COMMISSION INTERNATIONALE
DES GRANDS BARRAGES

VINGT- DEUXIÈME CONGRÈS
DES GRANDS BARRAGES
Barcelone, 2006

EILDON PROJECT ALLIANCE DELIVERS COST EFFECTIVE SPILLWAY CAPACITY UPGRADE (*)

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AUSTRALIA

1. INTRODUCTION

Goulburn-Murray Water (G-MW) is Australia's largest irrigation authority, with responsibility for the operation, maintenance, renewal and dam safety program for seventeen dams. Its region covers a large part of the State of Victoria, including the catchments of most of the State's north flowing rivers that enter the River Murray (figure 1).

The dams vary in age between 9 and 134 years and G-MW identified a need for a structured program to review the safety status of its dams in relation to contemporary standards. The first element of the program was to assign priorities for safety reviews based on preliminary calculations of the risk to downstream communities posed by each dam. Initially, five dams were selected for priority safety review.

(*) L'Enterprise d'Alliance d'Eildon remporte une amélioration économique de la capacité de l'evacuateur de crue.

Figure 1
Goulburn-Murray Water Region
(La Région de Goulburn-Murray Water)

1. Australia	*1. Australie*
2. State of Victoria	*2. État de Victoria*
3. Goulburn-Murray Water Region	*3. La Région de Goulburn-Murray Water*

Safety reviews for the first five priority dams revealed a number of deficiencies. The second element of the program was then to undertake a detailed risk assessment process to estimate the life safety and financial risks associated with each deficiency. The magnitude of the risks in comparison to the tolerable risk criteria in the Australian National Committee on Large Dams Guidelines on Risk Assessment (ANCOLD 2003) was then used to develop a program of works to progressively reduce risk over time, with the ultimate long-term objective of all dams meeting standards.

The risk-based approach is being used by G-MW to reduce the risks posed by its dams in a cost-effective way and in appropriate time frames in relation to the estimated risk, rather than reduce all risks at each dam on a dam by dam basis. This latter approach may result in communities being exposed to high levels of risk over too long a time frame, or the owner consuming scarce funds for dam safety on components of dams that may not meet standards, but do not pose high risks.

By 2003, G-MW had undertaken five separate interim dam safety upgrading projects. None of these projects had addressed all identified deficiencies at each of the dams, but did reduce high risks to a lower level in a timely manner and allowed funding to be applied to the areas of most need.

2. EILDON DAM

Eildon Dam is located on the Goulburn River, just upstream of the town of Eildon. The dam was originally constructed in the 1950s, replacing the smaller Sugarloaf Dam, which is located immediately upstream of the Eildon Dam embankment. Prior to the current upgrade works the embankment was about

80 m high, with a crest length of about 900 m. The dam crest, at level RL 291.65 m was 9.1 m wide, contained a 2 lane surfaced road and footpath. A small masonry wall was located on the upstream edge of the crest.

The spillway is located on the eastern (left hand) side of the embankment. It is a mass concrete structure with a height of 33 m in the ogee section and 40 m in the non overflow section and a crest width of 200 m. The overflow section of the structure consists of 3 bays each approximately 20 m in width. The approach channel to the spillway was excavated through the left abutment of the dam. It is unlined. The total length of this excavation is about 900 m with the structure located about 425 m from the upstream end. The discharge chute is fully concrete lined with a width varying from 65.5 m at the upstream end to 91.4 m at the downstream end. A concrete dissipater is located at the downstream end of the chute, where flows are discharged to the regulating pondage (figure 2).

Figure 2
Aerial View of Eildon dam
(Vue Aérienne du barrage d'Eildon)

The spillway flows are controlled by three vertical lift gates, each 20 m wide by 6.1 m high. These gates are currently operated from a gallery of the spillway road bridge.

Low level discharges from the reservoir are either through the outlet tower or the irrigation outlets through the base of the spillway structure. The intake tower for the outlet works is a 72 m high reinforced concrete structure located upstream of the right hand abutment of the embankment, within the spillway of the original Sugarloaf Dam. The tower discharges to a 7 m diameter steel lined tunnel, which is connected to the power station.

Eildon dam was one of five dams identified for priority safety assessment and then a detailed risk assessment process. The safety review revealed significant deficiencies in spillway capacity, embankment filters, particularly in the upper part of the embankment, together with unusual embankment performance and the potentially poor performance of the spillway structure under flood and earthquake loadings.

By 2003, Eildon dam was G-MW's highest priority for risk reduction. However, unlike other dams, the risk assessment process did not identify interim actions that could be taken to reduce risks to a tolerable level; rather the conclusion was that the project would need to address all deficiencies to meet contemporary standards, sometimes referred to as a "standards-based" upgrade.

Concept designs were prepared and evaluated using multi-attribute analysis. The preferred concept consisted of the following elements:

- Increasing the spillway capacity by both raising the main embankment and modifying the spillway to provide additional clearance to allow extreme flows to flow through the existing gate openings.
- The provision of pre-stressed anchors to stabilise the gravity spillway section.
- The reconstruction of the upper section of the main embankment, incorporating modern filters and providing a raised crest level to prevent overtopping of the embankment during an extreme flood event.
- Consideration of the merits of stabilising the outlet tower for its' Maximum Design Earthquake event.

3. PROJECT DELIVERY

3.1 Selection of procurement model

G-MW has very limited internal construction resources and had undertaken previous projects through a 'direct management' process. In this process, a consultant was engaged to design the works and then G-MW would manage the construction through a combination of directly engaging external resources and specialist sub-contractors. It was found that this approach allowed flexibility in dealing with ongoing dam operations whilst providing a means of dealing with project risk in a cost-effective way. Projects undertaken using this approach achieved good cost and time outcomes, in spite of several latent conditions that would have posed difficulties in a conventional contracting environment.

Although G-MW had a preference for continuing the 'direct management' project delivery process, it was recognised that the scope of the Eildon project would be significantly larger than any previous project undertaken and that this would present difficulties in being able to provide sufficient resources from within the organisation to support the project. G-MW therefore engaged a specialist management consultant to assist in determining the preferred project delivery model for the Eildon Project.

Procurement models considered included:

- Design then construction – the traditional project delivery method in Australia up until the early 1990's which can be completed either by a single contract constructor or by managing a number of specialist contractors,
- A single design and construct contract to deliver the project under one commercial arrangement, and
- A 'project alliance' where a single purpose organisation is formed between the owner and the design and construction organisations to deliver the project, with the risks and rewards shared amongst the 'alliance partners'.

The alliance form of project delivery is relatively new to the Australian dams industry, but has been used extensively in the oil and gas resources sector. It consists of a selection process to choose commercial partners to jointly deliver the project, then bringing design to a stage that an estimate of cost can be prepared. The owner and alliance team then independently estimate the cost and if agreement can be reached on the estimate, the project is implemented by an integrated team from the owner and commercial partners. Cost overrun or underrun is shared equally, but for the commercial partners any loss is capped at the value of profit and overhead.

The three project delivery methods were evaluated against thirteen criteria, viz – Security of Asset, Health and Safety, Budget and Time, Customer Service, Due Diligence, Competence of Team, Robust Communications, Environmental Care, Business as-usual, Emergency Procedures, Skills transfer, Expertise with Management System and Risk Allocation.

The alliance method was scored as preferred for the delivery of the Eildon Project. It was seen as having particular value in the following areas:

- The opportunity to commence works quickly (water levels were predicted to be just above historic lows in 2004 – providing an opportunity for the embankment to be reconstructed with low levels of risk).
- Sharing of risks.
- Ability to select the best available team from owner and commercial partners.
- Integration of key performance indictors to measure project health.
- Continuing owner's involvement in project decision making and ability to influence project outcomes.
- Team approach to problem solving.
- Ability to deal with unexpected change in a cooperative way.

- Potential for flexibility of scope – allowing works to be easily included or excluded from the project depending on available budgets and allowing the final project to provide dam safety risk levels compatible with others within the owners' dam portfolio.

3.2 Eildon Alliance Partner Selection Process

The Eildon Alliance partners were selected from eight consortia that expressed interest in the project. Selection criteria focussed on technical and project delivery strengths, but also considered less tangible criteria such as teamwork, ability to adapt to alliance principles of behaviour, involving joint responsibility for decision making, openness, and commercial criteria.

Following the selection of the design and construction alliance partners by the owner, the Eildon Alliance was formed using the best personnel available from each of the four partner organisations.

Therefore the Eildon Alliance had both experienced design and construction personnel available early on in the project, which helped the designers to explore innovative solutions whilst providing quick and valuable feedback on the cost and construction implications of these solutions.

3.3 Development of Project Target Cost

Like any contract, the Eildon Alliance required agreement on the scope of work and the cost to design and construct these works.

In this case preliminary design was completed to the extent needed for an accurate cost estimate to be prepared. This cost estimate was then compiled on an open book basis and independently reviewed. In this way it was possible for elements of the project (such as the spillway works) to be investigated further as the estimate proceeded. This process also allowed G-MW and the Eildon Alliance to complete value management reviews for elements of the work and assess various partial fix options for both cost and risk reduction provided with accurate cost and dam safety data.

4. SPILLWAY MODEL STUDY

4.1 Background

The peak flood discharge for which the spillway was originally designed is 3 400 m^3/sec. The criterion adopted by G-MW was that the dam should be able to pass the Probable Maximum Flood (PMF) with the initial reservoir level at Full Supply Level (FSL). This produced a peak discharge of 6 900 m^3/sec.

Four feasible options were investigated to pass the increased discharge including widening the spillway and building a secondary spillway. However the preferred solution was to pass the flood over the existing spillway with the maximum head increased from about 9 m to about 14.1 m.

The model study quickly showed that the geometry of the approach and the structure combined with the high head over the ogee was producing very turbulent flows that were generating very large waves and dynamic pressures in the chute. (Hampton, 2004 & Sunwater, 2004)

4.2 Model Runs

Prior to the formation of the Eildon Alliance, G-MW had commissioned a 1:70 physical model of the spillway. The model extended from about 750 m (prototype) upstream of the spillway structure to about 600 m downstream of the structure. The approach channel of the spillway was included to ensure that any conditions caused by the asymmetrical approach conditions could be modelled.

Upon formation, the model was handed over and the Eildon Alliance became responsible for all testing. The model was tested with the original arrangement of the spillway, but assuming the gate operation and bridge had been modified to such that the PMF could pass under the gates. In this arrangement the peak spillway discharge required was 7 100 m^3/sec, which is more than double the original design flood of 3,400 m^3/sec.

4.3 Model Flow Conditions

The initial runs indicated that for flows greater than about 1,000 m^3/sec the flow became very turbulent with very disturbed flow evident at the external piers particularly at the right hand pier. As the flow increased the turbulence increased significantly. The cause of the turbulence is thought to be a function of the approach geometry where three rotational flows are combining as the water is accelerated over the spillway. At higher discharges these circulating flows developed to the point that vortices occurred in the approach channel.

As the flow reached about 5,000 m^3/sec an additional phenomenon occurred. Vortices formed on the downstream face of the spillway and extended to the point of connecting with an air column that had formed in the gate slots. These vortices collapsed causing large airbursts occurring at the toe of the spillway structure.

A third phenomenon identified in the model was the large “bow waves” generated by the bull nosed piers. These waves resulted in rapidly converging flow on the downstream side of the piers, leading to a “rooster tail” which extended about 15 m (prototype) above the chute and impacting about 50 m downstream of the spillway structure.

The result of the turbulent flows, the collapsing vortices and the “rooster tails” was the development of large transient waves progressing down the chute, with some measured as high as 20 m at the peak discharge. The model included a series of manometers and pressure transducers to monitor both the static pressures and the pressure fluctuations along the spillway structure and the upper part of the spillway chute and chute walls.

If the highly turbulent conditions that have been predicted were to occur in the prototype, they are likely to cause the downstream chute to fail. The chute is constructed on relatively low strength sedimentary rocks (mudstones, siltstones and sandstones) of varying degrees of weathering from fresh to extremely weathered. Therefore any loss of the concrete chute would quickly result in the erosion of the underlying foundation. Loss of a section of the spillway chute concrete lining might lead to extensive back erosion, with the possible undermining of the spillway structure, leading to failure.

4.4 Flow Smoothing Options

The initial approach to overcome the turbulent flows was to incorporate flow smoothing features into the model. These included circular approach training walls, modifications to the piers to provide a less abrupt cross section and gate slot fillers to prevent the drawing of air into the slot. All these provided some improvement to flows for each of the above conditions.

To reduce the impact of the approach channel rotational flows, training walls were trialled to minimise the abrupt changes in direction the flow is forced to undergo. A number of different arrangements were modelled, however the final arrangement proposed incorporated full height solid walls with a 30 m radius, extending from upstream face of the spillway to the sloping sides of the approach channel. This arrangement significantly reduced the turbulence caused by the rotational flows in the approach channel.

Gate slot fillers were trialled to stop air being drawn down the gate slots. Once the supply of air was restricted it immediately stopped the collapse of the vortices and the resulting air burst in the chute.

Modifications to the piers included long extensions to both the upstream and downstream edges of the existing piers providing a very gradual transition in the flow. This was successful in reducing the rapidly converging flow and the resulting rooster tail.

4.5 The Design Process

Following trials of various arrangements to flow smoothing options a design was developed that could be constructed around the prevailing storage levels.

A significant constraint was that water harvesting could not be interrupted and therefore there was no control on the reservoir level. Reservoir level projections indicated that the range of depths would be between about 6 m and

20 m in the spillway approach channel over the proposed construction period. It was therefore necessary to identify construction methods which allowed construction of both the training walls and upstream pier extensions in up to 20 m of water. A very interactive process was put into place to develop a constructible design and an exhaustive range of options was considered including underwater construction, barges and coffer dams. All options considered had a significant risk to both cost and timeliness should the reservoir rise quicker or higher than had been predicted.

From a technical approach designs which would produce the desired improved flow conditions while resisting the applied loads were readily identifiable. However these solutions were expensive and difficult to construct and a value management review suggested other approaches be investigated.

4.6 The Alternative Approach

Given the high costs and risks associated with the construction of any upstream works it was clear that it would not be cost effective to significantly reduce the turbulent flows that would be generated. Instead it was decided to investigate means of strengthening the downstream chute such that should failure of the chute occur, any back erosion would be restricted and the spillway structure would not fail.

In consultation with G-MW the principle adopted was that for large to extreme floods damage to the chute would be acceptable provided that there is no failure of the spillway structure leading to uncontrolled releases from the reservoir.

5. SPILLWAY ASSESSMENT

5.1 Chute Condition

As originally constructed the chute is fully concrete lined from the spillway structure to the dissipater. The width of the chute varies from 65.5 m at the upstream end to 91.4 m at the downstream end. The chute floor consists of 10.5 m square reinforced concrete slabs, each keyed into the four adjacent slabs. The three most upstream lines of slabs are 900 mm deep while the remaining slabs are 300 mm thick. The slabs are reinforced by 19 mm bars at 300 mm centres on the top surface and are either unreinforced or reinforced with 12 mm bars at 300 centres at the bottom. The slabs have been anchored to the foundation by 19 mm bars at 1.5 m centres and 2 m into the rock. A grid of under drains exists along the joints between all slabs that connect to collector drains running down each side of the chute.

Reinforced concrete side walls run the full length of the chute. They vary from 4.6 m high at the structure to 3.3 m high 35 m downstream of the structure and for the remainder of the chute. These walls were not constructed as designed and some of the wall stems have deflected by up to 80 mm. At last two wall slabs had structurally “failed”.

The chute slabs have moved relative to each other, damaging the original sealing material between the joints. The slab dowel anchors were also found to be severely corroded and significant quantities of the original formwork were found to be within and below the concrete slabs.

5.2 Failure Mechanisms

A detailed risk analysis was undertaken for the Eildon dam including an assessment of the failure of the chute leading to undercutting of the spillway structure. For the purpose of this assessment the possible flood failure mechanisms were considered. These are discussed below:

Failure of the Dissipater – The dissipater was designed to be effective for the original design flood of 3 400 m^3/s, and the model testing showed that it is effective up to about 4 000 m^3/s, at which time it is swept out. Once the dissipater becomes ineffective it is expected to fail, leading to the failure of the chute. Back cutting would then occur rapidly along the chute due to the weathered materials beneath the chute at the downstream end.

Plucking of Slabs – With the chute joint sealant in very poor condition uplift pressures would be transmitted to the underside of the slabs. With very high transient pressures being developed down the chute it is likely that the slabs would start being lifted off at flows of 1 000 m^3/s to 2 000 m^3/s. For the purpose of the risk assessment it was assumed that all slabs are equally likely to be removed. Once the slabs are removed the foundation would be eroded.

Failure of the Side Walls – The chute walls are in poor condition, under designed by current standards and their stability with flood flows in the chute relies on the backfill behind the wall. The walls would be first overtopped at about 3 000 m^3/s by the occasional surge and would be continuously overtopped once the flow reaches about 5 000 m^3/s. This overtopping would rapidly wash out the granular backfill behind the walls causing them to fail. Once wall failure occurs the chute slabs would be undercut from flow along the side of the chute, causing the side slabs to be removed and subsequent erosion of the foundation.

Given that failure of the chute slabs occurs, the resulting exposure of the foundation would cause erosion to commence in both the downward and back cutting modes. The back cutting would cause the slab immediately upstream to be undercut to the point that it fails leading to the progressive failure to the point that the toe of the spillway structure is exposed, with further down and back cutting leading to spillway structure failure.

5.3 Erosion of the Chute Foundation

A number of methods have been developed for the analysis of erosion of the spillway structure. For the Eildon Dam spillway the likelihood of erosion was first assessed using the van Schalkwyk method (van Schalkwyk 1995). This method indicated that excessive erosion was likely. Further analysis was then undertaken using the "Erodability Index Method" proposed by Annandale (1995). This approach is based on the correlation of the rate of energy dissipation and the earth or rock mass erodability. Turbulence in the water flow is associated with energy loss and Annandale observed that this turbulence at the boundary will produce pressure fluctuations which could initiate erosion by causing jacking resulting in a unit of material being dislodged and then displaced. He stated that the correlation between the rate of energy loss and the resistance to erosion of a material can be expressed as the function:

$P = f(K_h)$ at the threshold of erodability. Equation 1

Where P = rate of energy dissipation (W/m^2)

and K_h = Erodibility Index

Annandale selected the Kirsten Ripability Index (Kirsten 1982) as the measure for the erodability of the material, although renamed and redefined it as the "erodability index", K_{Hz} . If $P < f(K_h)$ then erosion will probably not occur and if $P > f(K_h)$ then erosion will probably occur.

Based on research by others, Annandale adopted the rate of energy dissipation per unit width of flow as a reasonable indication of the pressure fluctuations caused by the turbulence. To establish a relationship between the rate of energy loss and the erodability, and thus define the erosion threshold, Annandale reviewed 150 case studies considering both erosion and no erosion cases. Based on analysis of data from various sources, Annandale's criterion for non-erosion is for the rate of energy dissipation, or stream power, P, to be below the value of the erodability index at a given point. The general form of the stream power per unit area is expressed as follows:

$P = \gamma.q.(\Delta E/\Delta L)$ Equation 2

Where:

γ is the unit weight of water (9810 N/m^3)

q is the unit discharge ($m^3/s/m$ width)

$\Delta E/\Delta L$ is the energy loss per unit length of flow (along the jet) (m/m)

and P has the units W/m^2.

The stream power per unit area, based on Annandale's analysis of data from many spillways, should fall in the region depicted by the inequality Equation 4 in order to avoid erosion:

$P < 0.95\ (EI)^{0.76}$ Equation 3

or, for all practical purposes,

$P < (EI)^{0.75}$ Equation 4

Annandale (1995) has developed a number of solutions to the energy loss at critical points such as head cuts, hydraulic jumps, changes in bed slope and for open channel flow. Although at the beginning of the erosion process any of these forms of energy loss could be the dominant one, it is expected that at Eildon the relatively weak foundation will commence to erode at which time the head cutting process will dominate. Using the procedures outlined by Annandale (1995) and further described in Wark, Granger, Lesleighter and Buchanan (2004), who incorporated work by Bollaert (2002) on jet dispersion, the unit power required for down cutting and for back cutting to occur can be determined and compared with the power available in the jet.

5.4 Assessment by the Annandale Method

The extent of possible erosion was assessed at four locations along the length of the spillway chute, coinciding where rock cores were taken from the foundation. These locations are the toe of the spillway structure, the mid point, 75% point and downstream end of the chute. The summary of typical values of the Erosion Index determined for the Eildon dam chute foundation are given in Table 1.

With the chute slabs still in place the rate of energy dissipation due to the water flowing down the chute varies from about 40 kW/m^2 to 80 kW/m^2 and is generally less than 100 kW/m^2. However if a chute slab is lost, then the energy available for down cutting and back cutting must be evaluated. For a given assumed step height, the analysis checks the total energy available and the energy available for down cutting and back cutting. If the resistance to either of these erosive processes is less than the applied energy, then down cutting and / or back cutting would occur. This process continues until the erosion reaches a stratum of foundation rock which has greater resistance to erosion than the available energy. The back cutting and down cutting may not necessarily cease at the same time. If a suitably erosion resistant bed is encountered to prevent down cutting it may still be possible for back cutting to occur on weaker beds above the stronger bed.

5.5 Discussion of Results

Given that the structure of the rock beneath the chute is primarily near-vertically bedded, weathering and alteration are the main controls to the erodability of these rocks. At the bottom end of the chute, the degree of alteration is quite significant at depth and only in the middle of the chute does this alteration reduce sufficiently to enable some indication that the fresher rock would be more resistant to erosion. There is however theoretically enough energy available for head cutting towards the main spillway crest structure to occur. The eroded surface that is likely to be generated will be rough and variable as it crosses over the different layers of vertically structured beds.

At the toe of the spillway crest structure the condition of the rock materials improves considerably. This improvement is confined largely to the region upstream of a fault that crosses the spillway about 100 m downstream of the spillway structure. Although there is potentially sufficient energy available to cause both some back cutting and down cutting, the rocks themselves are fresh to slightly weathered and rated as very high strength rock and will be relatively resistant to the eroding forces. The main defects are the lack of strength across the joints and the bedding. As shown in Table 1, it is predicted that at the toe of the dam, while there is enough energy available to enable down cutting to continue, the rock would be resistant to back cutting. This result suggests that provided suitable protection to the rock can be maintained, there is adequate resistance in the rock mass to limit the extent of back cutting that is likely to occur. Further down the chute, the loss of the chute slabs is highly likely to initiate both down and back cutting to the area at the toe of the spillway crest structure.

Table 1. Comparison of Average Power and Available Energy for Eildon Spillway

	Power Available to Erode kW/m^2		Power Required to Erode Rock kW/m^2			
Depth	Down cut	Back cut	Toe of Spillway	50% down chute	75% down chute	100% down chute
0 to 5	800	350	765	121	8	1
5 to 10	2000	350 to 250	1147	24	32	14
10 to 20	3100	250 – 230	1423	253	99	16
20 to 30	3600	230- 230	2448	1682	451	31

This analysis confirmed the design strategy of anchoring the chute, with anchorage zones below a depth at which no back cutting is expected to occur.

One of the key variables to be considered was the potential for the development of a water cushion at the base of the jet. At Eildon the downstream tailwater depths were unlikely to be high enough to provide any significant cushioning. However depths of as little as 3 m at Eildon would have been enough to start to see some effect from a water cushion in the plunge pool. As the depth of the plunge pool increases the unit power available for down cutting drops rapidly. In other situations this effect could be significant.

6. REMEDIAL WORKS

6.1 Design Principles

Given the design approach adopted where major damage caused by a extreme flood event is acceptable provided the dam does not fail, it was therefore accepted that it was not necessary to protect the full length of the spillway chute from failure and all that was required was to protect an adequate length of the chute to sufficiently retard the back cutting to protect the spillway structure.

A remedial works strategy was therefore required to produce works that improve the effective strength of the rock across the joints. The design adopted uses a system of vertical and angled post tensioned anchors to tie the rock mass into a unit. Based on the analysis undertaken the maximum anchor depth required is 20 m with an anchor zone below that depth. This depth gives some leeway for providing protection to unexpected weaker zones that may occur.

The vertical anchors are arranged on a 4 m grid for the full width of the chute to about 40 m downstream of the spillway structure, with two rows of inclined anchors from the downstream end of the anchored zone. These inclined anchors, which will be roughly normal to the bedding of the rock, have the dual benefit of tying the rock mass and securing the concrete chute slabs.

In addition to the anchoring, the construction joints between the anchored slabs have been modified to prevent high pressures being transmitted to the underside of the slabs and the chute walls adjacent to the anchored zone have been replaced with more robust mass concrete walls. Although only the same height as the existing walls, the new walls are highly unlikely to fail during a flood, the mass concrete heel should resist erosion if the walls be overtopped and the walls can be easily raised at a later date if required.

Although not providing any resistance to erosion, the opportunity has been taken to install a toe drain at the toe of the spillway to ensure that pore pressures below the gravity structure remain well controlled.

6.2 Staged Project Approval

Due to delays caused by the complexity of the full spillway upgrade design, G-MW chose to approve the project in stages. This approach was easily accommodated by the alliance process. The AUD14.8 million initial stage of the project encompassed embankment reconstruction to incorporate modern filters in the upper 10 metres of the embankment that, for public safety reasons, had to be undertaken while the reservoir level was low.

Staged approval allowed time for critical works to commence while the difficult design issues relating to the spillway were resolved and costs estimated.

Because indicative cost estimates for the standards-based solution were well in excess of the originally expected project cost, it was also decided to undertake a risk assessment to determine if there was a lesser scope of work that would achieve a reasonable level of residual risk (the staged risk reduction upgrade).

6.3 Use of Risk Assessment

Whilst risk assessment for dams is widely used in Australia and in some other countries, it remains a developing process within the dams industry and is not fully accepted as a mature technique by all practitioners. For these reasons, G-MW has a policy that risk assessment should not yet be used to 'sign-off' on the ultimate safety of a dam, but can be used for determining works priorities and interim risk reduction measures.

Therefore, to comply with G-MW policy, if a staged risk reduction upgrade (Stage 2) were adopted for Eildon dam, further work (Stage 3 of the project) would be required in the future to ultimately achieve the standards-based upgrade. The timing of Stage 3 work would be determined by the level of residual risk achieved by the risk reduction upgrade and risk priorities at other dams.

The standards-based upgrade would include all works necessary to ensure that the dam should survive a Probable Maximum Flood event occurring when the storage is at full supply level, although it was accepted that the spillway would be very badly damaged. Damage to parts of the dam, providing that the primary water retaining structures do not fail, is usually considered acceptable for very rare flood events.

The staged risk-reduction upgrade would include all works necessary to provide sufficient flood capacity so that public risk is reduced to a level that should be tolerable in the medium to long term, say for ten to fifteen years.

6.4 Risk Assessment Results

Figure 3 is a graph of estimated risk reduction that would be achieved as each element of the work is constructed. The upper horizontal line represents the first risk target used by the G-MW to prioritise dam safety works. It represents the limit of tolerability suggested by the Australian National Committee on Large Dams (ANCOLD) for existing dams. The lower horizontal line represents one order of magnitude less risk and is used as a guide for medium to long term staged risk reduction projects.

6.5 Agreed Scope of Works

The works approved for construction at an estimated project cost of AUD52.5 million works to reduce the risk were:-

- Raising of the embankment by approximately 5 m to increase the flood storage
- Inclusion of filters in the embankment down to a level approximately 17 m below the raised crest level
- Strengthening the spillway chute immediately downstream of the spillway structure to reduce the risk of erosion undercutting the structure
- Refurbishment of the gate mechanical components.

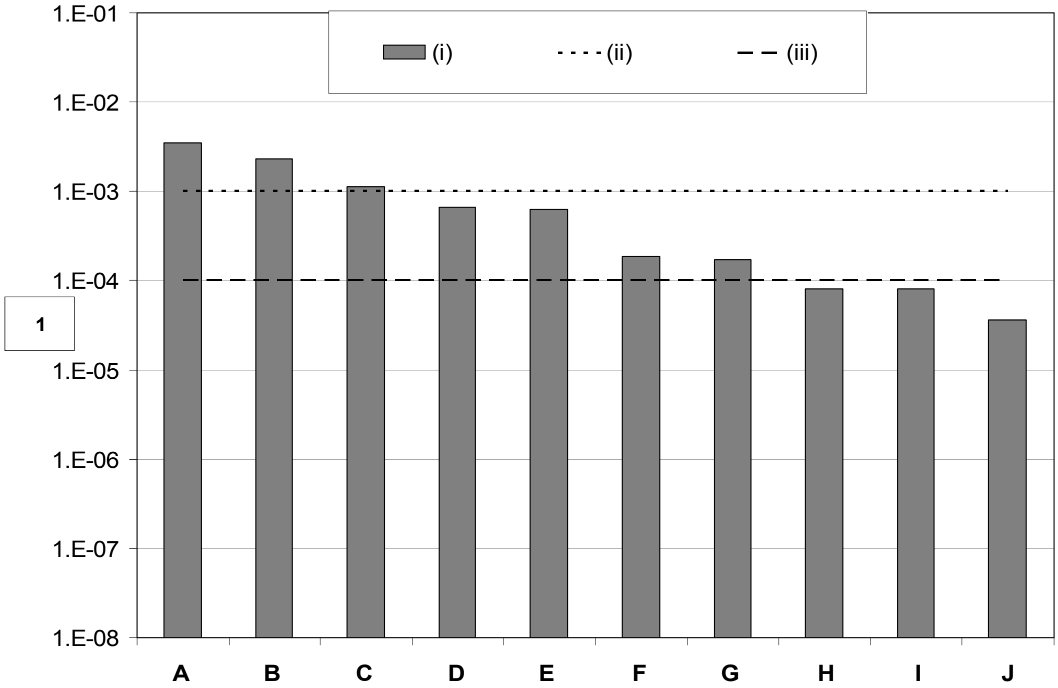

Figure 3
Risk Reduction Achieved by Work Elements
(Réduction des Risques atteindu par les éléments des travaux)

i	Total Societal Risk Value	*i*	*La valeur des risques à la société*
ii	First Interim Target	*ii*	*Premier objectif d'entre-temps*
iii	Second Interim Target	*iii*	*Deuxième objectif d'entre-temps*
1	Societal Risk Value	*1*	*La valeur des risques à la société*
A	Existing Dam	*A*	*Le barrage actuel*
B	Filters, Crest RL291.65 (existing)	*B*	*Filtres, Crête RL291.65 (actuel)*
C	Filters, Crest RL294	*C*	*Filtres, Crête RL294*
D	Filters, Crest & Parapet RL296.9	*D*	*Filtres, Crête et Parapet RL296.9*
E	Chute Joint Repairs	*E*	*Réparation du joint du coursier*
F	Strengthen Toe Slabs & Walls	*F*	*Renforcez les dalles et les murs du pied aval*
G	Upgrade Gate Operating Gear	*G*	*Améliorez l'équipment d'exploitation de la vanne*
H	Anchor Spillway Structure	*H*	*Installez le tirant de l'evacuateur de crue*
I	Downstream Training Walls	*I*	*Les murs guideaux d'aval*
J	Raise Bridge, Piers & New Operating Gear	*J*	*Levez le pont, les piles et le nouvel équipement d'exploitation*

Future works to bring the dam to the standards based fix are:

- Post tension anchoring of the spillway structure
- Replace and relocate the spillway gate hoists to allow a greater gate opening
- Modify the spillway bridge to allow the PMF flow to pass
- Raise the overflow structure training walls and spillway chute walls to contain the estimated peak discharge.

6.6 Risk and Costs

Once the design for the spillway had reached the stage where a target cost estimate could be prepared for both the standards-based and staged risk reduction upgrade options, it was then possible to compare the cost of risk reduction achieved by each part of the works.

Figure 4 is a graph of percentage risk reduction achieved as each element of the work is constructed, along with the cumulative construction cost.

The agreed estimated cost to achieve the standards-based upgrade and allow the dam to safely pass the PMF (with substantial damage to the spillway chute) and cope with the Maximum Design Earthquake was AUD70.8 million

6.7 Risk Reduction Upgrade

The review of risk reduction versus costs in Figure 4 indicated that the increments of risk reduction to be achieved by constructing additional works beyond the strengthening of the spillway toe slabs and chute walls were very small in comparison with costs of the additional works. However, the next works item to upgrade the operating gear was thought to be worthwhile as the operating gear was 50 years old and although significant risk reduction would not be achieved by upgrading, good practice indicated that the work should be considered.

The review of risk reduction in relation to the target levels of risk in Figure 4 indicated that the first risk target could be achieved by works to the main embankment only. However, this would only be by a small margin.

An additional consideration was the issue of the magnitude of improvement to the dam's flood capacity. If all work up to and including upgrade of the spillway operating gear were undertaken, then the following improvements in flood capacity relative to the PMF would be achieved:

• Current dam flood capacity	3,500 m^3/s	50% of PMF
• Staged risk reduction flood capacity	5,000 m^3/s	70% of PMF
• Standards-based flood capacity	7,050 m^3/s	100% of PMF

Table 2 shows the improvements in the annual exceedence probability of the dam failure flood.

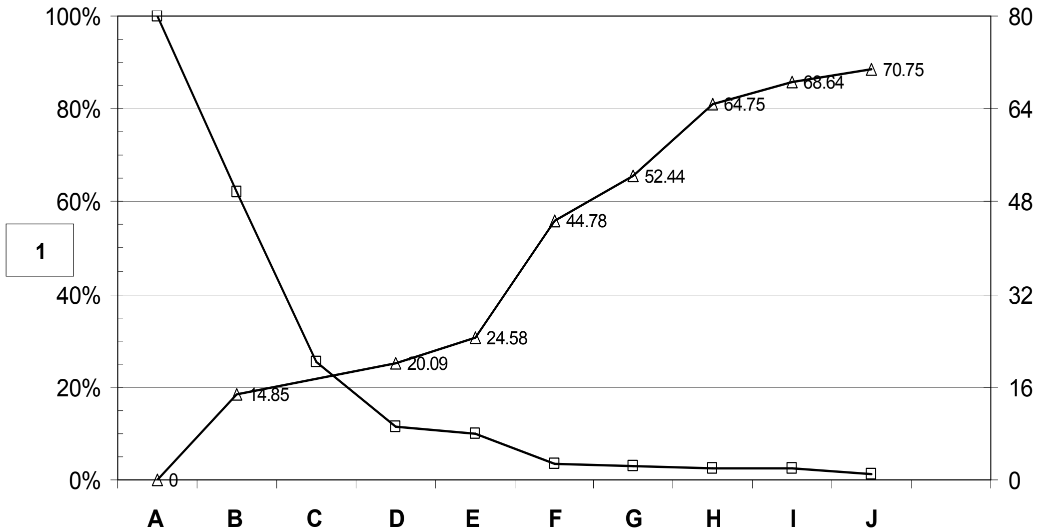

Figure 4
Risk Reduction vs Cost
(Réduction des Risques contre Coût)

1	Benefit (Percent of Original Risk)	*1*	*Actif (Pourcentage des risques originals)*
A	Existing Dam	*A*	*Le Barrage Actuel*
B	Filters, Crest RL291.65 (existing)	*B*	*Filtres, Crête RL291.65 (Actuel)*
C	Filters, Crest RL294	*C*	*Filtres, Crête RL294*
D	Filters, Crest & Parapet RL296.9	*D*	*Filtres, Crête et Parapet RL296.9*
E	Chute Joint Repairs	*E*	*Réparation du joint du coursier*
F	Strengthen Toe Slabs & Walls	*F*	*Renforcez les dalles et les murs du pied aval*
G	Upgrade Gate Operating Gear	*G*	*Aléliorez l'équipment d'exploitation de la vanne*
H	Anchor Spillway Structure	*H*	*Installez le tyrant de l'evacuateur de crue*
I	Downstream Training Walls	*I*	*Les murs guideaux d'aval*
J	Raise Bridge, Piers & New Operating Gear	*J*	*Levez le pont, les piles et le nouvel equipment d'exploitation*

Table 2 – Flood Capacity Probabilities

Flood Capacity Probabilities		
Dam Condition	Flood Capacity Annual Probability (variable reservoir level)	Flood Capacity Annual Probability (dam full)
Current	1: 100,000	1: 13,000
Stage 2 works	1: 500,000	1: 100,000

In considering all of the relevant data, G-MW decided to upgrade Eildon Dam by undertaking all work packages up to and including the spillway operating gear. This option provides for works that will allow the dam to pass 70% of the probable maximum flood, at a total project cost of AUD52.5 million. The estimated residual risk at completion of works is considered to be low enough that many other dam safety priorities within Goulburn-Murray Water's portfolio of dams would take precedence over Eildon and it is unlikely that further work would be justified in the medium term, say for ten to fifteen years

7 CONTRIBUTIONS OF THE PROJECT ALLIANCE MODEL

7.1 Project Outcomes

The success of the Eildon Dam Improvement Project was measured by a number of criteria including cost, time, safety record, effects on the environment, benefit to the community, security of asset, operational and documentation issues, experience gained by G-MW staff and whole of life cost. These criteria are detailed and weighted in an agreed procedure and measured quarterly throughout the project.

Based on benchmarks that have been established on similar projects the performance criteria are scored on a 0 to 100 scale, with 50% being the "business as usual" score; that is, the normal outcome given a competent and experienced project team.

The project achieved a performance score of over 80% on these criteria. The score is well above "business as usual" due to:

- An excellent safety record (only one lost time injury in over 180,000 hours worked),
- Good environmental performance,
- A healthy relationship with the local community,
- Project completion three months ahead of program, and
- Cost savings of over AUD5 million on the overall project

This success can be attributed to the benefits of the project alliance project delivery model which brought all parties together with a common goal, encouraged and rewarded innovation throughout the project and ensured that when technical issues arose they were resolved quickly and in the most cost effective manner for the project as a whole.

7.2 Benefits of a Project Alliance

The key to these excellent outcomes was teamwork. By bringing experienced construction people into the design process it was possible to eliminate costly, unsafe difficult approaches to problems before much design effort had been expended. A productive team was quickly in place and construction activity commenced less than eight months after the design team started preliminary designs and the spillway model study.

The collaborative approach that was developed in the design process continued through the construction phase where significant savings were generated. For example close liaison between design and construction personnel enabled savings of over AUD2 million to be made on the spillway anchoring contract. This saving was generated by refining the design to better suit the drilling contractor's equipment without compromising the design intent.

However the benefits provided by the alliance approach went beyond simple cost savings. By working closely with G-MW the Eildon Alliance was able to deliver added functionality such as maintaining public facilities that were scheduled for demolition, rehabilitation of previously degraded vegetation areas, reduced traffic impacts on the community during construction and improved facilities for the ongoing operational staff.

However the largest contribution to the success of the project was the avoidance of disputes through the construction phase. By taking on all risk during construction, including latent conditions, the Eildon Alliance was responsible to quickly solve problems as they arose without the ability to claim for the cost of these problems.

During the embankment works it was found that the original embankment had not been constructed as shown on the drawings, causing additional rockfill to be placed and requiring extensive repair work. In a normal contract situation the contractor would have charged heavily for these variations. In this case the Eildon Alliance team responded by reprogramming the works, immediately addressing these problems and getting on with the project. The outcome was that minimal time was lost and the overall project remained on target.

8. CONCLUSIONS

The project alliance delivery method facilitated innovative approaches that provided substantial cost and time savings to the upgrade of the Eildon dam. These savings were shared by the owner, designer and the constructors. The benefits were achieved through a fully integrated team that worked together to solve technical design issues, develop enhancements and optimise construction.

Specifically the Eildon dam upgrade project has included:

- An alliance project delivery model bringing owner, designer and constructors together as a single entity sharing project risks.
- The use of risk assessment to determine a scope of work that achieved a reasonable level of residual risk for the medium term and freeing up scarce resources to deal with high risks at other dams.
- A significant extension to the Annandale analytical approach to calculating the extent of spillway erosion that gave the owner confidence to allow the design effort to concentrate on a cost-effective means of securing the safety of the spillway block.

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SUMMARY

Eildon dam is an 80 metre high zoned earth and rockfill embankment. It has a 40 metre high mass concrete spillway structure with a 435 m long concrete lined spillway chute. The dam is 50 years old. Safety evaluations of Eildon Dam identified unusual embankment performance and deficient filters. Its spillway had inadequate flood capacity and doubtful structural performance for floods and earthquake.

The initial scope of the project had been to design and construct a "standards based" solution to these problems. However, as the design and construction planning progressed and the likely cost of the work required increased, it became apparent that the original budget expectations would not be met.

This paper describes a cooperative project delivery model that assisted the dam owner, the project designer and constructors to develop a cost effective spillway upgrade. The paper also details the use of risk based techniques and applies earlier work by Annandale and Bollaert to quantify the potential erosion at the rock beneath the spillway chute.

RÉSUMÉ

À une hauteur de 80m, le barrage d'Eildon a un digue en zoné de terre et d'enrochement. Il a un coursier de l'evacuateur de crue de 435m avec un revêtement en béton. Le barrage a 50 ans.

Des évaluations de sécurité du barrage d'Eildon ont identifié un fonctionment hors d'ordinaire du digue et des filtres défectueux . L'evacuateur de crue avait une capacité insuffisantée et une performance d'ouvrage discutable pendant des crues et des séismes.

Au début du projet, on a décidé d'étudier et de construire une solution fondée

sur les niveaux pour ces problèmes. Cependant à mesure que la préparation de l'étude et de la construction avancaient et le coût probable des travaux demandés augmentait, on a aperçu qu'il serait impossible de satisfaire les espérances du budget original.

Ce papier décrit un modèle coopératif du projet realisé qui a aidé le maître d'ouvrage, le concepteur de projet et les constructeurs à développer une amélioration économique de la capacité de l'evacuateur de crue. Le papier raconte en détail l'utilisation des techniques fondées sur les risques et il applique les travaux plus antérieurs d'Annadale et Bollaert pour évaluer l'erosion potentiel du rocher sous le coursier de l'evacuateur de crue.

REFERENCE CARD

1 EILDON PROJECT ALLIANCE DELIVERS COST EFFECTIVE SPILLWAY CAPACITY UPGRADE	2 2006 3 4
6 Anchorage, Concrete Dam, **Construction Methods**, **Design**, Design Flood, **Erosion**, Fissured Rock, **Risk Analysis**, **Safety of Dams, Scour Protection, Spillway, Stability,** Weathered Rock	5 English
7 This paper describes how the Eildon dam spillway was upgraded in a cost effective way through the use of an alliance project delivery method, risk assessment and a new approach to the calculation of the potential extent of spillway erosion caused by backward erosion.	
8 Eildon	
9 McGrath S., Buchanan P., Wark R., Fox S., Waller J. (Australia)	
10 (L'Enterprise d'Alliance d'Eildon remporte une amélioration économique de la capacité de l'evacuateur de crue)	
11 ICOLD 22, Barcelona 2006	

Item 6:
Tirant, Barrage en béton, Méthodes de Construction, Étude, Crue de Project, Erosion, Rocher Fracturé, Calcul des Risques, Sécurité des Barrages, Protection contre Chasse, Evacuateur de crue, Stabilité, Rocher altéré.

Item 7:
Ce rapport décrit l'amélioration économique de l'evacuateur de crue du barrage d'Eildon. On a utilisé une enterprise d'alliance, une évaluation des risques et une nouvelle approche au calcul de la mesure potentielle de l'erosion de l'evacuateur de crue à cause de l'erosion en rétro.