with very high pore pressures in the Core, the local gradient may also have been sufficiently high to initiate erosion of fines within the Splitter Dyke material. So while the gradation of the Splitter Dyke material is not materially different to the Transition it is more likely to have been subject to suffusion allowing some erosion of the Core (see Section 6.5.3.2). This is a mechanism proposed and presented by Stewart and Garner (2000) and the Panel finds it plausible.

However the Panel is of the opinion that the mechanism described under the first bullet point (erosion through cracks caused by hydraulic fracturing in the Core) is the most likely one to have contributed most to the transportation of fines with erosion occurring at several levels in the Core, not only at the base.

There are some indications from the interpretations of seepage analyses, from observations of water flow in the observation wells (OWs), and from cross-hole measurements of shear wave velocities, that the material in the construction layers (horizons) between construction seasons is less dense and more pervious than material below and above these horizons. Frost penetration during construction and Core placement in the late fall and early spring may be part of the explanation for lower density. Thus, these layers may be preferential paths for water flow and possibly transport of fines by backward erosion starting on the core-transition interface. However, as discussed in Section 6.1, based on the seepage pattern during first impoundment, one cannot confirm that the construction horizons have any markedly higher permeability than the Core material above and below.

The Panel does not believe that any significant suffusion can occur inside the Core (see Appendix F) or that any erosion has taken place through the construction horizons.

The formation of voids and loose zones in the sinkhole region due to wetting compaction occurred during and shortly after the first impoundment of the reservoir. Likewise, the formation of cracks in the Core due to hydraulic fracturing also occurred mainly during impoundment, and the transportation of fines took place during the period of high pore pressures in the Core. Since then the cracks have closed due to the lower pore pressures, swelling in the crack walls, consolidation and creep deformations. The potential sinkhole had existed for many years, and the transportation of fines out of the sinkhole region stopped several years ago. However, the surface settlement did not appear until 1996 due to arching of the soil in the upper part of the dam. Now a stable situation has been reached. During the period when internal erosion was occurring, some fines passed into the Transition zone until the grain size distribution of the Transition became such that it arrested further migration. From then on the Transition and Filter have performed as a filter system should and prevented, and continue to prevent, any unsafe situation from developing (Section 6.5).

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

6.5.1 The Bennett Dam filter and drainage system

Figures 2 and 3 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 between 2% and 8% fines passing 0.075mm.

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

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

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
Top of Core	2220ft
Top of Filter	2190 ft
Top of Drain/change in core section	2160 ft
Normal Maximum reservoir level	2205 ft
Normal Minimum reservoir level	2100 ft
PMF Flood Level, current operating rules	2207 ft spring flood; 2208 ft summer flood.

Above EL 2190, the Transition is the single line of filter defence. From EL 2190 to EL 2160 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.

Figures F5 to F8 in Appendix F show the as-constructed gradations for the Core, Transition, Filter and Drain. These are plots of every 5th test carried out during construction. It will be seen that the gradations vary from year to year. Figure F9 shows selected typical for the Upstream Random Fill.

The Transition and Upstream Random Fill are typically somewhat gap graded with a deficiency of coarse sand and fine gravel. The Filter has a somewhat different shaped gradation and is not gap graded.

The ability of the Transition and Filter to arrest erosion in the Core is affected 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. Figure F10 shows what is meant by “finer” and “coarser” fractions.

6.5.2 Assessment of the effectiveness of the filter system

6.5.2.1 Method of Assessment

(a) Allowing for the effects of suffusion.

The effects of suffusion have been assessed by using the method of Wan (2006), Wan and Fell (2004, 2008) which adapts the Burenkova (1993) method to allow the probability of a soil being internally unstable and hence subject to suffusion. Wan (2006) carried out tests on silt-sand-gravel soils similar to the Transition and Filter and is therefore applicable to those materials.

BC Hydro has used the Kenny and Lau (1985, 1986) method to assess internal instability. This method was developed for less broadly graded soils than the Transition and Filter at Bennett Dam, but the results are included for comparison. In their method Kenny and Lau (1985, 1986) indicate that for widely graded materials the limiting amount of finer fraction for internally unstable soils is 20%. That is the controlling factor for much of the Bennett Dam materials.

If a soil is subject to suffusion some or all the finer fraction may be eroded from the soil under the leakage flows in the dam. Wan (2006), Wan and Fell (2006) found that for the soils they tested about half of the finer fraction eroded but the actual amount was dependent on the soil gradation. For this report two amounts have been used: (a) Assuming 50% removed; and (b) assuming 100% of the finer fraction removed. The latter is almost certainly conservative. The method for determining the grading after loss of 50% of the finer fraction is shown in Figure F11.

(b) Allowing for the effects of segregation.

Broadly graded soils such as the Transition and Filter are subject to segregation as they are placed, with the coarser particles separating from the finer particles and collecting at the base of each layer.

Based on the gradations of the Transition and the Core, the relatively wide Transition, and even allowing for the fact the Constructors were aware of the problems of segregation, it is assessed that segregation of the Transition was possible. To see if this is critical, the filter capability of the Transition has been checked assuming complete separation of the coarser fraction from the finer fraction.

Figure F12 shows an example of how the gradation of the Transition or Filter has been adjusted to allow for segregation. This is a conservative approach as it assumes that the coarser fraction separates completely from the finer fraction.

(c) Filter criteria.

Many dam engineers use gradation-based criteria to design filters. These were first developed by Terzhaghi and rely on the D85 of the base soil, and the D15 of the filter. Sherard and Dunnigan (1989) refined these criteria. Their criteria are widely used and are summarized in Table C.1.

These are known as ‘no-erosion’ criteria although in fact they may rely on a small amount of erosion of the base soil to create the self filtering mechanism on the soil-filter contact.

Foster (1999) and Foster and Fell (2001) used the Sherard and Dunnigan (1989) test data and their own tests to refine the no-erosion criteria and to develop ‘excessive’ and ‘continuing’ erosion boundaries. These are summarized in Table F2. Filters which fall between the no-erosion and excessive erosion criteria will experience some erosion before erosion of the base soil is arrested. Those falling between the excessive and continuing erosion criteria will experience excessive erosion. Filters which are coarser than the continuing erosion criteria will not arrest erosion of the base soil.

Foster and Fell (2001) found that information from case histories of poor filter performance suggests the potential maximum leakage flows that could develop due to piping are as follows:

- Filters falling into the Some Erosion category – up to 100 L/sec before sealing

- Filters falling into the Excessive Erosion category – 100 to 1000 L/sec before sealing
- Filters falling into the Continuing Erosion category – flows of 1000 L/sec and increasing.

These criteria were developed with case data including many dams with glacial core materials and are considered appropriate for use with Bennett Dam materials.

6.5.3 Assessment of the filter system from as constructed gradations

6.5.3.1 Assessment of the likelihoods of internal instability

Table 6.3 summarizes the assessment of internal stability. Details are given in Section F3.1 in Appendix F.

Table 6.3: Assessment of the likelihood of internal instability of the Core, Transition and Filter

Zone	Likelihood of internal instability			
	1964	1965	1966	1967
Core	>98% negligible 2% very low	>98% negligible 2% very low	100% negligible	100% negligible
Transition	20% likely	20% likely	50% some chance 10% very likely	50% some chance 10% very likely
Filter	20% some chance	70% very likely	70% very likely	70% very likely
Drain	60% very likely	98% very likely	98% very likely	95% very likely

In this Table to say “20% likely” means that about 20% of the gradations in Figures F5 to F8 are likely to be internally unstable. The Transition samples gradations which are likely to be internally unstable are on the coarse side of the gradation plots in Figure F6.

Permeameter tests were done at UBC on samples determined to be representative of Core and Transition from Bennett Dam. These are reported in Moffat et al (2011) and Moffat and Fannin (2011). Figure F13 shows the gradations of the samples tested.

The Core samples had a gradation on the coarse side of the as-placed Core gradations and the Transition samples approximately at the centre of the as-placed gradations.

These tests showed no significant movement of finer soil within the Core samples until the average gradients were 27 in one test and 29 in a second; and for the Transition 11 in one test and 31 in a second. The gradation of the Transition sample is in the very unlikely range by the Wan and Fell (2008) method, so this result is not unexpected.

Given that the maximum measured gradients in the Core are no higher than around 3, these tests add weight to the assessment that the Core is internally stable and not subject to suffusion.

6.5.3.2 Assessment of the filtering capabilities of the filter system

Table 6.4 summarizes the assessment of the filter capabilities of the filter system using the Foster and Fell (1999) method. These are described in detail in Appendix F.

It can be seen that as placed the Transition is a “no-erosion filter” to all the Core. However suffusion of about 10% to 20% of the Transition is likely, and if this occurs, it is most likely to be only 50% of the finer fraction eroded, leaving the Transition for these gradations as potentially a “some erosion” filter.

The 100% loss of finer fraction and complete segregation are unlikely scenarios, but even if they do occur the Transition will remain a “some erosion” or less likely an “excessive erosion” filter. Hence erosion of the Core may occur but the Transition will arrest erosion for all potential scenarios.

The particular geometry of the Splitter Dykes where seepage flow may bank up against the dykes may make it more likely that suffusion of the Random Fill Zone 6 occurs there than elsewhere. That does not alter the assessment that the Splitter Dykes will act in the same manner as Transition in filtering the Core and act most probably as a “some erosion” filter, but with some chance of acting as an “excessive erosion” filter. There is no scenario that makes the Splitter Dykes act as a “continuing erosion” filter; that is erosion is not arrested.

6.5.4 Assessment of Filter System as shown by site investigations in 1996

Transition to Core

Figures F15 to F17 in Appendix F present gradations of the Core and Transition on samples taken from the boreholes drilled in 1996 and 1997.

From these it can be seen that:

(1) The gradations for the Core are within the range of gradations for the as-placed Core. In particular there are no finer gradations.
(2) The gradations for the Transition are within the range of gradations for as-placed Transition except that there are some finer gradations such as those shown in Figure F17. These gradations may be a result of breakdown of coarse particle due to the sonic drilling. This is discussed more in Section F7.
(3) The coarsest gradation in the Transition in DH 96-38 shown in Figure F17 is on the coarse boundary of the as-placed gradations.

Table 6.4: Summary of the assessment of the filter capabilities of the filter system using the Foster and Fell (1999) method

<table>
<tr><th rowspan="3">Filter interface</th><th colspan="5">Summary of Likely Filtering Performance</th></tr>
<tr><th rowspan="2">As Placed Gradations</th><th colspan="2">Effects of Suffusion</th><th rowspan="2">Effects of complete segregation</th><th rowspan="2">Comments</th></tr>
<tr><th>50% loss of Finer fraction</th><th>100% loss of Finer fraction</th></tr>
<tr><td>Transition as a filter to the Core</td><td>No-erosion for all as placed gradations of Core and Transition</td><td>Some erosion for internally unstable Transition materials</td><td>Some erosion for internally unstable Transition materials. Lesser likelihood of excessive erosion.</td><td>Some to excessive erosion for 1964 and 1965 Transition materials. Excessive erosion for 1966 and 1967 Transition materials</td><td>There is no combination of segregation or suffusion of the Transition which can lead to continuing erosion.</td></tr>
<tr><td>Filter as a filter to the Transition</td><td>No-erosion for all as placed gradations of Transition and Filter</td><td>No-erosion for all internally unstable gradations.</td><td>Some erosion for all internally unstable gradations.</td><td>Small possibility of some erosion for all years of construction</td><td>There is no combination of segregation or suffusion of the Filter which can lead to excessive erosion.</td></tr>
<tr><td>Drain as a filter to the Transition</td><td>No-erosion for 90% of Drain in 1964, and 75% in other years. Remainder some erosion</td><td>No-erosion or some erosion for all internally unstable gradations.</td><td>Some erosion for all internally unstable gradations. Small number of gradations in 1966 and 1967 excessive erosion</td><td>Small possibility of some erosion for all years of construction. A chance of excessive erosion in 1966 and 1967.</td><td>There is no combination of segregation or suffusion of the Drain which can lead to continuing erosion.</td></tr>
<tr><td>Splitter Dyke as a filter to the Core</td><td colspan="5">Splitter Dyke is Zone 6 material with gradations within the envelope for Transition. Hence filtering from Splitter Dyke to Core is as for Transition to Core. Some and possibly excessive erosion might occur between the Drain and Splitter Dyke depending on the actual gradations.</td></tr>
</table>

In view of points (1) and (3) above, there is nothing from the borehole samples which is worse from a filter compatibility viewpoint than discussed above for the as-placed Core and Transition.

Filter to Transition.

There is no data from boreholes for the Filter so the gradations for the Transition from the boreholes must be compared to the as-placed Filter gradations.

When the finest borehole Transition sample (as shown on Figure F17) is re-graded on the 4.75mm sieve it has a D_{85} of 1.6mm. As can be seen from Table F9 this is within the range of D_{85} for the as-placed Transition for 1965, so even though it is a lot finer than the as-placed transition the Filter will still be effective in arresting erosion if the gradation is real and not affected by the sonic drilling process.

6.5.5 Assessment of the likelihood the Transition will hold a crack.

(a) The effect of fines

The upper part of the embankment above EL 2190 ft has no Zone 3 Filter so the filter system relies only on the Zone 2 Transition to arrest erosion in the core above this level.

Transitions and filters which have a percentage of non-plastic fines passing 0.075mm may hold a crack and not perform as required. This is dependent on the percentage of fines and the degree of compaction. Using a method from Fell et al (2008) based on research carried out by Park (2003) and case data, it is assessed that the Core is likely to hold a crack. The Transition is unlikely to do so but there is some chance it may do so in the upper parts of the dam where the fines content of the Transition was higher (3% to 9% fines passing 0.075mm sieve) than elsewhere.

(b) The effects of high carbonate contents in Core, Transition and Filter.

The petrographic analyses reported in MEP 399, January 2000 indicate that the Core, Transition and Drain all contain significant percentages of carbonate rock. X-ray diffraction showed these to be calcite and dolomite in varying ratios from 0.42 to 0.64 for the Transition minus 200 sieve pan samples.

The carbonate contents of the Transition samples were dependent on the fraction being tested. It was highest for the minus 200 sieve fraction (wash), least for the fraction retained on the #60 and #100 sieves (150 micron and 300 micron). The pan and pan (wash) % were from 20% to 50%, the #60/#100 20% to 28%.The Core fines samples had 56% and 36% carbonate in approximately equal parts calcite and dolomite.

Based on these data and the description of potential cementation of dolomite and calcite rich aggregates in Fell et al (2005) there is a possibility that the Transition and Core may cement and hold a crack. The Filter and Drain are unlikely to be affected because they are too coarse.

For both these mechanisms if there was a common cause for cracking, such as cross valley differential settlement, the crack could persist through the Core and Transition.

6.5.6 Assessment of the High Fines Contents detected in the 1996/1997 Investigations

Gradations on samples taken from sonic drill holes drilled into the Transition during the 1996/1997 investigations showed higher fines content than specified or indicated from the construction testing. In particular there are very high fines contents in Instrument Plane 1 below EL 1900ft, and some high fines contents in the same elevations in holes drilled at Benchmark 2 and Instrument Plane 2.

There are a number of possible causes for these high fines contents as summarized in Section F7 of Appendix F.

- They are caused by the sonic drilling process. A small diameter (about 100mm dia.) core barrel was used in these strata. The gradations for the high fines content samples also are deficient in coarser particles so it is almost certain that breakdown has contributed to the creation of fines in the samples.
- They are a result of fines being transported within internally unstable horizons within the Transition. This is to be expected because as discussed above some of the Transition gradations are likely to be internally unstable, with the finer fraction being able to be transported within the Transition under quite low gradients (<1.0). Some finer fraction may also be eroded into the Filter. The fact that higher than as placed fines contents are observed throughout the Transition supports this.
- The fines have come from the Core by erosion. Throughout the dam if the Transition is subject to suffusion it will allow some erosion of the Core. The coarsest Transition after suffusion is for the 1965 construction season with some gradations of Core and Transition after suffusion potentially falling into the excessive erosion category. This coincides with the highest fines contents.

It seems most likely that both mechanisms are present. The very high fines contents are probably mainly due to drill breakdown, but some erosion of fines is likely a contributor as the Transition / Core compatibility is least good in this area.

It should be noted that the suffusion process may take some time. Laboratory tests on a marginally internally unstable soil from an Australian dam eroded for about 30 days under a single gradient. That sample was only 300mm high, so for erosion in the wide Transition it might take years.

6.5.7 Assessed performance of the filter system as evidenced by the 1996/1997 investigations

Most importantly the performance of the dam indicates that the filter system has worked satisfactorily. This is evidenced by:

(a) The piezometers have reached close to what seems a steady state condition.
(b) They have not shown sudden rises and drops as would be expected if the filter system was intermittently letting erosion occur.
(c) The multiport piezometers all show low pore pressures indicating the Transition to Core filter contact has arrested erosion. The fact that some multiport piezometers have small positive pressures and some follow reservoir fluctuations does not alter the conclusion that the Transition has arrested erosion.

(d) The measured seepage rates are within what might be expected given the relatively high permeability of the Core. They do not fluctuate rapidly as would occur if the filter system was only working intermittently.

6.5.8 Drainage capacity of the filter system and its effect on filtering capability

The capacity of the filter system to safely transmit leakage resulting from internal erosion is summarized in Table 6.5.

Table 6.5: Summary of drainage capacity of the filter system

Elevation	Drainage provided by	Likely capacity of the drainage system	Comments
Crest to EL 2190	Transition	Low. Controlled by the permeability of the relatively high fines content Transition.	Failure mode likely to be instability of the downstream slope due to saturation. May be concentrated leak and formation of a pipe if the Transition is cemented or hold a crack due to fines content
EL 2190 to EL 2160	Filter and Transition	Moderate. Controlled by the permeability of the Filter which should be reasonably high.	There is also a large volume of Shell downstream to accept leakage if the Filter cannot cope.
Below EL 2160	Drain, Filter and Transition	Large, controlled by the permeability of the drain which has been demonstrated to be high.	See Sections 3.5 and 3.7

As discussed above, case data from dams which have experienced internal erosion and piping incidents indicate that the maximum leakage rates which may be expected before the filters arrest erosion are:

- Filters falling into the Some Erosion category – up to 100 L/sec before sealing
- Filters falling into the Excessive Erosion category – 100 to 1000 L/sec before sealing
- Filters falling into the Continuing Erosion category – flows of 1000 L/sec and increasing.

It is likely that the small leaks which result from “some erosion” concentrated leak scenarios can be controlled safely at all levels. However leaks for filtering in the “excessive erosion” range may lead to saturation of some of the Transition in the upper part of the dam. This could result in instability of the upper part of the downstream slope. If the Transition were to hold a crack because of relatively high fines content and / or cementing from carbonate aggregates, the failure mode would potentially be gross enlargement of the concentrated leak through what would effectively be an unfiltered exit to the leak. This could lead to loss of freeboard or the mode may develop into slope instability of the upper downstream slope if the enlarged erosion pipe was to collapse.

The critical scenarios in the upper part of the embankment are likely to occur under high reservoir levels. They are most likely to occur in the areas where low stresses may remain in the dam from differential settlement during construction, or under earthquake loading.

The other vulnerable area is at Benchmark 1, where pore pressures in the upper part of the dam are near to reservoir level indicating the core is damaged to at least half way through the zone. The low stresses in the vicinity of this area may allow erosion to initiate due to hydraulic fracture or in existing cracks.

The other scenario which might lead to cracking or creation of low stresses in the upper part of the core is if instability of the upstream slope due to erosion of the rip-rap and underlying bedding leads to loss of part of the crest of the dam.

Lower in the dam below EL 2160 ft the capacity of the drainage system is substantial, and well in excess of the leakage rates expected from case data. The performance of the dam since first filling has also demonstrated its capacity to cope with the leakage / seepage resulting from the internal erosion which has occurred.

Given that the leakage rate is reservoir level dependent, it is conceivable that high reservoir levels could re-initiate erosion at the Core Transition interface low in the dam, resulting in some further erosion before the Transition arrests erosion. It could also lead to suffusion within the Transition leading to coarsening of the Transition, and some erosion of the Core before the Transition arrests erosion.

If the reservoir operating rules under floods were altered to allow storage of the floods and higher flood levels as was the case prior to 1988, the likelihood of internal erosion and piping incidents would be increased.

7 EVALUATION OF DAM PERFORMANCE AND SAFETY

7.1 OVERALL DAM PERFORMANCE

7.1.1 Deformations

The dam crest settlements from end of construction to present are small, indicate stable good performance, and are consistent with what is seen in other dams constructed of similar materials.

7.1.2 Seepage

The seepage flow rates appear to be stable and have been for some years. Seepage quantities respond to reservoir level. No turbidity has been detected, but it is possible to detect it only at Weir 6, draining the right bank terrace. It is not possible to collect all seepage from the canyon section (or reliably detect turbidity there, see Sections 6.1.2 and 8.2.1.8).

7.1.3 Pore pressures

The pore pressures in the Core have reached what seems to be a steady state condition. The seepage gradients within the dam are reasonable. Seepage gradients in central part of Core are generally between 0.7 and 0.8.

7.1.4 Internal erosion in the dam

Information obtained by drilling and sampling towards the downstream side of the Transition zone indicates that breakdown during drilling does not account for all the fines and there has been some internal erosion within the Transition and of the Core. This may be explained by some of the Transition being potentially internally unstable and subject to suffusion. If this happens the Transition becomes somewhat coarser and may allow ‘some erosion’ of the Core. Most of the Transition is a ‘no erosion’ filter to the Core. Some erosion in the Core may have initiated in cracks formed by hydraulic fracture during first filling. Hydraulic fracture and cracks are likely to occur at low stress zones, at and above the canyon sides in particular.

7.1.5 Sinkholes 1 and 2

There appears to have been localized internal erosion at Benchmark 1 (Sinkhole 1). It is not possible to explain all the sinkhole settlement by densification (wetting compaction) of the less densely compacted fill around the benchmark casing. A certain volume of the Core in the Benchmark area must have been lost by erosion. Erosion has occurred through concentrated leaks in cracks opened by hydraulic fracture resulting from stresses lower than pore pressure in the Core, and eroded material has entered the Transition and possibly Splitter Dyke 2. The erosion of Core probably occurred only for a short period until the Transition and similarly graded material in the Splitter Dyke arrested erosion.

At Benchmark 2 (Sinkhole 2), all the settlement observed may be explained by wetting compaction of the less dense fill around the benchmark casing.

7.1.6 Core internally stable and not subject to suffusion

The data on the Core shows that it is not internally unstable and therefore not subject to suffusion which involves detachment of particles throughout.

7.1.7 High early pore pressures and subsequent reduction

The high early pore pressures and the decrease of pore pressure with time in the Core and the Transition are most probably the result of occlusion and ex-solution of air. There was much air in the fill because it was placed at low degrees of saturation. Now that the air, including the free air in the Core and the air taken into solution in the early pore water, has been driven out, normal conditions apply. Some pore pressure changes may also have been due to fines movement in the Transition, the fines having been eroded locally from the Core. There does not appear to have been erosion into the foundation, as was once suspected, and the foundation grouting does not seem to have deteriorated.

7.2 OVERALL SAFETY OF THE DAM

7.2.1 High standards of design and construction

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

7.2.2 High standards of monitoring and surveillance

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

7.2.3 Effectiveness of filtering system

The dam has a good filter system consisting of the Transition, the 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. The Panel has made some suggestions for future investigations by BC Hydro to confirm this assessment (see Chapter 8).

7.2.4 High fines content and cementing of filters

Investigations into the Transition fines content have revealed that the Transition was constructed with fines contents up to 9% in the upper part of the dam. They have also shown that the moraine materials and the silt used as Core, Transition and in other Zones in the dam contain carbonates in the form of Calcite and Dolomite. In some circumstances carbonates can cause cementing of filters.

If the mechanism causing cracking in the upper part of the Core also causes a crack in the Transition, as may happen due to cross valley differential settlement or under seismic loads, water flowing in cracks and carrying eroded particles could pass unfiltered through the cracks, possibly allowing erosion to continue. The dam and foundation are very stiff and creep deformations have occurred for 45 years, so under static loads little movement can be expected in future. However, above the phreatic surface, cracking has been found in other dams many years after construction as they have not collapsed or swelled shut. As the upper part of Bennett Dam is generally still not saturated it therefore remains potentially susceptible to concentrated leak erosion in cracks or hydraulic fracture under high reservoir levels. Cracking or extension of low stress zones susceptible to hydraulic fracture in the upper part of the dam may also occur under seismic conditions. The Panel has recommended further investigations into the high fines content and cementation issues.

7.2.5 Filter system drainage capacity

The Drain has a large capacity to discharge leaks resulting from internal erosion and to prevent instability of the dam. This was confirmed by inflow tests in the drain carried out in 1996-97. The special berm constructed at the toe of the drain in 1999-2000 is designed to prevent unravelling of the toe of the blanket drain under large leak discharges.

However, near the crest of the dam, the drainage capacity is less. The top of the Drain is at EL 2160, and from EL 2160 to EL2190 drainage is provided by the Transition and the Filter. From EL 2190 to the dam crest at EL 2234, the filtering and drainage capability is from the Transition alone, so the drainage capacity is much lower at the highest levels in the dam.

7.2.6 Occasional high seepage flows and pore pressure variations

The Panel assesses that the seepage and internal erosion control systems provided by the Transition, Filter and Drain are adequate to maintain the integrity of the dam. It is however possible that there might be short periods of higher seepage flow and variations of pore pressures if erosion occurring at one location on the Core / Transition interface ceases and erosion commences temporarily at an adjoining area on the interface. This has been seen in dams elsewhere. It is also possible that under high and particularly prolonged high reservoir levels further suffusion of the Transition may occur, resulting in some further erosion at the Core / Transition interface, and changes in the pore pressures until the filter system reaches equilibrium again.

7.2.7 Instrument riser islands

Sinkhole 1 and Sinkhole 2 are directly related to the benchmarks, the lightly compacted Core fill around the benchmark tubes and their proximity to the canyon walls and the bedrock profile. Sinkhole 1 may also be related to its proximity to Splitter Dyke 2. Those conditions do not exist elsewhere in the dam. However, the instrument risers constructed on Instrument Planes 1 and 2 for construction year 1966 have similar lightly compacted Core fill surrounding them. Some wetting compaction settlement could therefore occur around them, and around the Observation Wells and Cross Arm unit nearby.

There is concern (but no evidence to date, see Section 8.2.1.4) that such settlement could conceivably result in a cavity from which a sinkhole could initiate and gradually progress upwards in the Core. Where the sinkhole might progress to is not clear. It should be noted, however, that the Instrument Planes are not close to the canyon walls, the region where low stresses might be present, so the likelihood that the risers may lead to internal erosion is less than in those locations.

For a sinkhole to develop Core fill dropping from the roof of the cavity would need to be carried through the Transition. However, this is likely to, be arrested by the filter action of the Transition.

The lesser degree of compaction could also have resulted in lower stress zones in the Core between the riser and the Core / Transition interface which might have increased the likelihood of hydraulic fracture on first filling. Any such erosion is also likely to be arrested by the Transition / Filter/ Drain system. All the twin tube hydraulic piezometers continue to operate successfully. Therefore, as an indication, any settlement that may have occurred in the risers has been too small to break the piezometer tubes.

It should be noted that there is no evidence of cavity formation or sinkhole development or other changes that could cause concern, as discussed in more detail, with recommendations for monitoring, in Section 8.2.1.4.

7.2.8 Hydraulic piezometers

All the twin tube hydraulic piezometers continue to operate successfully, and any settlement in the risers has been too small to break the piezometer tubes.

The method used to install the piezometers in trenches was not ideal, but there is no evidence to suggest this has led to localised seepage or internal erosion.

7.2.9 Observation wells and cross-arm settlement units

The Observation Wells and Cross-Arm Settlement Units were constructed by over-placing the Core, and excavating back through the compacted soil to locate the casing already installed. The soil in the trench was then placed and compacted by hand equipment. This process would be less likely to result in low stress zones surrounding the wells leading to settlement and the formation of a sinkhole. However, the water levels in the Observations Wells have been observed to fluctuate and to suddenly drop as much as 30 ft as the water is pressured out through the leaky casing joints. This may cause hydraulic fracturing and further loosening of the Core material around the casings. Their location on the downstream side of the crest make it unlikely that any sinkhole would result in overtopping of the dam.

The Observation Wells and Cross-Arm Settlement Unit 1 are important for cross-hole seismic monitoring to identify any further loosening of the lightly compacted Core fill in the sinkholes and riser islands. The Panel recommends that the Observation Wells be sealed on the inside by grouting, but install a smaller diameter casing which would allow crosshole seismic measurements to continue. An alternative that may also be considered is perforating the casings at intervals to prevent build-up of high water levels inside them.

7.2.10 Seismic stability investigations

Seismic stability investigations using the new seismic hazard assessment will be needed. There is particular concern about the crest of the dam because the seismic acceleration at height in a dam is usually amplified significantly above the acceleration at ground level. There is also concern about the liquefaction potential of a 50-ft deep scour hole filled with sands, gravels and boulders over the upstream third of the canyon floor. A lobe of uniform fine sand thought to be susceptible to liquefaction was removed from the surface during construction, but the alluvial sands and gravels below it were left in place.

7.2.11 Rip-rap contract and other issues at crest of dam

The Panel notes that a contract is in preparation to repair the rip-rap on the upper upstream slope of the dam. This upper part of the dam may also be vulnerable to damage during seismic events and may be vulnerable to internal erosion through concentrated leaks in cracks when reservoir level is high. This is particularly an issue at Sinkhole 1 because high pore pressures indicate the Core is damaged, and to a lesser extent at Benchmark 2. The Panel has suggested that the three issues 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.

8 RESPONSES TO QUESTIONS IN TERMS OF REFERENCE

This chapter responds to the requirements of the Terms of Reference, which are included in Appendix A.

8.1 EXPECTATIONS

The Expert Engineering Panel has been provided with the same raw data as is available to BC Hydro Engineering. So provided, the Panel is to arrive at an interpretation of the seepage flow control function of the dam's performance.

8.1.1 Independent interpretation of seepage control function of dam's performance

The Panel considers that Chapter 7 adequately covers the Panel's response to this question.

8.1.2 Basis for determining how BC Hydro's previous interpretations compare with the Panel's interpretation

The Panel is impressed by the thorough evaluations, investigations and analyses performed by BC Hydro Engineering. All important aspects seem to have to have been considered by the Team, and the differences in the interpretations by BC Hydro Engineering and the Panel seem to lie in the relative weight placed on the different aspects and mechanics of the behaviour. Some differences in the interpretation of data and opinions are listed below:

a) The Panel believes that air occlusion and ex-solution has been the main contributor causing the high pore pressures in the core. Air occlusion and ex-solution can also explain the gradual reduction in pore pressures to a stable, steady state situation which seems to have been reached. The Panel believes that fines migration, interface blocking, and subsequent degradation may have contributed, but the major reason is air occlusion (see Section 6.3.2).

b) Some internal erosion in the core has occurred, not by suffusion, but mainly through hydraulic fractures developed during first impoundment. These cracks have subsequently been closed due to the significant reduction in pore pressures in the core, swelling of fines in crack walls, consolidation and creep deformations during 45 years of dam operation (see Section 6.4).

c) The Panel believes 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, have played an important role in the sinkhole formations. Tensile stresses and strains have developed in the vicinity of the benchmarks and have facilitated the hydraulic fracturing discussed above. The tensile zones and crack formation during construction may have been more pronounced than at the end of construction (see Section 6.4).

d) Wetting compaction, internal erosion and the formation of loose zones under Sinkholes 1 and 2 occurred several years ago, mainly during impounding and a few years thereafter, but the sinkholes did not appear until 1996 due to arching in the soil above. For Sinkhole 2 wetting compaction may explain all of the sinkhole settlement (see Section 6.4).

e) The Panel puts much more emphasis than previous interpretations on the analysis and ability of the filter system to arrest any internal erosion from continuing, than on the initiation of the erosion process itself (see Section 6.5).

f) The Panel does not believe that the previous mathematical modelling of fines migration, simulating a suffusion process in the core and a resulting loosening of the soil creating an unstable and collapsible soil structure, may be used to explain the dam behaviour and the crest settlements that have occurred or to predict future settlements.

8.1.3 Determine what further information, analyses and/or performance indicator are required in order to evaluate if or when it would be appropriate to move from a reactive to a proactive approach in regards to remedial work at the dam

This question is covered by the discussion of risk mitigation works in Section 8.2 below.

8.2 OVERALL QUESTION: ARE THERE ISSUES THAT REQUIRE RISK MITIGATION OR INVESTIGATION AT THIS TIME?

What follows is the Panel's response to the question: "Does Bennett Dam present any issues that require risk mitigation or investigation at this time in the context of:

- The known distinct defects such as casings and instrumentation "islands" and trenches, winter horizons etc. and
- General flow control and filtration considerations."

8.2.1 Risk mitigation works and investigations at known defects

8.2.1.1 Repairs of rip-rap and reduction in the likelihood of upstream slope instability

The EEP understands that BC Hydro is progressing with the investigations and design of the repairs to the erosion and shallow slope instability which has occurred on the upstream slope by wave action. This work is important not only from the viewpoint of controlling damage to the upstream slope and crest of the dam, but also from the viewpoint of mitigating the risks from internal erosion. If instability of the upstream slope were to occur it would remove support for the Core and longitudinal cracking of the remaining Core would be likely. Furthermore, transverse cracks in the core might occur, forming a potential pathway for concentrated leak erosion under high reservoir levels.

Given the relatively poor drainage capacity available at the crest of the dam by the Transition, and there is no Filter or Drain, this could be a significant contributor to the likelihood of failure of the dam unless remediated.

Earthquake effects should also be considered when upgrading the top of Bennett Dam.

8.2.1.2 Internal erosion at Benchmarks 1 and 2

The EEP notes that piezometers sited in the upper part of the dam at Benchmark 1 are showing pressures virtually at reservoir level and following reservoir level fluctuations. This indicates that the Core is damaged and is of high permeability or cracked to at least half way through the Core. Pore pressures at Benchmark 2 are not so high, but there are too few instruments to get the complete picture. The crosshole seismic testing does indicate that some unfavourable changes may be occurring because the low velocity zone seems to be working upwards into the Random Shell. In view of this and their location in potentially lower stress zones due to cross valley differential settlement, these areas stand out as having a higher likelihood of internal erosion than other areas. If it occurs, the internal erosion is likely to be concentrated leak erosion, and while the Transition should arrest any erosion this may not occur before some erosion or conceivably even excessive erosion occurs, with resultant leakages which may overtax the drainage capacity of the Transition and there is no Filter or Drain in the upper part of the dam to back up this capability.

If the Transition holds a crack because of high fines content or cementation of carbonate aggregates in the Transition the filter capability may be compromised.

It is recommended therefore that as part of the investigations and design of the remedial works an assessment be made of the potential for internal erosion in these areas (see Section 8.2.2.1). If remedial works are needed, they could be implemented as part of the Upstream Slope Instability remediation project.

8.2.1.3 Investigation of the risks posed by casings and instrumentation and the feasibility of remediation work to mitigate risks

There are a number of casings and other instrument related singularities which warrant investigation to assess the risk they pose to dam safety, and the feasibility and cost of carrying out remedial works to mitigate the risks if warranted. These are:

8.2.1.4 Instrument risers

The data from construction and the crosshole shear wave velocity testing show that the Core fill surrounding the risers in Instrument Planes 1 and 2 has been less well compacted than the surrounding Core. Settlement due to wetting compaction of the lightly compacted Core fill around the risers may have occurred, the evidence is that this has not resulted in a cavity at the top of the risers or the forming of a sinkhole above the risers. If a cavity were to develop it could emerge on the dam surface as a sinkhole, it is not possible to predict where it might emerge but if it were to emerge in the upstream slope below reservoir level, there could be a large increase in pore pressure into the riser area. This could initiate internal erosion by hydraulic fracture. The filter system is likely to arrest erosion which might initiate before the drainage system is over-taxed because the filter system in these circumstances is at worst in the ‘excessive erosion’ range, and more likely the ‘some’ or ‘no erosion’ range.

The evidence from crosshole seismic tests indicates that there is no cavity at the top of the risers and that the low velocity zone (indicating less dense fill) is not extending above the risers so the situation seems to be stable. The risers are at a considerable depth in the dam, and seasonal variations in reservoir water level have very limited effects on conditions in and around them. In such constant conditions, changes that would result in cavity formation and the development of sinkholes seem unlikely, as discussed below.

The EEP has considered whether it is necessary and feasible to improve the degree of compaction of the Core surrounding the risers. The points to be considered include:

(a) There is evidence of loosening and / or lower stresses in the risers but the velocities are not particularly low at about 600 ft /sec.

(b) The seismic data indicates that there is not a cavity formed above the risers, and that the velocities have been stable since 1996/7.

(c) The data available regarding compaction around the risers is that the Constructors made an effort to compact the soil to around 95% density ratio, and probably achieved this except possibly immediately adjacent the instruments where it would have been difficult to get in even with small equipment.

(d) Given this and the experience at Benchmark 2 sinkhole, the likely amount of collapse settlement would be about 6 ft. Even if this formed a cavity 6 ft high that amount of strain could be absorbed in the overlying about 180 ft of Core and Upstream Shell without forming a sinkhole.

(e) To form a sinkhole would probably require erosion of the Core between the Risers and the Core / Transition interface. This is less likely to occur at the Risers as the Instrument Planes are in areas where cross valley stresses would be compressive, whereas Benchmarks 1 and 2 are in potential extension zones.

(f) The piezometer readings indicate that a stable condition has been reached and the area around the Risers are now saturated. These are conditions which will not lead to hydraulic fracture unless there is a major sudden change in the pore pressures in the Riser area. This would require the formation of the sinkhole and that is a low likelihood.

(g) The filter system can be relied upon to arrest erosion given erosion were to initiate.

(h) If a cavity was to begin forming it is likely to do so slowly and this should be detected by the crosshole seismic testing.

(i) Any attempt to densify the Core surrounding the Risers would almost certainly result in damage to the piezometer tubes in the riser with loss of the ability to monitor pore pressures in the upper parts of IP1 and IP2.

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 crosshole seismic testing at annual intervals. However, to be confident of this, the ability to do crosshole seismic between Cross Arm 1 and OW2 should be restored. If the device blocking Cross Arm 1 cannot be removed, a new hole should be drilled to allow the crosshole seismic testing within the Riser area to be carried out. Rather than drill a new hole in the less well compacted Core in the Riser the EEP suggests locating any new hole in the Core fill, close to, but not in, the riser area.

The crosshole seismic testing at 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 therefore 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 development of a cavity will be detected.

8.2.1.5 Observation Wells

The Observation Wells (OWs) have benefits in that they are used for the crosshole seismic testing, and provide some pore pressure data even though they are not piezometers because they do not have a sealed entry zone. They pose a potential hazard in that the Core surrounding them may not be as well compacted as the Core generally, but the amount of less densely compacted soil is less than at the Risers or the Benchmarks because they were installed in pits excavated in the Core and the backfill compacted with hand compactors. Sudden drops in water level of up 30 ft have occurred and this could conceivably cause local hydraulic fracture around the OWs.

If the Observation Wells are sealed on the inside by grouting, it would be desirable to install smaller diameter tubes in the casings which would still allow the cross-hole seismic surveys to be continued. Alternatively the casing could be perforated to make them leakier while allowing their continuing use for cross-hole testing and as a standpipe well for observation of water levels. This would have to be done in a way that did not allow erosion of the surrounding soil into the casing. An even better option would be to install piezometers in discrete sections of the casing so pore pressures could be measured as well as having the casing for the crosshole seismic testing. This may not be possible given the small diameter of the current casing.

It is suggested that BC Hydro investigate the feasibility and cost of these options, and whether the OWs pose a significant contribution to the likelihood of internal erosion with or without these upgrades.

The EEP does not feel that based on the available information, it is warranted to try to further compact the soil surrounding the OWs.

8.2.1.6 Piezometer Tube Trenches

As discussed in Section 5.3.1 the methods used to install the piezometer tubes were not ideal in that some were installed in quite deep trenches. The backfilling was not well compacted. Despite this there is no indication that the trenches are causing problems with the piezometer readings or are posing a hazard to internal erosion. This is probably because the cut-offs constructed across the trenches every 50 ft with bentonite core mix are effective. In any case the filter system should arrest any erosion which might initiate. Given this and that it is virtually impossible to do anything about the issue, the EEP suggests nothing be done other than to keep inspecting the piezometer data for signs of irregular readings.

8.2.1.7 Winter Construction Horizons

As discussed in Section 2.3.4 the EEP concludes that the winter construction horizons are not likely to be significantly more permeable than the rest of the Core.

8.2.1.8 Seepage monitoring system

It is highly desirable to have a monitoring system which is able to monitor all the seepage through the dam. This is particularly important at Bennett Dam because of the history of the piezometer fluctuations with time and the probability that some internal erosion has occurred.

The monitoring system is thought to collect all the seepage in the upper left and right abutments, but in the canyon zone inflow investigations in 2007 found that only about 50% of the seepage is measured by Weirs R1 and R1S. BC Hydro has spent significant sums of money in the past to try to improve the % of seepage measured in the canyon area with little success, and further improvements do not seem feasible. Turbidity measurements at Weirs R1 and R1S are not reliable, but sudden inflows in excess of about 300 L/min would be identified by the weirs and the piezometers in the canyon area. As this is less than the increased leakage expected from piping prior to sealing following a temporary malfunction of filters, further efforts to improve matters are not justified.

8.2.2 Risk mitigation and investigations relating to flow control and filtration

8.2.2.1 Investigations to confirm the filter and drain system is effective

The EEP recommends that the following investigations be carried out to 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. This will involve:

Testing for Internal Instability and Suffusion

(i) Tests on internal instability of the Transition, with emphasis on the coarser materials which are more likely internally unstable.

(ii) These tests will need to be large diameter so the full gradation can be tested, compacted at the moisture contents and density representative of what they were during construction of the dam. The tests should involve checking for movement of finer fraction within the sample as well as out of the sample. Some tests may need to be long term, possibly months to check time dependency. It is suggested that initially the tests be carried out in a test set-up without the ability to replicate the stress conditions in the dam. This is likely to yield somewhat conservative results for internal stability but will keep the testing equipment relatively less costly than if the stresses are to be replicated. If these tests give unexpectedly adverse results then further tests could be carried out under high stresses which may inhibit the movement of particles.

(iii) Actual gradations as in the dam should be tested, not average gradations as experience elsewhere with testing for internal erosion is that small variations in gradation affect the outcomes. It is essential to test samples representative of the coarser Transition as it is these which are likely internally unstable.

(iv) Tests should be carried out under gradients up to say 10 to check for internal instability. These should be supplemented by tests to assess the gradient(s) at which parts of the finer fraction are eroded.

Filter Tests

(v) No-erosion filter tests on representative samples of the Core, and Transition after suffusion. These also will have to be large diameter so as to test the full gradation of the Transition. The pre-formed hole in the sample used to simulate a crack may need to be larger than the normal 6mm.

It is suggested that BC Hydro engage Dr Mark Foster, URS Australia to assist in the design and to be present for the initial testing. He has extensive experience in these techniques as a researcher and later in practice with similar materials in New Zealand and Australia.

(b) Carry out investigations to determine whether the Transition in the upper part of the dam may hold a crack.

This is necessary because the as-placed % fines were as high as 9%. It is suggested that the modified Vaughan method developed by researchers in Iran and about to be published in ASTM Geotechnical Testing Journal titled "A review of the sand castle test for assessing collapsibility of filters in dams” is used. If test pits are excavated into the Transition to check on cementation by carbonates they may also yield useful information on the effect of high fines contents. The distribution of fines within the upper part of the dam should be determined from the construction records with a view to seeing the distribution spatially and to check on the potential continuity of higher fines content material.

(c) Carry out investigations to determine if the Transition may be cemented because it has a significant % of carbonate rock.

This would include excavation of test pits into the Transition from the crest of the dam.

(d) Assess the drainage capacity in the upper part of the dam.

This can be done by assembling the available information on the permeability of the Transition and Filter, relate this to the distribution of fines contents of the Transition materials used in the upper part of the dam above the top of the drain, and assess whether there is sufficient drainage capacity to cope with any foreseeable concentrated leak or other internal erosion scenario.

8.2.2.2 Investigations to develop a better understanding of the behaviour of the dam

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.

In particular re-assess the permeability of all Zones in the Dam, and the strata in the foundation. The EEP has looked at this as reported in Appendices D and E but BC Hydro needs to do this systematically.

(b) Use these data as the base properties to be used in seepage analyses.

(c) Further improvement in crosshole monitoring methodology and interpretation of measurements.

The Project Team are planning to seek advice from an experienced geophysicist to see whether more information can be extracted from the crosshole testing. The EEP supports this proposal although we are impressed by the level of expertise within BC Hydro.

(d) Further investigate the gas occlusion and ex-solution theory to explain the long term pore pressures in the dam.

Include in this assessment the progression of the wetting front on first filling, and take account of the actual degrees of saturation of the fill, likely anisotropy effects, foundation permeabilities which bound the problem, etc. Additional laboratory testing of representative samples of Core will be needed rather than relying only on the Laval University test.

(e) Complete the 2D numerical analysis of stresses and strains in the dam. At this stage the EEP considers that it is not necessary to progress to do 3D analyses as suggested earlier. The areas potentially subject to hydraulic fracture should be assessed for the first filling case using pore pressures as recorded then, and for the 2011/2012 pressures. For the longitudinal model particular attention should be paid to stresses at the base of and above the top of the steep slopes which form the canyon walls; and the sharp changes in profile at Stns 2+350 to Stn 2+600, 2+700 to 2+900; 3+00 to 3+100; 5+900; 6+700 to 6+850. The sensitivity of the outcomes of this analysis to the input parameters should be assessed. The results of the longitudinal model will be most sensitive to the assumed Poisson Ratio of the Core. The cross section models will be most sensitive to the relative moduli of the Core, Transition, Filter, Drain and Random Shell.

REFERENCES

Boncompain, B.,Pare, J.J. and Levay, J.(1989) Crest sinkholes related to the collapse of loose material upon wetting, Proc. 12thInternational Conference on Soil mechanics and Foundation Engineering, Rio de Janeiro, pp.1797 -1801

Fell, R., MacGregor, P., Stapledon, D. and Bell, G. (2005) *Geotechnical Engineering of Dams*. Balkema, Leiden ISBN 04 1536 440 x

Peck, R.B. (1990) Interface between core and downstream filter, H.Bolton Seed Memorial Symposium, May 1990, pp.237-251

Sherard, J. L. (1984) Proc. Of International Conference on Case Histories in Geotechnical Engineering, University of Missouri-Rolla, Vol. IV, pp.1613-1616

Sherard, J.L. (1986) Hydraulic fracturing in embankment dams, ASCE, Journal of Geotechnical Engineering, Vol. 112, No. 10, October

Sobkowicz, J.C., Byrne, P., Leroueil, S. and Garner, S. (2000) The effect of dissolved and free air on the pore pressures within the core of the WAC Bennett Dam, Proc. Of the 53rd Canadian geotechnical Conference, Montreal

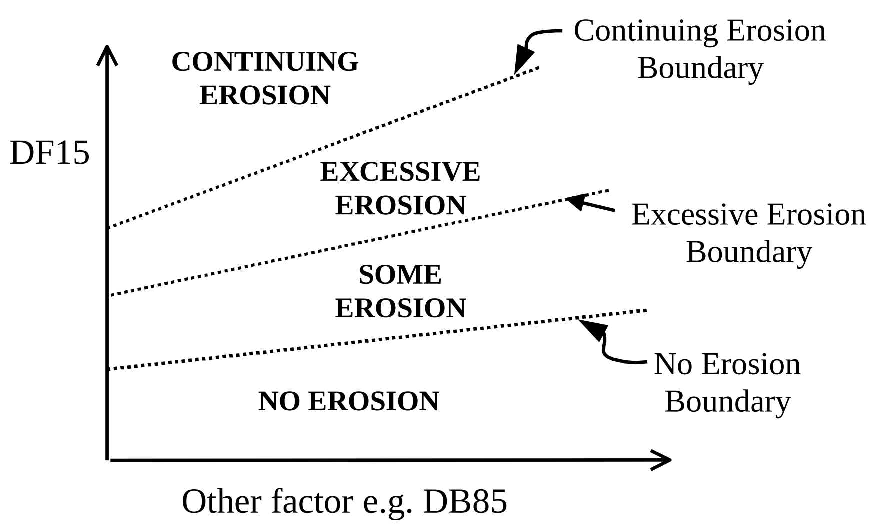
St.-Arnaud, G. (1995) The high pore pressures within embankment dams: an unsaturated soil approach. Canadian Geotechnical Journal, Vol.32, pp. 892-898

TERMINOLOGY used in relation to internal erosion

The following terminology is used in relation to internal erosion and piping. These are taken from the draft ICOLD Bulletin on internal erosion of dams and their foundations:

Backward erosion. Backward erosion involves the detachment of soils particles when the seepage exits to a free unfiltered surface, such as the ground surface downstream of a soil foundation or the downstream face of a homogeneous embankment or a coarse rock fill zone immediately downstream from the fine grained core. The detached particles are carried away by the seepage flow and the process gradually works its way towards the upstream side of the embankment or its foundation until a continuous pipe is formed.

Continuation (filtration). Continuation is the phase where the relationship of the particle size distribution between the base (core) material and the filter controls whether or not erosion will continue. Foster and Fell (2001) and Foster (1999) define four levels of severity of continuation; no erosion, some erosion, excessive erosion and continuing erosion. These are shown conceptually in Figure A1 and defined individually below.



Conceptual filter erosion boundaries (Foster, 1999), Foster & Fell (2001)

Continuing erosion. The filter is coarser than the continuing erosion criteria and is too coarse to allow the eroded base materials to seal the filter allowing unrestricted erosion of the base soil.

Detachment. Detachment is the first stage of the erosion process. Particle detachment occurs by the hydraulic shear forces developed by the seepage flow velocity. The mechanics are determined by whether the soil is cohesive or cohesionless.

Excessive erosion defines conditions where erosion of the base soil will be excessive before it seals. Filters between the excessive erosion boundary and the continuing erosion boundary will eventually seal but only after significant erosion of the base soil. In dams there may be large leakage flows before the filter seals by clogging of the surface of the filter by eroded base soil.

Hydraulic fracture occurs in the core or foundation of embankment dams when the effective minor principal stress in the core or foundation becomes zero or even slightly negative if the soil can withstand tensile stresses. The pressure of the water seeping through the core from the reservoir exceeds the remaining compressive stress and forms a crack or further opens an existing crack in which internal erosion by concentrated leak may initiate.

Initiation. Initiation is the first phase of internal erosion, when one of the phenomena of detachment of particles occurs. Four initiation phenomena are defined: concentrated leak, backward erosion, suffusion and soil contact erosion (each of which are defined separately in this section).

Internal erosion occurs when soil particles within an embankment dam or its foundation, are carried downstream by seepage flow. Internal erosion can initiate by concentrated leak erosion, backward erosion, suffusion and soil contact erosion.

Internal erosion path is the path of moving eroded particles inside the dam and/or its foundation.

Internal erosion phases (or **internal erosion mechanisms)** are the mechanisms during an internal erosion process leading to failure. Internal erosion of embankment dams and their foundations can be represented by four phases (or mechanisms):

1. **Initiation** of erosion.
2. **Continuation** of erosion (i.e. whether there are filters capable of stopping the erosion process).
3. **Progression** to form and sustain a pipe and/or to increase seepage and pore pressures in the downstream part of the embankment or its foundation.
4. **Breach** initiation resulting in uncontrolled release of the water from the reservoir.

Internal Instability. In soils subject to internal instability finer particles in the soil are able to move within the soil mass under the forces imposed on the particles by seepage flow. The phenomenon does not require a crack within the soil in which erosion may occur as is required in concentrated leak erosion, or a free surface from which particles detach and a roof of cohesive soil as is required for backward erosion. Internally unstable soils may be subject to suffusion if the hydro-mechanical conditions are suitable.

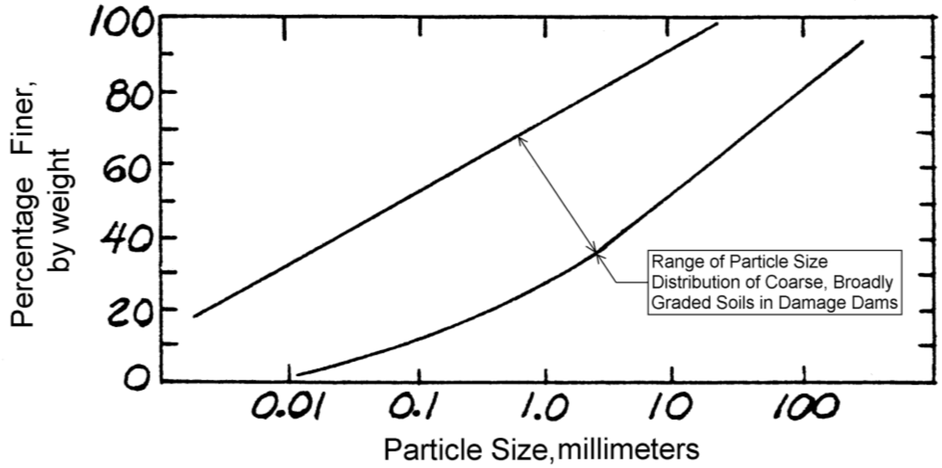
No erosion The filter is finer than the no erosion criteria and seals with no or practically no erosion of the base material. Filters designed and constructed according to modern filter design criteria will satisfy no erosion criteria.

Piping. Piping a potential progression phase of internal erosion which initiates by backward erosion, or erosion in a crack or high permeability zone, and results in the formation of a continuous tunnel called a 'pipe' between the upstream and the downstream side of the embankment or its foundation. Internal erosion is commonly described as 'internal erosion and piping' but piping is actually the culmination of a process of erosion in which a number of phases must occur and be sustained in order that a 'pipe' develops through the dam or its foundation and allows the passage of considerable quantities of water which may lead to a breach.

Progression. Progression is the phase of internal erosion, where hydraulic shear stresses within the eroding soil may or may not lead to the erosion process being on-going and in the case of backward and concentrated leak erosion to formation of a pipe. The main issues are whether the pipe will collapse, or whether upstream zones may control the erosion process by flow limitation.

Self-filtering. In soils which self-filter, the coarse particles prevent the internal erosion of the medium particles, which in turn prevent erosion of the fine particles. Soils which potentially will not self-filter include those which are susceptible to suffusion, and very broadly graded soils such as those which fall into the grading envelope shown in Figure A2.

Movement of the particles in such soils may be in concentrated leaks in cracks or openings caused by hydraulic fracture as proposed by Sherard (1979).



Examples of grading envelopes of some broadly graded soils which did not self filter (Sherard, 1979)

Sinkhole A sinkhole is a cavity formed in the core of a dam or in a dike which may form as a result of erosion initiated by a concentrated leak, contact erosion or suffusion, or erosion into open joints in a foundation or conduit. The sinkhole cavity is often near vertical and forms above the zone or point where erosion initiates. The progression of development of sinkholes is often a slow process with soil falling from the top of the sinkhole being carried away by the initiating erosion mechanism. Sinkholes may take years to manifest themselves at the crest of the dam or dike

Some erosion. The filter is between the no erosion and excessive erosion boundaries. The filter quickly seals after particles of the base material clog the surface of the filter.

Suffusion is a form of internal erosion which involves selective erosion of finer particles from the matrix of coarser particles, in such a matter that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. The volume of finer particles is such that they fit within the voids formed by the coarser particles. That is the voids are under-filled. Suffusion involves little or no change in volume of the soil mass. Suffusion occurs at vertically upward seepage gradients less than the Terzaghi critical gradient and the effective stresses are carried largely by the coarser particles. *This phenomenon is sometimes referred to as suffosion in the literature.*

APPENDICES IN VOLUME 2