DAMS AND EARTHQUAKES IN NEW ZEALAND

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ABSTRACT

New Zealand is located in the area of active seismicity on the boundary between the Australian and Pacific plates. Most dams in New Zealand are in the close proximity to active faults and some dams have been built across active faults.

The paper describes how earthquake considerations have been taken into account by dam owners in New Zealand over the last 25 years including:

- the development of modern earthquake engineering practices for dams,
- the provision of an 'earthquake' joint in a large concrete gravity dam,
- a re-assessment of lessons learnt from the near-failure of a rockfill dam from earthquake effects in 1987.
- the upgrading of a dam for a potential 3m of movement on an active fault in the foundation,
- the assessment of an embankment dam for an active fault in the foundation and the consequential strengthening works,
- the seismic safety assessment of a proposed dam and reservoir in a highly seismic area,
- the measures taken by dam owners to ensure post-earthquake availability of spillway gate systems and outlet facilities and,
- emergency planning.

Keywords: earthquakes, faults, seismic, dams, dam safety, spillway gates, emergency planning.

INTRODUCTION

New Zealand is located in the area of active seismicity on the boundary between the Australian and Pacific plates. Most dams in New Zealand are in the close proximity to active faults and some dams have been built across active faults. New Zealand has experienced a number of damaging earthquakes in its recent history. Earthquake engineering and hazard management is an accepted element in all infrastructure development and management.

About 25 years ago, in the early 1980's, the application of plate tectonic concepts was re-shaping the way faults and earthquakes were considered in New Zealand. These concepts were applied to the design of a large concrete gravity dam, the Clyde Dam, in the South Island of New Zealand. In 1987, the Matahina Dam located in the North Island of New Zealand experienced strong shaking from a nearby Richter magnitude 6.3 earthquake. The dam was damaged and subsequently required major repairs.

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These two dams were focal points in the development of current earthquake engineering and safety management practices for dams in New Zealand. The Clyde Dam was instrumental in introducing modern earthquake engineering dam design concepts including the issue of fault displacement in a dam foundation. The Matahina Dam earthquake damage experience highlighted the seismic safety assessment needs for existing dams including spillway gate reliability issues and important elements of dam safety management, particularly emergency planning.

Following an outline of New Zealand's seismicity, these two important case histories are described together with a description of current New Zealand practice and case history examples for the evaluation of the seismic safety for existing dams including foundation fault displacement, improvements to the reliability of spillway systems and emergency planning for earthquakes.

NEW ZEALAND SEISMICITY

New Zealand is located along the boundary of the Pacific and Australian plates (Figure 1). Along the eastern coast of the North Island the Pacific plate is being subducted under the Australian plate. In the south of the South Island the situation is reversed, with the Australian plate being subducted under the Pacific plate.

Displacement across the boundary varies as shown in Figure 1. Most of the countries deformation occurs on strike-slip and reverse faults in the axial tectonic belt (Figure 2). The most prominent of the faults in this belt is the Alpine Fault in the South Island. The Taupo Volcanic Zone (Figure 2) contains a number of large dams and is characterised by normal faults. The most active faults in New Zealand have a recurrence interval of a few hundred years. The epicentres of large shallow earthquakes in NZ between 1840 and 2000 are shown in Figure 3.

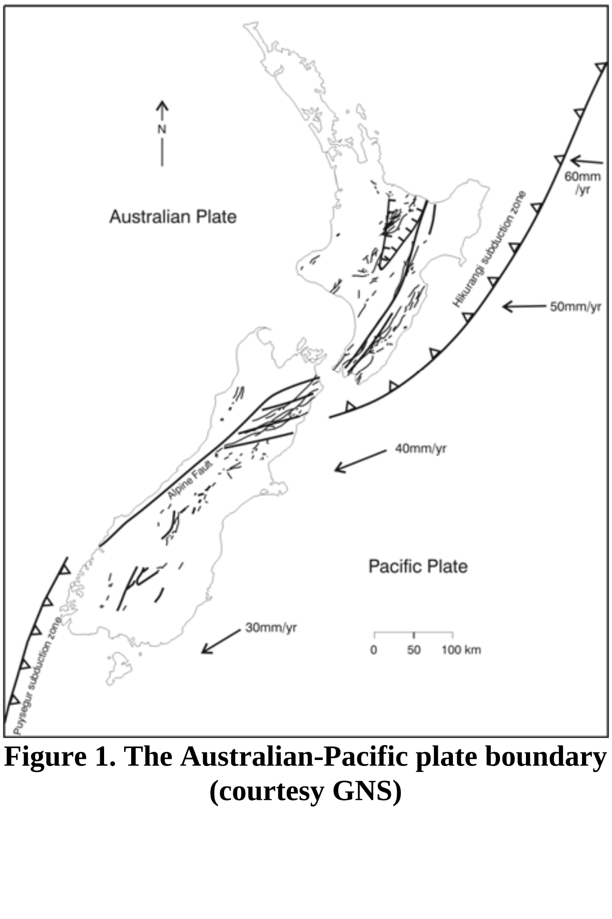

Figure 1. The Australian-Pacific plate boundary (courtesy GNS)

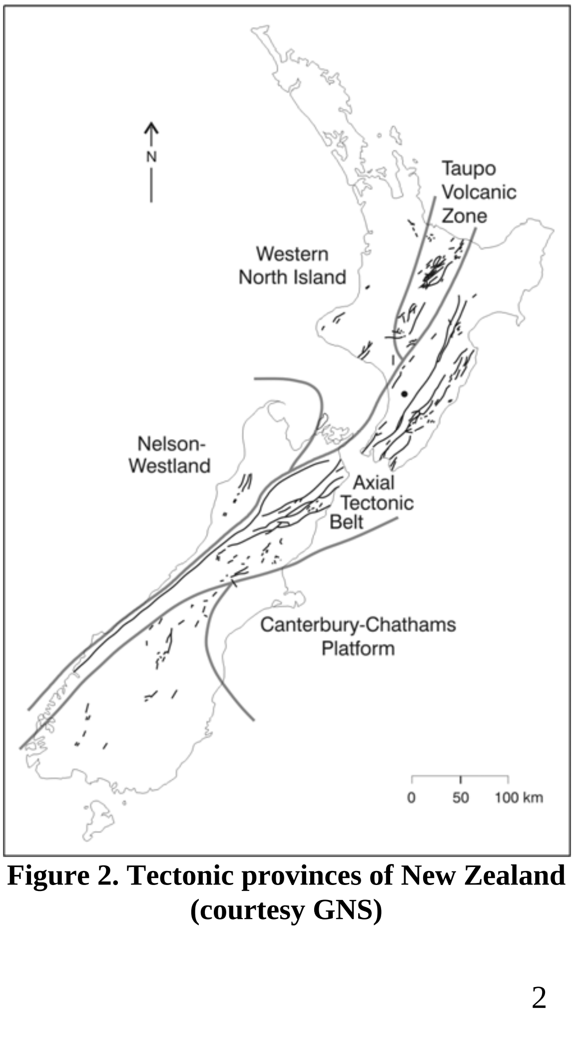

Figure 2. Tectonic provinces of New Zealand (courtesy GNS)

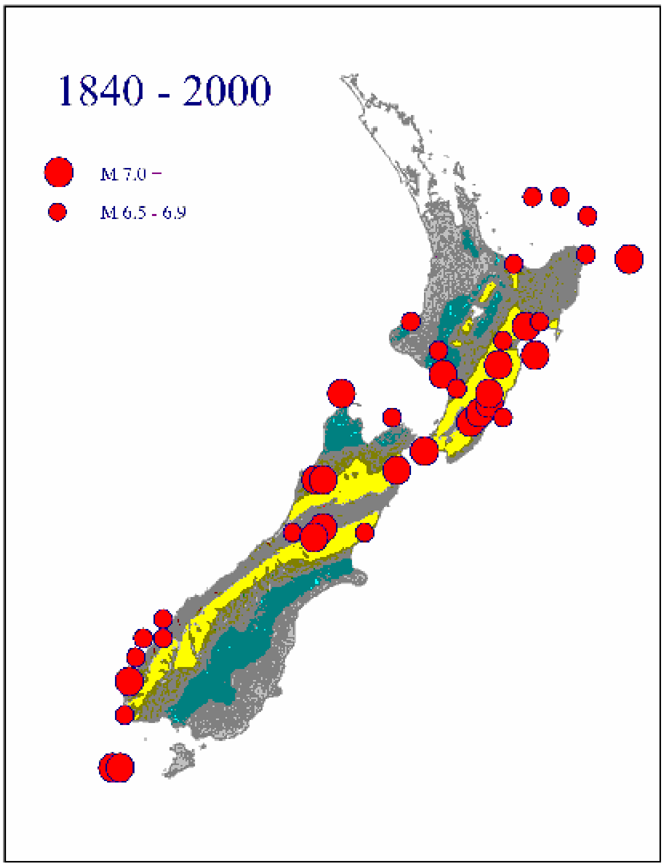

Figure 3. Epicentres of large shallow earthquakes since 1840 (courtesy GNS)

SEISMIC DESIGN OF THE CLYDE DAM

The Clyde Dam is located on the Clutha River in the South Island of New Zealand. It is a 100m high concrete gravity dam (Figure 4) for hydropower generation. A feature of the dam is the provision of a slip joint to allow up to 2m of movement on the plane of a fault passing through the dam foundation (Hatton et al., 1987a, 1987b).

A comprehensive seismotectonic hazard evaluation was carried out for the dam. The evaluation was based on US practices of that time (1982-83) and was the first study of its kind applied to a dam in New Zealand. The study considered seismic sources in the region, the magnitude and recurrence of the earthquakes associated with them and the characteristics of the ground motions that would result at the dam site. The study also considered the displacements that might be experienced at the site

An active fault, the Dunstan Fault passes within 3 km of the dam site. A major rupture of the fault was considered capable of generating a Richter magnitude 7 to 7.5 earthquake. The average recurrence interval was estimated to be about 8000 years based on evidence from fault trenching. The seismotectonic hazard evaluation identified the Dunstan Fault as the source of the controlling maximum credible earthquake and the maximum design earthquake for the dam is based on this event.

The study also identified that a major rupture of the Dunstan Fault could induce up to 200mm of sympathetic movement on a fault in the river channel passing through the dam site. The dam design was modified to provide a slip joint along the fault. The joint was designed to accommodate 2m of strike-slip movement and 1m of dip-slip movement (Figure 5). To keep the joint water tight during normal operation and to limit leakage in the event of fault displacement, a wedge plug has been provided at the upstream face. The joint continues down into a cutoff shaft excavated into the fault below the dam. A blanket of zoned backfill is provided upstream of the dam to control seepage and prevent washout of the fault material should movement occur.

The design criteria for the dam were that it should withstand the maximum design earthquake without any sudden or uncontrolled release of the reservoir and the operating basis earthquake, a lesser event with a return period of 100 years, with no damage. These criteria were consistent with dam design practices in the USA at that time. The design value peak ground accelerations for the maximum design and operating basis earthquakes were 0.54g and 0.19g respectively. A simplified probabilistic seismic hazard assessment was carried out to determine the operating basis earthquake peak ground accelerations.

Figure 4a. Clyde Dam (courtesy Contact Energy)

Figure 4b. Clyde Dam slip joint (courtesy Contact Energy)

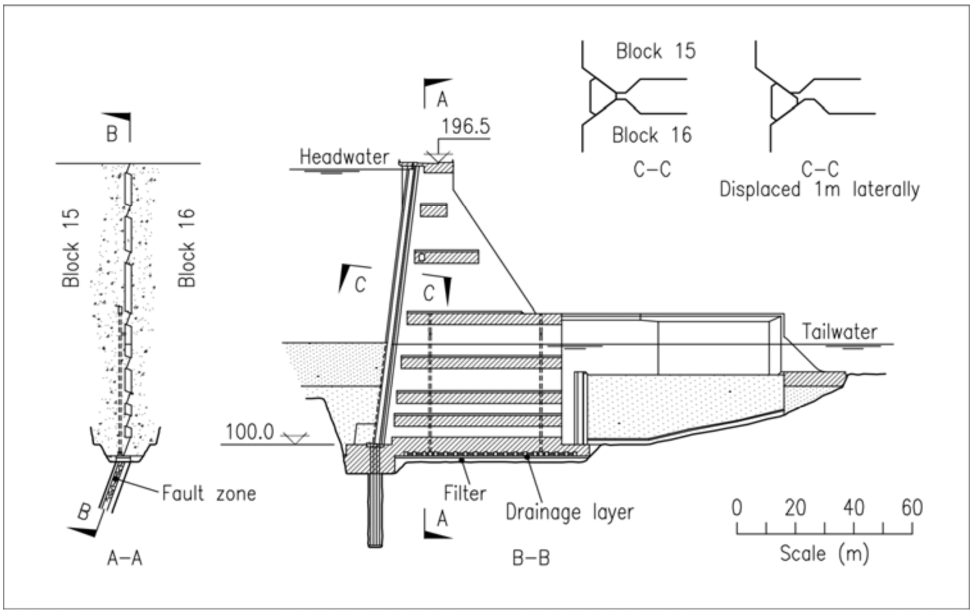

Figure 5. Clyde Dam slip joint (after Hatton et al., 1987b)

To ensure that the spillway remained operable, the bridge deck was designed as a continuous structure over the four spillway gates and the ends were keyed into the dam. Based on a linear elastic dynamic analysis, the spillway gates and their equipment were designed for 2g accelerations to allow for amplification effects at the dam crest.

LESSONS FROM THE EARTHQUAKE DAMAGE AT THE MATAHINA DAM IN 1987

Matahina Dam is located on the eastern boundary of the Taupo Volcanic Zone (Figure 2). It is an 80m high rockfill dam (Figure 6) used for hydropower generation. The dam experienced a serious internal erosion incident (not earthquake related) on the right abutment during the initial lake filling in 1967.

On 2 March 1987 it was subject to peak ground accelerations of 0.33g from a nearby magnitude (M_L) 6.3 earthquake. Subsequent investigation (Gillon, 1988) revealed internal erosion damage at the left abutment and in December 1987, while the damage was being assessed, an erosion cavity formed on the crest above the damaged area. Following an emergency drawdown of the reservoir, excavation revealed serious damage to the core (Figure 7) and major repairs were undertaken (Gillon and Newton, 1991). This internal erosion incident at the dam has recently been re-evaluated (Gillon, 2007 in press).

There were in effect two emergency situations associated with the earthquake. The first was in March 1987 at the time of the earthquake and the second was following the discovery of the erosion cavity in December 1987, nine months later. The details of these two events and the lessons learnt are described in Gillon, 1996. The lessons relating to dam safety programmes and emergency planning are as follows:

- Emergency events raise questions concerning the likely performance of the dam and the various outlet structures. The availability of design, construction and performance information together with engineering personnel familiar with both the dam and the information were essential in the management of both emergency events.
- The regular six yearly comprehensive dam safety examinations that are part of the dam safety programme provided the means for a new generation of engineers to become familiar with the dam. The dam safety examination report completed before the earthquake correctly identified the likely response of the dam.
- Instrument monitoring from the original lake filling became an important reference set against which to compare the settlements and seepage behaviour of the dam and abutments. The post earthquake operation of the lake below its normal operating levels meant that this was the only precedent information available. It is important therefore to record dam performance under a full range of reservoir conditions and maintain this record.
- At earthquake intensities at or above MMVII many people are affected by shock, including dam staff. Relief operators and engineers were required on-site as soon as possible to continue dam operations and inspections. Staff may also leave to attend to their own family needs at the time. The provision of relief staff, including senior personnel, must be a high priority in emergency plans.
- Duplicate power sources were necessary for the spillway gates. After the earthquake power was not available either from external lines or from station generation. There was a few hours delay before the spillway gates could be operated.
- Multiple communication means are necessary. Public telephone systems typically become overloaded during widespread emergency events like earthquakes and this occurred at Matahina.
- When the emergency lake drawdown was required, there was no pre-prepared plan and this had to be developed at short notice during a national public holiday. Fortunately an experienced dam engineer was available at the site and there was not an undue delay. The preparation of dewatering plans is an obvious part of good emergency planning.
- Also, in retrospect, it was considered that after the crest erosion cavity formed there had been insufficient liaison with authorities concerning the possible need to lower the lake. This caused some initial difficulties that were compounded when the notification was delegated to a person who was not known to the authorities. Communication plans need to consider the timing of communications with external parties and recognise the benefits of early communication between appropriately senior personnel.

Figure 6. Matahina Dam

Figure 7. Internal erosion in the dam core

ASSESSING THE SEISMIC SAFETY OF EXISTING DAMS

Dam Safety Programme Influences

Dam safety programmes have had a major influence (Gillon and Amos, 2007) on the seismic safety assessments of existing dams. New Zealand's major dam owners adopted the USBR SEED programme (USBR, 1983) as the basis for establishing their own programmes in the mid 1980's. The USBR SEED programme was widely used until, with the issue of the New Zealand Dam Safety Guidelines in 1997 and 2000 (NZSOLD, 2000), practices moved to conform to the New Zealand guidelines.

The USBR SEED programme and the New Zealand Dam Safety Guidelines both have as components of their programmes a regular dam safety review or evaluations typically at 5-6 year intervals. The initial reviews invariably made recommendations concerning the re-evaluation of the seismic safety of existing dams because with the changing knowledge about faults and earthquakes, the original dam design evaluations did not reflect current knowledge or practice.

In the late 1980's the New Zealand government commercialised the ownership of many of the large hydropower dams in New Zealand. As a new generation of commercial dam owners developed their understanding of dam safety management, the safety review recommendations started being actioned, initially focussing on spillway gates. In the mid 1990's the seismic safety evaluations of the Karapiro and Matahina Dams led in both cases to strengthening works.

The current situation is that many dam owners are systematically proceeding to determine the applicable seismic loads and undertake a structured programme of seismic safety evaluations. Dams where seismic safety deficiencies were most likely to be present have been examined. The remaining dams have a high degree of seismic safety. It has been recognised that seismic safety assessments may take several years, particularly where field investigations of faults are required.

Safety Criteria

Current criteria against which to evaluate the seismic safety of dams is based on the New Zealand Dam Safety Guidelines (NZSOLD, 2000). The maximum seismic loads in the guidelines are summarised in Table 2 in terms of the hazard or, in New Zealand terminology, the potential impact classification of the dam. The performance criterion for these loads is that '… some damage is allowable but it must not lead to a catastrophic failure." The New Zealand Dam Safety Guidelines refer to this load as the maximum design earthquake (MDE). This term is now increasingly reserved for new dam design and the term safety evaluation earthquake (SEE) used for existing dams. Note that as serviceability and not a safety load criteria, no reference has been made to the operating basis earthquake (OBE).

Table 2. Potential Impact Categories and safety related seismic loads for dams (NZSOLD, 2000)

Potential Impact Category	Potential Incremental Consequences of Failure		Seismic Load for Safety Evaluation (Maximum Design Earthquake or Safety Evaluation Earthquake)
	Life	**Socio-economic, Financial, & Environmental**	
High	Fatalities	Catastrophic damages	1 in 10,000 AEP ground motions or Controlling Maximum (Credible) Earthquake
Medium	A few fatalities are possible	Major damages	Less than the 1 in 10,000 AEP ground motions or Controlling Maximum (Credible) Earthquake
Low	No fatalities expected	Moderate damages	
Very Low	No fatalities	Minimal damages beyond owner's property	

The New Zealand Dam Safety Guidelines are not very specific in regard to the selection of ground motions to represent the MDE and SEE. To address this, two major New Zealand dam owners commissioned a review of international standards (Mejia et al, 2002). The results of this review are summarised in Table 3 and have been applied to a number of seismic safety evaluations.

Table 3. Summary of Seismic Load Safety Evaluation Criteria (Mejia et al., 2002)

Criteria Element	Evaluation Criteria		
Dam Potential Impact Category	High	Medium	Low
Evaluation Earthquakes	Safety Evaluation (SEE)	SEE	SEE
Earthquake Definition			
SEE	Controlling Maximum Earthquake (CME). Need not exceed controlling earthquake scenario (CES) for 10,000-year motions	CME. Need not exceed CES for 2,500-year motions	CME. Need not exceed CES for 500-year motions
Ground Motion Definition			
SEE	84^{th}-percentile level for CME. Need not exceed 10,000-year motions	50^{th} – 84^{th}-percentile level for CME. Need not exceed 2,500-year motions	50^{th}-percentile level for CME. Need not exceed 500-year motions
Other Ground Motion Considerations	Near-source, topographic, and seismic response effects	Near-source, topographic, and seismic response effects	Near-source, topographic, and seismic response effects
Foundation Fault Displacement			
SEE	84^{th}-percentile level for CME. Need not exceed 10,000-year displacements	50^{th} – 84^{th}-percentile level for CME. Need not exceed 2,500-year displacements	50^{th}-percentile level for CME. Need not exceed 500-year displacements

The application of these safety criteria is illustrated in the recently reported Aviemore Dam case history (Mejia et al, 2006) in the following section. The seismic safety assessment of Aviemore Dam includes both ground shaking and ground displacement effects.

ASSESSING THE SAFETY OF DAMS BUILT ON ACTIVE FAULTS

Karori Dams

There are two dams located across the active Wellington Fault in the suburb of Karori in the capital city Wellington at the south end of the North Island. The dams were built across the Karori Stream (Figure 8) for water supply to the city.

The upper dam is a concrete-gravity structure, 24m high and with a crest length of 107m. It was constructed between 1908 and 1911. The lower dam was constructed in 1874 and is a 20m high embankment dam with a crest length of 65m. The Wellington Fault passes directly through the foundations of both dams.

The recurrence interval for 4m displacement of the Wellington Fault is about 600 years. It has long been recognised and acknowledged that the dams would fail if there were such large movement on the fault. No dam stability studies were necessary to reach this conclusion! Alternative water supplies were developed and the upper reservoir was lowered in 1991 so that the volume of water released in an earthquake would be detained safely in the gully downstream of the lower dam without escaping into the urban area beyond. The dams are no longer used for water supply and now form the basis of a wildlife sanctuary.

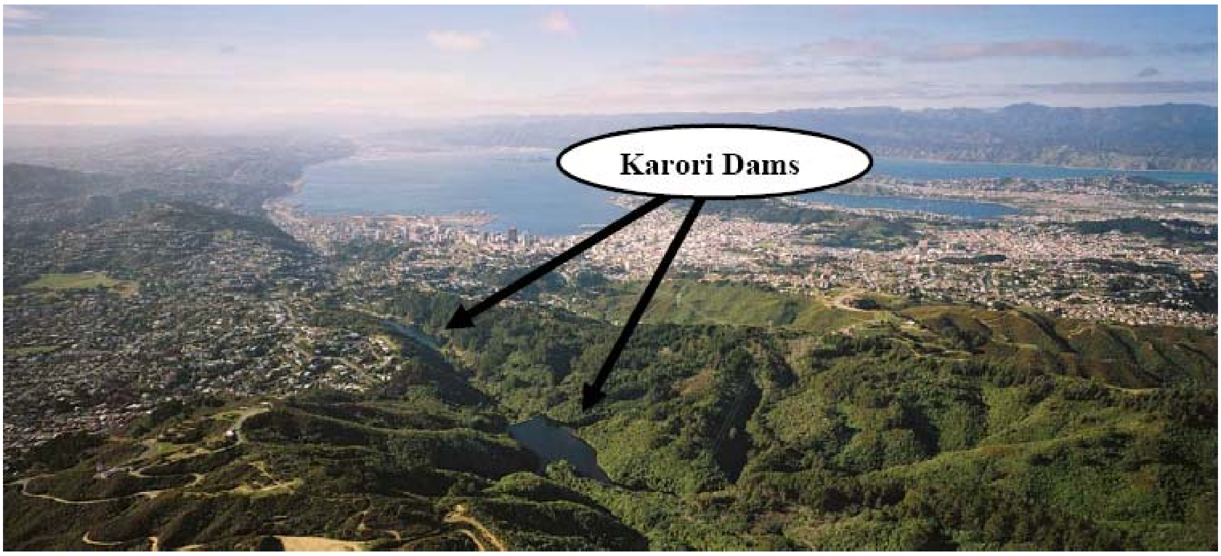

Figure 8. Karori Dams and the Wellington Fault
(courtesy of GNS, Photographer: Lloyd Homer)

Matahina Dam

The Matahina Dam and reservoir described previously are located along the trace of the Waiohau Fault[3]. Traces of the Waiohau Fault at the site are shown in Figure 9. A few years after the repair of the 1987 earthquake damage to Matahina Dam described above and in response to later safety review recommendations, the activity of the Waiohau Fault was investigated with trenches to determine the activity on the fault. The trenches showed that movement had occurred on the fault four times in the past 11,300 years. (Frequent historic volcanic eruptions in this area provided excellent dating control on fault movement.)

Studies demonstrated that the controlling maximum earthquake at the site was associated with the Waiohau Fault. The likely range of ground motions and fault surface displacements were estimated based on data from past earthquakes in New Zealand and other seismic regions of the world with similar tectonic environments.

[3] the 1987 earthquake was not associated with the Waiohau Fault

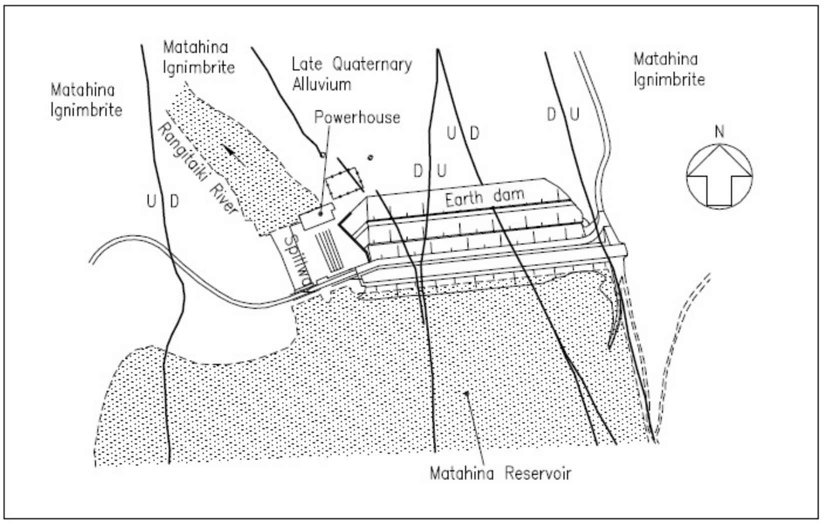

Figure 9. Traces of the Waiohau Fault at Matahina Dam

Because of settlement downstream of the dam, the ground motions and fault displacement parameters were estimated at the 84th percentile level (Gillon et al., 1997). The Safety Evaluation Earthquake (SEE) was estimated at M_w 7.2 with corresponding horizontal and vertical peak accelerations of 1.25g and 1.35g respectively and 3m of oblique slip on the fault. There are several fault traces present within the dam site and it was considered that the full displacement could occur on any trace.

The powerhouse and spillway are located on a rock spur (Figure 6) that forms the left abutment. This rock spur has been displaced, primarily by rigid body rotation, over the last 280,000 years. The best estimate of tilting in the SEE was 40-90cm per 100m of block length.

As the dam had already experienced serious internal erosion due to the lack of filter compatibility during lake filling in 1967 and again following the 1987 earthquake, it was readily demonstrated that the existing dam would fail in the event of the SEE and that failure would be sufficiently rapid to exclude reservoir drawdown as a mitigation measure. Following this finding, the results were made public and the reservoir was lowered in 1995 while mitigation measures were evaluated.

A wide variety of rehabilitation and mitigation measures were considered and the concept of strengthening the existing dam (Figure 10) with a leakage resistant buttress (Gillon et al., 1998) emerged as the most economic and robust solution with acceptable environmental impacts. The overall concept was that the leakage resistant buttress (Figure 11) controls post-SEE leakage through the ruptured dam core to quantities that would both flow safely through the rockfill and be contained within the downstream river channel. The wide filter, transition and drainage zones remain continuous after the fault movement. The sand filter restricts leakage flows and minimises the loss of core material from within the dam body.

In addition the crest was raised to increase the freeboard from 3m to 6m. This had the benefits of providing overtopping protection from spillway gate malfunction and seiches as well as allowing for possible crest subsidence due to internal erosion of core material.

Although the spillway was not expected to be affected by shear deformations, the structure was subject to severe ground motions and some strengthening was required. Control and power supply systems were also upgraded and strengthened (Ritchie et al, 2000) for the SEE ground motions plus an allowance for topographic amplification on the rock spur. In addition, bulkhead doors were installed on the earth dam gallery to prevent leakage and internal erosion occurring through the gallery in the event of it being ruptured by fault movement.

The strengthening work was started in 1997 and completed in 1998. The strengthening of Matahina Dam for potential movement of fault traces in the dam foundation is thought to have been a world first.

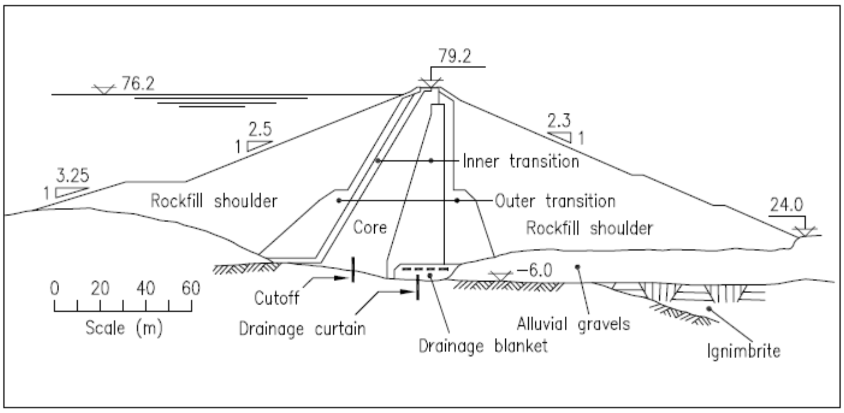

Figure 10. Matahina Dam cross section before strengthening

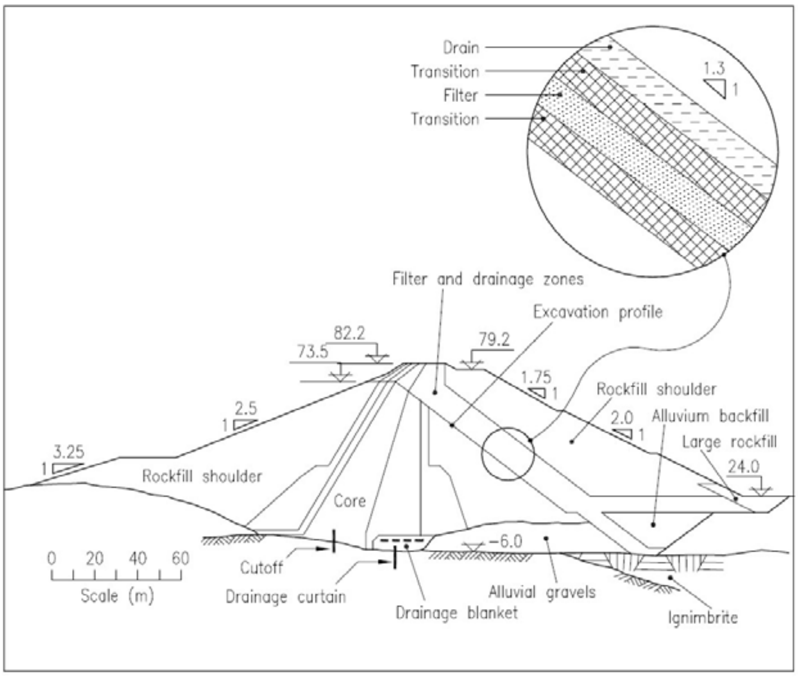

Figure 11. Matahina Dam cross section after strengthening

Aviemore Dam

Aviemore Dam is located on the Waitaki River in the South Island of New Zealand. Aviemore Dam (Figure 12) consists of a 364m long, 56 m high concrete gravity section abutting a 430m long, 49m high zoned earth embankment. The concrete gravity dam consists of spillway and power intake sections flanked by non-overflow sections at each end.

The dam was built in the late 1960's. The presence of the Waitangi Fault in the dam foundations was known at that time but it was not considered to be active. The fault crosses the foundation of the earth dam about 30m from its junction with the concrete dam (Figure 12) and strikes almost perpendicular to the dam axis. To the left of the fault the dam foundation is Mesozoic age greywacke rock. To the right of the fault the foundation is in weak Tertiary age claystone and sandstone. Safety reviews had recommended a re-evaluation of the seismic safety of the dam.

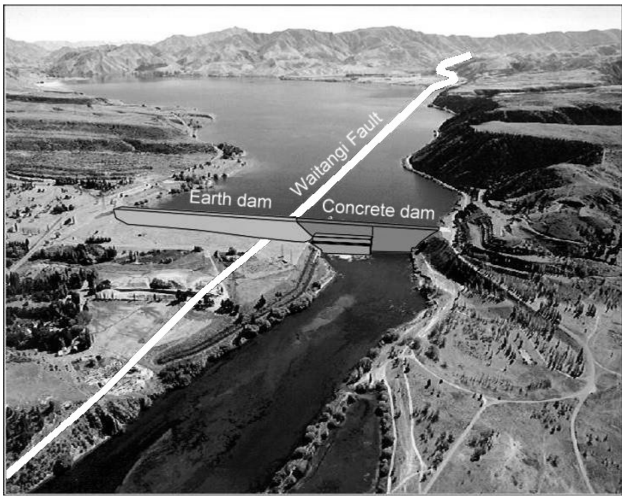

Figure 12. Aviemore Dam and the Waitangi Fault

As part of the seismotectonic hazard evaluation, trenches were excavated across the fault. These showed that the last movement occurred about 13,000 years ago and that at least two movements have occurred in the last 21,000 years. Other nearby faults were also investigated to determine their movement histories.

Deterministic and probabilistic analyses of ground motion and fault surface displacements showed the Waitangi Fault as inducing the largest seismic demand on the dam. The SEE was determined as being the maximum earthquake on the Waitangi Fault with a Richter magnitude of 7.0. This produced a horizontal peak ground acceleration of about 1g, a vertical displacement of 1.2m and a horizontal displacement of 0.4m.

The fault traverses the reservoir and a large area of the reservoir would be uplifted during surface rupture of the fault generating seiches in the lake.

Key aspects of the seismic safety evaluation of the dam were:

- stability of the concrete and earth dams under shaking
- integrity of the earth dam under fault displacement in the foundation
- potential for overtopping due to seiche action and a reduction in freeboard
- performance of the appurtenant works under shaking.

These evaluations are described in Mejia et al, 2005 and 2006. The seismic safety evaluation indicated that the dam will withstand the SEE without catastrophic, uncontrolled release of the reservoir. However, the earth dam is likely to be damaged by foundation fault rupture during the SEE and limited subsequent overtopping by seiche waves. As a result of the evaluation, a wave wall is being installed on a section of the dam and the spillway gates and spillway bridge deck are being strengthened. Specially developed instrumentation has been installed at the dam to assist in the post SEE performance monitoring (Amos et al., 2006).

ASSESSING THE SEISMIC SAFETY OF A PROPOSED DAM

Concrete gravity dams have a very good performance history in earthquakes. Earthquake shaking alone has never caused failure of a concrete dam.

For many years analysis techniques for seismic design of concrete gravity dams around the world have relied on stability criteria which include factors of safety during an earthquake. More recent trends documented by the US Federal Energy Regulatory Commission (FERC, 2002) adopt the following steps:

1. Static analyses for reservoir loads, normal uplift conditions and thermal stresses to understand the pre-earthquake condition of the dam.
2. Analysis of the dam during the earthquake to determine the extent of damage. The dam is assumed to be in its normal state at the time of the earthquake (i.e. normal reservoir level and uplift conditions).
3. Assessment of the post-earthquake condition of the dam and its stability. Post-earthquake conditions are applied, such as residual strengths for damaged areas and whether uplift is being controlled adequately. This is the key condition of the dam which determines if safety is adequate.

An 85m high RCC gravity dam (figures 13 and 14) is planned on the Mokihinui River on the West Coast of the South Island of New Zealand. The dam is in the preliminary stages of design and because of the proximity of major faults this has included seismic analysis of the proposed dam.

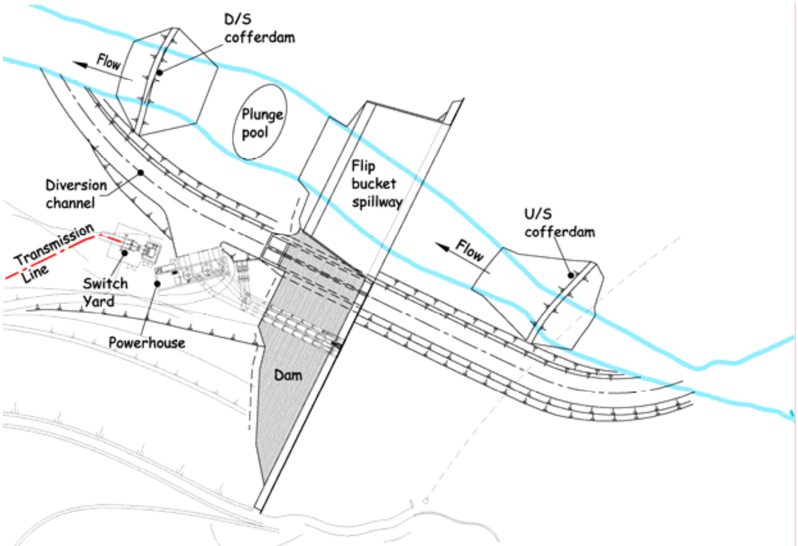

Figure 13. Dam Layout

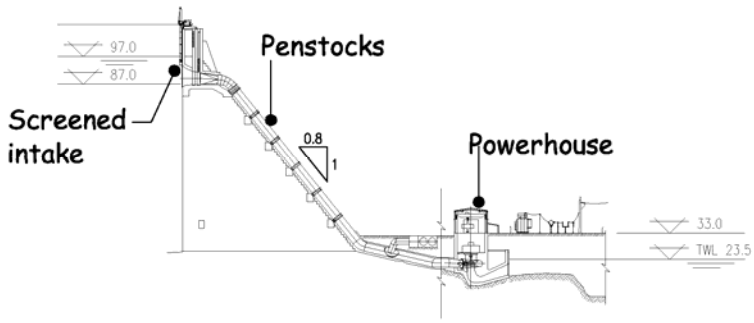

Figure 14. Dam Section

The Mokihinui dam has a high potential impact classification and therefore an earthquake from a fault source or a probabilistically derived earthquake with a return period of more than 1 in 10,000 years will be the Maximum Design Earthquake. In the case of the Mokihinui dam site, the nearest fault (the Glasgow Fault 1km from the dam site) is estimated to provide shaking with a peak ground acceleration of 0.91g and this is currently being used as the Maximum Design Earthquake for dam design.

Analysis of the Mokihinui RCC dam for earthquakes involved the following steps:

1. Sliding analysis on dam lift joints and foundation interface for horizontal accelerations. If foundation investigations had found horizontal seams in the foundation rock, then analysis would have included these weaker planes.
2. Two dimensional finite element analysis of 1m thick slices of cross section using response spectra to determine tensile stress concentrations. Two dimensional analysis is carried out to maintain simplicity when compared to three dimensional
3. Assessing the tensile stress concentrations to determine the likely extent of cracking in the dam foundation.
4. Applying residual strength parameters to cracked sections and determining post-earthquake sliding stability.
5. Applying an aftershock to the site. These are assumed to be one order of Magnitude less than the main earthquake.

Finite element modelling of gravity concrete dams typically shows that the dam body is largely a stiff cantilever with response having a high frequency and being dominated by the first mode as shown on Figure 15.

The response of concrete dams in earthquakes depends on a combination of the dam's size and shape, together with its foundations. The foundation stiffness is a key parameter for response analysis. Safety against sliding on a plane within the dam foundation (or at the interface surface) is dependent on strength represented by angle of internal friction and cohesion parameters. Sliding on a near horizontal plane of weakness is an important feature to check, but none were present at Mokihinui. Two rock types are present in the Mokihinui dam foundation (granite and greywacke) with subtly different parameters. Rock samples from cores were removed for laboratory tests to determine strength properties.

For the purposes of the preliminary study of the Mokihinui Dam, the NZ Society of Large Dams (NZSOLD, 2000) "Dam Safety Guidelines" was used to establish acceptable limits on the sliding stability of the dam. The New Zealand guidelines reflect internationally-accepted dam safety criteria at the time of issue (November 2000). Acceptable limits allow for some easing of requirements if the materials are well understood by testing, but for the preliminary assessment the more conservative limits were used.

Peak tensile stresses calculated at the base of Mokihinui Dam are shown on Figure 16 for various return period earthquakes analysed. Higher stresses occur at both the upstream and downstream faces because of the cyclic motion of the earthquake. The dam concrete is assumed to be capable of sustaining some tensile stresses before cracking. The dynamic tensile capacity of concrete has been researched by Raphael (Raphael, 1984) and others. It is typically considered to be in the order of 2 to 3MPa. Therefore cracking is only assumed to occur when tensile stresses during the earthquake exceed this.

Two dimensional analysis of stresses in the dam body showed that the dam will exceed a linear-elastic range of behaviour for earthquakes greater than about the 2,500-year event and horizontal lift joints will probably de-bond in high tensile stress areas. To estimate the order of concrete strength required to minimise the extent of de-bonding during these higher levels of earthquake, a three dimensional linear-elastic time-history analysis of the dam was then carried out. The 3-D analysis was also used to determine whether some load will be taken by the abutments, thereby reducing the demand at the dam foundation. 3-D analysis is more complex, hence it was important to correlate the 3-D analysis to the initial 2-D analysis.

The three dimensional finite element analysis calculated stresses in the dam body and the extent of tensile stresses during the earthquake indicated where there is potential for horizontal placement lift joints to debond. From the results it was clear that high cement paste RCC would be required to reduce the risk of joint debonding under the very high seismic loads at Mokihinui.

From the finite element analyses it will now be possible to refine the section shapes to reduce areas of stress concentrations. Sharp changes in section shape are not recommended for concrete dams during earthquakes. Utilising resistance from the abutments will be greatly enhanced by a slight plan curvature of the dam axis.

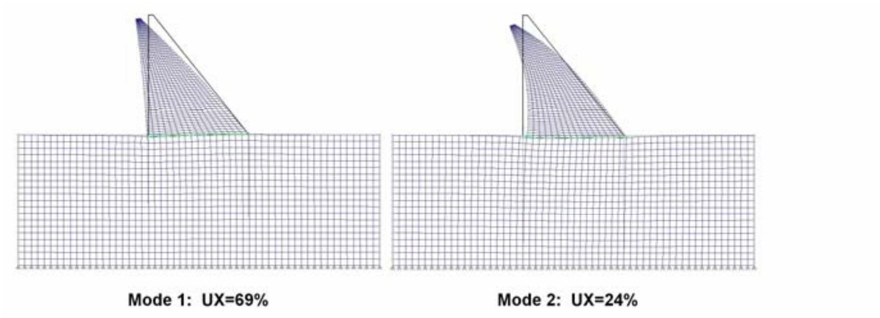

Figure 15. Modal Analysis

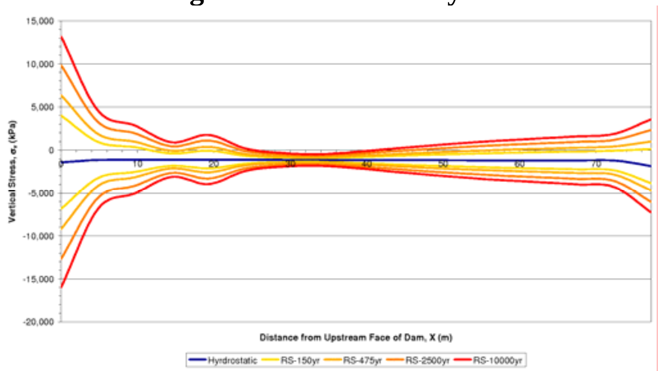

Figure 16. Peak tensile stresses at dam base for different return period earthquakes

ASSESSING THE SEISMIC SAFETY OF RESERVOIRS

Reservoir slope instability, including slope failures caused by earthquakes, can potentially affect dams in five main ways. The first is that a slope failure in the immediate vicinity of the dam could potentially damage the dam and its appurtenant structures or prevent the operation of facilities like intakes by blocking them. The second is that failure of the reservoir slopes has the potential to either cause damaging impulse waves if the failure is very rapid or, if the slide has sufficient volume, to block the valley causing a landslide dam to form within the reservoir. Subsequent failure of the landslide dam then has the potential to create damaging flood flows at the dam. The third is that a slope failure in the river valley upstream of the reservoir could also potentially create a landslide dam whose eventual failure could create damaging flood flows at the dam. The fourth is that slope failure where the reservoir rim is narrow has the potential to breach the reservoir wall and release damaging flows. The fifth effect is that the earthquake ground motions may cause a flow slide in the sediments deposited in the reservoir which then flow down the reservoir with potential to block intakes.

Slope stability hazard assessment is primarily based on a thorough understanding of the geology and the past and current landscape forming processes in the reservoir and the upstream river valleys. It is particularly important to identify and understand the slope failures that have occurred and whether any potential slope stability issues remain. This information is then used to identify the potential for future slope failures and to evaluate their effects at the dam. The stability of the slopes is considered for the applicable loads including the effects of inundation by the lake, lake level fluctuations, rainfall effects and earthquakes. ICOLD Bulletin 124 is the primary reference and guideline for the investigation and management of reservoir slope stability hazards.

In addition to the hazard of reservoir slope instability, the earthquake ground motions can cause seiching in the reservoir. This is usually a minor effect. Also, as discussed above in the Aviemore Dam seismic safety evaluation, an active fault within the reservoir has the potential to generate seiching in the event of fault displacement.

An example of utilising previous slope failures to assess the seismic safety of a proposed reservoir is illustrated in Figure 17. Known rockfalls (referred to as M11.5 and M12.5 on Figure 17) in the 1929 Murchison earthquake upstream of the proposed Mokihinui dam were used to estimate the possible maximum wave height due future earthquakes and rockfalls. The assessment used an accepted wave height estimation technique that was suited to narrow winding gorge scenarios such as at Mokihinui. Wave heights of up to approximately 8m (Figure 18) at the dam site were estimated for the conservative (worst case) instantaneous triggering rockslide event that was modelled. This wave height was then used as a load case in the structural analysis of the dam.

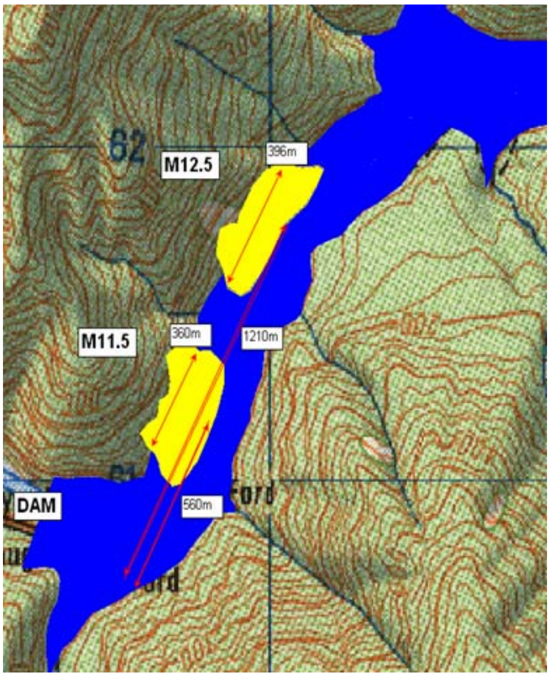

Figure 17 Rock Slide Seismic Hazard

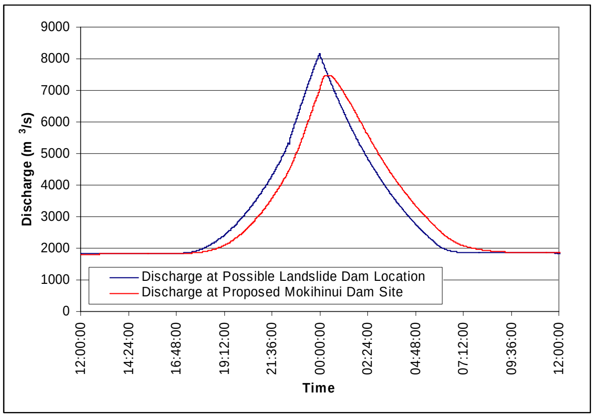

Figure 18 Flood Discharge Hydrograph

IMPROVEMENTS TO SPILLWAY SYSTEMS

At Matahina Dam, immediately following the 1987 earthquake, a lack of power supply delayed the opening of the spillway gates. This highlighted the importance of providing robust power supplies and lifting systems for spillway gate operation in emergencies. Another incident at about this time also highlighted the need to have safe access in adverse conditions for local operation of spillway gates. The particular incident involved severe storm conditions that prevented safe access to the elevated gantry where the controls for local operation of the spillway gates were located.

The requirement to lower the Matahina Dam reservoir for repairing the earthquake damage also highlighted the contribution of deep spillway gates and low-level outlets to the safety of smaller volume reservoirs following earthquakes. This had particular relevance to the cascade of dams on the Waikato River in the Taupo Volcanic Zone, where the safety of the whole chain is very dependent on the safe operation of the upstream reservoirs.

The Clyde Dam spillway design was instrumental in bringing attention to the amplification of earthquake ground motion effects at dam crest level. The spillway support structures, control and lifting equipment and the gates on older dams need to be robust enough for this load condition.

During the twenty years since the earthquake damage at Matahina Dam there has been a steady programme of upgrading spillways and outlet structures for extreme load conditions from floods and earthquakes. In many cases the spillway gates are now capable of remote operation as well as local operation, particularly for flood control.

At the Roxburgh, Ohakuri and Whakamaru Dams, all built in the 1950's, the spillway superstructure has been upgraded to ensure the seismic safety and post-earthquake operability of the spillways. At Whakamaru the gates are operated using winches mounted 12m above the dam deck on a reinforced concrete frame (Foster et al., 2003). The amplification effect of earthquake loads at the level of the winches was estimated to be 12 times the dam foundation accelerations. These columns were strengthened by adding reinforced concrete to change the former rectangular section into an I beam section as shown in Figure 19. A similar upgrade took place at Ohakuri Dam.

Figure 19. Whakamaru Dam strengthened spillway columns

Figure 20. Roxburgh Dam spillway gantry upgrading

The Roxburgh Dam was built with a similar reinforced concrete spillway superstructure as at Whakamaru Dam. Upgrading in this case consisted of spillway bridge deck strengthening to limit the displacement of spillway piers followed by progressive demolition of the superstructure and its replacement with a steel gantry. Hydraulic lifting cylinders replace the former winches and counterweights. In order to maintain sufficient spillway capacity during the work, a temporary lifting frame was installed on the centre gate. Figure 20 shows the work in progress. At both dams the spillway gates have also been strengthened and the former diversion sluices have been modified to operable low level outlets.

At the Karapiro Dam on the Waikato River the diversion tunnel has been modified to allow operation as a low level outlet (Grilli, 1994). The modification involved converting the tunnel to a partially steel lined pressure tunnel with a downstream sliding gate and outlet structure.

The spillway gate lifting equipment and power supplies at many dams have been upgraded. At the Matahina Dam, small individual petrol motors were installed on each gate to provide a lifting facility that was totally independent of electric power. In addition, as is now common practice, an independent diesel powered generator was provided at the spillway.

At several of the Waikato River dams, the upgrading of the lifting systems consisted of replacing the existing electric motor with a higher rated motor, plus a hydraulic motor at the opposite end of the primary shaft, with a diesel hydraulic power pack and valves for powering it.

On the Waikato River cascade the upgrading of power supplies and the spillway winch lifting equipment has typically involved:

- power supply from the station to have two sets of cables following different routes and terminated to a 400V bus with good segregation between the two sides of the bus
- a diesel generator located at the spillway
- a diesel/hydraulic set to power a hydraulic motor coupled to the primary winch shaft.

- the provision of a suitable torque limiter between the motors and the gearbox drive to the winch shaft.

The typical maintenance and test regimes for spillway gates and their ancillary equipment include:

- a monthly lift-off test for spillway gates and a small annual test discharge
- test operation of the various control systems during the monthly tests
- monthly starting of diesel generators and annual load tests
- regular testing and, when necessary, replacement of diesel fuel
- detailed tests of the gates and the normal and backup systems during the 5-6 yearly comprehensive safety examinations.

EMERGENCY PLANNING FOR EARTHQUAKES

Emergency planning is a specialised area of dam safety management. The following comments summarise some selected areas of New Zealand practice in relation to earthquake events.

- New Zealand practice in the preparation of emergency plans is to follow the NZSOLD Dam Safety Guidelines (NZSOLD, 2000) outline for these plans.
- There is almost total reliance on the national seismograph network to determine the location and magnitude of earthquakes. This is a very well developed service with fast automatic distribution of earthquake characteristics by pager, text messaging and email to subscribers. This information is used to prioritise inspections for owners with a wide geographic spread of dams.
- Post-earthquake damage inspection procedures are based on a simple check sheet type initial inspection and reporting with detailed inspections following where necessary. These simplified inspections allow inspections to be carried out by a wide range of operational staff with only minimal training required. Inspection procedures are typically activated at Modified Mercalli intensity 6 or greater.
- Plans allow for relief personnel to be brought into the affected area for both operations and inspection.
- Some owners assist staff with their own household planning for earthquake emergencies recognising that staff need to be sure of the safety of their families before being able to commit to lengthy inspection duties.
- A radio based communication system is maintained in addition to the now widely available national cell phone coverage.
- Dewatering plans are pre-prepared for dams where this is an applicable mitigation measure.
- Dam surveillance and inspection activities are utilised to ensure that there are always engineers who are familiar with the dams.

CONCLUDING REMARKS

New Zealand presents a challenging tectonic and geologic environment in which to build safe dams. We have been fortunate to inherit a stock of robust dams from our predecessors so that relatively few strengthening projects have been required as current earthquake understanding has been applied to the seismic safety assessment of existing dams.

AKNOWLEDGEMENTS

This paper is largely based on the similarly titled paper presented at the First Turkish Dam Safety Symposium May 2007 by Murray Gillon. The authors acknowledge the assistance of their colleagues in New Zealand in locating reference material and thank GNS Science for assistance with illustrations.

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