Damwatch
Dam Engineering Specialists

Safety Review of Birchville, Korokoro and Woollen Mills Dams

30 June 2006

Wellington Regional Council

Issue 1

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EXECUTIVE SUMMARY

Greater Wellington Regional Council, Parks and Forests Division (Greater Wellington) have requested Damwatch to carry out a safety review for the Birchville, Korokoro and Woollen Mill Dams to assess the risk of failure and the potential hazard should the dams fail.

The review is a high level evaluation of the dams with the main objectives being to assess the risk of failure and the consequences of potential failure and comprises of the following main elements:

- Visual inspection of the structures, foundations and abutments where observable
- Review of previous reports and documentation provided by Greater Wellington
- Assessment of the risk of failure based on the observed physical condition and structural stability analysis of the structures.
- Examination of the downstream environment to identify population, infrastructure and/or property at risk from the potential failure of the dams
- Based on the above assessment of consequences and using the criteria set out in the NZSOLD Dam Safety Guidelines assess the Potential Impact Category for the dams.

Two guidelines have been followed herein: the NZSOLD Dam Safety Guidelines (November 2000) and the draft regulations to the Building Act 2004. The Potential Impact Category for all three dams has been assessed as Low under both the proposed building regulations and the NZSOLD guidelines.

The Korokoro concrete gravity dam has a variable cross section when analysed as a two-dimensional gravity structure it is considered that the overall safety of the dam does meet accepted safety evaluation guidelines for a ‘low’ PIC structure.

The Woollen Mills concrete gravity dam has a constant cross sectional profile when analysed as a two-dimensional gravity structure, it falls slightly below guidelines for acceptable safety against sliding in the foundation. It is however considered that the overall safety of the dam does meet guideline recommendations for a ‘low’ PIC structure.

There is no quick preliminary analysis technique available for arch dams. The geometry of the Birchville dam for the given span and dam height are within general arch dam design guidelines. A simple two-dimensional analysis of potential rock wedge failure at the left abutment of the dam shows that there should be an acceptable safety margin against possible abutment failure. The analysis assumes the most adverse orientation of joints in the greywacke rock, and with reliance only on friction on the rock joints to resist the arch thrusts. A large sudden increase in the arch thrust from earthquake loading should not appreciably reduce the margin of safety because of the wedging effect the increased thrusts produces.

It is concluded that there is good confidence in the continuing safe performance of Birchville dam and that the dam meet acceptable safety guidelines for a ‘low’ PIC dam. A detailed static and seismic analysis of the dam is not considered to be warranted.

The main issue identified from the site inspection of the Korokoro and Woollen Mills Dams is that the spillways for both dams are designed for an annual flood event and hence the remaining dam is subject to frequent overtopping. The Korokoro Dam has extensive erosion of the downstream toe area. The Woollen Mills dam has similar erosion at the downstream toe with the toe area now flooded making it difficult to assess seepage under the dam.

The Korokoro dam has a continuous flow at the downstream toe of the order of 2.4 litres/minute observed during dry weather conditions. It appears this is related to flows from a draw off pipe (150mm dia.) located in the upstream face of the dam at the top water level, possibly a broken pipe. This flow should be investigated and repaired.

The Woollen Mills dam has been subjected to significant flooding and erosion in recent years resulting in scouring of the spillway and downstream channel immediately below the dam. The spillway performance and dam stability following loss of part of the spillway structure in 2004/05 flood event should be further investigated and repaired.

Erosion of the toe areas needs to be regularly monitored and if frequent floods lead to deterioration the area will require robust repairs.

Some minor maintenance is required to clear debris from the Korokoro Spillway.

Birchville Dam: Further studies and improvements for a more informed understanding of the left abutment failure mechanism were identified in the 1989 report by Tonkin and Taylor. No further detailed engineering geological logging of the abutments has been undertaken as part of this study. It is recommended that a more detailed understanding of the left abutment geology is undertaken.

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1.0 INTRODUCTION

Greater Wellington Regional Council, Parks and Forests Division (Greater Wellington) have requested Damwatch to carry out a safety review for the Birchville, Korokoro and Woollen Mill Dams to assess the potential for failure and the potential hazard should the dams fail. The locations of the dams are shown on Figure 1.1.

The review has been conducted in terms of the NZSOLD Dam Safety Guidelines (November 2000). The review also consider the proposed regulations of the Building Act (2004) (currently released for discussion) with respect to dam classification based on the potential impact of failure of the dams.

The review is a high level evaluation of the dams with the main objectives being to assess the dam condition, structural stability and the consequences of potential failure and comprises the following main elements:

- Visual inspection of the structures, foundations and abutments where observable
- Review of previous reports and documentation provided by Greater Wellington
- Assessment of the potential for failure based on the observed physical condition and structural stability analysis of the structures.
- Examination of the downstream environment to identify population, infrastructure and/or property at risk from the potential failure of the dams
- Based on the above assessment of consequences and using the criteria set out in the NZSOLD Dam Safety Guidelines assess the Potential Impact Category for the dams.

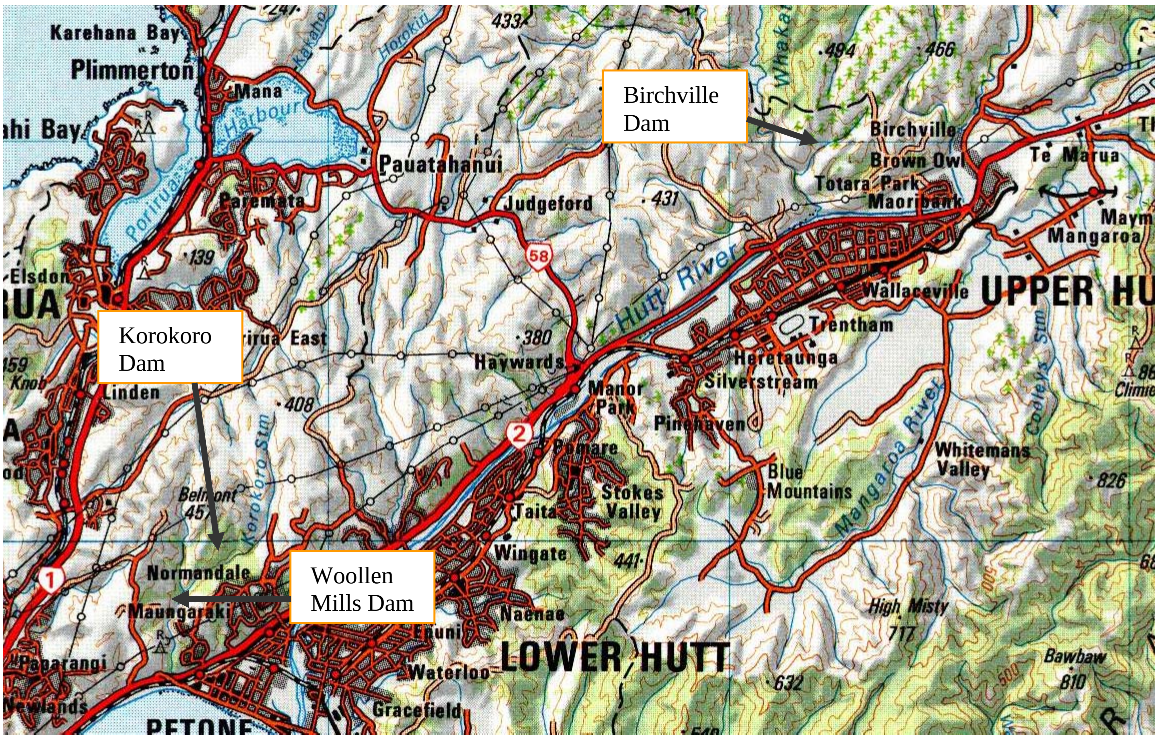

Figure 1.1: Locations of Birchville, Korokoro and Woollen Mill Dams

2.0 ASSESSMENT OF WELLINGTON REGIONAL COUNCIL STRUCTURES

The following structures owned by Wellington Regional Council are each assessed for in this report.

Wellington Regional Council Dams	Built	Height (m)	Crest Length (m)	Reservoir Volume 000 cu m	Dam Type
Korokoro Dam	1903	8	37	30 (estimated)	Concrete Gravity
Woollen Mills Dam	1903	6	15.5	<3 (estimated)	Concrete Gravity
Birchville Dam	1930	15	46	20	Concrete Arch

3.0 DAM POTENTIAL IMPACT CLASSIFICATION

The safety record of modern water retaining structures such as dams in New Zealand has generally been good, with a very low number of incidents or failures. This high level of safety has been achieved by close attention to and management of, the potential hazards and risks. An important part of the modern process is the identification of the potential impacts or consequences of dam failure. The consequence of failure and risk to the downstream population is used in dam safety guidelines to set the minimum loading conditions and acceptable dam safety practices for the dam. This is the basis of the NZ Society of Large Dams (NZSOLD) Dam Safety Guidelines and is a well established standard practice for dams in many countries.

The Building Act 2004 also requires classification of dams according to the potential impact of failure of the dam on persons, property and the environment by applying prescribed criteria. This classification is used to determine the level of dam safety assurance programme to be carried out by dam owner.

Two guidelines have been followed herein: the NZSOLD Dam Safety Guidelines and the draft regulations to the Building Act 2004. The two guidelines have slightly different procedures to determine potential impact classifications (PIC).

In both guidelines the potential impacts or consequences of a dam breach are described in terms of life, socio-economic, financial and environmental effects. As described subsequently in this section and shown in Table 2.1. The NZSOLD classification system places dams “high”, “medium”, “low” and “very low” potential impact categories depending on the severity of the potential consequences of dam breach. In the draft regulations to the Building Act 2004, “low” and “very low” categories are combined into one, namely “low”.

Table 2.1: Potential Impact Categories for Dams in Terms of Failure Consequences. (NZSOLD, 2000).

Potential Impact Category	Potential Incremental Consequences of Failure	
	Life	Socio-economic, Financial, & Environmental
High	Fatalities	Catastrophic damages
Medium	A few fatalities are possible	Major damages
Low	No fatalities expected	Moderate damages
Very Low	No fatalities	Minimal damages beyond owner's property

In the draft Building Act regulations risk to life is assessed in terms of “Population at Risk” (PAR). The Population at Risk includes all those persons who would be directly exposed to flood waters within the dam breach zone if they took no action to evacuate. The categories defined in the draft regulations are shown in Tables 2.2 and 2.3.

Table 2.2: Incremental Consequences for PIC Categories for use in Dam Classification Regulations 1. (Regulations for the Dam Safety Regime: Discussion Document, Department of Building and Housing, 2006).

<table>
<tr><th rowspan="2">Population at Risk</th><th colspan="4">Severity of damage and loss</th></tr>
<tr><th>Minimal</th><th>Moderate</th><th>Major</th><th>Catastrophic</th></tr>
<tr><td>0</td><td>Low</td><td>Low</td><td>Medium</td><td>High</td></tr>
<tr><td>1 – 10</td><td>Low (see notes 1 and 3 below)</td><td>Low (see notes 3 and 4 below)</td><td>Medium (see note 4 below)</td><td>High</td></tr>
<tr><td>11 – 100</td><td rowspan="2">See note 1 below</td><td>Medium (see notes 3 and 4 below)</td><td>High</td><td>High</td></tr>
<tr><td>More than 100</td><td>(See note 2 below)</td><td>High</td><td>High</td></tr>
</table>

Note 1: With a PAR of five or more people, it is unlikely that the severity of damage and loss will be “minimal”.
Note 2: “Moderate” damage and loss would be unlikely where PAR exceeds 100.
Note 3: Change to “medium” PIC where the potential for one identifiable life being lost is recognised or where the loss of itinerant lives is reasonably likely.
Note 4: Change to “high” PIC where it is reasonably likely for two or more non-itinerant lives being lost.

Table 2.3: Incremental Damage Descriptions Associated with Table 2.2 for use in Dam Classification Regulations 1. (Regulations for the Dam Safety Regime: Discussion Document, Department of Building and Housing, 2006).

Descriptor	Residential	Costs: socioeconomic and financial	Environmental	Recovery time
Catastrophic	More than 50 houses destroyed	Greater than $ 10 m	Permanent widespread ecological damage	Many years
Major	4 – 49 houses destroyed and a number of houses damaged	$ 1 – 10 m	Heavy ecological damage and costly restoration	Years
Moderate	1 – 3 houses damaged	$ 100,000 - $ 1 m	Significant but recoverable ecological damage	Months
Minimal	No damage	Less than $ 100,000	Short term damage	Days to weeks

The consequences of a hypothetical dam breach can be evaluated at varying levels of detail. For this study the level of detail is based on readily available data and general impressions and is likely to be conservative.

The assessment of hypothetical dam breach scenarios typically considers two different failure scenarios. The two scenarios are termed the 'sunny day' and 'rainy day' scenarios.

- The sunny day scenario is a sudden failure, which may result from an event such as a very large nearby earthquake (e.g. considering a 300-600 year return period event at the nearby Wellington Fault). In these scenarios the dam is assumed to be operating normally.
- The rainy day scenario is typically associated with extreme floods overtopping the embankment and causing failure. In assessing the potential hazard from the dam, only the incremental effect of the dam failure above the flood effects is applied when considering the consequences. For small dams this effect is usually minor.

In principle, the main differences between these two scenarios in terms of assessing the potential consequences are:

- There are different reservoir volumes associated with the two scenarios.
- The breach geometry and development time may be different.
- There is no warning of the sunny day event whereas the rainy day event takes place in the context of a developing flood with consequently more warning.

The consequences of a hypothetical dam breach are assessed in terms of the potential public safety, socio-economic, financial and environmental effects.

The risk to public safety downstream of the structures is assessed in accordance with the draft Building Act regulations by determining the "Population at Risk". The Population at Risk includes all those persons who would be directly exposed to flood waters within the canal breach zone if they took no action to evacuate. The draft regulations give an inundation depth of 0.3m as an indication of the area where the public is at risk. The draft regulations also include conditions for potential loss of life as detailed previously in Table 2.2. The NZSOLD Guidelines consider potential fatalities or loss of life but does not prescribe details of the methodology to be used to the same detail as the regulations. This assessment applies both the NZSOLD Guidelines and the draft regulations to the Building Act 2004 and tabulates both results.

The social consequences of a dam breach depend on the nature, location and extent of the community affected by the breach. For this consequence assessment, the social consequences are assessed from information derived from NZSS 1:50,000 mapping which locates buildings. Site reconnaissance determined which buildings are inhabited.

Environmental damage from a breach may affect urban areas, pastoral farmland, stock, roads and buildings. For this consequence assessment, the environmental effects are assessed from information obtained from site reconnaissance.

Financial impacts from a dam breach include repair of damage and compensation payments to third parties. In this assessment of PIC, losses suffered by the owner, Wellington Regional Council, have not been included in the assessment of financial impacts.

As noted previously the draft Building Act regulations on Assessment of Consequences of Dam Failure assess risk to public safety by determining the population at risk (PAR) in the area inundated by the breach outflow, whereas the NZSOLD Guidelines are based on possible fatalities. In this assessment of PIC both guidelines will be considered when assessing the public safety.

4.0 KOROKORO AND WOOLLEN DAMS

4.1 DESCRIPTION OF DAMS

Korokoro Dam

The Korokoro Dam is located approximately 4 km upstream of Cornish Street on the Korokoro Stream within Belmont Regional Park, refer to Figure 4.1.

The Korokoro Dam is an 8 metre high concrete gravity type dam with construction completed in 1903 to provide an additional water supply source to Petone's existing artesian water supply. The water supply from the dam was discontinued in 1962 and it remains as a focal point of the Belmont Regional Park. The dam has been identified as a heritage site being New Zealand's first concrete gravity type dam.

The dam is 37m in length at the crest and incorporates an uncontrolled chute type spillway with steps and is located on the true right side of the dam. During normal operations the crest has a freeboard of 0.7 metres above the spillway crest level. Records indicate that fine sediments have been gradually deposited in the reservoir over time. The local geology is generally anticipated to comprise sandstone-mudstone sequences of the Rakaia terrain, often referred to under the informal name "Wellington Greywacke".

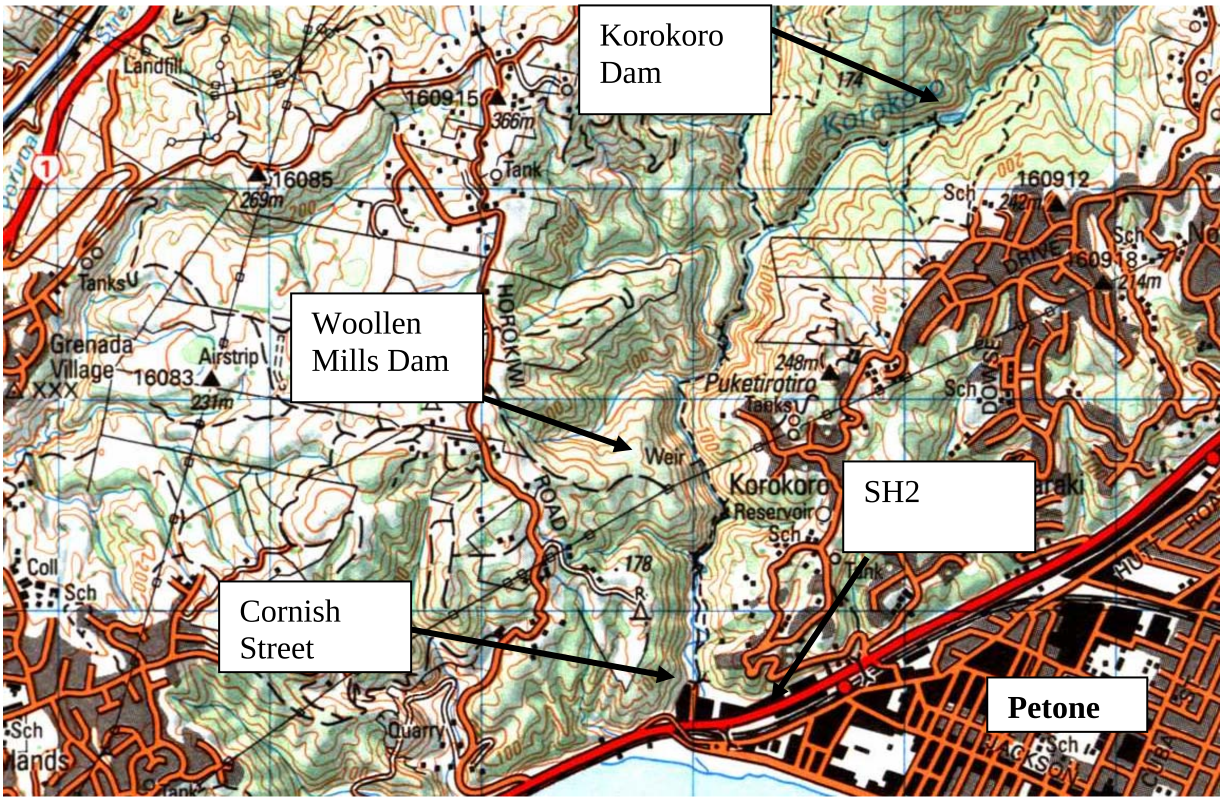

Figure 4.1: Locations of Korokoro and Woollen Mill Dams

Woollen Mills Dam

Prior to construction of the Korokoro Dam, as a water supply source for Petone, the Petone Woollen Mills was extracting water from a totara timber dam on the lower reaches of the Korokoro Stream to drive a turbine at the mill. A compensation package was agreed between the Petone Borough and the Petone Woollen Mills by means of a special Act of Parliament. The compensation package included the construction of a small concrete dam in the lower reaches of the stream and maintaining a minimum flow in the stream below the Korokoro Dam.

The Woollen Mills Dam is located approximately 2.5 km downstream of the Korokoro Dam within Belmont Park, refer to Figure 4.1. The dam is a small concrete gravity type dam constructed at the same time as the Korokoro Dam (1903) with river flows passed by an uncontrolled chute type spillway. The small lake behind the dam is now full of gravel. The local geology is generally anticipated to comprise the same sandstone-mudstone sequences as the Korokoro Dam often referred to under the informal name “Wellington Greywacke”.

4.2 FIELD INSPECTION

4.2.1 Korokoro Dam

The dam was inspected on the 29 May 2006, weather conditions described as overcast and dry. Observations from a visual inspection of the dam have been compiled in the table below together with a photographic record in Appendix A.

Dam Structural Element	***Observation Summary***
Crest	- The crest is 570 mm wide raised brick coping, 220 mm deep, on a concrete crest width of 450mm. - One brick close to the spillway has been removed; floods exceeding the spillway capacity may damage the competence of the brick coping.
Right Abutment	- No access to abutment for inspection
Left Abutment	- No apparent signs of movement / settlement at the abutments
Upstream Slope	- Vertical concrete wall – good condition - Overflow pipe observed at top water level (150mm dia) located on left of spillway
Downstream Slope	- Concrete condition appears reasonable, no cracking or defects observed
Downstream Toe	- Water ponding immediately downstream of the dam, the ponding was elevated above the normal stream level with a small continuous flow to the stream, seepage flow identified from spillway dam interface. - Heavy vegetation with difficult access. - Flows measured as 2.4 litres per minute - Flow rates at the toe appear related to those observed entering an overflow pipe in the upstream wall of dam located at lake level – not confirmed

Dam Structural Element	Observation Summary
	- Some recent ground slip / subsidence adjacent to the spillway due to seepage flows (as identified above) - A 450mm dia. clay drainage pipe which collects seepage from the downstream toe area discharging to the spillway plunge pool was observed as collapsed at outlet to plunge pool
Outlets	- Dam discontinued as water supply source, outlets no longer in use. - Sluice valves appear to be abandoned
Spillway	- Concrete stepped spillway, concrete in relatively good condition. - There appears to be some slippage of ground behind the spillway walls (true left) exposing the concrete, this appears a relatively recent event. - Floating branches and vegetation have limited the spillway capacity this should be cleared as part of the normal maintenance programme.
Plunge pool	- Well formed plunge pool no scouring of pool observed - A small amount of scour was observed below the base of the spillway (true right side).
Reservoir	- Sediment has been gradually deposited in the reservoir over the years with the clear depth of impounded water being 0.5 to 0.6m. - The inlet to the reservoir has filled with sediment reducing the area of the lake to approximately 5000 m^2 (rough order assessment).

In summary the issues arising from the site examination are:

1. Vegetation and debris build up at the spillway inlet reducing the spillway capacity – control by normal maintenance.
2. The source of erosion in the downstream toe area should be investigated. Possible sources include:
   - 150mm diameter inlet pipe on the upstream face at top water level
   - overflows over the crest – assess spillway capacity
   - seepage under / around the dam
3. The downstream toe area of the dam is not adequately drained back to the stream with ponding identified and damage to the existing drainage system, for example an existing clay pipe for toe drainage has collapsed.
4. Some minor scouring observed under the spillway outlet step.
5. Missing bricks in coping on crest could lead to further erosion of the coping in the event of floods overtopping the crest.

4.2.2 Woollen Mills Dam

The dam was inspected on the 29 May 2006, weather conditions described as overcast with light showers. Observations from a visual inspection of the dam have been compiled in the table below.

Dam Structural Element	***Observation Summary***
Geometry	- The dam width at the crest is approximately 15.5m - Crest width is 400 mm - Spillway width approx. 3.5 m - Spillway walls 400mm wide - Crest at two levels ▪ high level crest 900mm above TWL crest length 5.6 m ▪ low level crest 700mm above TWL crest length 5.45 m (emergency spill)
Crest	- The dam crest consists of 400 mm wide raised brick coping, 220 mm deep, on a concrete crest width of 450mm. - Some minor cracking observed
Right Abutment	- See observations on spillway below
Left Abutment	- Rock outcrops visible on both abutments - No evidence of dam movement observed - No evidence of seepage observed - No evidence of erosion observed
Upstream Face	- Concrete face in reasonable condition - Inlet control gates and cages in poor condition
Downstream Face	- Concrete face in reasonable condition
Downstream Toe	- Difficult to assess any seepage as downstream toe flooded with water backing up from river flows and a small continuous seep through spillway walls.
Outlets	- Pipework use has been discontinued.
Spillway and Plunge pool	- Flooding in 2004 /05 caused a rock slip directly below the spillway. This resulted in loss of part of the spillway structure including a section of the concrete base and part of the right abutment wall - Plunge pool is well formed however scour issues remain with the downstream channel exposed to scour during large floods
Reservoir	- The small reservoir is filled with river gravels - Water depth increases close to the dam up to 700mm but generally less than 300mm depth with gravel build up to above reservoir level in places. - The original reservoir is estimated to have been 15m wide by approximately 20 to 30 m long.

In summary the issues arising from the visual assessment of the site are:

1. Scouring of spillway and downstream channel immediately below the dam.
2. Spillway performance and dam stability following loss of part of the spillway structure in 2004/05 flood event.
3. Seep observed through the spillway wall is minor and does not represent a dam safety issue.

4.2.3 Summary of Issues Arising from Field Inspections

The main issue identified from the site inspection is that the spillways for both dams are designed for an annual flood event and hence the remaining dam is subject to frequent overtopping. The Korokoro Dam has extensive erosion of the downstream toe area; refer to historical photographs in Appendix B. The Woollen Mills dam has similar erosion at the downstream toe with the toe area now flooded making it difficult to assess seepage under the dam.

The Korokoro dam has a continuous flow at the downstream toe of the order of 2.4 litres/minute observed during dry weather conditions. It appears this is related to flows from a draw off pipe (150mm dia.) located in the upstream face of the dam at the top water level, possibly a broken pipe. This flow should be investigated and repaired.

The Woollen Mills dam has been subjected to significant flooding and erosion in recent years resulting in scouring of the spillway and downstream channel immediately below the dam. The spillway performance and dam stability following loss of part of the spillway structure in 2004/05 flood event should be further investigated and repaired.

Erosion of the toe areas needs to be regularly monitored and if frequent floods lead to deterioration the area will require robust repairs.

Some minor maintenance is required to clear debris from the Korokoro Spillway.

4.3 Flood Assessment and Downstream Impact

4.3.1 Flood Data

The Korokoro Stream flood frequency at the Woollen Mill Dam Site has been reviewed by Opus[1] in June 2003 following an earlier assessment by Wellington Regional Council[2] in 1979. This report has considered the recommendations proposed by Opus as being the most current and best estimate of flood conditions in the Korokoro Stream. The approximate recommended flood frequencies for the Korokoro Stream at the Woollen Mill Dam site from both assessments are summarised in the table 4.2 below.

Flood Return Period	Wellington Regional Water Board 1979 (cumecs)	OPUS 2003 (cumecs)
$Q_{2.33}$	-	27
Q_5	42	36
Q_{10}	55	44
Q_{20}	66	51
Q_{50}	82	60
Q_{100}	96	67

Table 4.2: Recommended flood frequencies for the Korokoro Stream at the Woollen Mill Dam site

[1] H.J. Freestone, ‘Korokoro Stream Flood Frequency’ ,Opus, letter dated 23 June 2003
[2] P. Purves, ‘Korokoro Stream Scour Potential – For Rahui Reservoir Rising Main’, WRC Internal Memorandum, 31 May 1996

The catchment area above the Woollen Mills Dam site has been reviewed and flood frequency estimates interpolated for the Korokoro Dam site. The catchment area between the Woollen Mills Dam and Korokoro Dam sites has been estimated at 38% of the total catchment area above the Woollen Mills Dam site (15.9 km^2).

The flood frequency for the Korokoro Stream at the Korokoro Dam site has been estimated based on the Woollen Dam site data and summarised in the table 4.3 below.

Flood Return Period	Estimated based on Wellington Regional Water Board 1979 data (cumecs)	Estimated based on OPUS 2003 data (cumecs)
$Q_{2.33}$	-	17
Q_5	26	22
Q_{10}	34	27
Q_{20}	41	32
Q_{50}	51	37
Q_{100}	60	42

Table 4.3: Estimated flood frequencies for the Korokoro Dam site

4.3.2 Spillway Capacity

Spillway discharges have been assessed using the weir formula

$$Q = C\,B\,H^{3/2}$$

Where: Q is spillway discharge (m^3/s) C is a constant
B is spillway breadth (m) H is depth over the spillway (m)

Table 4.4 below estimates the spillway capacity at each dam site. It is evident that the spillway capacity for both sites is not designed for flood frequencies greater than an annual flood event. It is therefore highly likely that both dams have overtopped frequently during their life to date and this will continue.

Dam Site	**Spillway Discharge Capacity (m^3/s)**
Korokoro Dam	4.9
Woollen Mills Dam	5.5

Table 4.4: Estimated spillway capacities for the Korokoro Stream Dams

The safety of the dam where overtopped by large floods depends on the following, these will be discussed further below:

1. whether the toe area is prone to scour
2. the stability of the concrete dam under high reservoir level

4.3.3 Downstream Flood Impact Assessment

The stream downstream of the Korokoro comprises of a steep sided v-shaped valley used by trampers and cyclists as part of Belmont Regional Park. The existing tracks are well maintained and incorporate a number of points where the stream is crossed by bridges. The valley above Cornish Street is naturally vegetated with no industrial, residential development or farming.

The Korokoro Stream flows into a flood channel in the industrial area of Cornish Street from there under State Highway 2 and railway bridges and beneath the Petone esplanade / motorway on / off ramp and watermain to the outlet culvert.

The existing channel has a nominal capacity of between 40 and 45 m^3/s however flood flows greater than 35 m^3/s will exit the channel. These flows cannot re-enter the channel until they have traversed the full length of Cornish Street and crossed SH2. This flooding is caused by inadequate channel capacity and backwater effects from downstream constrictions.

The industrial area in Cornish Street, Petone, 3.7 km below the Korokoro Dam, has been inundated by flood waters from the Korokoro Stream on a number of occasions, notably 20 December 1976 and 22 November 1977. It has been estimated[3] that the existing flood waterway from the industrial area to the stream outlet is grossly inadequate giving only in the order of a one in five year return period flood discharge capacity.

4.4 Dam Break Assessment

4.4.1 Sunny Day Event

A sunny day type failure is assumed to occur under dry weather (non-flood) conditions. For a concrete dam this would typically occur following an earthquake or foundation failure. Failure of the dam would allow the contents of the reservoir behind the dam to spill through the resulting breach until the reservoir level dropped sufficiently for this process to stop. The loss of reservoir content would be rapid and the resulting flooding of the downstream stream bed sudden.

A reasonable maximum breach discharge can be estimated based on information from historical dam failures or on assumptions of breach parameters and applications of hydraulic principles. The following relationship[4] below has been established to estimate the peak breach discharge in concrete dams and assumes the peak discharge develops instantaneously for these small dams:

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[3] Wellington Regional Water Board 'Korokoro Stream Flood Control Proposals', November 1979

[4] 'Engineering and Design – Hydrologic Engineering Requirements for Reservoirs', US Army Corps of Engineers, October 1997.

$$Q_P = (8/27).\, W_b.\sqrt{g}.Y_0^{3/2}$$

Where Q_P = peak discharge
W_b = width of the breach
g = gravity coefficient
Y_0= initial depth

The relationship above only considers water storage in the dam. The sediment stored behind both the Woollen Mills and Korokoro Dams will affect the peak dambreak discharge for each dam. The storage behind each dam has been assessed as two components including the live storage and the deposited sediment. Each dam has a water only component but an assessment of the sediment behaviour during a breach is necessary to determine the stored fluid volume on which to base calculations for the peak discharge volume.

The behaviour of sediment in small reservoirs under dam failure conditions has been investigated[5] in the UK and found to be strongly dependant on the rate of flow into the reservoir. If the dam fails under 'Sunny Day' conditions when the flow into the reservoir is small , then only a small proportion of even the very low strength deposits found in small reservoirs can be expected to 'flow' with the escaping contents of the reservoir. If there is a large flow into the reservoir when a breach occurs then more sediment is entrained. It would be expected that more sediment would subsequently be suspended by the action of the stream after the reservoir water had escaped which may have environmental impacts on the river system but would not constitute part of the 'escapable contents' of the reservoir under dam failure conditions.

The Korokoro dam reservoir deposits are composed mainly of a fine sediment deposition which can be described as silty sand in nature. The total Korokoro original reservoir volume is estimated to be 30,000 m^3 but much of this has been reduced by sediment deposition over the years and today 2,500 m^3 would probably be released with sunny day failure of the dam. This is a conservative estimate based on the present water storage depth (0.6 metres) and expected behaviour of the deposited material (sediment).

The Woollen Mills dam reservoir sediment deposit is mainly river gravels (i.e. coarser particles). The total reservoir volume is estimated to be 3,000 m^3 and today 600 m^3 would probably be released with a sunny day failure of the dam. This is a conservative estimate based on the water storage depth (0.2 metres) and expected behaviour of the deposited material (gravels).

[5] Halcrow Water, 'Dept of the Environment - Sedimentation in Storage Reservoirs', February 2001 (UK).

The peak flood discharge will attenuate (i.e. gradually decrease) as it travels downstream. Because of the small volumes of both the reservoirs, for the purposes of hazard classification only the peak discharge is routed.

There are a number of methods used to analyse routing of dambreak floods. The generalised flood attenuation curves[6] have been used for this assessment. These curves are based on historical dam break records and attenuation is described in terms of peak discharge at the dam site and peak discharge at some distance downstream according to reservoir storage. This is a recognized approach that does not require sophisticated numerical modelling and is considered appropriate for such small dams.

The most important location for downstream impact was assessed as the Korokoro Stream at the top of Cornish Street, approximately 3.7 km below the Korokoro Dam. Table 4.6 summarizes the flood attenuation estimates at the Cornish Street entrance to the Belmont Regional Park for both Korokoro and Woollen Mills dam sites. Channel dimensions, downstream channel slope (0.013) and Manning's number (0.04) were used and hence flood depth calculated at the downstream site. The existing channel depth at the top of Cornish Street is in the order of 2 metres deep.

A dam break flood from either the Korokoro dam or the Woollen Mills dam sites has been assessed to pass within the existing channel capacity at Cornish Street; the findings are summarized in Table 4.5 below.

Dam	Modified Storage Volume (m3)	Q_P Peak discharge (m^3/s)	X Distance downstream from dam site (km)	Q_x Peak discharge at Cornish Street (m^3/s)	Cornish St channel capacity (m^3/s)	Flood wave height at stream in Cornish Street (m)
Korokoro Dam	2,200	69	3.7	18	35	1.3
Woollen Mills Dam	600	39	1.2	27	35	1.6

Table 4.5: Estimated dam break discharges and attenuation for the Korokoro Stream Dams at top of Cornish Street

4.4.2 Rainy Day Event

A rainy day type failure is assumed to occur as a result of the dam being overtopped by an extreme flood with consequential sliding or foundation failure. In this case we have assumed

[6] USBR, 'Guidelines for defining inundated areas downstream from Bureau of Reclamation Dams' Bureau of Reclamation, Denver, CO June, 1982

that the extreme flood is the Probable Maximum Flood (PMF) event. The Probable Maximum Flood (PMF) is an estimate of the hypothetical flood that is considered to be the most severe "reasonably possible" at a particular location. Table 4.6 below summarises the estimated PMF events at both dam sites (rough order estimate).

Dam	Q_{100} (m^3/s)	PMF (m^3/s)	Peak Dambreak Discharge (m^3/s)
Korokoro	42	105	174
Woollen Mills	67	170	209

Table 4.6: Estimated Rainy Day Flood Magnitude (PMF) at dam site

The consequences in Rainy Day that need to be considered are the incremental effects above PMF. The magnitude of the PMF will clearly increase with distance downstream and increases in the catchment area. Incremental effects of the Korokoro Dam dambreak on Cornish Street are equivalent to 65% increase in flood over the PMF flood at the dam site. It has been assessed that this corresponds to an incremental flood height of less than 200mm in Cornish Street.

Similarly, incremental effects of the Woollen Mills dambreak on Cornish Street is equivalent to 23% increase in flood over the PMF flood at the dam site. It has been assessed that this corresponds to an incremental flood height of less than 300mm in Cornish Street.

The PMF event duration will be gauged in terms of hours with the flood magnitude building gradually to its maximum discharge level. In contrast the dambreak flood will have a much shorter duration, assessed in minutes rather than hours considering the storage capacity of these small reservoirs.

The 1976 flood event has been accepted as a 100 year return period flood with no damage recorded at either dam site. The maximum recorded flood depth in Cornish Street was about 1.5 metres. No direct record of persons at risk was made but it appears to be low from reports of the event.

4.4.3 Cascade Scenarios

The consequence assessment has considered the potential inundation resulting from the hypothetical failure of the Korokoro Dam. There is a possibility that the water released due to breach of the Korokoro Dam will cause overtopping at the downstream Woollen Mills Dam and consequently, its failure.

A cascade scenario needs consideration on any river with a sequence of dams, where the river valley focuses the breach flow over the downstream structures.

For a Sunny Day failure scenario at the Korokoro dam, the peak discharge from has been assessed as 69 m^3/s. Considering attenuation of this flood, the peak discharge will reduce to less than the Q_{100} flood determined for the Woollen Mills Dam of 67 m^3/s (Opus). It has been estimated that the attenuated flood will be in the order of 31m^3/s at the Woollen Mills Dam. The Woollen Mills Dam has been subject to overtopping flood flows up to about the 100 year event without any evidence of instability or adverse performance. It is therefore assumed that a Sunny Day cascade type failure scenario is unlikely.

A Rainy Day failure scenario at the Korokoro dam has a combined peak dambreak discharge of 174 m^3/s. The incremental dambreak flood is considered to attenuate similarly to the Sunny Day scenario described above and hence the Rainy Day cascade type failure scenario is equivalent to the Rainy Day failure scenario discussed for the Woollen Dam failure in section 4.42 above.

4.5 Potential Impact Classification (PIC)

In the draft Building Act regulations risk to life is assessed in terms of “Population at Risk” (PAR). The Population at Risk includes all those persons who would be directly exposed to flood waters within the dam breach zone if they took no action to evacuate. The population considered most at risk during a sunny day failure is trampers and cyclists. It is considered unlikely that trampers or cyclists will use the walking tracks during a rainy day failure event.

Incremental floods during a rainy day failure event may cause damage to the industrial and commercial units in Cornish Street due to the stream channel overtopping. The Cornish Street area is a known flood hazard area, refer to Section 4.3.3, and it can be reasonably assumed that the risk has been identified to the working population and that they will have been evacuated before a dam break occurs. However the consequential impact of a dam failure during a PMF event is considered minimal.

Environmental effects from the Woollen Mills dam failure are considered minor as the retained river gravels will be redistributed in the lower reaches of the stream. The environmental effects of sediment deposition from a Korokoro Dam breach would be significant and it is likely to be months for the lower reach of the Korokoro Stream to recover.

A dam break flood from either the Korokoro dam or the Woollen Mills dam sites has been assessed to pass within the existing channel capacity at Cornish Street.
Hence the Korokoro Dam and Woollen Mills Dams PIC are assessed as Low under both the proposed Building Act regulations and the NZSOLD Dam Safety guidelines; refer to Table 6.1 for a summary of the assessment.

The NZSOLD guidelines advise that for a Low PIC rating, the dam owner needs to implement adequate operational and surveillance procedures including:

- regular inspection by the operator or owner of the general condition and the consistency of aspects such as identifiable seepage
- routine maintenance of dam surfaces and spillway paths
- periodic inspections by an appropriate Technical specialist (e.g. 1 to 2 years)
- a dam safety Emergency Action Plan is not mandatory

4.6 Stability of Dams

4.6.1 Description of Dams

Korokoro dam has a variable sectional profile. At its centre, it has an effective crest width of about 3.5m and a downstream slope of about 0.6:1. The profile becomes progressively more slender towards both abutments. Beside the spillway on the right bank, the crest width reduces to about 1.5m and the downstream slope becomes steeper to a slope of about 0.3:1. This profile continues through the spillway section of the dam. But there is considerable buttressing provided by the fill placed against the downstream face of this section as a foundation for the 3:1 concrete-lined spillway channel. Early photographs show that fill was also placed against the dam section adjacent to the spillway channel, but this has apparently been eroded away – probably from flood flows that overtopped the spillway sidewalls or the dam crest. There are no vertical contraction joints in the dam wall, and no prominent vertical cracks are apparent. The reservoir is filled with silt to within about 0.6m of the water surface.

The Woollen Mills dam has a uniform sectional profile, with an effective crest width of 1.5m and a downstream slope of about 0.5:1. It has no vertical contraction joints and appears to be free from any significant vertical cracking. The section through the right bank spillway is not known, but probably comprises a mass concrete section that extends up the sloping right abutment slope. Fill has probably been placed as required against the mass concrete section and left hand spillway sidewall to form the concrete-lined spillway slope. As noted in Section 4.2.2, the bottom right-hand corner of the spillway channel (founded on rock) has been eroded away. The reservoir is filled with gravel to within about 0.6m of the water surface.

4.6.2 Performance to Date

Both Dams have performed satisfactorily to date over a period of 100 years. They have been subject to overtopping flood flows up to about the 100 year event without any evidence of instability or adverse performance. No significant earthquake has been experienced to date.

4.6.3 Stability Considerations

The basic stability requirement for the dams under all conditions of loading is that they should be safe against sliding on any horizontal or near-horizontal plane within the dam, at the base, or on any rock seam in the foundation. Experience for such gravity structures shows that if this criterion is met, they will be safe against overturning and excessive concrete stresses.

The sliding stability is based on a factor of safety (FS) as a measure of determining the resistance of the structure against sliding. For potential horizontal sliding planes in the dam or its foundation, the factor of safety is given by:

$$FS = (\Sigma V \tan\Phi + cL)/\Sigma H$$

Where

ΣV = resultant of vertical forces acting on the assumed sliding plane

Φ = angle of internal friction

C = cohesion

L = length of sliding plane for a unit strip of dam

ΣH = resultant of horizontal forces on section above the assumed sliding plane.

The NZSOLD Dam Safety Guidelines provide the following recommended minimum values for the factors of safety against sliding:

- For usual loading: 3.0
- For design flood conditions: 2.0
- For maximum safety evaluation earthquake loading: 1.1
- For post-earthquake loading conditions: 2.0

Some regulatory authorities in other countries allow lower factors of safety in certain circumstances. For example, the US Federal Energy Regulatory Commission (FERC) has the following recommended factors of safety for dams having a low potential impact as follows:

- For usual loading: 2.0
- For design flood conditions: 1.25
- For maximum safety evaluation earthquake loading: Greater than 1.0
- For post-earthquake loading conditions: 1.25

To assess the stability of the dams against the above sliding safety criteria, the following sections have been analysed:

- The centre of Korokoro dam (Figure 1)
- The right hand side of Korokoro dam adjacent to the spillway (Figure 2)
- The typical Woollen Mills dam section (Figure 3).

At each section, two horizontal sliding planes are considered:

- In the rock foundation
- Through the concrete near the base of the dams.

In addition, a section at mid-height of the right hand side of the Korokoro dam is assessed.

In each case, the factors of safety against sliding on the selected sliding planes are calculated and compared with the recommended minimum values provided by NZSOLD and FERC.

4.6.4 Loading

Self Weight

The dams resist the lateral hydrostatic and earthquake loads by their self-weight. The self-weight comprises the mass of concrete and rock immediately above the assumed sliding plane.

Water and Flood Loads

At Korokoro, under normal conditions, the headwater level is 0.7m below the dam crest level. The 100-year flood at Korokoro is about 42m^3/s (refer Section 4.3.1). It is estimated that this would overtop the dam to a depth of about 0.9m (comprising 12m^3/s spillway discharge and 30 m^3/s overtopping discharge over the crest of the dam section). Hence the design flood load level would be about 1.6m above the usual reservoir level.

At the Woollen Mills dam, under normal conditions, the headwater level is 0.9m below the dam crest level. The 100-year flood is 60m^3/s and, as at Korokoro, the dam will be overtopped. In this case the overtopping depth, because of the shorter length of dam and the higher flood discharge, is estimated to be about 1.5m. Hence the design flood load level would be about 2.4m above the usual reservoir level.

Sediment Loads

At Korokoro the loading from the silt is considered to be equivalent to fluid with a density of 13.6kN/m^3 (SG = 1.36). This includes the effects of the water in the silt.

At Woollen Mills the gravel load is calculated assuming it is an at-rest backfill material with a submerged density of $20kN/m^3$. The hydrostatic load is additional to this sediment loading.

Uplift

It is assumed that full uplift conditions would be present on the horizontal sliding planes. These conditions correspond to a linear triangular transition from full headwater uplift pressure at the upstream face to zero uplift at the downstream toe of the dams.

Earthquake Loading

Both dams have a ‘low’ PIC rating (refer Section 4.5). The NZSOLD guidelines do not provide specific recommendations for the level of earthquake shaking such dams should safely withstand without failure. Major dam owners in New Zealand have adopted the 1:500 AEP event as the safety evaluation earthquake for ‘low’ PIC structures[7]. It is considered that this design guideline should also be adopted for Korokoro and Woollen Mills. The NZS earthquake loading code (NZS 1170.5:2004) can be used to determine the earthquake shaking the dams would be subject to in a 500-year earthquake, and this shows that a 0.4g peak ground acceleration would be appropriate for the dams.

A simple pseudo-static analysis of the dams is acceptable. The seismic inertia load on the dams is simply the mass of the dam multiplied by the seismic coefficient of 0.4g.

The water pressure on the Korokoro dam would be increased during an earthquake by an added hydrodynamic pressure, resulting from the dynamic interaction of the dam and the stored water/silt. Westergaard’s[8] formula was used to determine the hydrodynamic forces, assuming the water/silt fluid has a specific gravity of 1.36. At Woollen Mills, which is filled with gravel sediment, there is unlikely to be any hydrodynamic loading. Instead, in an earthquake, the gravel will exert an additional load on the rigid wall of the dam in a similar manner to backfill against a retaining wall. This has been assumed for assessing the seismic load on the dam.

It is assumed at both dams that during the earthquake the reservoir is at its normal level (that is, at spillway crest level).

4.6.5 Rock and Concrete Properties

The local geology is generally anticipated to comprise sandstone-mudstone sequences of the Rakaia terrain, often referred to under the informal name “Wellington Greywacke”.

[7] L.Mejia et al, “Criteria for Developing Seismic Loads for the Safety Evaluation of Dams of Two New Zealand Owners”, NZSOLD/ANCOLD Conference 2001
[8] US Army Corps of Engineers, ‘Gravity Dam Design’, EM 1110-2-2200, 1995

It is considered that appropriate Hoek-Brown strength parameters for such a rock are:

- Uniaxial compressive strength = 30MPa
- GSI (Geological Strength Index) = 30
- m_i (Hoek-Brown constant) = 15

These parameters were analysed using the ROCLAB program to determine an appropriate cohesion and friction angle for the foundation rock (refer Figure 4). This shows that at a normal stress of about 150 KPa (typical average value for the sliding planes in the foundation) the friction angle (φ) for the rock is 54.28º and its cohesion (c) is 0.1079MPa. The following shear strength parameters are assumed for the rock foundations:

φ_R = 55º, and

c_R = 100kPa

Seismic loading may cause cracking in the dams' foundations. To account for this, the post-earthquake sliding stability is assessed using the following reduced shear strength parameters:

φ_R = 45º, and

c_R = 50kPa

For sliding planes on concrete within a dam, typical peak effective cohesion values used for concrete are $0.07f'_c$ MPa. If it assumed that the compressive strength of the concrete is a relatively low 15MPa, the cohesion would be 1.05MPa. Because of the age of the dams and the uncertainly of the quality of the concrete, particularly at horizontal construction joints in the dam, a cohesion of 500kPa is considered appropriate. The friction angle normally used for concrete is 45^0. Hence, the following shear strength parameters are used for the concrete:

φ_C = 45º, and

c_C = 500kPa

For assessing the post-earthquake stability on sliding planes through the dam, some cracking of the concrete is usually assumed. The friction angle should remain unchanged at 45º, but the effective cohesion is probably reduced. Assuming the crack extends half-way through the dam on the sliding plane, the following post-earthquake shear strength parameters would be applicable:

φ_C = 45º, and

c_C = 250kPa

4.6.6 Resultant Loads

The calculated loads acting on the sliding planes for the two sections considered for the Korokoro dam and the typical Woollen Mills dam section are provided below in Tables 4.6 to

4.9. The loads are for 1m wide sectional strips through the dams. For practical purposes, it is assumed that the sliding plane at or near the base of the dam could either be through rock or through concrete. Hence the loadings on these planes are the same.

Table 4.6: Korokoro Dam, Centre Section.
Loads (per metre width) on Sliding Plane at Base of Dam

Load Case	Self Weight (kN)	Uplift (kN)	ΣV (kN)	Hydro-static (kN)	Silt (kN)	Seismic Inertia (kN)	Hydro-dynamic (kN)	ΣH (kN)
Usual Static	810	270	540	320	100	-	-	420
Design Flood	810	320	490	460	100	-	-	560
Earthquake	810	270	540	320	100	320	180	920
Post-Earthquake	810	270	540	320	100	-	-	420

Table 4.7: Korokoro Dam, Right Hand Section.
Loads (per metre width) on Sliding Plane at Base of Dam

Load Case	Self Weight (kN)	Uplift (kN)	ΣV (kN)	Hydro-static (kN)	Silt (kN)	Seismic Inertia (kN)	Hydro-dynamic (kN)	ΣH (kN)
Usual Static	470	160	310	320	100	-	-	420
Design Flood	470	190	280	460	100	-	-	560
Earthquake	470	160	310	320	100	190	180	790
Post-Earthquake	470	160	310	320	100	-	-	420

Table 4.8: Korokoro Dam, Right Hand Section.
Loads (per metre width) on Sliding Plane at Mid Height of Dam

Load Case	Self Weight (kN)	Uplift (kN)	ΣV (kN)	Hydro-static (kN)	Silt (kN)	Seismic Inertia (kN)	Hydro-dynamic (kN)	ΣH (kN)
Usual Static	170	50	120	80	20	-	-	100
Design Flood	170	70	100	160	20	-	-	200
Earthquake	170	50	120	80	20	70	50	220
Post-Earthquake	170	50	120	80	20	-	-	100

**Table 4.9: Woollen Mills Dam, Typical Section.
Loads (per metre width) on Sliding Plane at Base of Dam**

Load Case	Self Weight (kN)	Uplift (kN)	ΣV (kN)	Hydro-static (kN)	Gravel (kN)	Seismic Inertia (kN)	Gravel EQ surcharge (kN)	ΣH (kN)
Usual Static	460	150	310	170	150	-	-	320
Design Flood	460	210	250	340	150	-	-	490
Earthquake	460	150	310	170	150	170	270	760
Post-Earthquake	460	150	310	170	150	-	-	320

4.6.7 Stability Analysis Results

The factor of safety (FS) against sliding failure of the intake blocks is calculated using:

- the relationship given in Section 4.5.2 (that is, FS = (ΣVtanΦ + cL)/ΣH),
- the ΣV and ΣH values given in tables 4.8 to 4.9, and
- the Φ and c values given in Section 4.5.4 for concrete and rock **.**

Tables 4.10 to 4.13 show the factors of safety calculated for the four load cases and for:

- The base of the centre section of Korokoro dam for sliding planes in the concrete and rock (Table 4.10)
- The base of the more slender right hand section of Korokoro dam for sliding planes in the concrete and rock (Table 4.11)
- Mid-height of the right hand section of Korokoro dam (table 4.12).
- The base of Woollen Mills dam for sliding planes in the concrete and rock (Table 4.13)

Table 4.10: Korokoro Dam, Centre Section, Sliding Factors of Safety at Base of Dam

Load Case	FS for Sliding Plane in Rock	FS for Sliding Plane in Concrete	NZSOLD Minimum FS	US FERC Minimum FS (for Low PIC)
Usual Static	3.4	9.4	3.0	2.0
Design Flood	2.5	6.9	2.0	1.25
Earthquake	1.6	4.3	1.1	>1.0
Post-Earthquake	2.1	5.3	2.0	1.25

Table 4.11: Korokoro Dam, Right Hand Section, Sliding Factors of Safety at Base of Dam

Load Case	FS for Sliding Plane in Rock	FS for Sliding Plane in Concrete	NZSOLD Minimum FS	US FERC Minimum FS (for Low PIC)
Usual Static	2.0	5.5	3.0	2.0
Design Flood	1.4	4.1	2.0	1.25
Earthquake	1.1	2.9	1.1	>1.0
Post-Earthquake	1.2	3.1	2.0	1.25

Table 4.12: Korokoro Dam, Right Hand Section, Sliding Factors of Safety at Mid-Height of Dam

Load Case	FS for Sliding Plane in Concrete	NZSOLD Minimum FS	US FERC Minimum FS (for Low PIC)
Usual Static	13.7	3.0	2.0
Design Flood	6.8	2.0	1.25
Earthquake	6.2	1.1	>1.0
Post-Earthquake	7.5	2.0	1.25

Table 4.13: Woollen Mills Dam, Centre Section, Sliding Factors of Safety at Base of Dam

Load Case	FS for Sliding Plane in Rock	FS for Sliding Plane in Concrete	NZSOLD Minimum FS	US FERC Minimum FS (for Low PIC)
Usual Static	2.9	8.8	3.0	2.0
Design Flood	1.8	5.6	2.0	1.25
Earthquake	1.2	3.7	1.1	>1.0
Post-Earthquake	1.8	4.9	2.0	1.25

4.6.8 Assessment of Results

Sliding planes in concrete

The results show that there are appreciable margins of safety against possible sliding failure on planes through the concrete of the dams. These results are also based on a conservative value for the cohesive strength of the concrete on the sliding planes. The assumed 500kPa cohesion takes account of the uncertainties in the quality of the original concrete construction. A typical value for mass concrete, where there is more confidence in the concrete construction, would be at least 1,000kPa.

Korokoro dam foundation

At the foundations of the central section of Korokoro dam (Figure 1), the recommended sliding stability criteria of both NZSOLD and FERC are met.

At the minimum dam section at the right hand end of the structure (Figure 2), the factors of safety against sliding are mostly below the minimum NZSOLD–recommended values but meet the FERC values for a low PIC dam. There should also be some three-dimensional structural behaviour of the more slender right hand section of the dam. This is because there are no vertical contraction joints in the dam wall, or any evidence of vertical cracks in the dam. This section will therefore be restrained from undergoing its full cantilever deflection (as assumed in the two-dimensional analysis) because of side restraint from the stiffer centre section of the dam to the left and the spillway dam section with its downstream buttress to the right. Hence the factors of safety as shown in Table 11 should be higher. On the other hand the factors shown in Table 4.10 for the centre of the dam will be less, although there is a margin available for these values to be less than as given.

Overall, it is considered that the dam meets acceptable foundation sliding stability criteria.

Woollen Mills dam foundation

Three of the factors of safety against sliding at the foundation of the Woollen Mills dam are slightly below the NZSOLD recommended minimum values, but are well above the FERC values. It is considered that the dam meets acceptable foundation sliding stability criteria.

5.0 BIRCHVILLE DAM

5.1 DESCRIPTION OF DAM

The Birchville dam is a concrete arch dam located on Clarke's Stream which is a west bank tributary of the Hutt River at Birchville north of Upper Hutt. The dam was originally built as part of Upper Hutt's water supply and is thought to have been completed in 1931. The dam remained in operation until 1954 when the Kaitoke scheme commenced operation. The dam is still used to retain water as a feature of the Park's facilities.

The dam is a constant radius concrete arch dam of 27.5 metres radius with a crest (chord) length of 40 metres (arch length 46 metres) and is approximately 15 metres high above lowest foundation level. The infill of boulders at the foot of the dam (to protect against scour) makes the dam appear lower. The lake behind the dam is partially silted up, in 1988 the reservoir was drained and silt levels measured to 6.3m below the spillway.

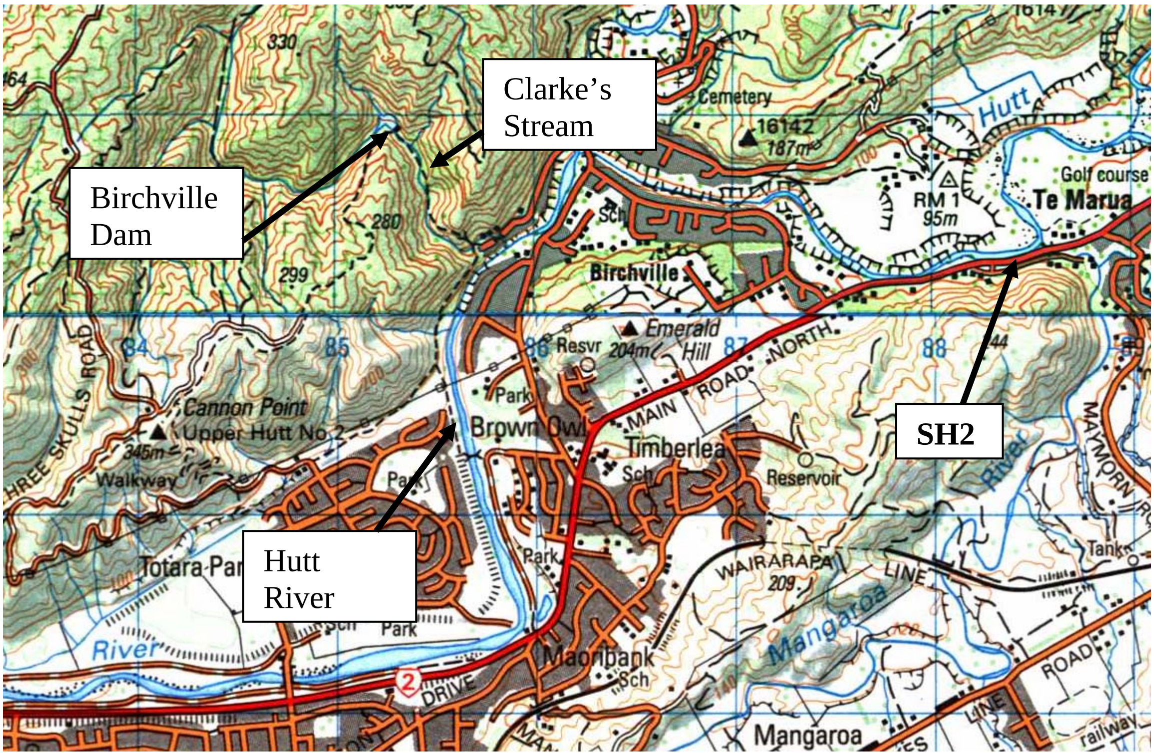

Figure 5.1: Location of Birchville Dam

The local geology is generally anticipated to comprise sandstone-mudstone sequences of the Rakaia terrain, often referred to under the informal name "Wellington Greywacke". Inspections were undertaken by Tonkin & Taylor in November 1998 whom generally considered the rock mass conditions at the embankment to be good, albeit with concerns raised on the area immediately downstream of the left abutment.

The abutments remain covered by vegetation and no further detailed engineering geological logging of the abutments has been undertaken as part of this study. Based on the observations described by Tonkin & Taylor[9] (1998) and site visit undertaken on 26 May 2006 no large scale deformations of the left abutment have been reported. The jointed nature of the rock could however be exploited by vegetation with root jacking possibly causing future deterioration of the abutment.

5.2 FIELD INSPECTION

5.2.1 Birchville Dam

The dam was inspected on the 26 May 2006, weather conditions described as overcast and dry. Observations from a visual inspection of the dam have been compiled in the table below together with a photographic record in Appendix C.

Dam Structural Element	***Observation Summary***
Crest	- No access to left crest for inspection - The concrete crest has an overall width of 1200 mm and including a 750mm wide walkway and 1050mm high wave wall - Concrete is generally in a good condition, no deformation or cracking was observed
Right Abutment	- No apparent signs of movement / settlement at the abutments - No signs of slope movement - Open rock joints observed dipping toward the dam centre line – likely to be stress relief of valley walls - No evidence of slide or seepage observed
Left Abutment	- Not able to fully access left abutment for inspection – inspect downstream vertical interface concrete wall / abutment - Rock joints fractured but not much area can be inspected readily due to heavy vegetation - No evidence of slide or seepage observed
Upstream Slope	- Vertical concrete wall – good condition
Downstream Slope	- Concrete condition appears reasonable, no cracking or defects observed - Horizontal construction joints appear less than 0.3mm with some efflorescence observed (indicator of past seepage).
Downstream Toe	- The rip rap rock fill at the toe was measured as approximately 8 metres below the crest level - No indicators of scour erosion - Observe pipe 375 mm dia on left abutment side, possibly by pass outlet. - Ground conditions very wet and slippery
Outlets	- Valve tower internal not inspected, it was noted that inlet spindles were highly corroded at the surface below valve tower - Inlet appears submerged by sediment (not confirmed) - Exposed pipes downstream of dam are corroded and generally in a very poor condition

[9] Tonkin and Taylor Ltd., (January 1989): “Birchville Dam – Stability Review – Stage 1”

Dam Structural Element	***Observation Summary***
	- Dam discontinued as water supply source - Sluice valve was closed at time of inspection (WRC maintain operation as part of maintenance procedures – not demonstrated)
Spillway	- Overflow – vertical drop type spillway - Concrete in relatively good condition. - No blockages observed – spillway discharge at time of visit was in the order of 0.6 m³/s. - Flow reasonably distributed over crest during inspection - Emergency spillway operation is to overtop the 1050mm high crest wall with downstream flood path as normal spillway operations
Plunge pool	- Well formed plunge pool. No scouring of pool observed - Rock rip rap type energy dissipater directly below spillway
Reservoir	- Sediment has been gradually deposited in the reservoir over the years with the clear depth of impounded water being 1.5 to 5.7m. - The inlet to the reservoir has filled with sediment reducing the area of the lake to approximately 20,000 m^2 (rough order assessment by T&T). - Site measurements were: ▪ 5.7 to 5.5 m below top of crest @ right side spillway ▪ 4.4 m below top of crest @ right side of valve tower ▪ 1.4 m below top of crest @ right abutment - Upper lake area was observed as very shallow with high proportion of weed growth

In summary the issues arising from the site examination are:

- No safe access to assess the condition of the left abutment geology

5.2.2 Summary of Issues Arising from Field Inspections

Further studies and improvements for a more informed understanding of the left abutment failure mechanism were identified in the 1989 report by Tonkin and Taylor. No further detailed engineering geological logging of the abutments has been undertaken as part of this study.

It is recommended that a more detailed understanding of the left abutment geology is undertaken.

5.3 Flood Assessment and Downstream Impact

5.3.1 Flood Passage

Tonkin and Taylor's report assessed design flood peaks for the 3.75 km^2 catchment area. This design data is summarised in the table 5.2 below.

Flood Event	Estimate Design Flood (m^3/s)	Tolerance
Q_{100}	20	±20%
PMF	45	±20%

Table 5.2: Design Flood Peaks, Tonkin and Taylor 1989

Spillway capacity has also been assessed by Tonkin and Taylor's report (1989). It was reported that 'on routing the upper limit flood peaks past the dam our calculations show that the 100 year flood would just be contained within the spillway section and under PMF conditions the whole crest would be overtopped by approximately 300mm'. Damwatch check calculations of spillway capacity concur with this assessment.

5.3.2 Downstream Flood Impact Assessment

The Birchville dam is located approximately 900 metres from the confluence with the Hutt River. The stream downstream of the dam comprises of a v-shaped valley used by trampers as part of Wellington Regional Councils Park facilities.

The valley below the dam is naturally vegetated with no industrial, residential development or farming. There are no houses or buildings close to the stream level which would be at risk were a dam break flood to occur.

5.4 Dam Break Assessment

5.4.1 Sunny Day Event

A reasonable maximum breach discharge can be estimated based on information from historical dam failures or on assumptions of breach parameters and applications of hydraulic principles as discussed in Section 4.4.1. The Peak breach discharge has been estimated as 170 m^3/s assuming that the worst case dam failure scenario will develop at an abutment with the maximum breach width equal to half the crest length and that the depth of stored water reduces rapidly as the breach develops. This analysis is based on a hypothetical case which has high consequence but low probability of actually occurring.

The Birchville dam reservoir deposits are composed mainly of a fine sediment deposition which can be described as silty sand particle size. It is assumed that in an initial peak discharge some of the sediment will be mobilised and the remaining sediment may be eroded over a longer duration. The Birchville reservoir capacity was estimated to have a live storage of 20,000 m^3 in 1989 and today following continued deposition of sediment it is estimated that 10,000 m^3 would probably be released with a sunny day failure of the dam. The component of sediment that would be mobilised in a dam break scenario is expected to be negligible due to the downstream height of the dam above rock fill.

Peak discharge will attenuate (i.e. gradually decrease) as it travels downstream however considering the valley shape and short distance to the Hutt River, the attenuation will be minimal. Normally for the purposes of hazard classification only the peak discharge is routed.

Dam	Modified Storage Volume (m^3)	Q_P Peak discharge (m^3/s)	X Distance downstream from dam site (km)	Q_x Peak discharge at top of falls (m^3/s)	Approx. flood wave height at top of falls (m)	
					above stream bed	above track
Birchville Dam	10,000	170	0.7	162	2.1	1.0

Table 5.3: Estimated dam break discharges and attenuation for the Birchville Dam at top of falls

This assessment considers that an important point for downstream impact is the stream at the top of water falls, approximately 0.7 km below the dam, at this point the stream diverges from the walking track route and then drops in level by approximately 15 metres. Table 5.3 summarises the flood attenuation estimates at the top of the falls. Channel dimensions, downstream channel slope (0.031) and Manning's number (0.04) were assessed and using a simple computer model (HEC-RAS) to calculate the flood depth at the downstream site.

A sunny day dam break flood from the Birchville dam site will flood the existing stream channel and the valley including the walking track by about 1.0 m depth of water this is a conservative estimate. The sunny day dam break flood event will have a very short duration (minutes) considering the reservoirs stored capacity and the peak discharge.

Considering the impact on the Hutt River, the peak discharge represents approximately 40% of the available top of bank channel capacity of the Hutt River, this allows for some bifurcation of the flood which will further attenuate flow. The channel capacity of the Hutt River is in the order of 300-400 m3/sec, and typical daily flow in the Hutt River at Birchville is in the order of 15-20 m^3/s. It is assumed that there will be significant attenuation in the channel of the Hutt River from Birchville to its outlet into Wellington harbour. Hence the risk to river users is assessed as low.

5.4.2 Rainy Day Event

A rainy day type failure is assumed to occur as a result of the dam being overtopped by an extreme flood with consequential sliding or foundation failure. In this case we have assumed that the extreme flood is the Probable Maximum Flood (PMF) event. The Probable Maximum Flood (PMF) is an estimate of the hypothetical flood that is considered to be the most severe "reasonably possible" at a particular location. Table 5.4 below summarises the estimated PMF events at Birchville dam (rough order estimate).

Dam	Q_{100} (m^3/s)	PMF (m^3/s)	Peak Dambreak Discharge (m^3/s)
Birchville	20	45	170

Table 5.4: Estimated Rainy Day Flood Magnitude (PMF) at dam site

The PMF event duration will be gauged in terms of hours with the flood magnitude building gradually to its maximum discharge level. In contrast the dambreak flood will have a much shorter duration, assessed in minutes rather than hours, considering the storage capacity of these small reservoirs. The dam break flood is assumed to occur closer to the peak PMF flood discharge and is therefore assessed to have minimal additional impact on top of the rainy day flood event.

5.5 Potential Impact Classification

The New Zealand Dam Safety Guidelines (NZSOLD, 2000) require the overall Potential Impact Classification (PIC) for a dam to be based on the worse case for either a sunny day or rainy day dam failure. The worst case considered is the sunny day failure scenario.

In the draft Building Act Regulations risk to life is assed in terms of “Population at Risk” (PAR). The Population at Risk includes all those persons who would be directly exposed to flood waters within the dam breach zone if they took no action to evacuate. The population considered most at risk during a sunny day failure is trampers and cyclists. It is considered unlikely that trampers or cyclists will use the walking tracks during a rainy day failure event. In the event of a sunny day dam break and considering the worst case scenario including the expected duration of the flood event it is considered that the population at risk will be less than five.

There are no houses or buildings close to the stream level which would be at risk were a dam break flood to occur.

The environmental effects of sediment deposition from a Birchville Dam breach would be significant and it is likely to take months for recovery.

The Birchville Dam PIC is assessed as Low under the proposed building regulations and Low under the NZSOLD guidelines. It is judged that the overall PIC is for a Low PIC structure.

5.6 Stability of Dam

5.6.1 Description of Dam

Birchville dam is a constant-radius arch dam with a radius of 27.4m and a maximum height of 14.8m. The effective span across the valley at the crest is about 40m. Its crest width is 0.6m and its maximum base width is 2.3m. The drawings show that it is keyed into the foundation for a nominal depth of 1.2m. There are no apparent vertical contraction joints in the dam wall, and the original drawings do not show any such joints.

The reservoir is filled with sediment to a depth of about 5.5m below dam crest level. The downstream toe of the dam is protected by rip rap and backfill placed to a level that is 10m below the dam crest.

5.6.2 1989 Stability Review

The stability of the dam was assessed in 1989 by Tonkin & Taylor consulting engineers. No detailed analysis of the structure was performed, but it was considered that the dam concrete was unlikely to be overstressed. The main concern was with the left abutment foundation, where the orientation of joints in the rock is such that potential instability could develop if the current usual arch thrust loads should increase (under earthquake loading for example).

5.6.3 Methods of Assessing Stability

Unlike a concrete gravity dam, there is no rapid and simple method of assessing the safety of an arch dam. This is because an arch dam obtains its stability by both its self weight and to a large extent by transmitting the imposed loads by arch action into the valley walls. Detailed numerical computer analyses are normally carried out for complicated arch structures however, because Birchville dam is a simple single-curvature arch dam of modest height, a reasonable assessment of loads from arching can be obtained by simple ring theory. The dam in this case is considered to be subdivided into discrete horizontal arch elements with vertical cantilever action neglected. Such an assumption is probably only applicable to the top 5m to 10 m of the dam where arching action predominates. Using this method, an estimate of the thrusts at the abutments from water and earthquake loads can be obtained. The thrusts – from usual, flood and earthquake loading conditions – can be used to assess the stability of potential rock wedges in the dam abutments. Experience has shown that unstable wedges of rock in the abutment are the most likely feature to endanger the safety of an arch dam.

Another indirect method for assessing the stability of the dam is to compare its geometry with other arch dams and to examine the performance record of arch dams when subject to earthquake shaking. If Birchville dam has proportions similar to most other arch dams, and if the performance record of such dams has been good, this would provide some confidence in the continuing safety of the dam.

5.6.4 Arch Dam Thrusts and Abutment Stability

On the left abutment, the joints in the greywacke rock may be unfavourably aligned to produce a potential rock wedge failure mechanism as shown in Figure 6. Only friction can be relied on for potential sliding on the joint surfaces. It should also be assumed that destabilising hydrostatic pressures from the reservoir would be present along the potential failure surface of the rock wedge. Because a three-dimensional wedge mechanism needs to develop for abutment failure, an effective cohesion can be assumed along the sliding surface if the analysis is limited to the simple two-dimensional model as shown. The indicated angle of 10º that the failure wedge makes with the plane of the left abutment (refer Figure 6) will give least resistance to potential abutment failure and is therefore the most probable failure plane geometry.

Residual sliding factors of safety can be assumed for the abutment mechanism shown.
NZSOLD recommends the following minimum residual factors of safety against sliding on joints or seams when no cohesion is relied on:

- For usual loading: 1.5
- For design flood conditions: 1.3
- For maximum safety evaluation earthquake loading: 1.0

As for Korokoro and Woollen Mills dams, the relationship FS = $(\Sigma V \tan\Phi + cL)/\Sigma H$ can be used to assess the abutment stability. In this case:

ΣV = resultant of forces acting normal to the assumed sliding plane
= (Tcos30º - hydrostatic pressure on joint) - where T is the horizontal dam thrust.
Φ = residual angle of friction on rock joint surface (including effects of joint asperities)
= 45º (estimated)
c = effective cohesion on joint surface from 3-D action of failure wedge
= 100kPa (estimated)
L = length of rock wedge
= 10m nom
ΣH = resultant of forces acting along the assumed sliding plane
= Tsin30º + hydrostatic pressure on upstream face of rock wedge.

The horizontal thrusts acting on the abutment can be estimated assuming arch action only of the dam occurs (refer Section 5.6.3). This assumption would probably only apply to the top 10m or so of the dam.

At depths more than about 10m below the dam crest the water loads would be mainly transferred to the foundation at the base of the dam by vertical cantilever action. With the keying in of the dam to the rock along the base, and with the stabilising effect of the fill and rip rap placed against the toe of the dam, it is considered that the dam foundation across the valley floor should be reasonably safe from potential instability.

For arch thrusts under earthquake loading, it is assumed that the dam would be subject to 0.4g PGA shaking (as for the Korokoro and Woollen Mills dams – refer Section 4.5.3). The increase in thrust is estimated to be approximately 80% of the usual load thrusts. This takes account of the seismic inertia force from the self-weight of the dam and the hydrodynamic water pressure. During large floods the dam is only overtopped to a depth of about 0.3m. This would have little effect on the usual arch thrusts, with the pressure on the arch elements increasing by only 3kPA.

Using the simple ring or cylinder method, with an adjustment made for the included angle arch of the dam makes with the two abutments, the estimated dam thrusts (T) at 5m and 10m depth for various load conditions are shown in Table 5.4 below.

Table 5.4: Birchville arch dam
Estimated abutment thrusts per metre height

Load Case	At 5m depth (kN)	At 10m depth (kN)
Usual Static	2,000	5,000
Design Flood	2,200	5,200
Earthquake	3,500	9,000

Using the relationship provided above for the residual factor of safety against sliding, the factors for the various thrust cases are provided below.

Table 5.5: Birchville arch dam
Estimated residual sliding factors of safety at left abutment

Load Case	At 5m depth (kN)	At 10m depth (kN)	NZSOLD Minimum FS
Usual Static	2.0	1.5	1.5
Design Flood	2.0	1.5	1.3
Earthquake	1.8	1.5	1.0

These results show that there should be reasonable confidence in the stability of the left abutment. The important consideration is that under earthquake loading, when the arch dam thrusts are increased, there should be little reduction in the stability of the potential failure wedges of rock in the abutment. This is because the component of thrust normal to the failure surface increases in the same proportion as the driving force parallel to the failure surface. The cohesion (or, in this case, effective cohesion from three-dimensional effects) and the water pressures remain constant regardless of the magnitude of the thrust, and these comprise a relatively small proportion of the total resisting and driving forces.

This partially explains the inherent abutment stability of arch dams when subject to earthquake and flood loading, and their excellent performance record under such unusual conditions (refer Section 5.6.5). Given that Birchville dam has safely withstood all loads for the past 75 years, including the development of sediment loads, it is considered that the abutments should withstand earthquake loads up to at least 0.4g PGA level and also all expected levels of flood overtopping.

5.6.5 General Earthquake Performance of Arch Dams

Reports on the prototype performance of arch dams under earthquake conditions show that they have performed very well to date. More than 40 arch dams in 14 countries are known to have been subject to significant earthquake shaking. Except for Pacoima Dam in California, a 1929 dam which suffered some cracking during earthquakes in 1971 and 1994 from shaking of up to about 0.8g PGA, all of the other arch dams experienced very little or no damage. It is apparent that arch dams perform even better than concrete gravity dams, and no concrete dam (arch or gravity) has failed from earthquake effects to the extent where part or whole of the reservoir has been lost.

The Birchville dam has relatively conventional dimensions for an early constant radius arch dam of about 15m height and 40m straight line span between abutments. Initial layouts usually assume a radius of the dam axis of 0.6 times the span, giving a value of 24m for Birchville. The actual radius is 27m. Other empirical equation used for preliminary estimation of the dam crest and base thicknesses (based on the height and span of the arch dam) give dimensions of 0.64m for the crest and 2.1m for the base. The actual crest and base thicknesses for Birchville are 0.6m and 2.3m respectively. Hence the dam geometry is similar to other single curvature dams built throughout the world early in the 20th century.

Tonkin & Taylor in their 1989 stability review considered that on the basis of Schmidt hammer tests the concrete strength is likely to be in the range of 15MPa to 20 MPa. A typical recommended value for the dynamic tensile strength of concrete is 16% of the concrete compressive strength, giving an estimated strength for Birchville of about 3MPa. Under 0.4g shaking, it is probable that some cracking of the dam would occur. But it is considered that dam failure from excessive cracking in the concrete dam wall should not occur.

6.0 CONCLUSIONS

6.1 Potential Impact Classification

The Korokoro Dam and Woollen Mills Dams PIC's are assessed as Low under both the proposed Building Act regulations and the current NZSOLD Dam Safety guidelines. The Cornish Street area is a known flood hazard area with flood depths recorded at about 1.5m for a 1 in 100 year flood, it can be reasonably assumed that the risk has been identified to the working population and that they will have been evacuated before the significant flooding occurs and any dam break occurs.

Similarly, the Birchville Dam PIC is assessed as Low under the proposed Building Act regulations and Low under the current NZSOLD Dam Safety guidelines.

The criteria for and assessment of the PIC for Korokoro, Woollen Mills and Birchville Dams are summarized in Table 6.1 below.

The NZSOLD Dam Safety guidelines advise that for a low PIC rating the dam owner needs to implement adequate operational and surveillance procedures including:

- regular inspection by the operator or owner of the general condition and the consistency of aspects such as identifiable seepage
- routine maintenance of dam surfaces and spillway paths
- periodic inspections by an appropriate technical specialist (e.g. 1 to 2 years)

Under the proposed Building Act regulations the dams with low PIC's will not require mandatory Dam Safety Assurance Plans.

6.2 Korokoro Dam Safety

The Korokoro concrete gravity dam has a variable cross section, with the profile becoming more slender towards both ends of the structure. When analysed as a two-dimensional gravity structure, the central half of the dam meets or exceeds accepted stability criteria. At the rock foundation of the two end quarter spans, factors of safety against sliding fall below minimum values recommended by NZSOLD but meet US Federal guidelines for safety for a 'low' PIC structure. Taking account also of the three-dimensional load transfer that would take place in the unjointed dam wall and its satisfactory performance to date under flood loading, it is considered that the overall safety of the dam meets accepted safety evaluation guidelines.

6.3 Woollen Mills Dam Safety

The Woollen Mills concrete gravity dam has a constant cross sectional profile. It falls slightly below NZSOLD recommendations for acceptable safety against sliding in the foundation, but easily meets US Federal guidelines. The dam has safely withstood overtopping flood flows during its 100 year life. It is considered that the overall safety of the dam meets accepted evaluation guidelines for a ‘low’ PIC structure.

6.4 Birchville Dam Safety

There is no quick preliminary analysis technique available for arch dams. For assessing the safety of Birchville arch dam in an earthquake, reliance has been placed on the world-wide performance of arch dams that have been subject to seismic shaking. The geometry of the dam (radius of curvature and stem thickness) for the given span and dam height are within general arch dam design guidelines. Only slight damage has ever been recorded at arch dams that have been subject to high levels of earthquake ground motion.

An increase of only about 0.3m in the normal reservoir level is required to pass extreme floods at the site. Hence the incremental flood surcharge loading on the dam will be small.

A simple two-dimensional analysis of potential rock wedge failure at the left abutment of the dam shows that there should be an acceptable safety margin against possible abutment failure. The analysis assumes the most adverse orientation of joints in the greywacke rock, and with reliance only on friction on the rock joints to resist the arch thrusts. Because of the wedging effect an increased arch thrust produces, a large sudden increase in loading on the dam from earthquake loading should not appreciably reduce the margin of safety.

It is concluded that the following factors contribute to a good level of confidence in the continuing safe performance of Birchville dam:

- Its satisfactory performance to date over a period of 75 years.
- The conventional geometry of the dam (for a 1930’s design at least).
- The excellent performance of arch dams in earthquakes.
- The assessed acceptable level of safety against sliding of rock wedges in the dam abutments for the range of arch thrusts expected.
- The keying-in of the perimeter of the dam to the foundation.
- The presence of stabilising fill placed at the toe of the dam.
- The relative insignificance of possible future extreme flood loadings on the dam.

Taking account of the above factors, it is concluded that the dam meet acceptable safety guidelines for a 'low' PIC dam. A detailed static and seismic analysis of the dam is not considered to be warranted.

6.5 Issues Arising from Field Inspections

6.5.1 Korokoro and Woollen Mills Dams

The main issue identified from the site inspection is that the spillways for both dams are designed for an annual flood event and hence the remaining dam is subject to frequent overtopping. The Korokoro Dam has extensive erosion of the downstream toe area, refer to historical photographs in Appendix B. The Woollen Mills dam has similar erosion at the downstream toe with the toe area now flooded making it difficult to assess seepage under the dam.

The Korokoro dam has a continuous flow at the downstream toe of the order of 2.4 litres/minute observed during dry weather conditions. It appears this is related to flows from a draw off pipe (150mm dia.) located in the upstream face of the dam at the top water level, possibly a broken pipe. This flow should be investigated and repaired.

The Woollen Mills dam has been subjected to significant flooding and erosion in recent years resulting in scouring of the spillway and downstream channel immediately below the dam. The spillway performance and dam stability following loss of part of the spillway structure in 2004/05 flood event should be further investigated and repaired.

Erosion of the toe areas needs to be regularly monitored and if frequent floods lead to deterioration the area will require robust repairs.

Some minor maintenance is required to clear debris from the Korokoro Spillway.

6.5.2 Birchville Dam

Further studies and improvements for a more informed understanding of the left abutment failure mechanism were identified in the 1989 report by Tonkin and Taylor. No further detailed engineering geological logging of the abutments has been undertaken as part of this study.

It is recommended that a more detailed understanding of the left abutment geology is undertaken.

Table 6.1: Criteria for and Assessment of PIC for Korokoro, Woollen Mills and Birchville Dams

Dam	Socio-economic Impacts				Impact on People				Overall Dam
	NZSOLD and Building Act				NZSOLD		Building Act (Composite Socio + PAR assessment		
	Facilities	$ value	Environmental	PIC	Fatalities	PIC	Population at Risk (PAR)	PIC	PIC
Korokoro Dam	Moderate 1 – 3 houses damaged	<$100,000	Significant – months recovery	**Low**	No fatalities expected	Low	0 - 5	Low	**Low**
Woollen Mills Dam	Moderate 1 – 3 houses damaged	<$100,000	Short term damage	**Low**	No fatalities expected	Low	0 - 5	Low	**Low**
Birchville Dam	No damage	<$100,000	Significant – months recovery	**Low**	No fatalities expected	Low	0 - 5	Low	**Low**

APPENDIX A: FIGURES

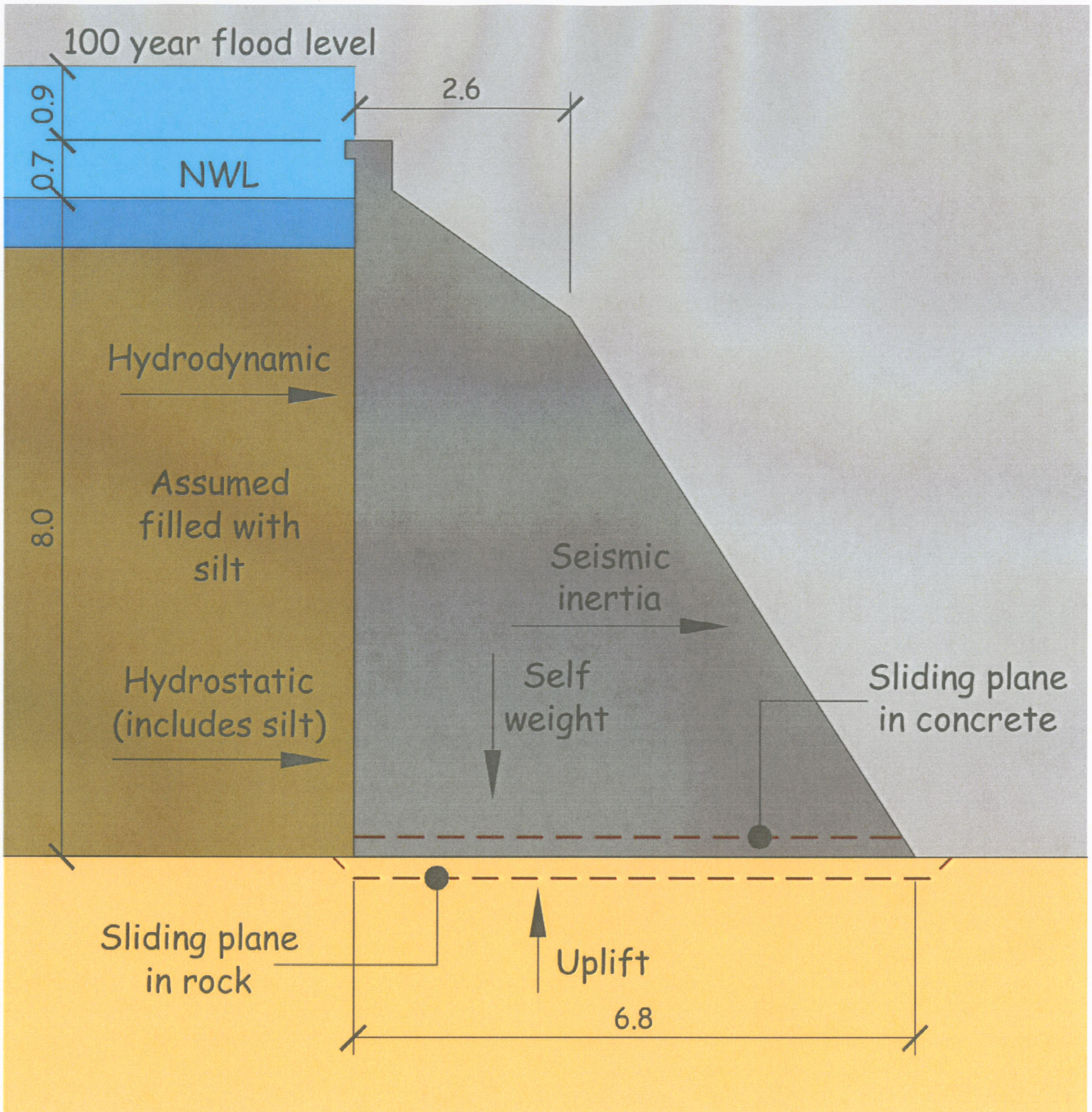

**Figure 1: Korokoro Dam – Section at centre of dam
Loads and assumed sliding planes**

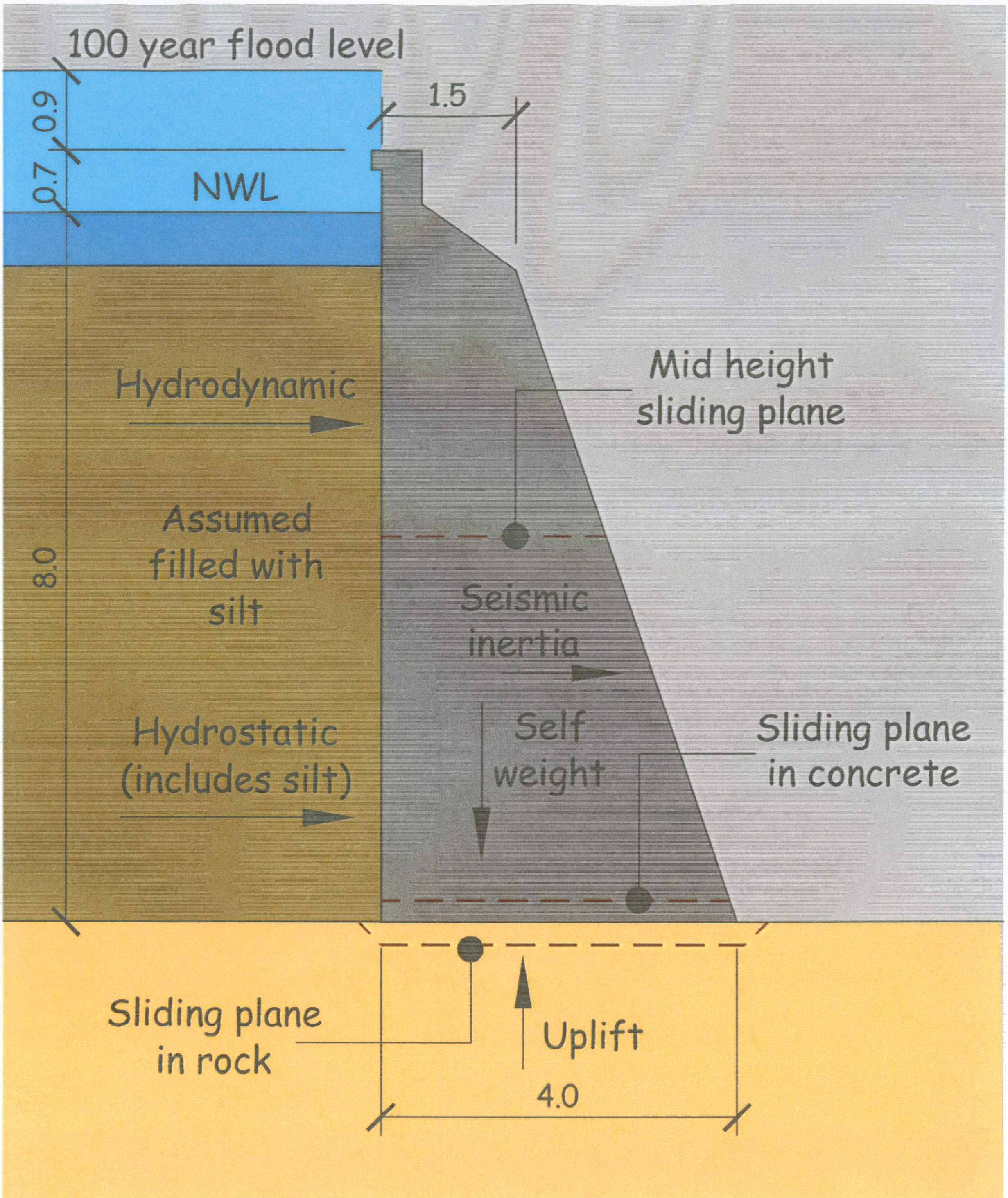

Figure 2: Korokoro Dam – Right hand section
Loads and assumed sliding planes

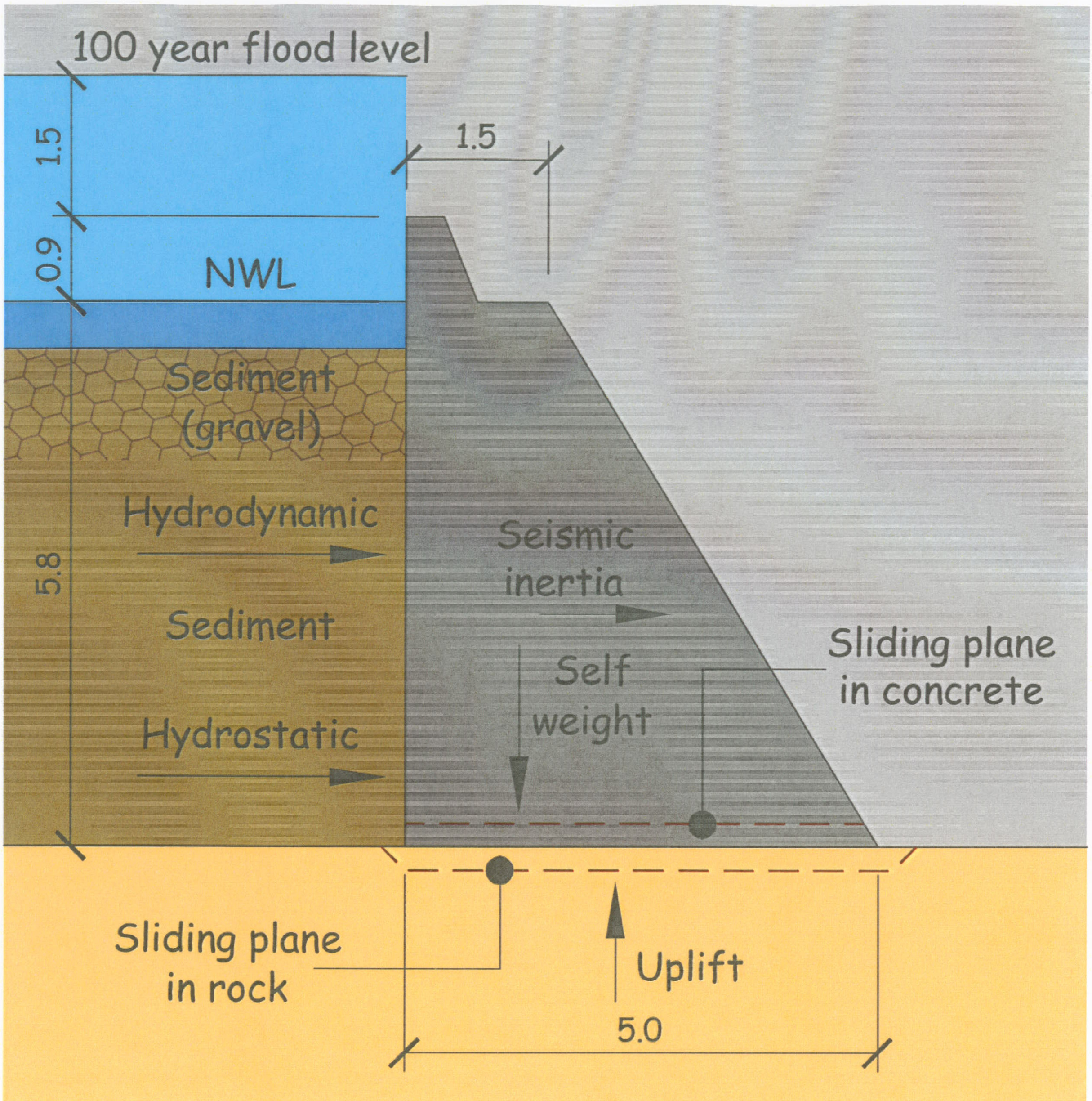

Figure 3: Woollen Mills Dam – Typical section
Loads and assumed sliding planes

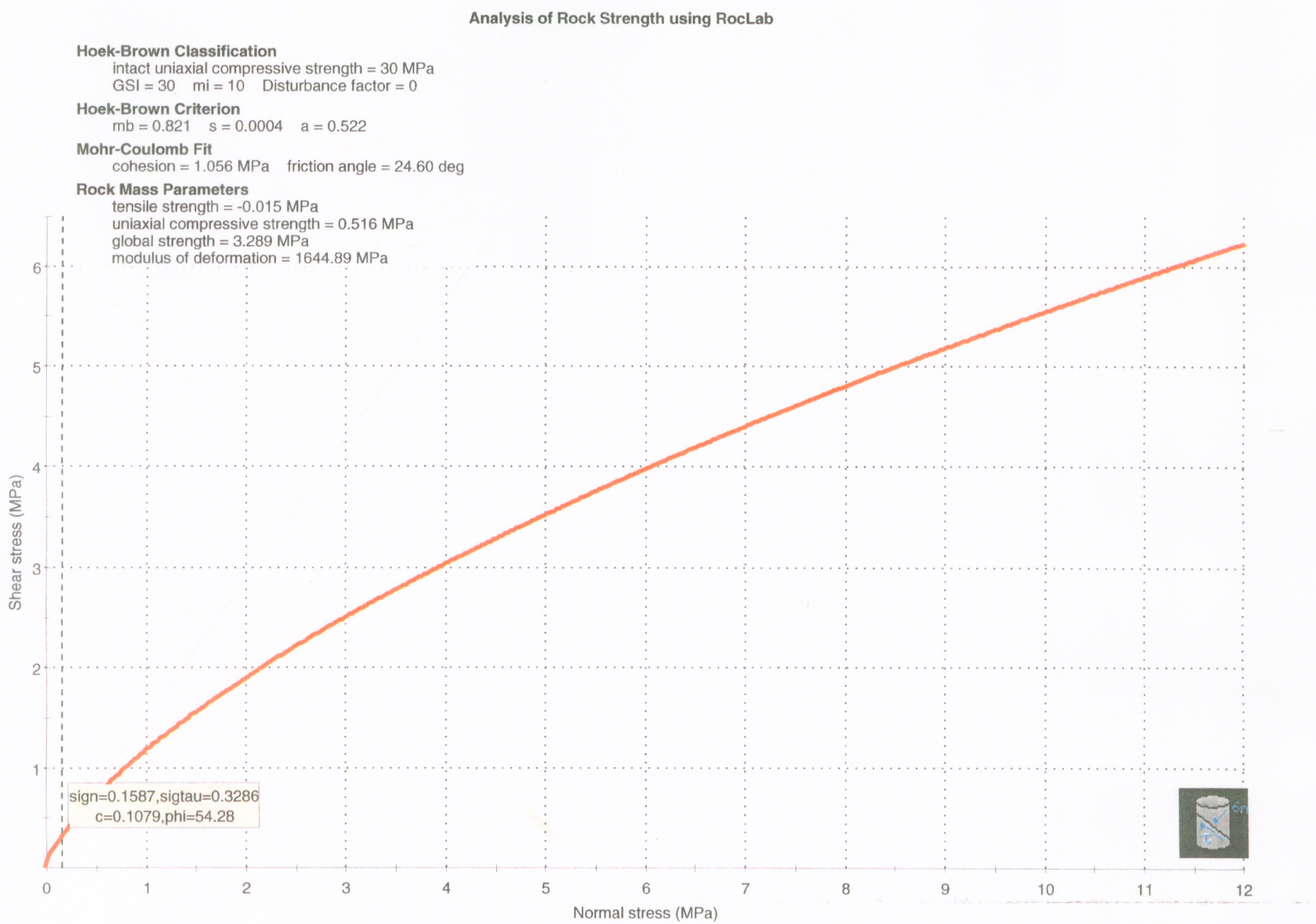

Figure 4 Korokoro and Woollen Mills Dams

Shear Strength of Greywacke Foundation Rock

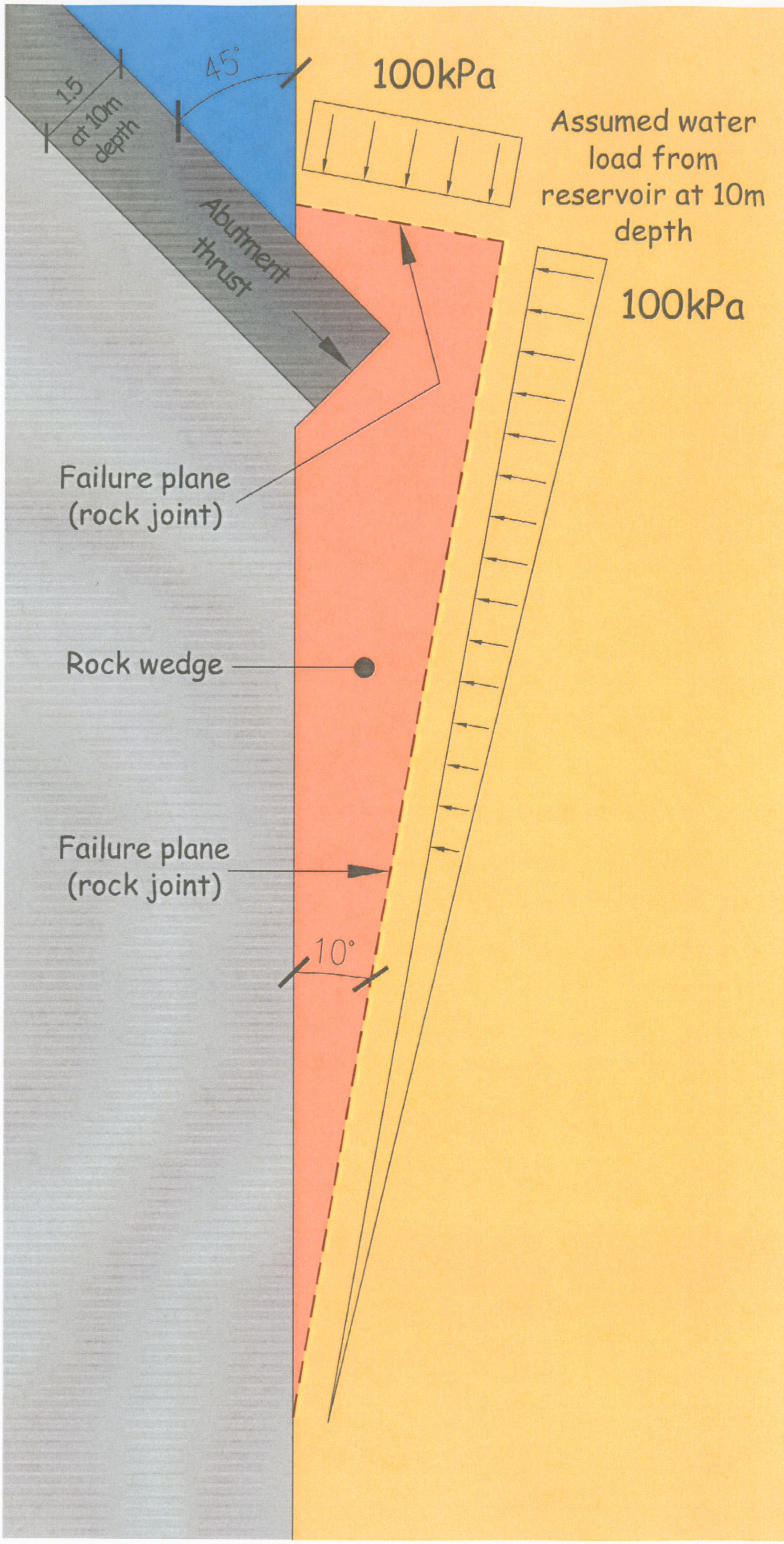

Figure 5: Birchville Arch Dam - Left Abutment
Plan section showing potential sliding of rock wedge

APPENDIX B: PHOTOGRAPHS – KOROKORO AND WOOLLEN MILLS DAM

The Korokoro Dam

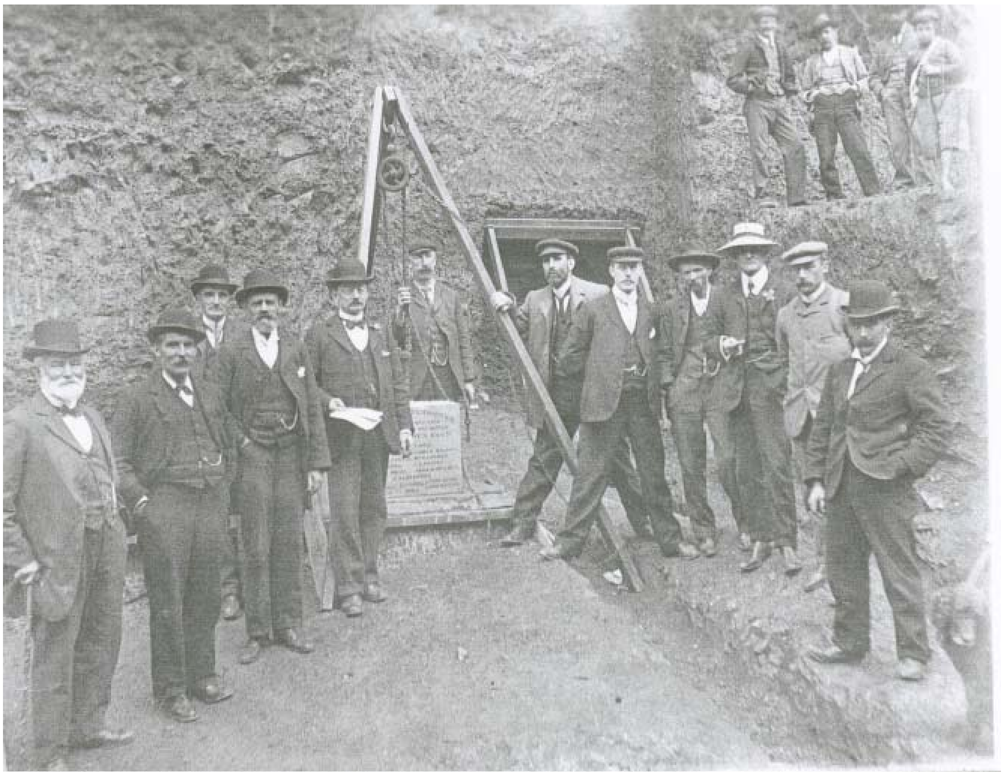

Fig. B-1: Laying the foundation stone at Korokoro Dam Construction 1903

Fig. B-2: Foundation stone located in the Korokoro Dam spillway 2006

Fig. B-3: Foundation stone at Korokoro Dam Construction 1903

Fig. B-4: Korokoro Dam and Spillway, 1906.

Note the fill level against downstream face and spillway walls.

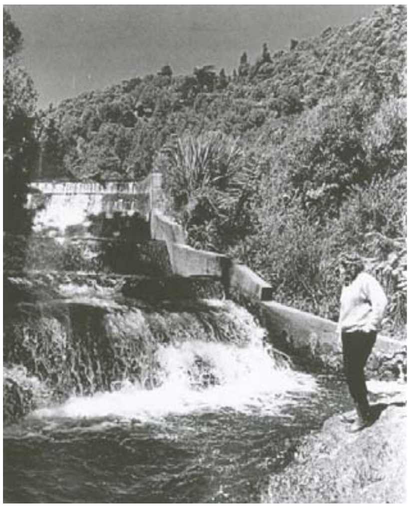

Fig. B-5: Korokoro Dam Spillway, November 1986.

Note the fill level against the spillway and downstream face has reduced significantly.

Fig. B-6: Korokoro Dam Spillway, 2006

Fig. B-7: Korokoro Dam and Reservoir, 1912

Fig. B-8: Korokoro Dam and Reservoir, 2006. Shallow depth of reservoir is clearly visible.

Fig. B-9: Extent of Korokoro Dam Reservoir, 2006.

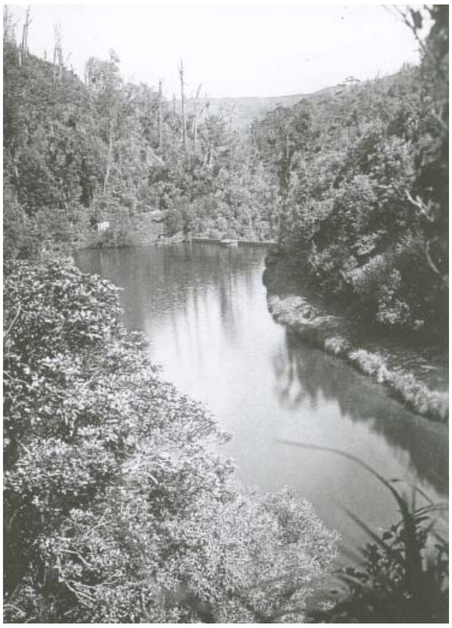

Fig. B-10: Extent of Korokoro Dam Reservoir, 1907.

Fig. B-11: Extent of Korokoro Dam Reservoir, 2006.

Fig. B-12: Extent of Korokoro Dam Reservoir, 2006.

Fig. B-13: Korokoro Dam looking toward left abutment

Fig. B-14: Korokoro Dam looking toward right abutment

Fig. B-15: Korokoro Dam Spillway inlet. Note: The build up of debris at the spillway inlet and missing brick on crest wall

Fig. B-16: Korokoro Dam downstream face of concrete dam

Fig. B-17: Damaged drain outlet pipe from downstream area to spillway plunge pool.

Fig. B-18: Ponding in downstream toe area

Fig. B-19: Signs of erosion at downstream toe area behind spillway wall(refer to Fig. B-4)

Fig. B-20: Closer view of erosion behind spillway wall, at Fig. B-19

Fig. B-21: Further signs of erosion at downstream area of Korokoro Dam, ground slip in remaining filled area.

The Woollen Mills Dam

Fig. B-22: General view of Woollen Mills Dam from downstream.

Fig. B-23: General view of Woollen Mills Dam from downstream.

Fig. B-24: Significant rock slip due to flood event in 2004 /05.

Fig. B-25: Significant rock slip due to flood event in 2004 /05.

Fig. B-26: Woollen Mills Dam upstream face

Fig. B-27: Woollen Mills Dam impoundment – high level of deposited material can be seen.

Fig. B-28: Downstream toe area permanently submerged.

Fig. B-29: Right abutment – dam cast against rock.

Fig. B-30: Left abutment.

Fig. B-31: Flood damage to spillway

Fig. B-32: Flood damage to spillway

The Korokoro Stream at Cornish Street

Fig. B-33: Debris Arrestor above Cornish Street

Fig. B-34: Korokoro Stream channel immediately above Cornish Street (Contherm Scientific Ltd)

Fig. B-35: Korokoro Stream channel immediately above Cornish Street (Contherm Scientific Ltd)

Fig. B-36: Cornish Street, entrance to Belmont Regional Park (view from upstream track).

Fig. B-37: Concrete access bridge across Korokoro Stream at top of Cornish Street.

The bridge acts as a flow restrictor, overflow to Cornish Street occurs to right, max design discharge 35 m^3/s.

Fig. B-38: Korokoro Stream channel parallel to Cornish Street, max design discharge 40 m^3/s.

Fig. B-39: Cornish Street, entrance to Belmont Regional Park.

Fig. B-40: Cornish Street, view from top of street to SH2 (entrance to Belmont Regional Park).

APPENDIX C: PHOTOGRAPHS – BIRCHVILLE DAM

The Birchville Dam

Fig. C-1: Birchville Dam and spillway

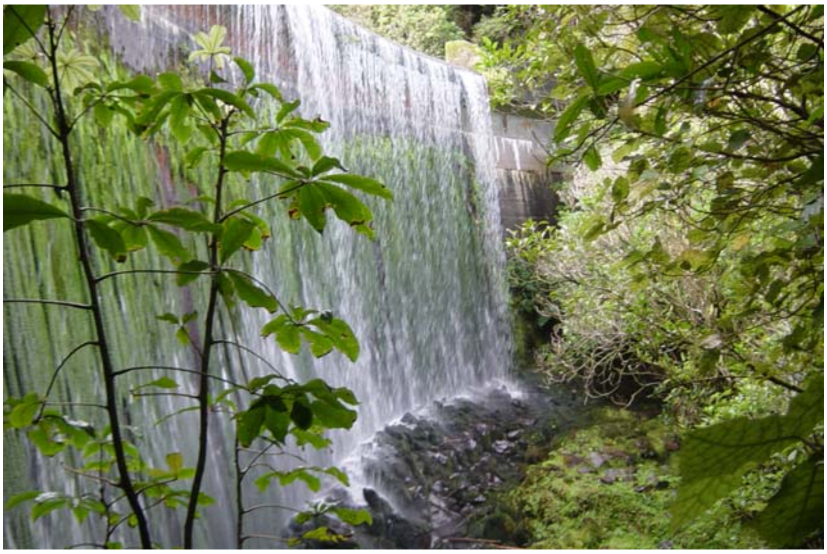

Fig. C-2: Birchville Dam and spillway

Fig. C-3: Birchville Dam and spillway – energy dissipation on rockfill

Fig. C-4: Right abutment with dam crest

Fig. C-5: Right abutment – open joints in rock

Fig. C-6: Downstream face of left side of concrete dam

Fig. C-7: Downstream face and abutment interface of left side of concrete dam

Fig. C-8: Dam crest right side – intake tower

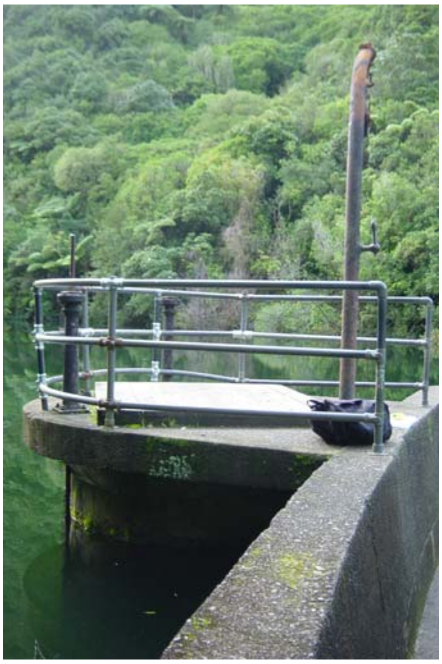

Fig. C-9: Dam crest right side – intake tower

Fig. C-10: Intake tower with control valve

Fig. C-11: Badly corroded control valve spindle

Fig. C-12: Birchville Dam Reservoir

Fig. C-13: 375mm dia. cast iron by-pass pipe outlet, some continuous leakage is occurring in the pipe.