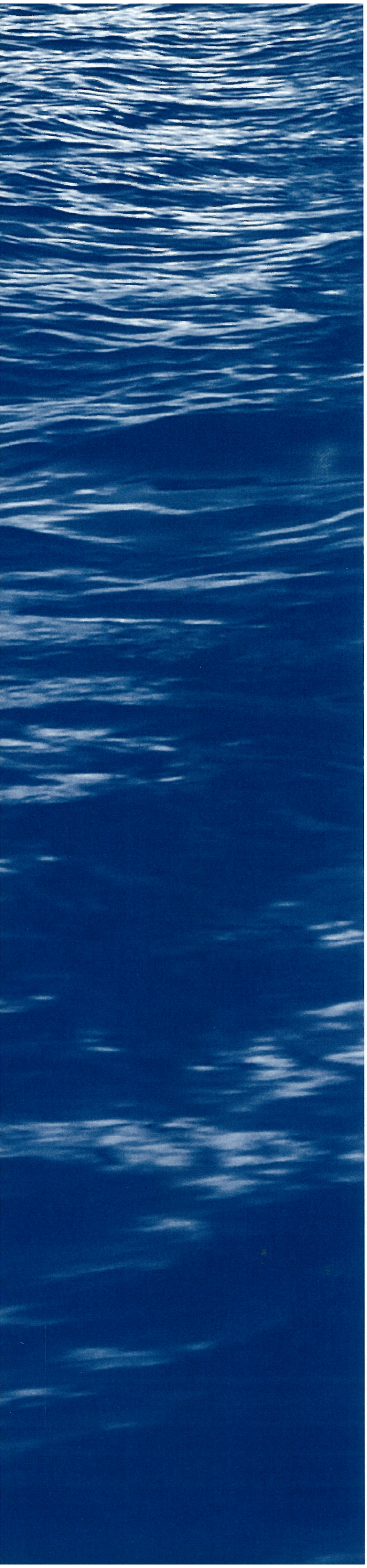

Damwatch
Dam Engineering Specialists

KOROKORO, WOOLLEN MILLS AND BIRCHVILLE DAMS

SAFETY REVIEW

March 2013

Greater Wellington Regional Council (Parks)

Revision 0 for Client Comment

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Approved for issue: PP .... 13-03-13 ....
Peter Amos Date

EXECUTIVE SUMMARY

This safety review is an independent engineering review of the safety of Korokoro, Woollen Mills and Birchville dams relative to current practice. It is the second safety review carried out for these dams and is based on a review of available documentation, site visits and discussions with Greater Wellington Regional Council staff. The first safety review was in 2006 and this report fits within the recommended 10-yearly frequency for such reviews. The safety review has been carried out in accordance with the New Zealand Society on Large Dams (NZSOLD) Dam Safety Guidelines requirements for safety review of Low potential impact category (PIC) dams.

Korokoro and Woollen Mills dams are located in Belmont Regional Park near Petone and were constructed in 1903. Birchville Dam is located in Akatarawa Forest near Upper Hutt and was constructed in 1930. The three dams were previously used for municipal water supply but now have public amenity and recreation as their primary use. The dams remain operational in the sense that they continue to impound their reservoirs.

Key dam safety status conclusions and recommendations are presented for each of the dams as follows;

<u>Korokoro Dam</u>

The dam is performing well except for significant seepage at the toe near to the spillway. It appears that the rate of seepage has increased with time and may be suggestive of erosion of the dam's foundation. Whilst this would likely not deteriorate rapidly nor lead to a rapid failure, a recommendation has been made to undertake regular monitoring of the seepage rate to characterise its behaviour with time and serve as an early warning of foundation piping.

There are no indications of gross movement of the dam or signs of instability in the foundation and abutments.

Whilst the dam has obviously been overtopped on a regular basis it has performed well. The most significant erosion is of what was either placed fill or natural ground against the downstream face of the dam. A recommendation has been made to monitor this and if undermining occurs then remediation will be required. In the meantime it is beneficial to see and measure the seepage that is emerging at this location.

The dam meets acceptability criteria for stability under all loading conditions.

There are no indications of any of the dam's failure modes developing, although better understanding of the toe seepage is required to confirm that this is not related to foundation piping.

Korokoro Dam has a low likelihood of failure and the best and most appropriate risk management is to perform regular routine surveillance inspections including the monitoring of key features, i.e. toe seepage and erosion. The value of this is significantly improved by keeping vegetation short on the toe and downstream abutment contacts.

Local capture and regular measurement of the seepage quantity at the dam toe (near to the spillway) and adjacent to the spillway plunge pool (dam 'total' seepage) will be highly beneficial to ongoing surveillance of the dam and should be implemented as a priority.

<u>Woollen Mills Dam</u>

The dam is performing well except for major erosion beneath and at the toe of the spillway. This appears to have been initiated by rock-fall damage to the spillway chute and subsequent undermining by normal and flood flows. The extent of undermining has worsened significantly over the last six years and if it continues will eventually lead to the dam's integrity being compromised through removal of base and toe support. A recommendation has been made to undertake regular monitoring of the extent of undermining in the short team and remediation should be planned for in the near future.

There are no indications of gross movement of the dam or signs of other instability in the foundation and abutments.

The dam meets acceptability criteria for stability under all loading conditions.

There are currently no indications of any of the dam's failure modes developing, however continued spillway undermining will eventually lead to dam failure through loss of base or toe support.

Woollen Mills Dam has a low to moderate likelihood of failure due to the worsening condition of the spillway. The most immediate risk management is to perform regular routine surveillance inspections including the monitoring of its key feature, i.e. spillway toe erosion. Remediation of the spillway and its toe will return the dam to its intended function and reduce the likelihood of failure to low. This should be done as a matter of priority.

<u>Birchville Dam</u>

The dam is performing very well with no indication of stress within the dam or its abutments. There is no discernible leakage or seepage, although the toe is covered with large riprap meaning that seepage in this area would not be visible. A recommendation has been made to undertake regular monitoring of the abutments, with particular attention to the left abutment to ensure that there are no indications of instability.

There are no indications of gross movement of the dam or signs of other instability in the foundation and abutments.

The dam meets acceptability criteria for stability under all loading conditions using basic analysis methods. A more complex analysis is not necessary or warranted given the dam's Low PIC and good performance over eighty three years.

There are currently no indications of any of the dam's failure modes developing, however ongoing monitoring of its abutments is important.

Birchville Dam has a low likelihood of failure. The best and most appropriate risk management is to perform regular routine surveillance inspections including the monitoring of key features, i.e. the left and right abutments.

<u>Greater Wellington Regional Council dam safety activities</u>

Greater Wellington Regional Council (GWRC) has ongoing dam safety activities for Korokoro, Woollen Mills and Birchville dams. Importantly, these include monthly routine inspections by maintenance rangers that have been trained in dam safety and surveillance. Routine dam safety inspections, in conjunction with appropriate technical support and review, are widely known to be the most effective method for monitoring dam safety.

Fundamentally, GWRC's dam safety activities include the items recommended in the NZSOLD guidelines and are being completed at the recommended, or in some cases greater, frequency.

Due to the impending regulations for the NZ Dam Safety Scheme, *from July 2014 GWRC will need to review the dam potential impact categories (PIC's) in accordance with the new Building Act methodology. If Medium PIC's are determined (currently Low), the current dam safety programme will require alignment with the dam safety assurance programme (DSAP) format specified in the regulations.*

Recommendations

The following table is a summary of recommendations made in this report, per dam and generic to GWRC's dam safety activities;

Recommendation	Report ref.	Timeframe to address
Korokoro Dam		
Clearing the vegetation on the downstream face, toe and abutment contacts, and drainage of the toe, will vastly improve ability to perform regular visual inspection.	5.2.2	2-3 months
Local capture and regular measurement of the seepage quantity at the dam toe (near to the spillway) and adjacent to the spillway plunge pool (dam 'total' seepage) will be highly beneficial to ongoing surveillance of the dam and should be implemented as a priority.	5.2.2	2-3 months
The downstream toe should be monitored during and after flood events and if undermining of the toe develops, remediation will be required. In the meantime it should remain uncovered so that toe seepage can be observed and measured.	5.2.2	During and after floods
Woollen Mills Dam		
Clearing the vegetation on the toe and abutment contacts will vastly improve ability to perform regular visual inspection.	5.3.2	2-3 months
The spillway toe undermining has worsened in the last six years and will need to be remediated before erosion removes material supporting the dam resulting in dam failure. In the meantime the extent of undermining should be regularly monitored, particularly after significant rainfall events.	5.3.2	Remediate in next 12 months
Birchville Dam		
Clearing a 2m vegetation buffer on the dam to abutment contacts will vastly improve ability to perform regular visual inspection.	5.4.2	2-3 months
It is important to inspect the downstream abutments for any signs of deep-seated instability or seepage, with particular attention to the left abutment. Surface weathering is not an issue.	5.4.2	Monthly during routines
It would be valuable to conduct a basic bathymetric survey of the reservoir using depth sounding from a boat. A significant increase in the depth of sediment against the arch dam would warrant a check of the sediment load and arch dam thrust loads.	5.4.2 & 6.3.2	1-2 years
It would be beneficial to lower the storage level temporarily to allow removal of the weed and moss, which in turn would allow a visual inspection of the downstream face concrete.	5.4.2	1-2 years
GWRC dam safety activities		
Due to the impending regulations for the NZ Dam Safety Scheme, *from July 2014 GWRC will need to review the dam potential impact categories (PIC's) in accordance with the new Building Act methodology. If Medium PIC's are determined (currently Low), the current dam safety programme will require alignment with the dam safety assurance programme (DSAP) format specified in the regulations.*	1.3.2 & 6.4.2	2 years
It would be beneficial to perform special surveillance inspections of all GWRC (Parks) dams during and after flood events to establish a documented record of the dams' performance under flood conditions. Depth of water over the dam crest supported by photography and video footage will be particularly beneficial.	6.1.1	During and after floods

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1 INTRODUCTION

1.1 Background

Damwatch has been engaged by Greater Wellington Regional Council (Parks) to perform the 2013 Safety Review of Korokoro, Woollen Mills and Birchville dams.

Korokoro and Woollen Mills dams are located in Belmont Regional Park near Petone and were constructed in 1903. Birchville Dam is located in Akatarawa Forest near Upper Hutt and was constructed in 1930. The three dams were previously used for municipal water supply but now have public amenity and recreation as their primary use. The dams remain operational in the sense that they continue to impound their reservoirs.

The purpose of this Safety Review is to provide an independent assessment of the dams' safety status relative to current practice. The Safety Review has been carried out in accordance with the New Zealand Society on Large Dams (NZSOLD) Dam Safety Guidelines (NZSOLD, 2000).

This is the second Safety Review for Korokoro, Woollen Mills and Birchville dams following the NZSOLD Guidelines. The first Safety Review was carried out in 2006 (Damwatch, 2006) and is used as reference point in assessing the current condition, performance and dam safety status of the three dams.

The Safety Review dam inspections were made on 10th of December 2012. No testing of dam valves was carried out however none of these is considered to be dam safety critical plant.

<u>Safety Review Team</u>

The Safety Review team comprised:

Dan Forster	Lead Examiner and Author	Senior Dam Safety Engineer, Damwatch Services Ltd.
Andrew Balme	Support Examiner and Author	Dam Safety Engineer, Damwatch Services Ltd.

Bronek Kazmierow, Principal Ranger, Assets and Maintenance, Parks, Greater Wellington Regional Council (GWRC) is the client for the project. During the site inspections the review team was also assisted by:

Lawrence Silas	GWRC (Maintenance Ranger)
Chris Sanders	GWRC (Maintenance Ranger)
Bryn Menzies	GWRC (Maintenance Ranger)
Joel Revill	GWRC (Maintenance Ranger)
James Craig	GWRC (Engineering Technician)

Jeremy Patterson (Belmont Regional Park Ranger) and Joanne Hunwick (Assets Coordinator) were unable to attend the site inspections however attended the dam safety awareness training the following day and provided valuable input.

The Safety Review team gratefully acknowledges the helpful and efficient assistance given by Greater Wellington Regional Council personnel.

1.2 Review scope

The activities involved in this Safety Review are:

- Visual inspection of the dam and associated hydraulic structures.
- Assessment of the construction, design, monitoring, physical condition and maintenance of the structures with reference to current acceptability criteria.
- Review and update the likelihood of failure for each dam.
- Review and update the Potential Impact Category (PIC) for each dam.
- Develop routine inspection checklists for each dam.
- Preparation of a report.

No significant new calculation or studies are carried out in this Safety Review. References have been made to existing documentation, records and any work carried out since the last Safety Review in 2006.

Internal inspections of intake towers and outlet conduits were not part of the Safety Review brief.

1.3 Context of this report

1.3.1 **Building regulations**

The Building Act (2004) introduced a draft regulatory framework for dam safety, where previously dam owners had self-regulated dam safety practices. The regulatory framework for dams is set out in the Dam Safety Scheme of the Building Act. Formal regulations that will allow the Dam Safety Scheme to operate are expected to become mandatory on 1 July 2014.

The Building Act Dam Safety Scheme requires all dam owners to have a Potential Impact Classification (PIC) for each dam over 3 m high and 20,000m^3 stored volume (Government is currently considering introduction of a higher threshold), prepared in accordance with the Building Act's classification methodology. The PIC identifies the impacts of failure of the dam in terms of population at risk and damage to environment and property. The Building Act has introduced a new methodology for determining PIC which will become compulsory on 1 July 2014. A Medium or High PIC dam is required by the Building Act to have a dam safety assurance programme (DSAP) for safe management of the dam. The DSAP will include routine and periodic activities including monitoring, gate testing, O&M plans and safety review timetables. Emergency action plans (EAP) are part of the DSAP. A Low PIC dam is not required by regulation to have a DSAP or EAP.

1.3.2 **Greater Wellington Regional Council dam safety activities**

Greater Wellington Regional Council has ongoing dam safety activities in place for the Korokoro, Woollen Mills and Birchville dams. These activities are aligned to NZSOLD dam safety guideline requirement for dams with Low PIC (refer to Section 6.4).

From 1 July 2014, GWRC will need to formally review the dam PIC's in accordance with the new Building Act methodology. If Medium PIC's are determined, the current dam safety program will require alignment with the DSAP format specified in the Regulations. The Regulations require the PIC assessment to be authorised by a Recognised Engineer.

1.4 Industry practice

Dam safety practice in New Zealand follows the NZSOLD Dam Safety Guidelines (NZSOLD, 2000). Where the NZSOLD Guidelines do not provide detail, international dam design guidelines are available. Typically reference is made to dam guidelines from the International Commission on Large Dams (ICOLD), USA, Canada and Australia.

Seismic hazard data is provided by GNS Science using a model that is based on internationally accepted techniques. GNS has recently completed a comprehensive assessment of the Wellington Fault, the closest active fault to the three GWRC dams. GNS has also been closely involved in assessing information from the Canterbury earthquakes and has just recently updated its national seismic hazard model in light of new information. This is discussed further in Section 6 stability assessments.

2 POTENTIAL IMPACT CATEGORY (PIC) REVIEW

2.1 Classification Systems

2.1.1 NZSOLD Guidelines

Under NZSOLD Dam Safety Guidelines (NZSOLD, 2000) the Potential Impact Category (PIC) is assessed based of the incremental impact (consequences) of a dam failure using the worst of sunny day and flood dambreak scenarios. The incremental consequences considered are people (life), economic losses and environment. The PIC for dams in terms of failure consequences are presented in Table 2-1.

Table 2-1 *PIC for Dams in terms of Failure Consequences (NZSOLD, 2000)*

Potential Impact Category	Potential Incremental Consequences of Failure: Life	Potential Incremental Consequences of Failure: 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

2.1.2 Building Act

In 2014 the Dam Safety Scheme of the Building Act (Department of Building and Housing, 2008) will introduce a PIC classification methodology that considers population at risk rather than expected fatalities. Population at risk (PAR) will be defined as "*the number of people likely to be affected by inundation greater than 0.5 metres in depth if they took no action to evacuate*".

As for the NZSOLD guidelines, the Building Act methodology will also consider the incremental impact of a dam failure on property and environment for the worst of sunny day and flood scenarios. The methodology is summarised in Table 2-2 and Table 2-3.

Table 2-2 *Determination of Assessed Damage Level (Department of Building and Housing, 2008)*

DAMAGE LEVEL	SPECIFIED CATEGORIES				
	Residential Houses[1]	Critical or Major Infrastructure[2]: Damage	Critical or Major Infrastructure[2]: Time to Restore to operation[3]	Natural Environment	Community Recovery Time
Catastrophic	More than 50 houses destroyed	Extensive and widespread destruction of and damage to several major infrastructure components	More than one year	Extensive and widespread damage	Many years
Major	4 to 49 houses destroyed and a number of houses damaged	Extensive destruction of and damage to more than one major infrastructure component	Up to 12 months	Heavy damage and costly restoration	Years
Moderate	1 to 3 houses destroyed and some damaged	Significant damage to at least one major infrastructure component	Up to 3 months	Significant but recoverable damage	Months
Minimal	Minor damage	Minor damage to major infrastructure components	Up to one week	Short-term damage	Days to weeks

Notes:

1. In relation to residential houses, destroyed means rendered inhabitable.
2. Includes:

(a) lifelines (power supply, water supply, gas supply, transportation systems, wastewater treatment, telecommunications (network mains and nodes rather than local connections)), and
(b) emergency facilities (hospitals, police, fire services), and
(c) large industrial, commercial or community facilities, the loss of which would have a significant impact on the community, and
(d) the dam if the service the dam provides is critical to the community and that service cannot be provided by alternative means.
3. Estimated time required to repair the damage sufficiently to return the critical and major infrastructure to normal operation.

Table 2-3 *Determination of Potential Impact Classification (PIC) (Department of Building and Housing, 2008)*

ASSESSED DAMAGE LEVEL	**POPULATION AT RISK (PAR)**			
	0	**1 to 10**	**11 to 100**	**More than 100**
Catastrophic	High	High	High	High
Major	Medium	Medium/High (see note 4)	High	High
Moderate	Low	Low/Medium/High (see notes 3 & 4)	Medium/High (see note 4)	Medium/High (see notes 2 & 4)
Minimal	Low	Low/Medium/High (see notes 1, 3 & 4)	Low/Medium/High (see notes 1, 3 & 4)	Low/Medium/High (see notes 1, 3 & 4)

Notes:

1. With a PAR of five or more people, it is unlikely that the PIC will be Low.
2. With a PAR of more than 100 people, it is unlikely that the PIC will be Medium.
3. Use a Medium PIC if it is highly likely that a life will be lost.
4. Use a High PIC if it is highly likely that 2 or more lives will be lost.

2.2 Korokoro, Woollen Mills and Birchville PIC's

2.2.1 2006 determination

The last assessment (Damwatch, 2006) determined a Low PIC for all three dams on the basis of moderate damages (socio-economic, financial and environmental), population at risk of less than 5 and no fatalities expected.

Both sunny day and incremental flood dam-break scenarios were assessed in detail and the PIC was determined using both classification systems described above. Both systems resulted in the same PIC.

2.2.2 Review in 2013

Since 2006 there is nothing to warrant changing the assumed dam-break characteristics, nor has there been an appreciable change to the downstream channel or population at risk. The population considered most at risk remains itinerant recreational walkers or cyclists on the tracks adjacent to the streams downstream of each dam. Under flood conditions it is unlikely that walkers or cyclists will be using the tracks so the governing case is sunny day failure.

The following reasoning and conclusions are supported by dam-break modelling undertaken in the 2006 PIC assessment (Damwatch, 2006).

Under sunny day condition the time factored number of itinerant walkers and cyclists can be estimated as a population at risk (PAR) of less than 5. This is to say that *on average* during daylight hours it is unlikely that at a given time there will be more than 5 people downstream of the dam in question. Remembering that the definition of PAR is "*the number of people likely to be affected by inundation greater than 0.5 metres in depth if they took no action to evacuate*" it is important to consider the likely downstream track inundation depths for each dam-break and whether or not walkers or cyclists would have sufficient warning and ability to evacuate.

In the case of all three dams the duration of sunny day dam-break flood would be very short due to the small volume of stored contents, i.e. in the order of minutes to an hour. On this basis the exposure to itinerant downstream persons is very low on a time weighted basis. Regarding dam-break flood inundation depth; the track below Korokoro and Woollen Mills dams is relatively elevated above the stream channel and generally dam-break flows will be contained within the stream channel. In comparison the track below Birchville dam is relatively close in height to the stream and in some areas the track would be inundated by up to 1m in a sunny day dam-break. However as mentioned above the released volume of water would be relatively small and the breach flood would pass in a matter of minutes. Whilst the valley is generally narrow and steep sided meaning that total evacuation would be difficult, there are a number of options to relocate above dam-break inundation levels including trees and local high points in the track.

Further downstream of all three dams there is no residential or industrial population at risk under sunny day or incremental rainy day dam-break scenarios. The respective streams exit into the Hutt River (Birchville dam) and the Wellington Harbour (Korokoro and Woollen Mills dams) shortly after emerging from the bushland.

Therefore we conclude that *(in the terminology of the Dam Safety Scheme)* it is not highly likely that a life will be lost at Korokoro, Woollen Mills and Birchville dam sunny day dambreak scenarios, nor for incremental flood rainy day scenarios.

From Table 2-2 the assessed level of damage for all three dams would be Moderate with the dominant factor being environmental damage that would take in the order of months to recover. Furthermore, this damage would be no worse than would occur during an infrequent natural flood in the valley.

In conclusion, using Table 2-3 with a Moderate level of damage and PAR of 1 to 10 (and less than 5), *the PIC determination for Korokoro, Woollen Mills and Birchville dams remains as Low (Building Act methodology).* The same PIC is determined using the existing NZSOLD methodology (refer Table 2-1), with Moderate level of damage and no expected fatalities.

It is useful to test the sensitivity of these determinations by considering what condition would result in a Medium PIC. Effectively this would be if PAR was assessed to be 5 or greater or if there was one fatality expected. At present this is considered to be unlikely, however it is something that should be reviewed in subsequent years as the tracks downstream of the dams become more popular with walkers and cyclists.

3 DAM DETAILS

3.1 General

There is limited design and construction information available for the three dams; particularly Korokoro and Woollen Mills given the year of construction. GWRC has endeavoured to search historic archives for useful reports and drawings, and thus this safety review is undertaken using the best available existing information. Basic drawings only exist for Korokoro and Birchville dams and are provided in Appendix A.

3.2 Korokoro Dam

Korokoro Dam is an 8m high concrete gravity dam on the Korokoro Stream located in the Belmont Regional Park, approximately 4 km upstream of Cornish St in Petone. The dam was built in 1903 to provide high pressure water supply for fire-fighting in Petone and was the first dam of its type to be built in New Zealand. The dam has a 37m crest length and incorporates a free overflow stepped spillway at its contact with the right abutment.

The local geology is generally anticipated to comprise sandstone-mudstone sequences of the Rakaia terrain, often referred to under the informal name 'Wellington Greywacke' (Damwatch, 2006).

The impounded reservoir has significant accumulation of soft sediment such that the active clear water storage is of the order of half a metre deep. The reservoir has also reduced significantly in surface area as the upstream extent has advanced downstream again through sedimentation. The total original Korokoro reservoir volume is estimated to be 30,000 m^3, but today only 2,500 m^3 would likely be released in a sunny day failure. This is a conservative estimate based on the present water storage depth and expected behaviour of the deposited sediment (Damwatch, 2006).

3.3 Woollen Mills Dam

Woollen Mills Dam is a 6m high concrete gravity dam on the Korokoro Stream, approximately 2.5km downstream of Korokoro Dam, within Belmont Regional Park. The dam was built in the same year as Korokoro Dam (1903) as part of a compensation package to the Petone Woollen Mills who had previously collected water in the lower reaches of the Korokoro Stream utilising a timber dam. The dam has a crest length of approximately 16m and incorporates a free overflow spillway at its contact with the right abutment.

The local geology is generally anticipated to comprise the same sandstone-mudstone sequences as Korokoro Dam often referred to under the informal name 'Wellington Greywacke' (Damwatch, 2006).

The impounded reservoir has significant accumulation of gravel sediment such that active clear water storage is almost non-existent. The original Woollen Mills reservoir storage was estimated as 3,000 m^3, but today only 600 m^3 would likely be released in a sunny day failure. This is a conservative estimate based on the present water storage depth and expected behaviour of the deposited gravels (Damwatch, 2006).

3.4 Birchville Dam

Birchville Dam is a 15m high single curvature arch dam on Clarke Stream, which is a west bank tributary of the Hutt River at Birchville. The dam was built as part of Upper Hutt's water supply in 1930. The dam has a constant radius of 27.5m and a crest (chord) length of 40m (arch length 46m). Whilst the dam is 15m high above its lowest foundation the use of boulders at the toe for spill protection give the appearance of a lower dam height.

The local geology is generally anticipated to comprise sandstone-mudstone sequences of the Rakaia terrain, often referred to under the informal name 'Wellington Greywacke' (Damwatch, 2006).

The impounded reservoir is partially silted with 1988 measurements showing silt levels 6.3m below the spillway crest. The Birchville reservoir storage volume was estimated to be 20,000 m³ in 1989 and today following continued deposition of sediment it is estimated that 10,000 m³ would likely be released in a sunny day failure of the dam (Damwatch, 2006).

4 FAILURE MODES DETERMINATION

4.1 General

The understanding of a dam's credible failure modes provides the basis for comprehensive assessment of its ongoing safe performance. Damwatch has applied this methodology for a number of years and it is now recommended in a number of jurisdictions. Failure modes are the most likely credible ways that a dam will fail, if it was to fail, and therefore do not imply that failure is imminent by any of the defined modes. Understanding a dam's failure modes allows operation, surveillance and maintenance activities to be targeted directly at the areas of key importance and thus provide best prevention, or at worst, early detection of the initiation of a dam failure mechanism. The same applies to safety reviews of dams in ensuring that issues relevant to the safety of the dam are not overlooked.

Korokoro, Woollen Mills and Birchville dams have not had failure modes formally developed as an independent exercise that reviews all historical information on the subject dam and forms a collective judgment of its credible failure modes. For the purpose of this safety review we have carried out a preliminary determination as an aid to describing issues relevant to the dams. We consider this level of assessment appropriate given that the dams are Low PIC and there is limited design and construction information available.

4.2 Korokoro Dam

4.2.1 Failure modes

The following credible failure modes have been determined for Korokoro Dam;

FM1: Flood overtopping leading to dam instability and/or undermining
FM2: Piping failure in the dam foundation under normal loading
FM3: Earthquake leading to significant structural damage

This determination is based on the following influencing factors;

- Korokoro Dam is a concrete gravity dam with no foundation relief drains and no uplift instruments (typical for its era), meaning that the potential for instability is increased where reservoir head is transmitted to the base of the dam.
- The spillway is sized for an annual flood (approx.) meaning that the dam is regularly overtopped. There is evidence of erosion at the toe of the dam.
- There is a significant seep emerging at the downstream toe adjacent to the spillway, indicating foundation piping erosion as a possible causal mechanism.
- Korokoro Dam is located very close to the Wellington Fault meaning that in a seismic event, earthquake shaking loads will be very high.

4.2.2 Surveillance

Regular visual inspection and measurement of any seepage or leakage are the best forms of early detection of these failure modes. The key performance indicators are;

- Toe erosion *(FM1)*
- Increasing and/or turbid (muddy) leakage *(FM2)*
- Earthquake damage *(FM3)*

Special inspections during and after floods and after earthquakes are also important given the unusual load condition. Inspections should include observation of flood overtopping flow paths, erosion/undermining, structural damage, unusual seepage and reservoir rim instability.

4.3 Woollen Mills Dam

4.3.1 Failure modes

The following credible failure modes have been determined for Woollen Mills Dam;

FM1: Flood overtopping leading to dam instability and/or undermining
FM2: Piping failure in the dam foundation under normal loading
FM3: Earthquake leading to significant structural damage

This determination is based on the following influencing factors;

- Woollen Mills Dam is a concrete gravity dam with no foundation relief drains and no uplift instruments (typical for its era), meaning that the potential for instability is increased where reservoir head is transmitted to the base of the dam.
- The spillway is sized for an annual flood (approx.) meaning that the dam is regularly overtopped.
- There is major undermining of the spillway plunge pool that has regressed significantly upstream over the last six years. This will likely lead to dam instability through removal of toe support or foundation piping due to shortened seepage length increasing hydraulic gradient.
- Woollen Mills Dam is located very close to the Wellington Fault meaning that in a seismic event earthquake shaking loads will be very high.

4.3.2 Surveillance

Regular visual inspection and measurement of the extent of plunge pool undermining and any seepage or leakage is the best form of early detection of these failure modes. The key performance indicators are;

- Toe erosion (including spillway plunge pool) *(FM1)*
- New/increasing and/or turbid (muddy) leakage *(FM2)*
- Earthquake damage *(FM3)*

Special inspections during and after floods and after earthquakes are also important given the unusual load condition. Inspections should include observation of flood overtopping flow paths, erosion/undermining, structural damage, unusual seepage and reservoir rim instability.

4.4 Birchville Dam

4.4.1 Failure modes

The following credible failure modes have been determined for Birchville Dam;

FM1: Earthquake leading to significant structural damage and/or left abutment failure
FM2: Left abutment failure under normal loading

This determination is based on the following influencing factors;

- Birchville Dam is a single curvature arch dam with no foundation or abutment relief drainage. Arch dams are typically very robust and perform well in all loading conditions provided the abutments continue to support dam loads.
- The dam toe is resilient to flood overtopping on the assumption that it is well embedded into the foundation and protected with extensive riprap (indicated on design drawing and supported visually). The dam is also indicated to be keyed in to the abutments, providing a degree of resistance to erosion under flood overtopping.

- The left abutment rock is adversely jointed, weathered and re-entrant. These factors present the potential for abutment instability, however there is no indication of deep seated instability or seepage.
- Birchville Dam is located very close to the Wellington Fault meaning that in a seismic event earthquake shaking loads will be very high.

4.4.2 **Surveillance**

Regular visual inspection with particular attention to the abutments is the best form of early detection of these failure modes. The key performance indicators are;

- Slope distress in the left abutment *(FM1 and FM2)*
- New/increasing and/or turbid (muddy) leakage in the left abutment *(FM1 and FM2)*
- Structural damage to the dam under normal or earthquake loading *(FM1 and FM2)*

Special inspections during and after floods and after earthquakes are also important given the unusual load condition. Inspections should include observation of flood overtopping flow paths, erosion/undermining, structural damage, unusual seepage and reservoir rim instability.

5 INSPECTION FINDINGS

The safety review dam inspections were made on 10th of December 2012. No testing of dam valves was carried out.

5.1 Safety Review Team

The Safety Review team comprised:

Dan Forster	Lead Examiner and Author	Senior Dam Safety Engineer, Damwatch Services Ltd.
Andrew Balme	Support Examiner and Author	Dam Safety Engineer, Damwatch Services Ltd.

The dam inspections were also attended by:

Bronek Kazmierow	GWRC (Principal Ranger)
Lawrence Silas	GWRC (Maintenance Ranger)
Chris Sanders	GWRC (Maintenance Ranger)
Bryn Menzies	GWRC (Maintenance Ranger)
Joel Revill	GWRC (Maintenance Ranger)
James Craig	GWRC (Engineering Technician)

5.2 Korokoro Dam

5.2.1 Key findings summary

The dam is in remarkably good condition considering it is now one hundred and ten years old. The key issues identified during the inspection of Korokoro Dam were;

1. Significant vegetation on the dam groins, downstream face and toe. *Clearing the vegetation, particularly on the toe, will vastly improve ability to perform regular visual inspection.*
2. Water ponded on the toe, particularly against the left abutment. *Drainage of the toe will vastly improve the value of regular visual inspection and seepage monitoring.*
3. Significant seepage emerging from the toe of the dam adjacent to the spillway. *Local capture and regular measurement of the seepage quantity will be highly beneficial to ongoing surveillance of the dam. Likewise capture and regular measurement of dam 'total' seepage adjacent to the spillway plunge pool will be highly beneficial.*

5.2.2 Inspection detail

(Referenced site inspection photos are included in Appendix B)

<u>Reservoir</u>

The reservoir has significant accumulation of soft sediment such that the active clear water storage is of the order of half a metre deep. The reservoir has also reduced significantly in surface area as the upstream extent has advanced downstream also with accumulation of sediment (photo 1).

The reservoir rim is heavily vegetated and has no signs of instability.

Abutments

Both left and right abutments are heavily vegetated with no signs of instability (photos 1 and 2). There is no sign of erosion at the upstream contact of dam to abutment on either abutment. The downstream left groin is vegetated right to the dam contact (photos 2 and 9). *It is recommended that a 2m clear width be provided to allow ongoing inspection of this key area.* The downstream right abutment is vegetated right to the contact with the spillway chute (photos 3, 4 and 11). *It is recommended that selective trimming of overhanging vegetation be undertaken to allow ongoing inspection of this key area.*

Upstream face, crest and downstream face

The dam upstream face, crest and downstream face concrete are sound with no significant defects. There is a brick missing from the crest detail near to the spillway, however this is of low consequence (photos 2 and 3). A large portion of the downstream face has accumulated sediment and grass growth. *It is recommended that this be removed to allow clear view of the downstream face.*

The crest accommodates appendages in the form of a wooden walkway and viewing platform for visitors (photos 1 and 2). These are in reasonable condition and are only of dam safety significance during major floods when they will catch debris and reduce the discharge capacity over the dam crest.

Spillway

The spillway comprises a free overflow crest and stepped chute at the right abutment contact (photos 3, 4, 5 and 11). The spillway is in fairly good condition with no evidence of major erosion in the chute steps or plunge pool. The chute walls are upright with no signs of damage. Anecdotal evidence suggests that the spillway chute, and potentially the dam also, are frequently overtopped. This is supported by evidence of significant erosion of what is either placed fill or natural ground adjacent to the spillway chute and above the toe of the dam.

Toe

The dam toe is heavily vegetated with ferns, trees and grasses (photos 5 and 9-11). *It is recommended that the trees and grasses be removed to allow regular visual inspection of this key area.*

There is significant seepage emerging from beneath the spillway chute and from the toe of the dam adjacent to the spillway (photos 6-8). This has increased from approximately 2.4 litres per minute in 2006 to approximately 12 litres per minute at the time of inspection (December 2012). Whilst these measurements were made without an accurate instrument or set-up, it is clear there has been a notable increase in the rate of seepage over recent years and this is concerning. *Local capture and regular measurement of the seepage quantity will be highly beneficial to ongoing surveillance of the dam and should be implemented as a priority. Likewise capture and regular measurement of dam 'total' seepage adjacent to the spillway plunge pool will be highly beneficial.*

There is ongoing evidence of erosion at the dam toe of what is either placed fill or natural ground. *This should be monitored during and after flood events and if undermining of the toe develops remediation will be required. In the meantime this area should be uncovered so that toe seepage can be observed and measured. Comparative photography is a useful tool in determining changes to erosion.*

There is water ponded on the toe, particularly against the left abutment (photo 2). Drainage of the toe will vastly improve the value of regular visual inspection.

There is a large diameter pipe of unknown origin exiting adjacent to the spillway plunge pool (photo 11). It is thought that this may be the outlet for the dam outlet works, which is believed to be out of service. Clearing of vegetation on the dam toe may confirm this.

5.3 Woollen Mills Dam

5.3.1 Key findings summary

The dam is one hundred and ten years old and is in good condition except for the spillway toe. The key issues identified during the inspection of Woollen Mills Dam were;

1. Major damage to the lower extent of the spillway chute and regression/undermining upstream of the spillway plunge pool. *This has worsened in the last six years and will need to be remediated before erosion removes material supporting the dam resulting in dam failure. In the meantime the extent of undermining should be regularly monitored, particularly after significant rainfall events.*
2. Significant vegetation on the dam groins and toe. *Clearing the vegetation, particularly on the toe, will vastly improve ability to perform regular visual inspection.*

5.3.2 Inspection detail

(Referenced site inspection photos are included in Appendix B)

Reservoir

The reservoir is almost non-existent with significant accumulation of gravels (photos 13 and 14). The active clear water storage is less than half a metre deep and of the order of 10 metres upstream extent.

The upstream valley is very narrow and steep sided with numerous instances of rock-fall (photo 12). There is a moderate to high likelihood of the occurrence of rock-fall leading to temporary blockage of the Korokoro Stream, impoundment of a reservoir and likely subsequent breach. This would likely be damaging to Woollen Mills dam through sudden overtopping and the impact of debris.

Abutments

Both left and right abutments are steep with moderate vegetation (photos 13-17). The left abutment is stable (photos 21 and 22) whereas the right abutment exhibits significant rock-fall downstream of the dam alignment (photos 13, 16 and 17). It appears that rock-fall damaged the spillway chute and contributed to regression/undermining of the spillway toe (19 and 22). However, rock instability on the right abutment does not appear to be deep seated and as such does not compromise the dam.

Upstream face, crest and downstream face

The dam upstream face, crest and downstream face concrete are sound with no significant defects (photos 13, 17 and 21). There is a crack that extends down the spillway wall, across the crest and diagonally down the upstream face toward the spillway crest. The crack is not open or displaced and does not affect the structural integrity of the main dam body (photo 18).

Spillway

The spillway comprises a free overflow crest and chute at the right abutment contact. The spillway chute and toe are in very poor condition with significant loss of the chute slab and major undermining of the toe (photo 22). Probing with a stick under the remaining downstream chute slab indicates an eroded cavity of 1.5m horizontal extent and 2m depth approximately. *This has worsened significantly in the last six years and will need to be remediated before erosion removes material supporting the dam resulting in dam failure. In the meantime the extent of undermining should be regularly monitored, particularly after significant rainfall events.*

Toe

The dam toe is heavily vegetated with weeds (photos 15 and 23). Inspection of the immediate toe following cursory weed-clearing indicated no signs of erosion. *It is recommended that the weeds be removed to allow adequate visual inspection of this key area.*

At times there is water ponded on the toe of the dam, however drainage of this area could be difficult due to the relatively flat ground/streambed slope.

5.4 Birchville Dam

5.4.1 Key findings summary

The dam is eighty three years old and is in very good condition. The key issues identified during the inspection of Birchville Dam were;

1. Significant vegetation on the dam groins and toe. *Clearing a 2m buffer on the groins will vastly improve ability to perform regular visual inspection.*
2. *Importance of regular visual inspection of the downstream left abutment checking for any signs of deep-seated instability or seepage. Surface weathering is not an issue.*
3. *It would be valuable to conduct a basic bathymetric survey of the reservoir using depth sounding from a boat.*
4. *It would be beneficial to lower the storage level temporarily to allow removal of the weed and moss, which in turn would allow a visual inspection of the downstream face concrete.*

5.4.2 Inspection detail

(Referenced site inspection photos are included in Appendix B)

Reservoir

The reservoir is reported to have significant accumulation of sediment, with sediment measured 6.3m below the spillway crest in 1988. In the years since this measurement it is expected that the depth to sediment will have decreased, thus decreasing the active water storage volume. *It would be valuable to conduct a basic bathymetric survey of the reservoir using depth sounding from a boat.*

The reservoir rim is steep and heavily vegetated with no signs of instability (photo 24).

Abutments

Both left and right abutments are steep with heavy vegetation (photos 25, 27 and 30). Both abutments are visibly stable although there has been previous question raised over the stability of the left abutment due to jointing and re-entrant nature (Tonkin and Taylor, 1989). *Inspection of the upper and downstream left abutment reveals that whilst there is significant surface weathering there are no indications of deep-seated instability or leakage. This key area should continue to be monitored during routine visual inspections.*

Both left and right ends of the dam make positive contact with their respective abutments. There is no reservoir erosion at the upstream contacts (photo 28). *To improve visual inspection of the downstream contacts a 2m strip clearance of vegetation is recommended (photos 29 and 31).*

Upstream face, crest and downstream face

The dam upstream face, crest and downstream face concrete are sound with no significant defects (photos 25, 27 and 29). The downstream face is covered with weed and moss due to the constant presence of spill flow (photos 26 and 30). *It would be beneficial to lower the storage level temporarily to allow removal of the weed and moss, which in turn would allow a visual inspection* *of the downstream face concrete.* Based on other observations of the dam concrete it is expected that the concrete beneath the weed and moss is in reasonable condition.

Spillway

The spillway comprises a free overflow crest central to the dam with toe erosion protection in the form of riprap (photos 25 and 26). The spillway crest length is approximately half the dam crest length at an estimated 23 m. The spillway is in good condition with the toe riprap intact and continuing to provide erosion protection to the dam toe.

Toe

The immediate dam toe is clear of vegetation and well protected from spill flow by substantial depth of riprap (photos 26 and 30). There are no indications of dam or foundation leakage.

6 ASSESSMENT WITH RESPECT TO CURRENT ACCEPTIBILITY CRITERIA FOR LOW PIC DAMS

6.1 Korokoro Dam

6.1.1 Flood passage capacity

Flood hydrology for the Korokoro Stream was last assessed in 2003 with flood return periods and discharges given for the Woollen Mills dam up to Q_{100} (Opus, 2003). This remains the current and best estimate of flood conditions in the Korokoro Stream.

NZSOLD dam safety guidelines (NZSOLD, 2000) defines the flood capability acceptance criterion for a Low PIC dam is, "between 1:100 and 1:1,000 AEP (annual exceedance probability)", termed the 'inflow design flood' (IDF).

Flood return periods and discharges were subsequently estimated for the Korokoro dam site with a one hundred year return period flood of 42 m^3/s. The 2006 safety review estimated Korokoro dam spillway as having a capacity of 5 m^3/s, which is approximately an annual flood event (Damwatch, 2006). Thus on a current standards basis, Korokoro Dam does not have sufficient flood capability using its spillway. It is expected that the dam is regularly overtopped by floods and this is supported by the evidence of significant erosion of the downstream toe area.

This being said, the dam has withstood what is believed to be a significant degree of overtopping remarkably well for its lifetime of one hundred and ten years. On a historical performance basis the dam performs satisfactorily under flood overtopping and has low incremental consequence of failure by this mode (it is unlikely there will be recreational users downstream during major flooding).

The area that has undergone the most significant erosion is at the dam toe adjacent to the spillway chute wall, where significant material has been removed. It is not certain whether this material was natural ground or fill placed during construction. The dam toe or spillway chute are not currently undermined, however there is significant seepage emerging from this location, including from bony concrete beneath the spillway chute. *It is recommended to continue to monitor the dam toe and spillway wall area for undermining and if it develops to remediate and install a form of protection or energy dissipation.* In the meantime this area should be uncovered for ongoing monitoring of seepage flows.

It would be beneficial to perform special surveillance inspections during and after flood events to establish a documented record of the dam's performance under flood conditions. Depth of water over the dam crest supported by photography and video footage will be particularly beneficial.

6.1.2 Dam stability assessment

The 2006 safety review (Damwatch, 2006) assessed dam stability in some detail. The assessment remains current on the basis that the parameters used for foundation and dam characteristics and the assessment loads still stand. This safety review provides a summary of the 2006 Korokoro dam stability assessment to demonstrate the approach used and present key results. Further details can be found in the 2006 safety review.

<u>General</u>

A key failure mechanism to be assessed for a concrete gravity dam is sliding on any horizontal or near-horizontal plane within the dam, at the base, or on any rock seam in the foundation.

The NZSOLD dam safety guidelines (NZSOLD, 2000) provide the following recommended minimum values for the factors of safety against sliding.

- For normal 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 normal 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

Sediment

The 2006 safety review (Damwatch, 2006) took into account the degree of sediment stored in the dam's reservoir (approx. 7.5m depth of sediment used) to calculate sediment loads on the dam. This is assumed not to have changed significantly in six years (relative to the lifetime of the dam) and has not been recalculated.

Flood loading

As discussed in Section 6.1.1, the 1:100 AEP flood is the minimum requirement for flood loads. The 1:100 AEP flood of 42 m^3/s is estimated to overtop the dam to a depth of 0.9m, which is approximately 1.6m above the normal reservoir level (Damwatch, 2006).

Earthquake loading

For a Low PIC dam 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 annual exceedance probability (AEP) event as the safety evaluation earthquake for Low PIC structures (Mejia L. et al, 2001). This is considered appropriate for this dam. The New Zealand Standard earthquake loading code (Standards New Zealand, NZS 1170.5:2004) can be used to determine the earthquake shaking the dams would be subject to in a 1:500 AEP earthquake, and this shows that a 0.4g peak ground acceleration would be appropriate for this dam. A pseudo-static analysis was used (Damwatch, 2006).

The loading code is based on the National Seismic Hazard Model developed by GNS Science. GNS has very recently reviewed and updated the model, with particular attention taken to apply learnings from the Canterbury earthquakes to other regions of the country. *Damwatch has spoken with GNS regarding any changes to the seismic hazard model and there has been no change in the Wellington area to the short period motions that affect dams. Therefore the peak ground acceleration taken from the 2004 earthquake loading code remains appropriate.*

The seismic hazard model may also be used to estimate historic ground accelerations (modelled values) that the dam sites have been exposed to in the past.

Dam sections

For Korokoro Dam two sections were analysed in Damwatch (2006); the centre of the dam and the right hand side adjacent to the spillway, with three horizontal sliding planes considered, namely; in the rock foundation, in the concrete near the base of the dam, and at mid height (right hand side of dam only).

Assessed factors of safety under all loading conditions

Table 6-1 is a summary of factors of safety against both NZSOLD and FERC recommended values, as determined in the 2006 safety review (Damwatch, 2006).

Table 6-1: *Korokoro Dam sliding factors of safety (Damwatch, 2006)*

Load case	In rock foundation	In concrete near base	In concrete mid height	NZSOLD min. (Low PIC)	FERC min. (Low PIC)
Centre section					
Normal	3.4	9.4	-	3.0	2.0
Flood	2.5	6.9	-	2.0	1.25
Earthquake	1.6	4.3	-	1.1	>1.0
Post-eq	2.1	5.3	-	2.0	1.25
Right hand section					
Normal	2.0	5.5	13.7	3.0	2.0
Flood	1.4	4.1	6.8	2.0	1.25
Earthquake	1.1	2.9	6.2	1.1	>1.0
Post-eq	1.2	3.1	7.5	2.0	1.25

Assessment of results

The results show that in the foundation; both NZSOLD and FERC minimum factors of safety are achieved in the centre section, whereas only the FERC factors are met in the right hand section. However, there is anticipated to be some three dimensional structural contribution at the more slender right hand section because there are no contraction joints or vertical cracks. This would increase the factors of safety determined in the two-dimensional analysis. The reverse is true for the centre section, however the available margins of safety are greater.

Overall, it is considered that Korokoro Dam meets acceptable foundation sliding stability criteria (Damwatch, 2006).

6.2 Woollen Mills Dam

6.2.1 Flood passage capacity

Flood hydrology for the Korokoro Stream was last assessed in 2003 with flood return periods and discharges given for the Woollen Mills dam up to Q_{100} (Opus, 2003). This remains the current and best estimate of flood conditions in the Korokoro Stream.

Woollen Mills dam site has a one hundred year return period flood of 67 m^3/s. The 2006 safety review estimated Woollen Mills dam spillway as having a capacity of 5.5 m^3/s, which is approximately an annual flood event (Damwatch, 2006). It is expected from this that the dam is regularly overtopped by floods. This is supported by anecdotal evidence, however apart from the spillway chute and plunge pool damage there is no significant damage observed at the dam toe.

As a general rule, the dam has withstood what is believed to be a significant degree of overtopping remarkably well for its lifetime of one hundred and ten years. The area that has undergone the most significant erosion is at the spillway chute and plunge pool where significant undermining exists. This is believed to have been contributed to by rock-fall causing damage to the spillway chute slab, however is undoubtedly worsened by continued frequent floods and less frequent major floods.

It would be beneficial to perform special surveillance inspections during and after flood events to establish a documented record of the dam's performance under flood conditions. Depth of water over the dam crest supported by photography and video footage will be particularly beneficial.

NZSOLD dam safety guidelines (NZSOLD, 2000) defines the flood capability acceptance criterion for a Low PIC dam is, "between 1:100 and 1:1,000 AEP (annual exceedance probability)", termed the 'inflow design flood' (IDF). Thus on a current standards basis, Woollen Mills Dam spillway does not have sufficient flood capacity. However, on a historical performance the dam performs fairly well under flood overtopping and has low incremental consequence of failure by this mode (it is unlikely there will be recreational users downstream during major flooding). In support of this, the very small volume of remaining reservoir reduces the consequences of dam break. In this respect a dam 'failure' is unlikely to result in an incrementally significant release of stored water content.

6.2.2 Dam stability assessment

The stability assessment for Korokoro Dam (refer Section 6.1.2) provides introductory commentary on the method used for the 2006 stability assessment for Korokoro and Woollen Mills dams (the same methodology was used for both dams).

As far as Woollen Mills dam stability is concerned the 2006 assessment remains current on the basis that the parameters used for foundation and dam characteristics and the assessment loads still stand. This safety review provides a summary of the assessment to demonstrate the approach used and present key results. Further details can be found in the 2006 safety review (Damwatch, 2006). Comment is provided on the worsened spillway toe erosion in the context of dam stability.

Sediment

The 2006 safety review (Damwatch, 2006) took into account the degree of sediment stored in the dam's reservoir (approx. 5m depth of sediment used) to calculate sediment loads on the dam. This is assumed not to have changed significantly in six years (relative to the lifetime of the dam) and has not been recalculated.

Flood loading

As stated in Section 6.2.1, the 1:100 AEP flood is approximately 60 m^3/s and is estimated to overtop the dam to a depth of 1.5m, which is approximately 2.4m above the normal reservoir level (Damwatch, 2006).

Earthquake loading

As for Korokoro Dam, a 1:500 AEP peak ground acceleration of 0.4g was considered appropriate for Woollen Mills dam and a pseudo-static analysis was used (Damwatch, 2006).

Dam section

A typical Woollen Mills dam section was analysed with two horizontal sliding planes considered, namely; in the rock foundation, in the concrete near the base of the dam.

Assessed factors of safety under all loading conditions

Table 6-2 is a summary of factors of safety against both NZSOLD and FERC recommended values, as determined in the 2006 safety review (Damwatch, 2006).

Table 6-2: *Woollen Mills Dam sliding factors of safety (Damwatch, 2006)*

Load case	In rock foundation	In concrete near base	NZSOLD min. (Low PIC)	FERC min. (Low PIC)
Typical section				
Normal	2.9	8.8	3.0	2.0
Flood	1.8	5.6	2.0	1.25
Earthquake	1.2	3.7	1.1	>1.0
Post-eq	1.8	4.9	2.0	1.25

Assessment of results

In the foundation three of the factors of safety are below the NZSOLD minimum values, but are well above the FERC values. This is acceptable on the basis that the analysis assumes a flat shear surface in the foundation, whereas in reality this surface is rough and provides much greater resistance to shear failure.

In the time since 2006 there has been further loss of foundation at the spillway toe through undermining. However, this does not affect the stability assessment on the basis that a typical section was used and the spillway chute continues to provide some buttressing support to the dam section at this location. It has been recommended earlier in this report that the spillway toe undermining be remediated to prevent dam instability.

Overall, it is considered that Woollen Mills Dam meets acceptable foundation sliding stability criteria (Damwatch, 2006).

6.3 Birchville Dam

6.3.1 Flood passage capacity

Tonkin and Taylor's stability review of Birchville Dam in 1989 assessed the 1:100 AEP flood peak as being approximately 20 m^3/s (Tonkin and Taylor, 1989). Their report also assessed spillway capacity and concluded that the 1:100 AEP flood would just be contained within the spillway and the PMF (45 m^3/s) would overtop the dam crest by approximately 300mm. Damwatch check calculations in 2006 concurred with this assessment.

It would be beneficial to perform special surveillance inspections during and after flood events to establish a documented record of the dam's performance under flood conditions. Depth of water over the spillway crest supported by photography and video footage will be particularly beneficial.

NZSOLD dam safety guidelines (NZSOLD, 2000) defines the flood capability acceptance criterion for a Low PIC dam is, "between 1:100 and 1:1,000 AEP (annual exceedance probability)", termed the 'inflow design flood' (IDF). Thus based on current standards, Birchville Dam has sufficient flood capacity in that it can safely pass a 1:100 AEP flood and would likely also pass a PMF without severe damage. This is supported by eighty three years of dam performing well under flood loading, with no indications of erosion or instability that could compromise the dam. Further, on a risk basis the dam has low incremental consequence of failure by flood overtopping. This is on the basis that it is unlikely there will be recreational users downstream during major flooding.

6.3.2 Dam stability assessment

There is not a rapid and simple method for assessing the structural stability of an arch dam. For complicated arch dams detailed computational analysis is used. However, the 2006 safety review made a reasonable assessment of loading for Birchville dam using simple ring theory. The method provided an estimate of the thrusts at the abutments from water and earthquake loads and in turn the thrust loads were used to assess the stability of the abutments. Experience shows that unstable wedges of rock in abutments are the most likely feature to endanger the safety of an arch dam. The 2006 safety review considered that the left abutment may have unfavourably aligned jointing that could cause a potential wedge failure mechanism. It then assumed that only friction in the joint could be relied on to resist thrust and that the joint would be pressurised by the reservoir (Damwatch, 2006).

The following residual factors of safety for sliding in foundation joints or seams recommended in the NZSOLD guidelines (NZSOLD, 2000) were used;

- Normal loading: 1.5
- Design flood: 1.3
- Maximum safety evaluation earthquake: 1.0

Sediment load

An approximate depth of sediment of 5m was used to calculate sediment loads on the dam. A recommendation has been made to undertake a bathymetric survey of the reservoir to confirm the present depth of sediment. *A significant increase in the depth of sediment against the arch dam would warrant a check of the sediment load and arch dam thrust loads.*

Flood loading

Flood loading on the dam was assumed to be 0.3m overtopping resulting in only a small increment in thrust loading.

Earthquake loading

As for Korokoro and Woollen Mills dams the earthquake loading used was 0.4g (peak ground acceleration). The increase in thrust was estimated to be approximately 80% of the normal thrust loads. This takes into account the seismic inertia force and from the self weight of the dam and the hydrodynamic water pressure.

Abutment assessed

The left abutment conditions were deemed to be the least favourable and hence this was the dominant feature that was assessed.

Assessed factors of safety under all loading conditions

Table 6-3 is a summary of the factors of safety against NZSOLD recommended values, as determined in the 2006 safety review (Damwatch, 2006).

Table 6-3: *Birchville left abutment residual sliding factors of safety (Damwatch, 2006)*

Load case	At 5m depth	At 10m depth	NZSOLD min.
Left abutment			
Normal	2.0	1.5	1.5
Flood	2.0	1.5	1.3
Earthquake	1.8	1.5	1.0

Assessment of results

The results give confidence in the stability of the left abutment under all loading conditions. The right abutment was not assessed specifically as its conditions are more favourable than the left abutment.

The two-dimensional assessment that was used is conservative in that three-dimensional effects would improve the abutment resistance (provided by plan area of failure wedge), and a level of cantilever action by the arch dam would result in lower thrust loads. The assessment is also conservative in that it considered factors of safety at point depths, whereas in reality the thrust and resistance applies to a depth range of dam to abutment contact, thus the overall factors of safety would be greater.

Importantly, under earthquake loading when arch dam thrusts are increased, there is expected to be minimal reduction in the stability of the potential abutment rock failure wedges. Arch dams and their abutments are inherently stable under both earthquake and flood loading and this is supported by their worldwide historical performance.

The 2006 safety review considered that, whilst some cracking of the dam concrete would be probable under earthquake shaking of 0.4g, dam failure by excessive cracking was not expected.

Overall, it is considered that Birchville Dam meets acceptable stability criteria (Damwatch, 2006).

6.4 Dam Safety Programme

6.4.1 NZSOLD dam safety guidelines

NZSOLD dam safety guidelines recommend that Low PIC dams be subject to the following dam safety and surveillance activities;

1. Regular (routine) inspection by the operator or owner of the general condition of the dam and the consistency of aspects such as identifiable seepage. Monthly to 4-monthly recommended.
2. Routine maintenance of dam surfaces and spillway paths.
3. Intermediate (e.g. 1 to 2 yearly) inspections by an appropriate technical adviser.
4. Comprehensive (10 yearly) inspection and review by an appropriate technical dam specialist. This may be met by augmenting the intermediate report at the 10 year interval.

For Low PIC dams a dam safety assurance programme (DSAP) and an emergency action plan (EAP) will not be mandatory for compliance with the new Building Act Dam Safety Scheme.

6.4.2 Assessment of Greater Wellington Regional Council dam safety activities

As introduced in Section 1.3.2, Greater Wellington Regional Council (GWRC) has ongoing dam safety activities for Korokoro, Woollen Mills and Birchville dams. Importantly, these include monthly routine inspections by maintenance rangers that have been trained in dam safety and surveillance. Routine dam safety inspections, in conjunction with appropriate technical support and review, are widely known to be the most effective method for monitoring dam safety.

Fundamentally, GWRC's dam safety activities include the items recommended in the NZSOLD guidelines and are being completed at the recommended, or in some cases greater, frequency.

Due to the impending regulations for the NZ Dam Safety Scheme, *from July 2014 GWRC will need to review the dam PIC's in accordance with the new Building Act methodology. If Medium PIC's are determined (currently Low), the current dam safety programme will require alignment with the dam safety assurance programme (DSAP) format specified in the regulations.*

7 DAM SAFETY STATUS

7.1 General

In this final section conclusions are drawn on dam safety status by bringing together the key components of the review. This is a process that forms an understanding of the dam structures and their failure modes; forms a hypothesis for their performance; assesses their performance through observation and analysis; and concludes with a determination of the overall safety status.

7.2 Korokoro Dam

7.2.1 Understanding of the dam and failure modes

Korokoro Dam is an 8m high concrete gravity dam that is one hundred and ten years old. There is limited design and construction information available, however indications are that the dam has been well built on competent foundation.

The dam does not have foundation (uplift) relief drains and is regularly overtopped by flood. The dam's reservoir is predominantly filled with silt with limited active storage remaining.

The dam's credible failure modes are;

FM1: Flood overtopping leading to dam instability and/or undermining
FM2: Piping failure in the dam foundation under normal loading
FM3: Earthquake leading to significant structural damage

7.2.2 A hypothesis for performance

A dam of this height and construction is fairly robust and is expected to perform well, however instability through foundation uplift and/or sliding needs to be the primary concern. Given the absence of foundation drains it has to be assumed that full reservoir pressure exists in the foundation, decreasing to downstream pool level at the dam toe.

The dam and foundation are expected to be relatively resistant to erosion through piping and overtopping.

7.2.3 Assessment of performance

The dam is performing well except for significant seepage at the toe near to the spillway. It appears that the rate of seepage has increased with time and may be suggestive of erosion of the dam's foundation. Whilst this would likely not deteriorate rapidly nor lead to a rapid failure, a recommendation has been made to undertake regular monitoring of the seepage rate to characterise its behaviour with time and serve as an early warning of foundation piping.

There are no indications of gross movement of the dam or signs of instability in the foundation and abutments.

Whilst the dam has obviously been overtopped on a regular basis it has performed well. The most significant erosion is of what was either placed fill or natural ground against the downstream face of the dam. A recommendation has been made to monitor this and if undermining occurs then remediation will be required. In the meantime it is beneficial to see and measure the seepage that is emerging at this location.

The dam meets acceptability criteria for stability under all loading conditions.

7.2.4 Dam safety status

The dam is in relatively good condition and is performing well for its age and the prevalent dam technology of the day. There are no indications of any of its failure modes developing, although better understanding of the toe seepage is required to confirm that this is not related to foundation piping.

Korokoro Dam has a low likelihood of failure and the best and most appropriate risk management is to perform regular routine surveillance inspections including the monitoring of key features, i.e. toe seepage and erosion. The value of this is improved significantly by keeping vegetation short on the toe and downstream abutment contacts.

7.3 Woollen Mills Dam

7.3.1 Understanding of the dam and failure modes

Woollen Mills Dam is a 6m high concrete gravity dam that is one hundred and ten years old. There is limited design and construction information available, however indications are that the dam has been well built on competent foundation.

The dam does not have foundation (uplift) relief drains and is regularly overtopped by flood. The dam's reservoir is predominantly filled with gravel with almost no active storage remaining.

The dam's credible failure modes are;

FM1: Flood overtopping leading to dam instability and/or undermining
FM2: Piping failure in the dam foundation under normal loading
FM3: Earthquake leading to significant structural damage

7.3.2 A hypothesis for performance

A dam of this height and construction is fairly robust and is expected to perform well, however instability through foundation uplift and/or sliding needs to be the primary concern. Given the absence of foundation drains it has to be assumed that full reservoir pressure exists in the foundation, decreasing to downstream pool level at the dam toe.

The dam and foundation are expected to be relatively resistant to erosion through piping and overtopping.

7.3.3 Assessment of performance

The dam is performing well except for major erosion beneath and at the toe of the spillway. This appears to have been initiated by rock-fall damage to the spillway chute and subsequent undermining by normal and flood flows. The extent of undermining has worsened significantly over the last six years and if it continues will eventually lead to the dam's stability being compromised through removal of base and toe support. A recommendation has been made to undertake regular monitoring of the extent of undermining with time and remediation should be planned for in the near future.

There are no indications of gross movement of the dam or signs of other instability in the foundation and abutments.

The dam meets acceptability criteria for stability under all loading conditions.

7.3.4 Dam safety status

The dam is in good condition and performing well with the exception of erosion beneath the spillway. There are currently no indications of any of its failure modes developing. However, continued spillway undermining will eventually lead to dam instability through loss of base and toe support.

Woollen Mills Dam has a low to moderate likelihood of failure due to the worsening condition of the spillway. The most appropriate immediate risk management is to perform regular routine *surveillance inspections including the monitoring of its key feature, i.e. spillway toe erosion. Remediation of the spillway and its toe will return the dam to its intended function and reduce the likelihood of failure to low. This should be done as a matter of priority.*

7.4 Birchville Dam

7.4.1 Understanding of the dam and failure modes

Birchville Dam is a 15m high single curvature arch dam that is eighty three years old. There is limited design and construction information available, however indications are that the dam has been well built on competent foundation.

The dam incorporates a free overflow spillway that can pass a flood of approximately 1:100 AEP, with further capacity possible through overtopping of the dam. The dam's reservoir contains a large amount of accumulated sediment.

The left abutment is re-entrant and exhibits adverse jointing and weathering.

The dam's credible failure modes are;

FM1: Earthquake leading to significant structural damage and/or left abutment failure
FM2: Left abutment failure under normal loading

7.4.2 A hypothesis for performance

Arch dams are typically robust and perform very well provided that their abutments and foundation continue to support dam thrust loads.

Birchville Dam has been well constructed on what appear to be competent abutments and foundation. There has been past concern raised over the left abutment stability, however there is no indication of deep seated instability or seepage.

7.4.3 Assessment of performance

The dam is performing very well with no indication of stress within the dam or its abutments. There is no discernible leakage or seepage, although the toe is covered with large riprap meaning that seepage in this area would not be visible. A recommendation has been made to undertake regular monitoring of the abutments, with particular attention to the left abutment to ensure that there are no indications of instability.

There are no indications of gross movement of the dam or signs of other instability in the foundation and abutments.

The dam meets acceptability criteria for stability under all loading conditions using basic analysis methods. A more complex analysis is not necessary or warranted given the dam's Low PIC and good performance over eighty three years.

7.4.4 Dam safety status

The dam and its abutments are in good condition and performing well. There are currently no indications of any of its failure modes developing, however ongoing monitoring of its abutments is important.

Birchville Dam has a low likelihood of failure. The best and most appropriate risk management is to perform regular routine surveillance inspections including the monitoring of key features, i.e. the left and right abutments.

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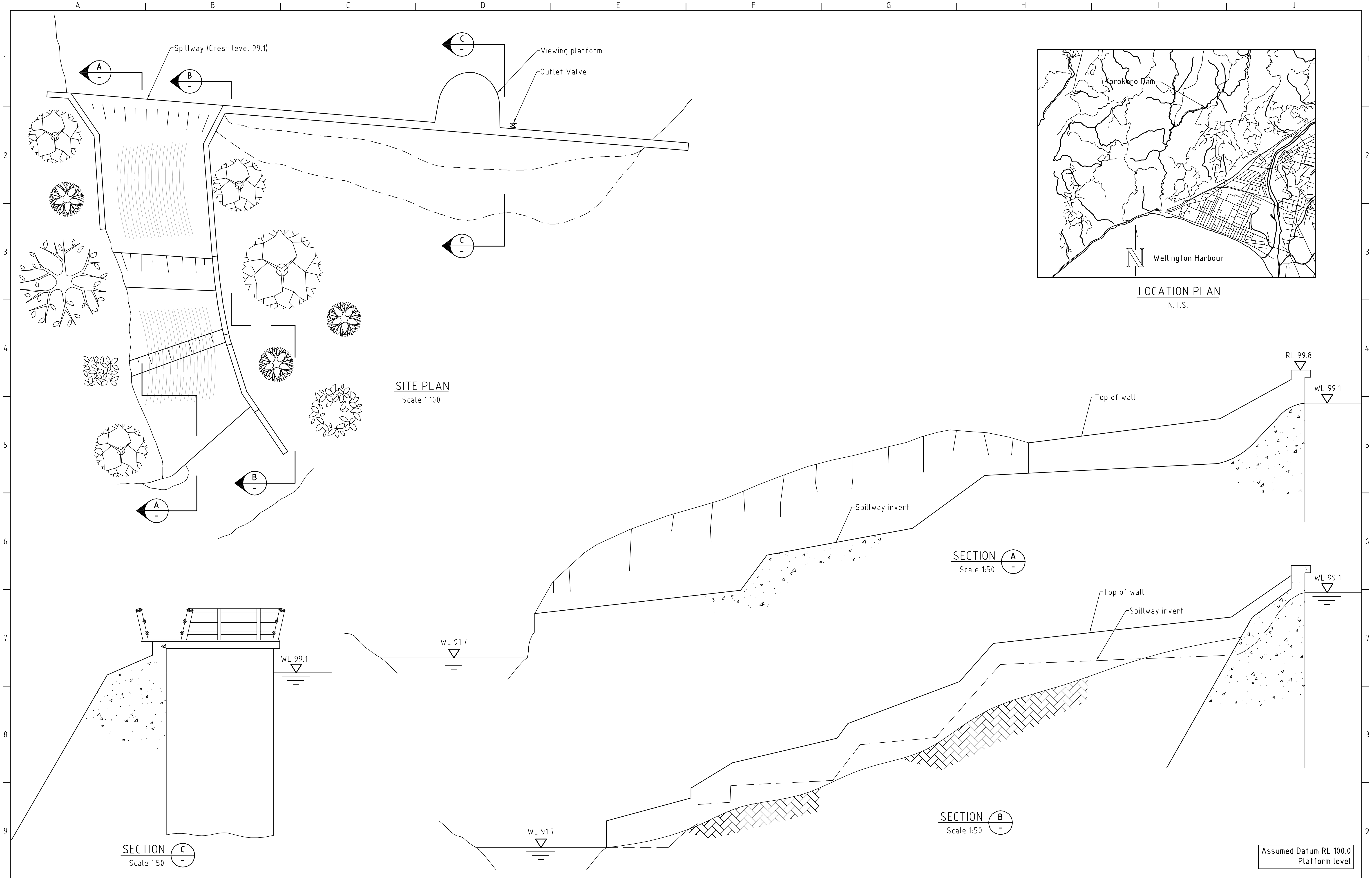
APPENDIX A: DRAWINGS



AMENDMENT	DWN	CHK	DATE	AMENDMENT	DWN	CHK	DATE	AMENDMENT	DWN	CHK	DATE

THE REGIONAL COUNCIL

UTILITY SERVICES DIVISION
ENGINEERING CONSULTANCY GROUP

SURVEYED	R. DALE / R. ASPDEN	08/01
DRAWN	J. FORSYTH	09/01
CHECKED		
RECOMMENDED	J. DUGGAN	11/01
APPROVED	J. MORRISON	11/01

KOROKORO DAM

SITE PLAN & CROSS SECTIONS

ISSUE –
M.F.
SCALE 1:50 1:100
DRAWING Nº A1-10030/01-RL
FILE Nº
SHEET Nº 1 OF 1 SHEETS.

A1-10030-01-RL.DWG

— PLAN —
Centre peg
Chord 133'6"
90 ft radius to face of dam
Stream
To be flumed during construction
12" main
Valve chamber
Intake chamber
Rock
Face
River Flat
Spillway
Line of Bores
31'0"
70'0"
Check peg for line
Bye pass 15" x 7" cast iron pipe, set in dam, with 15" x 20" mannesmann pipe on down stream side 15" SV fitted between pipes & covered with … arises to suit ground level. Cast iron surface box & lid set in c.o.n. block
Arc 151 Ft
Crest of spillway RL 234·0
Sec "A"
Sec "B"
Sec "C"
Sec "D"
Bore
Bye pass
River Flat
16'6"
31 Ft
24 Ft
19 Ft
53 Ft
— ELEVATION OF FACE OF DAM —
Scale:- 1/16" = 1'
SEC "A"
SEC "B"
SEC. "C"
SEC "D"
Crest
level
R.L. 234·00
W.L. 204·0
Spillway to be made as directed with specially selected rocks, minimum size 1 cwt. each
— CROSS SECTIONS —
Scale:- 1/8" = 1'
Crest R.L. 234·00
Parapet
BACK VIEW - SPILLWAY
Parapet 2'6" wide
Spillway
— PART PLAN —
Scale 1/8" = 1'
(4) 3/4" horiz bars to extend full length of dam, to break joint minimum overlap 3 ft. No 6 g double wire stirrups at 12" cens
— SPILLWAY —
— PARAPET —
— SECTIONS - TOP OF DAM —
1/4" = 1'
8" rag bolts
(2) circular rings 3/8" diam
3/8" ring 3/8" x 4 ft
Three for each standard at 12 centres
Control valves to be supplied by Council & fitted by Contractor
Ships davit, supplied by council, to be erected as shown
Manhole Cover
Frame 4 bars 3/8" x 2·3
LADDER RUNGS
(supplied by contractor)
1" diam
diameter 5'0"
Ladder
Filter cage
3" cone cover
12" pipe
12" valve
Davit guides 9" tubes of 1/4" m.s. plate, bell mouthed, set in concrete. Bottom tube to rest on m.s. plate 12" x 12" x 1/2"
WALL OF INTAKE CHAMBER
3/8" horiz rings, 9" cens.
3/8" x 23 ft vertical at 12" cens
6'0"
8'0"
7'3"
3" conc floor
Valve chamber cover 2" heart totara on 4" x 3" ht frames, to be built in three sections
— SECTION OF INTAKE —
Scale 1/4" = 1'
Davit
Ladder
Rock to be cut back 4'6" clear of chamber.
— PLAN OF INTAKE —
Scale 1/4" = 1'
Fence to be standard 7 wire. Wire No 8 g, 6 posts (totara) per chain. Approx length 40 chs. Posts 6'6" long, set 3 ft in ground. Two totara battens between each post.
New main to junction with old main. Lay 8" x 3" & rigid collar, & 12" to 8" reducer & valve … on new main.
THINK
Old Tank
Shed
Stream
Old intake
Foot of hills
Two 1/16 bends & One 1/32 bend supplied. Contractor to bend other pipes to suit pipe line where required
— PLAN OF PIPE LINE —
Horiz. Scale 1" = 60 Ft
Vertical " 1" = 20 "
— SECTION ALONG 12" PIPE LINE —
Cut
RL invert
RL ground
RL peg
Radius 90'
Centre peg
Dam
Intake
Fence to be pegged by Engineer before erection
New fence
30 Ft contour
Narrow Gully
New Fence
Reservoir site to be cleared to new fence
Stream
PLAN OF RESERVOIR
Scale 1" = 1 ch.
482-15
J.H.A.
1951
UPPER HUTT BOROUGH COUNCIL
WATER SUPPLY HEADWORKS
NEW 30 FT. DAM & 12" PIPE LINE
H.F. Toogood
Consulting Engineer
234 The Terrace, Wellington
File No C/68/1
6-3-30

APPENDIX B: PHOTOS FROM 2012 SAFETY REVIEW SITE INSPECTIONS

KOROKORO DAM

Photo 1: *Korokoro Dam looking upstream to head of reservoir. Spillway entrance in middle left.*

Photo 2: *Korokoro Dam crest and left abutment. Note vegetation on dam downstream face and groin.*

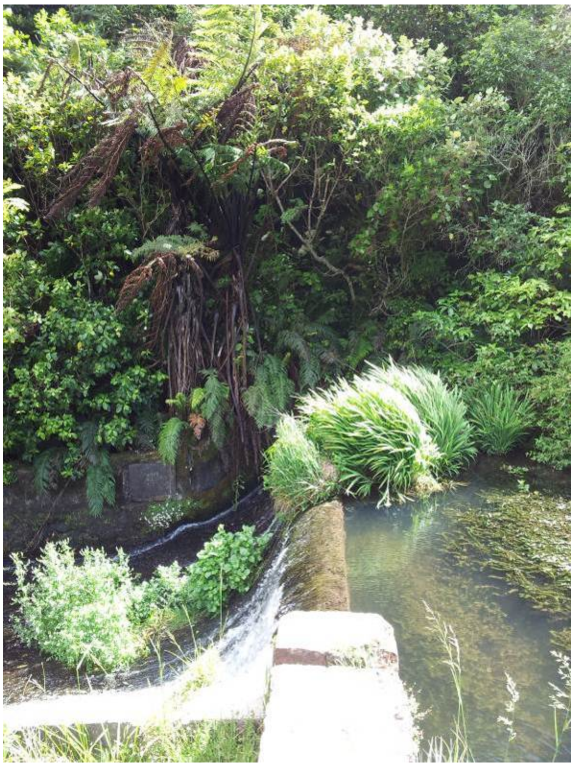


Photo 3: *Korokoro Dam spillway crest and right abutment. Note vegetation in spillway chute and on abutment.*

Photo 4: *Spillway crest and right abutment contact.*

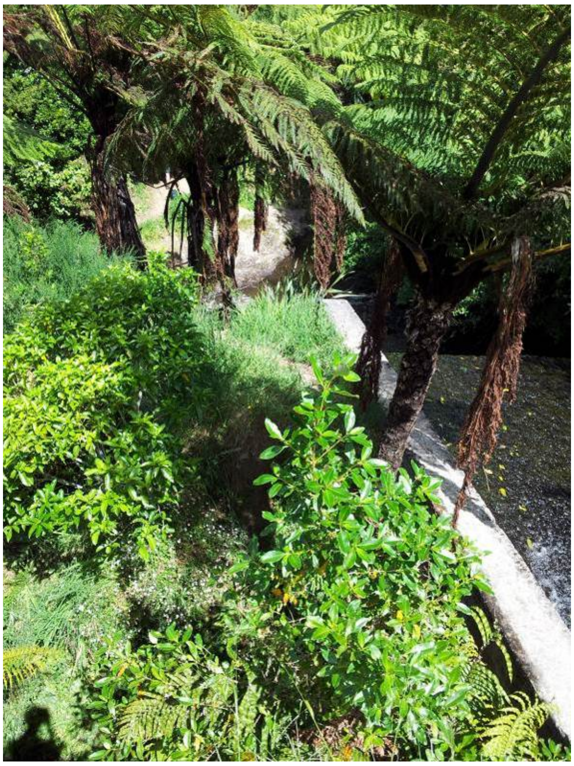


Photo 5: *Looking down spillway chute and adjacent to left wall.*

Photo 6: *Seepage emerging from poor quality concrete beneath spillway chute.*

Photo 7: *Seepage emerging from poor quality concrete beneath spillway chute.*

Photo 8: *Seepage flow from beneath spillway chute and dam toe approximately 12 litres per min.*

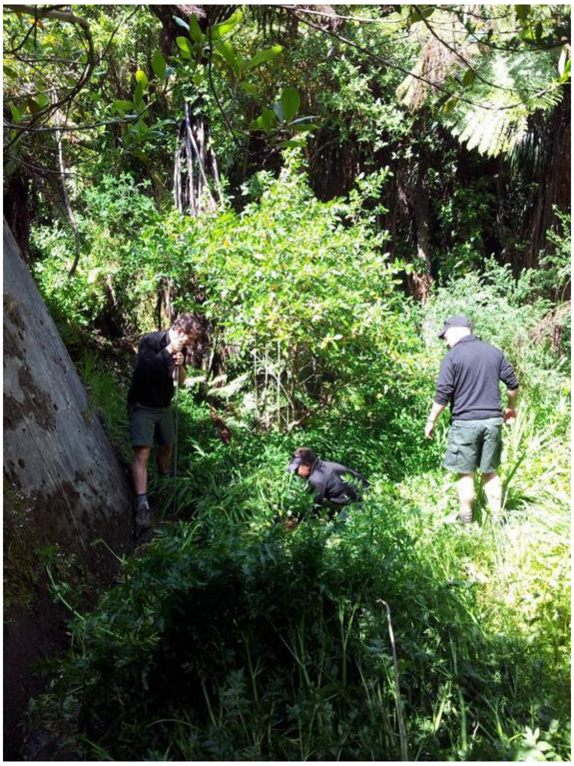


Photo 9: *Vegetation cover on dam toe looking toward left abutment.*

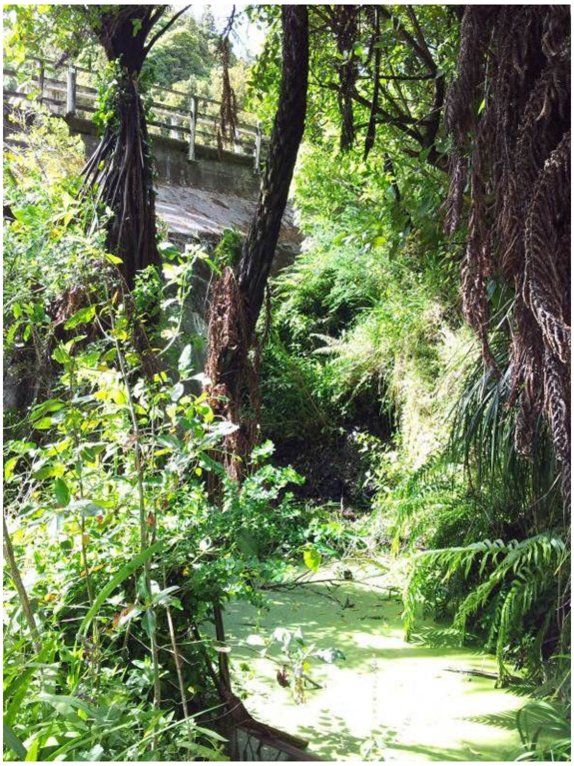


Photo 10: *Water ponded at left side toe.*

Photo 11: *Spillway chute, plunge pool and downstream toe. Outlet pipe amongst people standing right of middle.*

WOOLLEN MILLS DAM

Photo 12: *Typical valley profile downstream of Korokoro Dam and upstream of Woollen Mills Dam. Photo taken in downstream direction.*

Photo 13: *Woollen Mills dam and reservoir looking from upstream left abutment toward downstream right abutment. Note that the impounded reservoir is very small.*

Photo 14: *Woollen Mills dam upstream face, spillway crest, and right abutment. Note shallow depth to impounded gravels (less than 0.5m).*

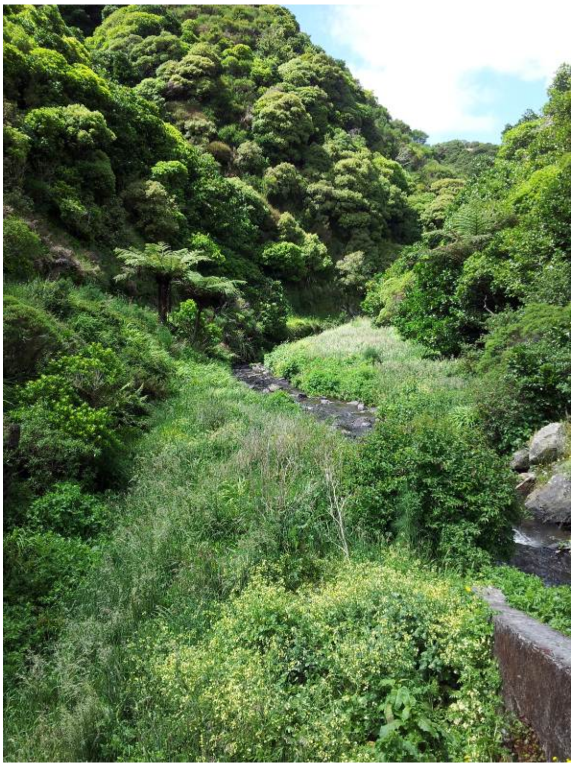


Photo 15: *Looking downstream between left abutment and spillway chute wall.*

Photo 16: *Woollen Mills spillway chute and wall looking downstream. Note rock-fall upper right in photo.*

Photo 17: *Woollen Mills dam crest, spillway chute wall and right abutment. Note rock-fall middle left in photo.*

Photo 18: *Woollen Mills spillway crest and chute. Crack extends down spillway wall, across dam crest wall and down upstream face toward spillway crest.*

Photo 19: *Spillway chute looking toward plunge pool. Note chute damage and overhanging vegetation.*

Photo 20: *Spillway chute and chute wall. Dam toe obscured by vegetation.*

Photo 21: *Dam downstream face and left abutment contact.*

Photo 22: *Woollen Mills spillway and dam looking upstream from toe of rock-fall. Note damaged spillway chute and major undermining. Probing with a long stick indicated a very large cavity under the remaining chute slab in centre photo. Cavity horizontal extent approx. 1.5m and depth approx. 2m.*

Photo 23: *Woollen Mills spillway and dam looking upstream from downstream left abutment. Note damaged spillway chute and visible extent of upstream regression.*

BIRCHVILLE DAM

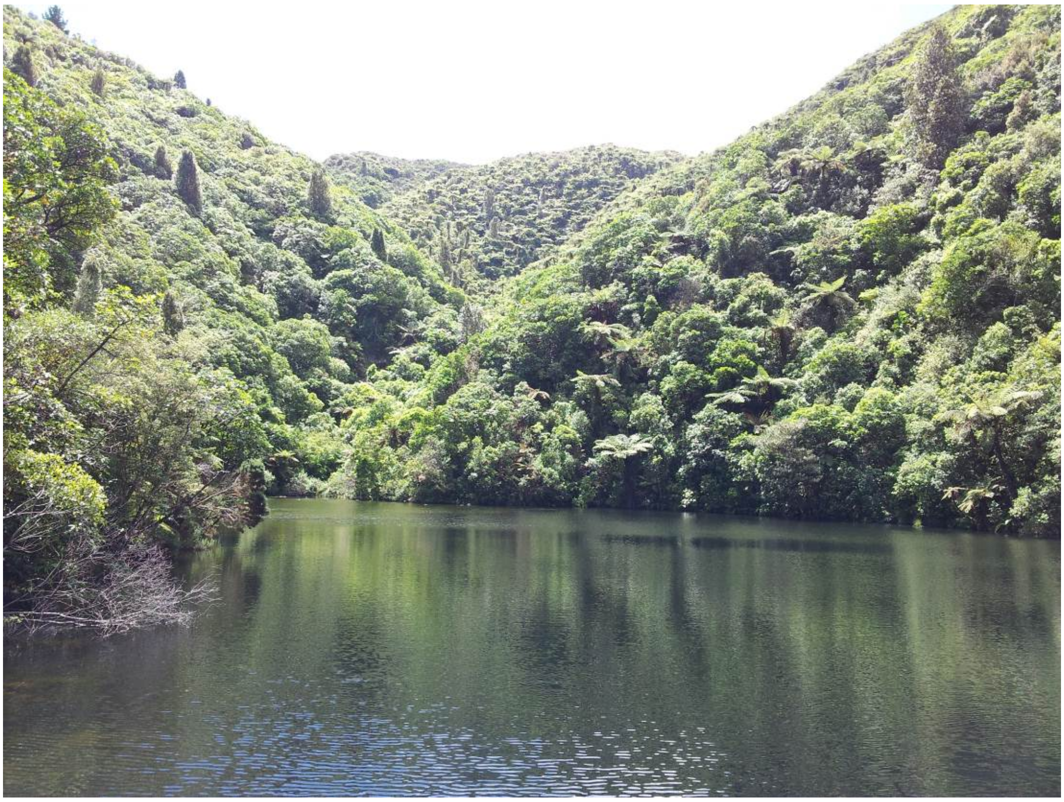


Photo 24: *Birchville Dam reservoir*

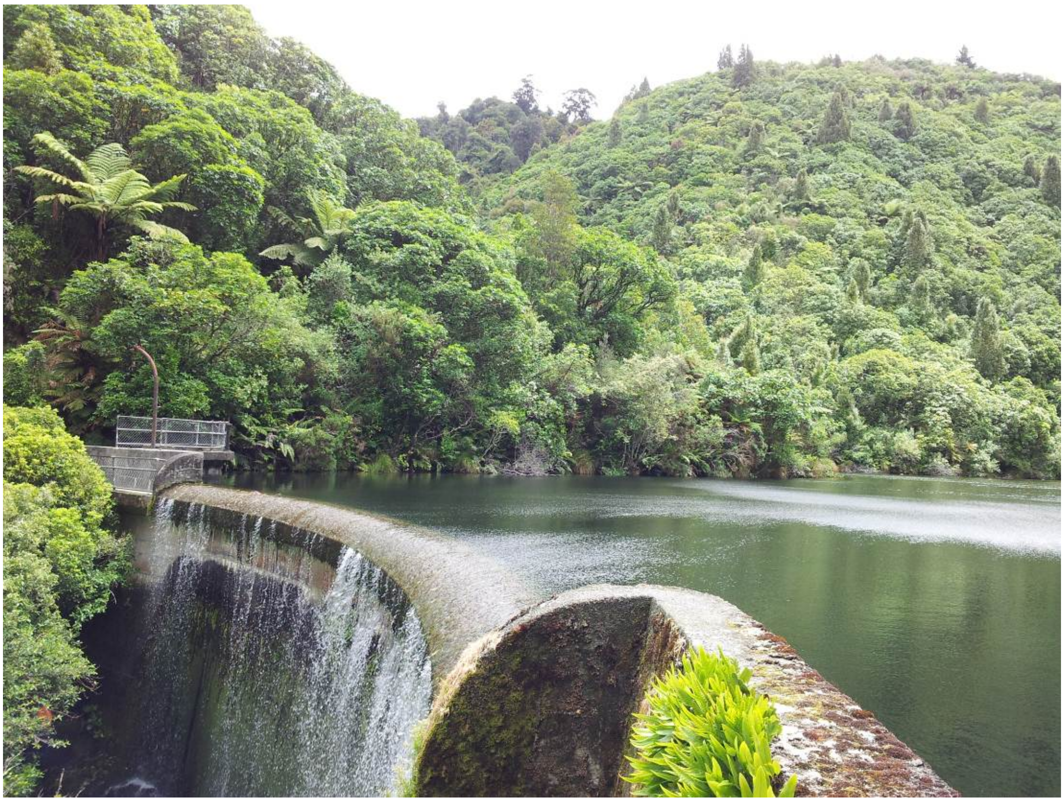


Photo 25: *Birchville Dam spillway crest and right bank reservoir*

Photo 26: *Birchville Dam spillway crest and toe from left abutment.*

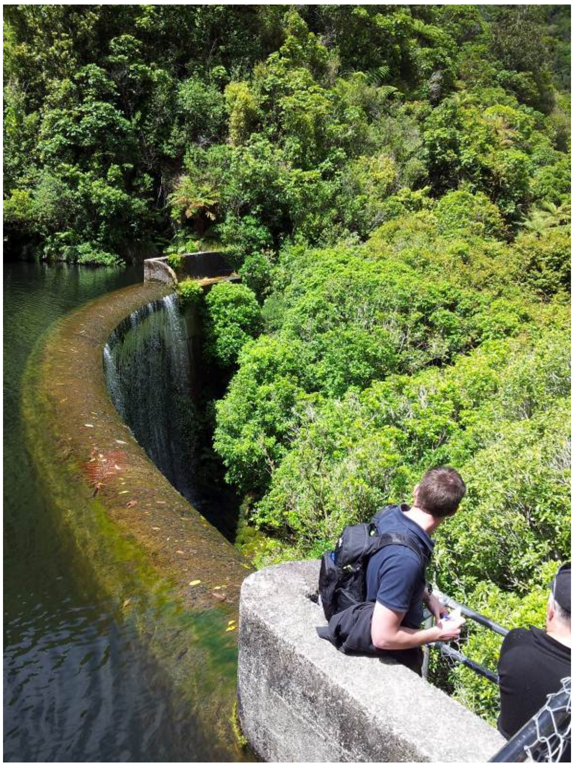


Photo 27: *Spillway crest and downstream left abutment.*

Photo 28: *Dam contact with right abutment*

Photo 29: *Downstream contact with right abutment.*

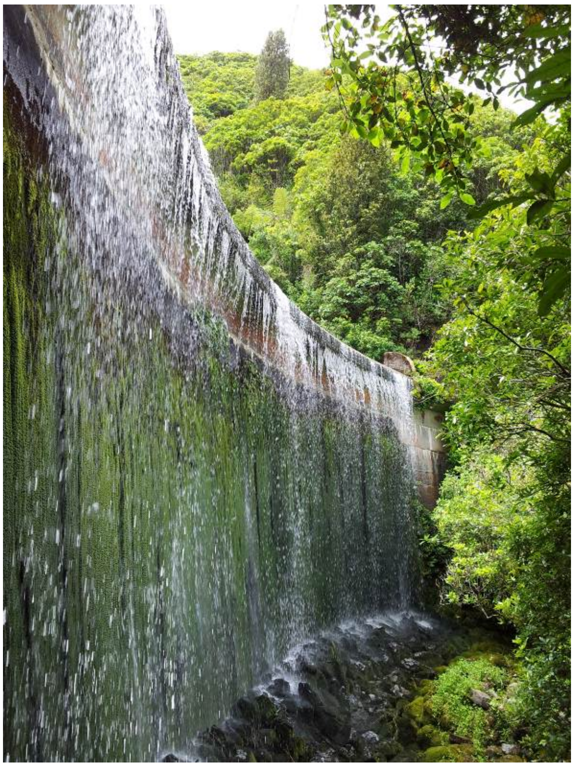


Photo 30: *Downstream toe looking towards left abutment.*

Photo 31: *Downstream contact with left abutment.*

APPENDIX C: ROUTINE INSPECTION CHECKLISTS

Korokoro Dam Routine Inspection Checklist

Summary information:

Type	Concrete Gravity Dam
Height	8m
Storage volume	Approx 30,000 m3 (estimated in 2006).
Year constructed	1903
Key failure modes	FM1: Flood overtopping leading to uplift and/or undermining instability FM2: Piping failure in the dam foundation under normal loading FM3: Earthquake leading to significant structural damage
Special feature	**Significant seepage at toe adjacent to spillway (FM2). Has increased since safety review in 2006. Continue to monitor with numerical measurement of seepage flow at collection point. Also monitor total dam seepage adjacent to spillway plunge pool.**

Inspected by:

Date and time of inspection:

Weather:

Inspection checklist:

Line	Identifier	Item	Enter; 0 for Ok/Normal, 1 for Changed, or numerical value if measurable.	Comment if Changed or other
1	KOROLAKE	Lake level in metres below (-) or above (+) spillway invert at crest		
2	KORO-RES	Reservoir rim stability		
3	KORO-RABT	Right abutment stability		
4	KORO-LABT	Left abutment stability		
5	KORO-CREST	Crest of dam		
6	KORO-SPILL	Spillway		
7	KORO-PLUNGE	Spillway plunge pool undermining.		
8	KORO-DSRABT	Downstream right abutment stability and dryness		
9	**KOROFLOW1**	**Total dam seepage measured adjacent to spillway plunge pool. Time (seconds) to fill 1 litre.**		
10	**KOROFLOW2**	**Seepage flow from dam toe adjacent to spillway. Time (seconds) to fill 1 litre.**		
11	KORO-TOE	Dam toe dryness and soundness		
12	KORO-DSLABT	Downstream left abutment stability and dryness		

Woollen Mills Dam Routine Inspection Checklist

Summary information:

Type	Concrete Gravity Dam
Height	6m
Storage volume	Approx <3,000 m3 (estimated in 2006).
Year constructed	1903
Key failure modes	FM1: Flood overtopping leading to uplift and/or undermining instability FM2: Piping failure in the dam foundation under normal loading FM3: Earthquake leading to significant structural damage
Special feature	**Extensive undermining of lower spillway chute (FM1 & FM2). Continue to monitor for gross worsening. Likely to require remediation in future.**

Inspected by:

Date and time of inspection:

Weather:

Inspection checklist:

Line	Identifier	Item	Enter; 0 for Ok/Normal, 1 for Changed, or numerical value if measurable.	Comment if Changed or other
1	WOOLLAKE	Lake level in metres below (-) or above (+) spillway invert at crest		
2	WOOL-RES	Reservoir rim stability		
3	WOOL-RABT	Right abutment stability		
4	WOOL-LABT	Left abutment stability		
5	WOOL-CREST	Crest of dam		
6	WOOL-SPILL	Spillway		
7	**WOOL-PLUNGE**	**Spillway plunge pool. Check for worsening undermining.**		
8	WOOL-DSRABT	Downstream right abutment stability and dryness		
9	WOOL-TOE	Dam toe dryness and soundness		
10	WOOL-DSLABT	Downstream left abutment stability and dryness		

Birchville Dam Routine Inspection Checklist

Summary information:

Type	Concrete Arch
Height	15m
Storage volume	Approx 20,000 m3
Year constructed	1930
Key failure modes	FM1: Abutment failure under normal loading FM2: Earthquake leading to significant structural damage and/or abutment failure
Special feature	**Left abutment to be monitored for signs of seepage and instability (FM1 & FM2). Surface weathering (e.g. scree) is ok.**

Inspected by:

Date and time of inspection:

Weather:

Inspection checklist:

Line	Identifier	Item	Enter; 0 for Ok/Normal, 1 for Changed, or numerical value if measurable.	Comment if Changed or other
1	BRCHLAKE	Lake level in metres below (-) or above (+) spillway invert at crest		
2	BRCH-RES	Reservoir rim stability		
3	BRCH-RABT	Right abutment stability		
4	**BRCH-LABT**	**Left abutment stability**		
5	BRCH-CREST	Crest of dam and spillway		
6	BRCH-DSRABT	Downstream right abutment stability and dryness		
7	BRCH-TOE	Dam toe soundness (e.g. erosion protection intact)		
8	**BRCH-DSLABT**	**Downstream left abutment stability and dryness.**		