Quipolly Dam safety upgrade

Peter A. Ballantine
Dams Service Group Manager, GHD, NSW

In view of the need for a safety upgrade for the Quipolly Dam and the plans of Liverpool Plains Shire Council for future growth in water supply, the Council took the opportunity to increase the storage capacity of the dam by raising the full supply level by 2.0 m. In 2009, the Council appointed GHD to design the upgrade in accordance with the ALARP principle.

The design of the dam included an innovative vertical crest wall, embedded into the embankment, a concrete-lined auxiliary spillway placed over the embankment adjacent to the existing spillway and the installation of Hydroplus Fusegates in the existing spillway channel.

This paper describes the design of the upgrade works.

Keywords: *ALARP, crest wall, safety upgrade, increase spillway capacity, societal risk, Hydroplus Fusegate.*

Background

Liverpool Plains Shire Council (LPSC), which was formed in 2004 through the amalgamation of portions of three existing shires, inherited Quipolly Dam. Soon after amalgamation, LPSC received communication from the NSW Dams Safety Committee (DSC), making them aware of the need to upgrade the safety of Quipolly Dam, which is a prescribed dam with a High C Consequence Category classification. The dam, which is located about 100 km south east of Tamworth and 10 km north of Quirindi in New South Wales, is a source for the supply of potable water to consumers.

LPSC implemented a dam safety upgrade programme, which included the following principle activities:

- Commissioning of a Risk Assessment and Options Study (2006).
- Appointment of a Consultant (2009) to review available upgrade options and undertake detailed design for the upgrade of the dam, including a storage augmentation by increasing the full supply level by a nominated 2.0 m.
- Appointment of a contractor to construct the upgrade works (2012).

Risk assessment (2006)

The risk assessment (URS 2006) identified the following principle deficiencies for Quipolly Dam (Figure 1):

- Potential piping through the upper section of the dam embankment due to drying out and associated micro-cracking (found in the upper 1.5 to 2.0 m) of the earthfill (the cracking was found in the core, but due to the similar nature of the materials, it was assumed that cracking could occur across the full dam crest width);
- Potential piping through the embankment alongside the vertical concrete spillway chute retaining wall;
- Inadequate spillway capacity (the dam crest flood had an estimated 1:800 Annual Exceedance Probability, or AEP).

The total maximum individual risk to persons downstream of the dam was estimated at 2.3 x 10^{-4}, which was higher than the ANCOLD 2003 criterion, which required the risk to be less than 1 x10^{-4}.

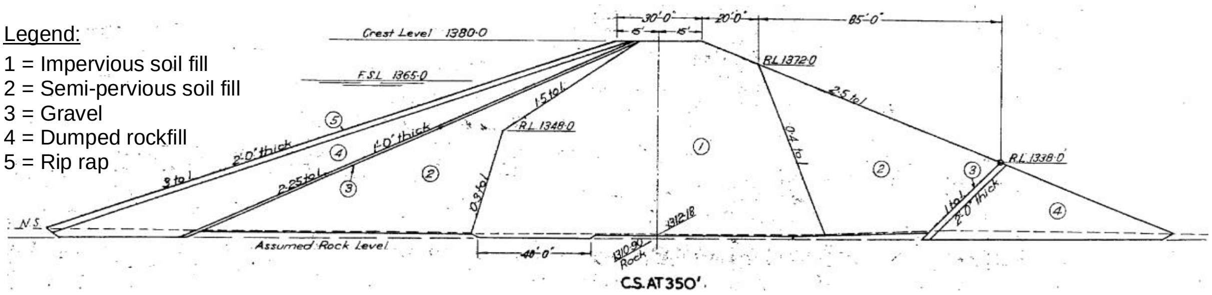

Figure 1. Original embankment profile

The Risk Assessment recommended the implementation of measures to reduce the principle risks identified so that remaining societal risks were acceptably low.

Scope of work for design of dam upgrade works

The scope of work for the detailed design stage consisted of the following:

- Improvement of the spillway capacity;
- Implement measures to reduce the risk of piping through the upper sections of the embankment and adjacent to the spillway chute retaining wall;
- Review of the hydrology, in line with a recommendation made in the Safety Review Report, in particular to review the unusually low RORB (Monash University and Sinclair Knight Merz software) calibration factor k_c (a k_c factor of 1.6 was used in comparison to the ARR (Engineers Australia 2001) recommended k_c factor of between 6.7 and 10.2), to determine whether the design flood could be reduced;
- Investigate whether the reservoir storage capacity could be increased by 2.0 m at minimal additional cost to the safety upgrade;
- Replace the wooden piers on the bridge to the intake tower;
- Improve the Occupational Health and Safety (OHS) conditions at the intake tower, including replacing internal ladders, minor upgrade of valves and pipework and providing a working platform at the top of the intake tower;
- Assist LPSC in the environmental approvals process and to conduct a public awareness campaign to inform the community of the risks posed by the dam and explain the proposed upgrade works.

In order to limit initial expenditure, the dam safety upgrade works were to be implemented in terms of ANCOLD and DSC "ALARP" (as low as reasonably practicable) principles.

Concept design stage

A concept design study was undertaken to identify options to address the principle dam safety concerns, i.e., piping risk and substandard spillway capacity and to provide storage augmentation. The hydrology review was undertaken before commencing with the Options Study.

Comparative costs for dam upgrade without increased storage capacity and dam upgrade options, including a 2.0 m increase in full supply level, were developed.

Hydrology review

Based on a calibration of the RORB model against actual flood events, a k_c factor of 4.0 was recommended. This reduced the design floods slightly compared to the floods used at the time of the Safety Review, but the reduction was not as significant as was hoped.

Improvement of spillway capacity

In terms of the ANCOLD guidelines and NSW Dams Safety Committee (DSC) requirements for a High C Consequence Category Dam, Quipolly Dam was required to pass a design flood with an annual exceedence probability (AEP) of 1:100,000. However, the DSC was prepared to accept phased upgrades over a period to be agreed with the Dam Owner (e.g., 10 to 20 years). Dam Safety upgrade options evaluated (excluding the provision of additional storage) included the following:

- Raise the embankment crest without changing the spillway configuration;
- Increase the spillway crest length without raising the embankment crest;
- Increase the spillway crest length by excavating into the left spillway abutment, associated with embankment crest raising;
- Improve spillway efficiency by deepening the spillway channel and installing an ogee profiled spillway, associated with embankment crest raising;
- Provide a second spillway on the right abutment, associated with embankment crest raising.

Upgrade options for raising the full supply level included the following:

- Similar options to those listed above;
- Installation of Hydroplus Fusegate units, associated with raising of the embankment crest.

Raising of embankment crest

As indicated above, raising of the embankment crest was required for most spillway upgrade options considered. Options for raising of the embankment crest included:

- Raising of the embankment crest by placing additional earthfill, by locally steepening the upstream and downstream slopes;
- “Reinforced earth” structures on the crest of the embankment;
- Reinforced concrete cantilevered crest wall;
- Various combinations of the above, associated with buttressing of the downstream embankment by the placement of additional fill, if required for stability.

The following challenges resulted from the raising of the embankment crest:

- Stability criteria limited the height to which the embankment could be raised without buttressing of the downstream face;
- Increasing the embankment height with earthfill subjected the spillway chute retaining wall to additional load, which resulted in unacceptable stability of the spillway chute retaining wall.

Options considered to improve the stability of the mass concrete spillway chute retaining wall included replacement of the wall (with associated concerns regarding erosion of the embankment should a flood be experienced before the concrete wall was constructed), installation of stressed anchors to anchor of the wall, installation of piles or caissons and the provision of a set-back retaining wall (associated with a concrete-lined section over the crest of the embankment) on the embankment to ensure that the load from the raised embankment did not impact the existing spillway chute retaining wall.

Reduction of piping risk

The following options to reduce the risk of piping through the embankment were considered:

- Installation of a 300 mm to 500 mm wide sand filter on the downstream side of the core within the upper 3 m of the existing embankment crest to intercept and filter seepage through micro-cracks;
- Installation of a sand filter, supported by fill on the downstream side of the existing embankment, for options which required buttressing of the downstream side of the embankment;
- Installation of a reinforced concrete or bentonite "cut-off" wall through the upper 3 m of the embankment crest, to prevent flow through the zone of micro-cracking.

The following options to reduce piping risk at the embankment/spillway chute retaining wall interface were considered;

- Installation of a filter, approximately 1.0 m deep directly adjacent to the retaining wall from the crest to the toe of the embankment;
- Installation of a full-depth concrete or bentonite cut-off wall.

Comparative options

All options proposed in the options study report included raising of the embankment crest. The amount by which the crest was raised varied from 1.75 m to pass a 1:10,000 AEP flood with no increase in the full supply level, to a height of 5.2 m for an increase in storage level of 2.0 m with a fixed spillway, which would pass a 1:100,000 AEP flood.

ALARP and CSSL evaluation

Comparative cost estimates for options were prepared. The reduction in the risk to the safety of persons downstream of the dam was evaluated and the "cost to save a statistical life" (CSSL) was determined in accordance with ANCOLD and DSC guidelines. The evaluation concluded that the CSSL for an upgrade option which could pass a 1:10,000 AEP flood (without increasing the full supply level of the dam), at an estimated construction cost of $2.86 million, had a CSSL of approximately $115 million. On the basis of the ANCOLD guideline (Ref 2), it was concluded that this cost was only marginally to poorly justified. The level of residual risk after implementation of this option would be nearly one tenth of the ANCOLD limit of tolerability values for existing dams and the level of flood capability would fall within the range of the ANCOLD fall-back criteria for an existing dam with a High C Consequence Category.

The DSC is prepared to consider a staged upgrade approach to achieve safety upgrade objectives (e.g., the fall-back design flood) by the implementation of successive upgrades. The 1:10,000 AEP flood potentially satisfies the ANCOLD (2000) guideline for existing dams, although the DSC fall-back design flood is the 1:100,000 AEP flood (DSC 2010). For this reason, the Options Study Report recommended that a 1:10,000 AEP design flood would be appropriate for the upgrade of the dam.

Review and comment on options by DSC

The report was submitted to DSC by Council, together with a request that the 1:10,000 AEP flood be adopted for the design of the upgrade works.

While the DSC was in general agreement, they requested Council to ensure that the upgrade option implemented should in no way hinder a future upgrade of the spillway capacity to pass the 1:100,000 AEP flood.

Detailed design

In view of DSC requirements, the options were reviewed. The maximum crest raise that could be achieved without buttressing of the dam was re-evaluated. It was concluded that the crest could be raised sufficiently by a combination of steepening of the embankment slopes locally and the construction of an embedded reinforced concrete crest wall, associated with the installation of Hydroplus Fusegate units to increase storage. The cost difference to construct the crest wall with a height sufficient to pass the routed 1:10,000 AEP flood compared to the 1:100,000 AEP flood was marginal and it was recommended that the 1:100,000 AEP option be implemented, which obviated the need for a phased upgrade.

Increased spillway capacity and storage

In line with requirements of LPSC (to increase available storage to facilitate future development), the storage level was increased by an effective 2.0 m (40% capacity increase) by the installation of 2.6 m high Hydroplus Fusegate units (see Figure 2). The Fusegates were designed to tip at flood AEPs between 800 and 1,000, which LPSC considered to present a reasonable economic risk. The main advantage of the Fusegates is the increased spillway area available after washout, which in turn reduces the total height to which the embankment needs to be raised. New reinforced concrete Fusegate foundations were placed slightly upstream of the original spillway channel crest wall. Steel anchor bars were embedded into the foundation rock to resist horizontal forces imposed by the Fusegates on their foundations. New side walls were provided against which the Fusegate units sealed.

Figure 2. View of upstream side of embankment and Fusegate units in spillway (note extended upstream retaining wall on Auxiliary Spillway)

Raising of embankment crest

The embankment crest was raised by a combination of earthfill and reinforced concrete crest wall. The original crest was approximately 9 m wide. By locally steepening the crest profile, both upstream and downstream, and reducing the crest width to 3.0 m, the earthfill embankment crest was raised by 2.5 m. The embedded crest wall provided an additional 1.65 m, bringing the total height by which the crest was raised to 4.15 m. This was sufficient to pass the 1:100,000 AEP design flood with a freeboard of 0.4 m after washout of the

Fusegate units. Earthfill obtained from local borrowpits was placed on the downstream side of the crest wall, while earthfill supported by filters and rip rap was placed on the upstream side. Details of the crest layout are shown in Figure .

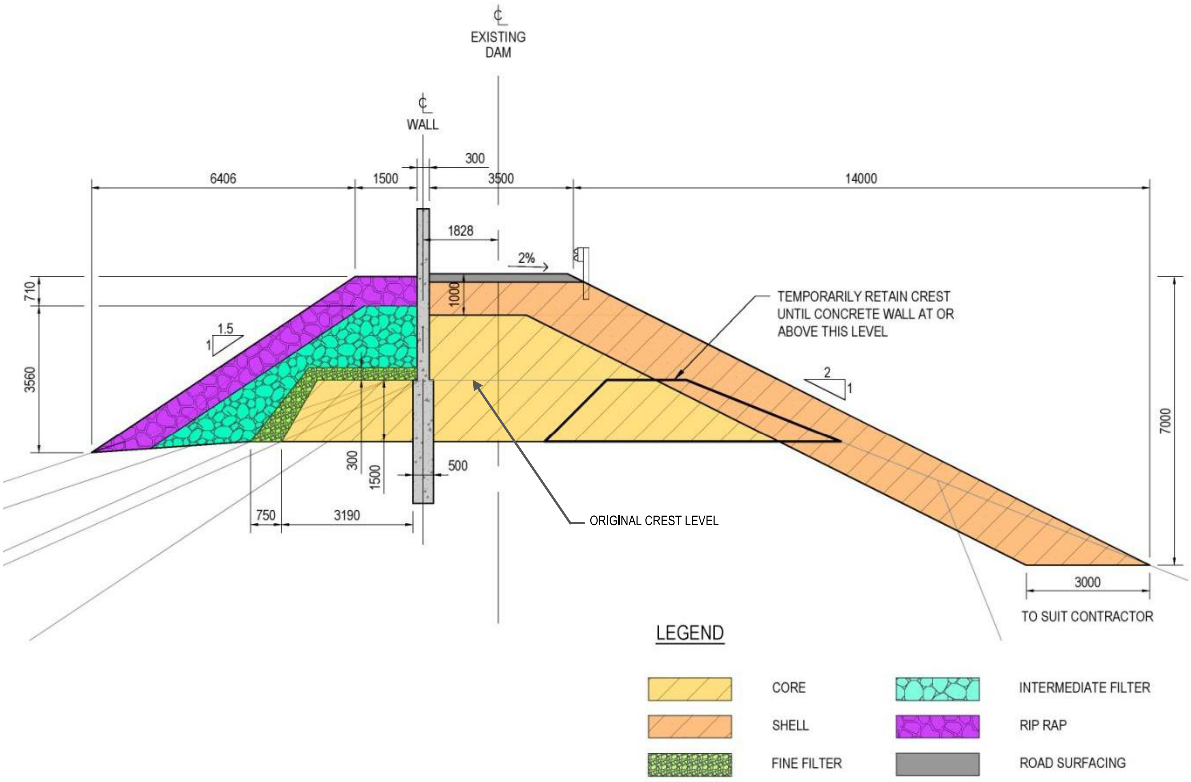

Figure 3. Embankment crest profile

The upper 1.5 m section of the embankment was reconstructed in order to remediate the micro-cracking which was present. After excavation of the upper 1.5 m and before reconstruction thereof, the reinforced concrete crest wall was embedded 1.5 m into the embankment. This effectively provided an impermeable barrier within the upper sections of the embankment where micro-cracking was present and reduced the risk of piping. The details of the crest wall are shown in Figure above (see the Construction Stage section in regard to mitigation of the risk of overtopping during construction).

Stability analyses were undertaken to confirm the factors of safety. The following material properties were used for the embankment materials (Table 1):

Table 1. Embankment material properties

Geotechnical Units/Zones	Unit Weight (kN/m^3)	Friction angle (Ø)	Effective Cohesion (kPa)	Su (kPa) (See Note 1)
Impervious Core	20	30	0	120
Shell	20	30	0	120
Transition Gravel	18	40	0	
Dumped Rockfill / Rip Rap	18	40	0	

Note 1. An undrained strength of 120kPa for the core and shoulder material was estimated based on the stiff consistency likely in compacted clayey fill located at the slip surface. A sensitivity analysis was performed by varying Su values, to check the criticality of the earthquake stability analysis.

The applicable factors of safety, from the software program SlopeW (developed by GEO-SLOPE International Ltd, Canada), were as follows (Table 2):

Table 2. Embankment factors of safety

Load Case	Description	Min Acceptable FOS	Critical FOS (See Note 1)
1	Steady state seepage at FSL 418.1m, functioning toe drainage, (a) downstream slope (b) upstream slope	1.5	(a) 1.73 (b) 1.89
2	Steady state seepage at FSL, partially functioning toe drainage, downstream slope	1.5	1.52
3	Flood at RL 424.4 m, functioning toe drainage, downstream slope. (a) RC wall within slip surface (b) Slip downstream of RC wall	1.3	(a) 1.55 (b) 1.52
4	Rapid drawdown (a) From RL 424.4 m to RL 416.1 m (b) From RL418. 1 m to RL 400.0 m	1.3	(a) 1.49 (Note 2) (b) 1.82
5	Earthquake MDE with FSL, pseudo static analysis, functioning toe drainage. (a) Downstream slope (b) Upstream slope	1.0 (to avoid need for deformation analysis as per ANCOLD (1998))	(a) 1.47 (Peak Su = 120 kPa) (a) 0.99 (Peak Su = 75 kPa) (b) 1.65 (Peak Su = 120 kPa) (b) 1.47 (Peak Su = 75 kPa)

Note 1 : Critical FOS are those slip surfaces that are likely to affect the structural and watertight integrity of the dam.

Note 2: Phreatic surface was conservatively taken as following the upstream face of the rip rap. In reality the water would drain from the rip rap.

Auxiliary spillway

Methods of strengthening the spillway chute retaining wall to carry the loads of the raised embankment section, such as post-tensioned anchors, piling/caissons, reconstruction of the wall, etc., were discounted due to either cost or embankment safety concerns. To limit the loads imposed by the higher embankment fill from being transferred to the spillway chute retaining wall, a new retaining wall to support the fill was placed on the embankment crest at a point at which the influence of loading would not affect the existing spillway chute retaining wall. To protect the embankment between the new retaining wall and the spillway chute retaining wall from being eroded during flooding, a 500 mm thick reinforced concrete lining, which tapered in width towards the downstream, was provided. This essentially serves as an auxiliary spillway and provides limited additional spillway capacity.

To mitigate the risk of piping adjacent to the spillway chute retaining wall, the reinforced concrete crest wall was extended to rock foundation level to limit seepage. In addition, a waterstop was placed between the spillway chute retaining wall and embedded concrete crest wall. The profile of the crest wall adjacent to the spillway chute retaining wall is shown in Figure 4.

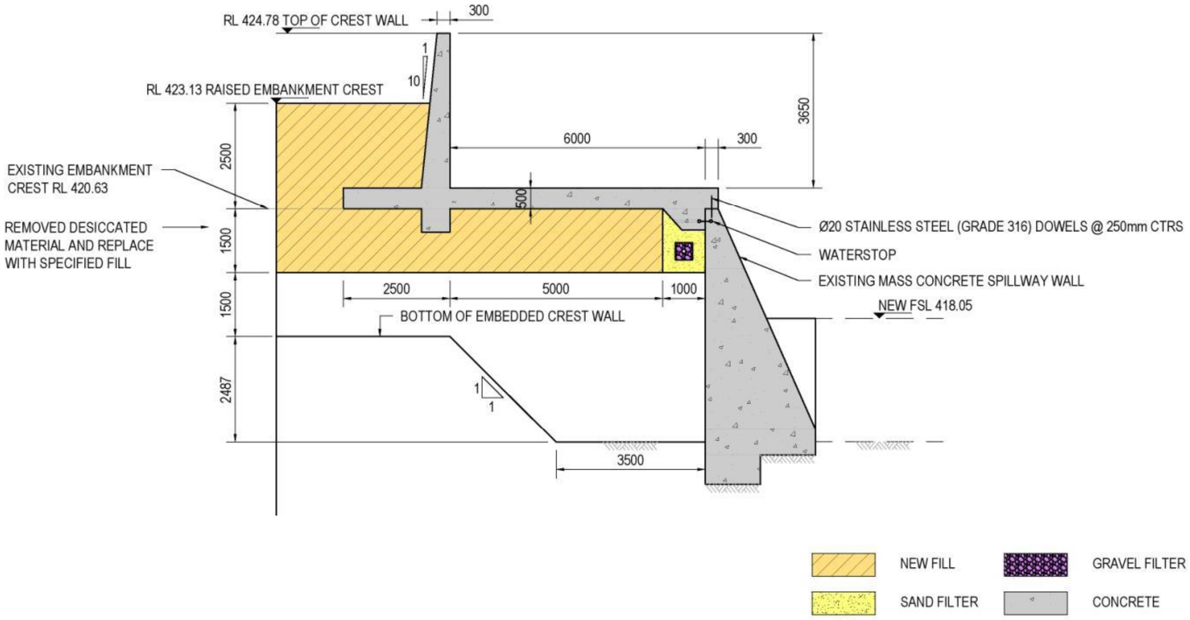

Figure 4. Embedded crest wall adjacent to spillway chute retaining wall

The removal of micro-cracked material in the upper 1.5 m of crest introduced the risk of post-construction settlement of material below the auxiliary spillway lining and separation between the lining and the supporting fill. The low loads imparted to this fill by the concrete was not expected to result in significant settlement. To resist cracking, a 500 mm thick reinforced concrete lining was provided.

To mitigate residual risk of piping associated with seepage, a sand strip drain was placed below the auxiliary spillway lining immediately downstream of the embedded crest wall and at intervals thereafter and a strip drain was placed adjacent to the existing spillway chute retaining wall. The locations of the drains are shown in Figure 5.

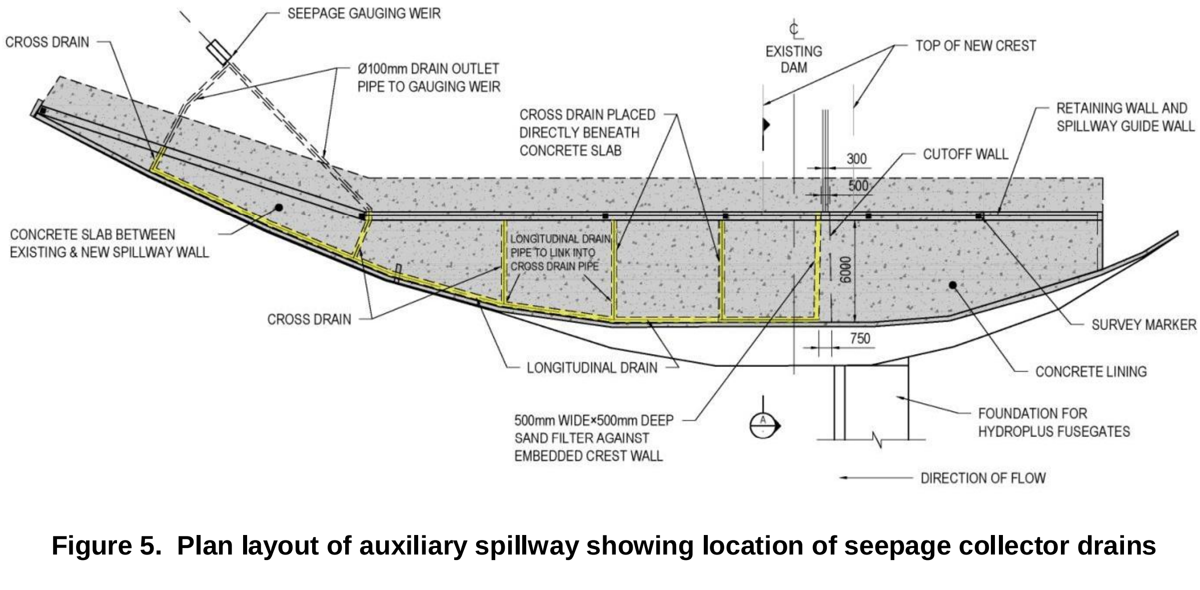

Figure 5. Plan layout of auxiliary spillway showing location of seepage collector drains

The height of the embankment retaining wall was determined by modelling the 1:100,000 AEP flood through the spillway with the software program, HECRAS (US Army Corps of Engineers). The retaining wall was provided to the downstream point of the existing spillway chute retaining wall, which ensured that the embankment works were protected against impact by high velocity water during flood flows through the spillway.

A model study conducted by Hydroplus to test operation of the Fusegate units found that significant flow occurred parallel to the embankment, which resulted in unsteady flow conditions through the spillway. In order to improve flow conditions, the auxiliary spillway retaining wall was extended on the upstream side. This detail can be seen in Figure 2.

Figure 6. Aerial view of completed embankment and spillway

Upgrade of intake tower access bridge and tower platform

The intake tower, which is a wet tower, has a top level equal to the original dam crest. Although the full supply level and dam crest were raised, the top of the intake tower remains above the revised full supply level and can be accessed for maintenance purposes under normal operating conditions. No purpose would be served by gaining access to the intake tower during a flood event. It was therefore recommended that the height of the intake tower not be increased. The top of the tower, however, only had a 2.0 m diameter platform, which was inadequate in terms of OHS requirements. A new steel platform was therefore provided.

The intake tower bridge piers were made of wood, which had become severely weathered. It was therefore decided to replace these wooden piers. Two bridge pier foundations were below full supply level and would have required divers to place new piers. It was therefore decided to support the existing bridge with new beams, which could span this distance and provide only one pier for the bridge (see Figure 6).

Construction stage

To manage the risk of overtopping of the embankment during floods, the Contractor was required to maintain the pre-existing embankment crest level, by retaining a portion of the embankment at original crest level until the new reinforced concrete crest wall was brought up to the same or higher level. In addition, installation of the Fusegates was only permitted once the embankment had been raised to a specified height.

To provide sufficient area for construction of the steepened downstream embankment profile, the Contractor was permitted to excavate a working platform within the embankment, at the point at which the embankment was steepened (refer to note “to suit Contractor” in Figure 3).

No major unforeseen conditions were encountered during construction. An aerial view of the embankment after completion of construction is shown in Figure 6. Figure 2 shows a view of the upstream side of the completed embankment.

Concluding remarks

The Quipolly Dam safety upgrade works have been successfully implemented. Innovative solutions were sought in order to limit costs of the upgrade, while at the same time complying with DSC requirements and accepted stability criteria. In addition, the LPSC took the opportunity to increase the storage capacity of the dam. The upgrade now complies with DSC requirements to limit the societal risks of dam failure to acceptable norms.

The project has raised the profile of Council amongst both the local and wider community and other local Councils, particularly through recognition by the local engineering community for the project. The dam capacity has been increased by 40% to provide water to additional water consumers, which will facilitate continued growth within the local economy.

Dam factual information

Comparative factual information on the dam is given in Table 3.

References

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US Army Corps of Engineers, HECRAS flow modelling software

Table 3. Dam factual information

Item	Original Dam	Upgraded Dam
Year Constructed	1955	2012
Design Engineer	NSW Dept. of Public Works	GHD
Contractor	NSW Dept. of Public Works	Leed Construction/Hydroplus
Dam Type	Zoned Earthfill	Zoned Earthfill with Crest Wall
Maximum Height of Dam	21 m	25.18 m
Length of Embankment	200 m	210 m
Full Supply Level	RL 416.1 m	RL 418.1 m
Dam Crest Level	RL 420.6 m	RL 424.78 m
Length of Spillway	37 m (including bridge piers)	37 m (no bridge piers)
Type of Spillway	37 m long partially lined side canal excavated into rock	37 m excavated side canal & Fusegates (main spillway) 6 m lined spillway on crest of dam (auxiliary spillway)
Maximum discharge capacity of spillway	600 m^3/s	1660 m^3/s (routed 1:100,000 AEP)
Dam consequence category	High C	High C
Storage Capacity	5,300 ML	7,430 ML
Intake Tower	6 foot diameter wet reinforced concrete intake tower with 300 mm diameter pipework and a floating trunnion intake pipe	