COST EFFECTIVE STORAGE CAPACITY INCREASE FOR ALUMINA TAILINGS DISPOSAL AREA THROUGH SPILLWAY OPTIMISATION

Lonie I[1*]

[1]*Tailings and Dams, GHD Brisbane, QLD, Australia*

Abstract

Queensland Alumina Limited operates an alumina refinery located at Parsons Point, Gladstone, Central Queensland. Tailings from the refinery, commonly referred to as red mud, are deposited in two dams, RMD#1 and RMD#2, which have an area of approximately 400Ha and 600Ha respectively.

Continued placement of red mud in RMD#2 had progressively reduced the volume available within the dam. Action needed to be taken to increase storage capacity (based upon water storage and mud storage capacity). Traditionally the volume would be created by raising the embankments, an expensive and time consuming process. An innovative solution was proposed to upgrade the spillway only and avoid the need to raise the embankment.

A workshop was carried out with the stakeholders to assess different spillway options with consideration of spillway performance, downstream impacts, freeboard impact and embankment impacts. A 1.0m high cast in-situ concrete labyrinth spillway, founded on the existing spillway and within the existing spillway walls, was adopted as the final solution. A risk assessment was carried out to verify the acceptability of the spillway raise in accordance with the Australian National Committee on Large Dams risk acceptance criteria.

The solution gave an equivalent storage capacity to an embankment raise of approximately 0.9m.

The project, including design and construction, was completed within 3 months and provided a cost effective solution for the capacity increase which was implemented before the 2010/2011 wet season.

This paper presents the process used for evaluating the labyrinth spillway option including wind and wave overtopping evaluation, risk assessment, labyrinth weir design and the advantages and disadvantages of the solution.

Notation and units
Mt – mega tons = 1×10^6t
Mtpa - mega tons per annum

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1. Introduction

The Queensland Alumina Limited (QAL) red mud dams, referred to as RMD #1 and RMD #2 cover an area of approximately 400Ha and 600Ha respectively. Both dams are located on Boyne Island in Gladstone, approximately 8 km south of the refinery site at Parsons Point, Gladstone.

The RMD #2 embankments comprise a zoned earthfill with sloping upstream clay core and rocky and clayey general fill downstream shoulder with filter as shown on Figure 1. The dam has been in operation since the 1970's and has been subjected to six downstream raises with the latest 'Stage 6' raising the embankment crest level to RL 22.0m. On the upstream side of the embankments a rip rap protection layer is provided to protect against wave action. The downstream slopes of the embankments are protected by weathered coarse rockfill and slightly weathered high strength rockfill on the northern and eastern sides respectively. The downstream slope angles are approximately 1V:1.8H.
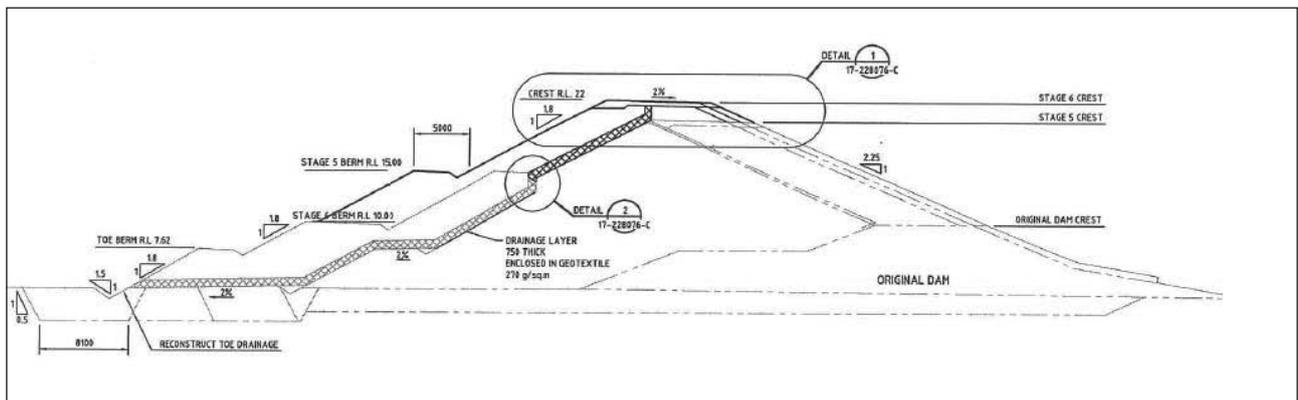

Figure 1 – RMD #2 Typical Embankment Section

The spillway existing prior to the upgrade comprised a 30m wide broad crested weir with a concrete lined chute and stilling basin. Downstream of the spillway is a series of gabion energy dissipation structures leading into a maze style polishing pond and eventual discharge into South Trees Inlet leading to the ocean.

2. The Situation Prior to Spillway Raise

Red Mud production from the refinery is in the order of 3.4 Mtpa, which, is deposited in either RMD #1 or RMD #2. Over time continued placement within the RMD #2 decant pond has reduced the available rainfall storage capacity and in early 2010 the available volume became less than that required for the design 1 in 50 year storm event. This required volume is based on a daily outflow volume of 90,000m3 and equates to approximately 5.5 Mm3. Figure 2 presents graphically the reduction in volume with time.
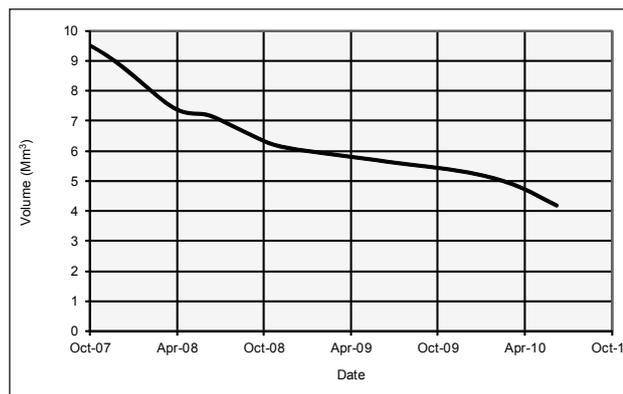
Figure 2 – RMD #2 Available Rainfall Water Storage Volume

This reducing rainfall water storage volume prompted the need to increase the storage capacity within the dam. Traditionally at QAL, the capacity increase would be achieved through raising the perimeter embankments with a spillway raise, however, for this project a series of options were developed which would allow QAL to keep operating RMD #2 by providing the required storage.

3. Options Developed

The options developed for assessment comprised:

- Raising the embankment with a raise to the spillway crest
- Widening and raising the existing spillway to reduce the wet freeboard
- Raising the existing spillway and increasing efficiency with a labyrinth or piano key to reduce the wet freeboard
- Fuse Gate Option

The options were developed to a point to allow a comparison of basic material quantities which required hydraulic and hydrological design to confirm that the water capacity increase was sufficient to allow continued operation of the dam. The options were compared on the basis of:

- Increase in water storage capacity
- Design and construction time with respect to onset of the wet season
- Construction cost

The labyrinth spillway was selected as the preferred option because of the low cost benefit ratio compared with the other options, this was due to:

- the relatively simple construction when compared to a piano key system which was meant existing, on-site, contractors were able to be used
- No need to raise the dam embankments reducing the required construction period

- No need to construct an additional spillway which reduces the complexity and time of construction
- Low cost solution compared with the embankment raising or spillway duplication and raising options

4. Labyrinth Spillway Design

The hydraulic performance of the labyrinth weir design was assessed using Tullis et al. (1995) and Falvey (2002) and a discharge rating curve was developed for use in the hydrological assessment of flood rise giving the required wet freeboard. The rating curve is presented in Figure 3.

The labyrinth weir geometry was optimised to maximise efficiency by minimising depth of water flow over the weir within the existing spillway width of 30 m and with a base level of R.L. 19.65m.

The final design is shown in Figure 4 and comprised the design parameters shown in Table 1.

Table 1   Labyrinth Weir Design Parameters

Design aspect	Value
No. Cycles	8
Cycle Width (w)	3.75 m
Height (P)	1.0 m
Effective Crest Length (L)	129.0 m
Existing Spillway width	30.0 m

The hydrological assessment was carried out using rainfall totals for the 1 in 10,000 Annual Exceedance Probability (AEP) design flood event. These were determined by interpolating between the 1 in 1,000 and 1 in 2,000 AEP rainfalls and the Probable Maximum Precipitation (PMP) rainfall as specified in Book VI of Australian Rainfall and Runoff: A Guide to Flood Estimation (1999).

The 1 in 10,000 AEP design storm event was routed through the spillway using XP-RAFTS for storm durations ranging between 3 hours and 72 hours. The storage elevation curve was taken from decant pond bathymetry and aerial survey.
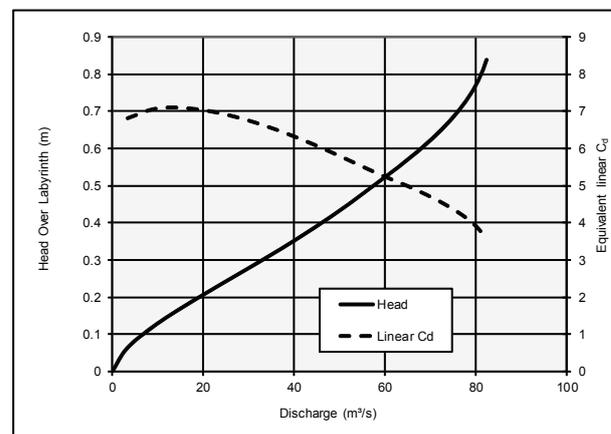
Figure 3 – Labyrinth Discharge Rating Curve and Equivalent Linear Coefficient of Discharge

As a comparison between the efficiency labyrinth spillway and the broad crested weir, Figure 3 shows a linear equivalent coefficient of discharge (Linear Cd) which ranges from 7 at low discharge flows to approximately 5.2 at the peak outflow. This compares to a typical coefficient of discharge of 1.45 for a wide broad crested spillway.

The peak outflow was calculated to be 59 m³/s which is lower than the broad crested weir outflow of 66 m³/s for which much of the downstream works was designed. This is due to the increased spillway length of 129 m compared to the previous spillway length of 30m meaning the spillway is able to discharge a greater volume of water earlier in the storm event leading to a lower peak. This meant that no alterations were required to the downstream outlet works.
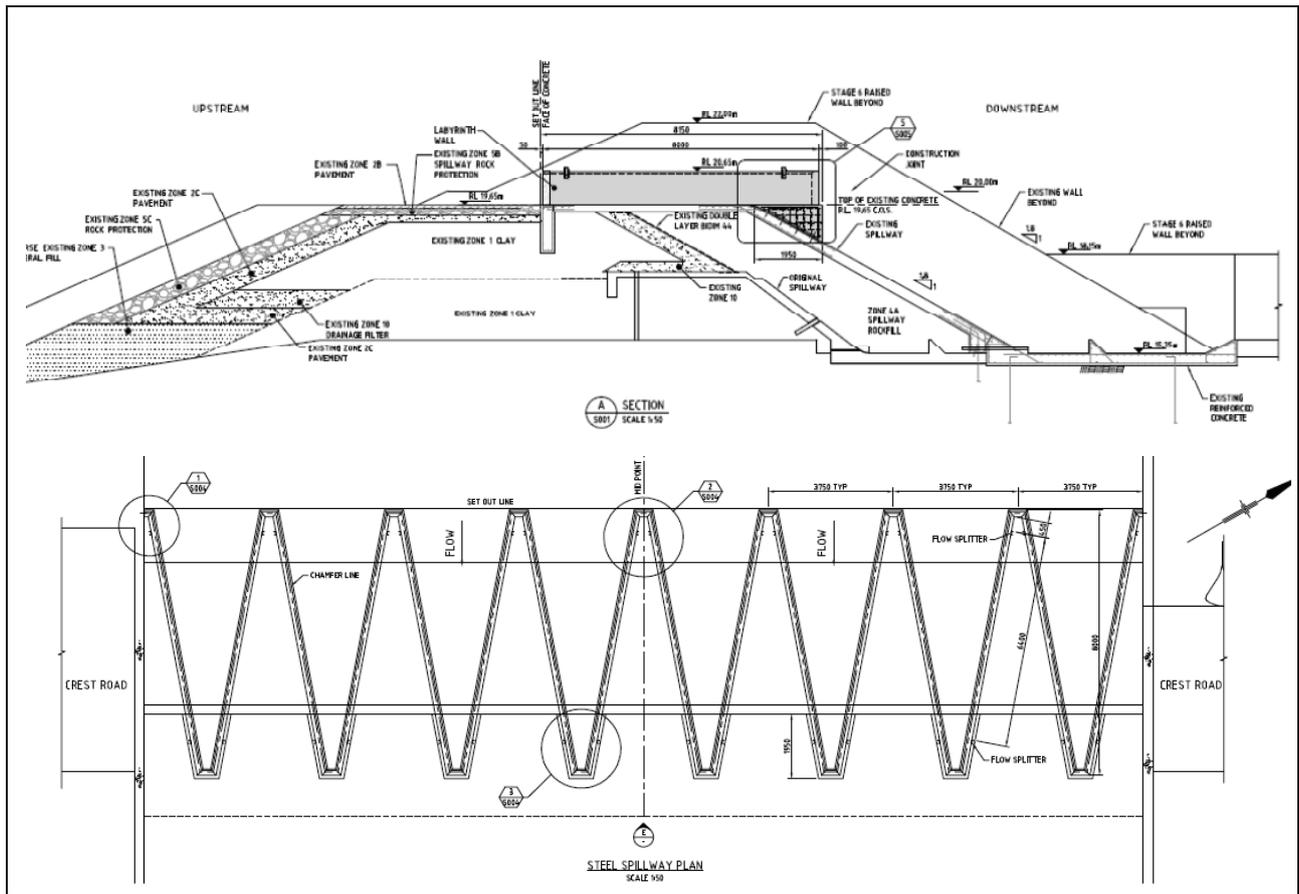

Figure 4 – Schematic of Labyrinth Design

5. Wind and Wave Considerations

The freeboard assessment was carried out using two previously adopted scenarios:

- 10,000 year ARI rainfall with a 10 year ARI wind
- 50 year ARI rainfall with a 200 year ARI wind

The wet freeboard levels were taken from:

- 1 in 50 AEP flood based on the design requirement for the available water storage capacity within the decant pond to exceed the 50 year ARI rainfall volume
- 1 in 10,000 AEP flood level from the spillway hydraulic analysis

The maximum fetch adopted for the freeboard assessment was calculated to be 2.6km as shown on Figure 5. The calculation to determine the maximum fetch length were carried out using radials drawn at 3° intervals, 12° either side of a central radial (ie resulting in nine radials total).The average water depth across the maximum fetch was determined from bathymetric survey of the RMD #2 decant pond.

Wind speeds were adopted from the Australian Standard Structural Design Actions, Part 2 Wind Actions, AS1170.2:2002, using Wind Region C and are:

- 200 year ARI wind speed 61m/s
- 10 year ARI wind speed 39m/s

Since the fetch and water depth varies around the dam perimeter the dry freeboard assessment was carried out at six locations (named A through F) each with a varying fetch and water depth. The highest total freeboard requirement was found to be for the 50 year ARI rainfall and the 200 year ARI wind. The results are presented in Table 2 with the site locations, and critical overtopping areas shown in Figure 5.

Table 2   Freeboard Assessment Results – 10,000 year ARI rainfall 10 year ARI wind

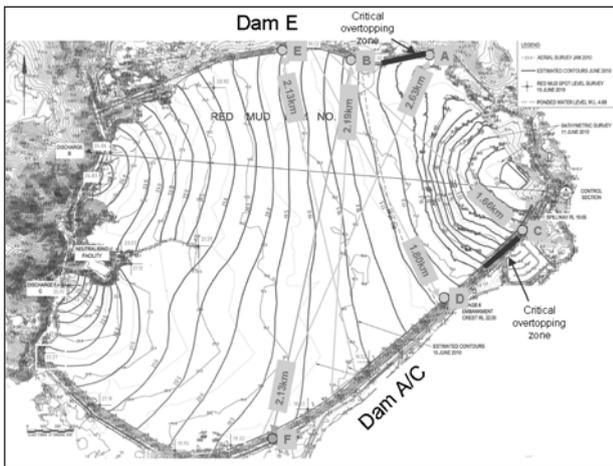| Site	Flood Level (R.L. m)	Fetch (km)	Water Depth (m)	Wave Height (m)	Run-up (m)	Set-up (m)	Required freeboard (R.L. m)
A - Dam E	20.65	2.63	3.00	1.76	2.01	0.44	23.10
B - Dam E	20.65	2.19	2.30	1.38	1.56	0.38	22.59
C- Dam A	20.65	1.66	4.25	1.41	1.61	0.20	22.46
D - Dam A	20.65	1.80	2.00	1.20	1.44	0.45	22.54
E and F - limit of decant water	20.65	2.13	1.50	0.90	1.24	0.71	22.60

Figure 5 – RMD #2 Fetch Locations and Critical Overtopping Areas

The maximum required freeboard was calculated to be R.L. 23.10 m at site A which is located beside the spillway on dam E. This exceeds the safety bund level of R.L. 22.5 m and, therefore, further assessment was carried out to assess the risk of embankment failure due to overtopping flows from wave action.

The discharge resulting from overtopping of the embankment by waves was estimated using the methods given in USACE (2002). Overtopping occurs when the run-up exceeds the freeboard above the still water level (SWL). For these analyses, the SWL was defined by the peak flood level.

The discharge calculated is an average value (temporally and spatially) and is dependent on the wave characteristics and the freeboard. The method adopted was that proposed by Owen (included in USACE (2002)) which is relevant to embankments which are:

- Impermeable
- Smooth or rough
- Straight-sloped or bermed

Allowable average overtopping discharges are given in USACE (2002) and reproduced on Figure 6. The figure shows that damage to an embankment is likely to commence at overtopping flows of between about 2 l/s/m and 10 l/s/m (0.002 m3/s/m and 0.01 m3/s/m). The downstream slopes of the embankments are generally gravelly fill, which has a high resistance to erosion and erosion is likely to initiate for flows in excess of 10 l/s/m.
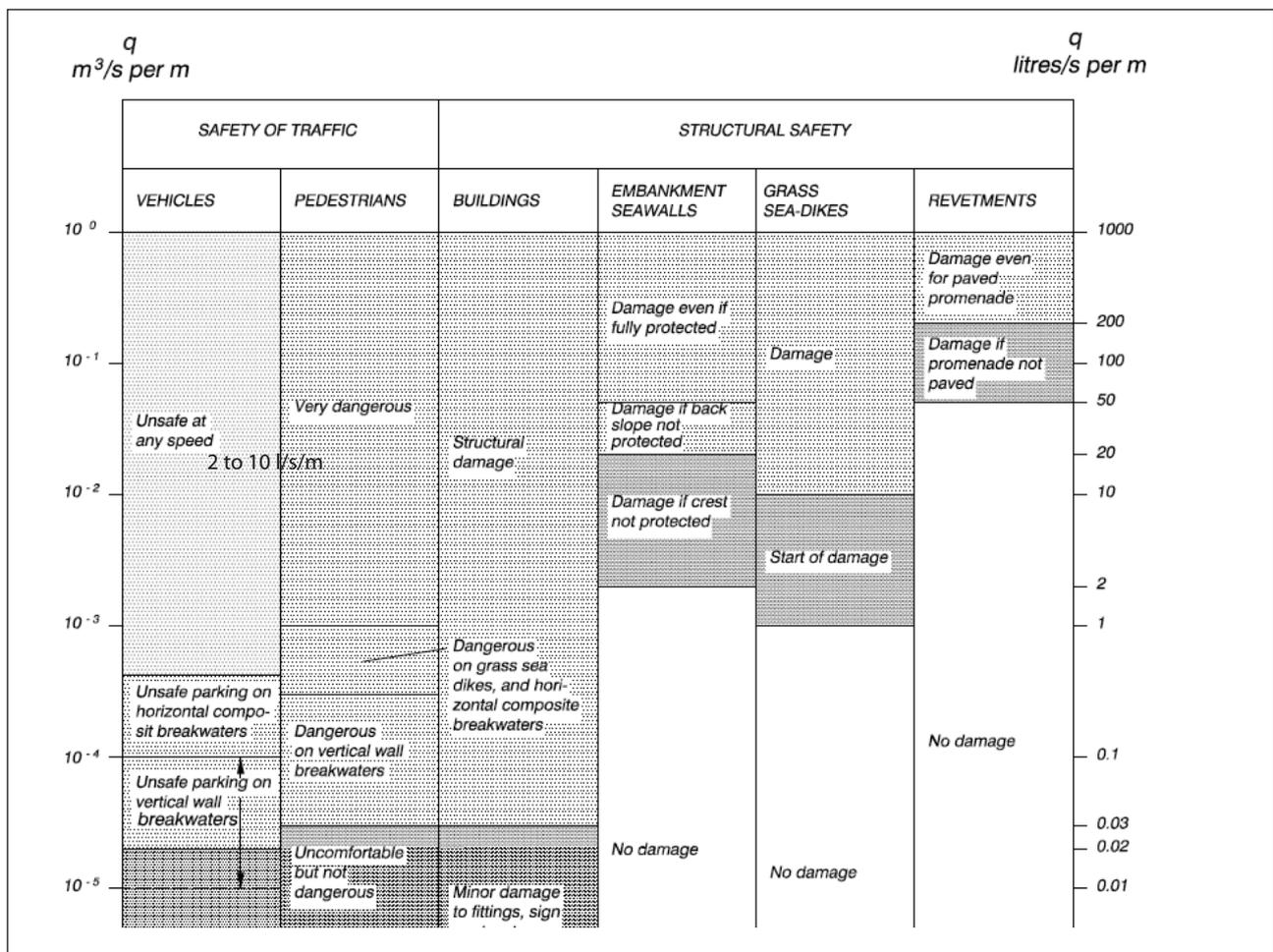

Figure 6 – USACE CEM, Critical Values of Overtopping Discharge

Table 3 presents the results of the wave overtopping assessment, the critical scenario in all cases was the 1 in 50 AEP flood with a 200 year wind ARI.

Table 3   Wave Overtopping Assessment Results

Site	Flood ARI (Years)	Wind ARI (Years)	Fetch (km)	Wave Height (m)	Run-up (m)	Set-up (m)	Overtopping Flow (l/s/m)	
							Crest	Bund
A - Dam E	50	200	2.63	1.76	2.01	0.44	9.0	1.8
B - Dam E	50	200	2.19	1.38	1.56	0.38	1.7	0.2
C- Dam A	50	200	1.66	1.41	1.61	0.20	2.1	0.3
C- Dam A	10,000	10	1.66	1.00	0.07	1.00	1.4	0.1
D - Dam A	50	200	1.80	1.20	1.44	0.45	1.0	0.1
E and F - limit of decant water	50	200	2.13	0.90	1.24	0.71	0.3	0.0

The risk assessment found that, in all cases, the overtopping flows over the 500 mm safety bund are small in relation to the lowest damage level and the likelihood of breaching through the bunds was, therefore, considered to be low. Further analysis showed that the expected discharge velocities are well below the velocities likely to initiate erosion of the crest or downstream face of the embankment.

On this basis it was considered that the risk of damage due to overtopping by wave action with the existing 0.5 m high bund is minimal and additional bund height was not necessary for the raised spillway design.

6.   Storage Capacity Increase

By providing a 1.0m high raise to the spillway level, additional storage of 4.0 Mm^3 was realised, comprising:

Table 4   Additional Storage

	Volume (Mm^3)
Decant Water Storage below Spillway Level	4.0
50 year rainfall storage	1.3
Additional Red Mud Storage below Spillway Level	2.7

The additional red mud storage beneath the spillway level allows further storage above the spillway level since only part of the discharged mud reaches the decant pond, the rest being deposited on the mud beaches. From the bathymetry and aerial survey, the overall increase in mud storage (on the mud beach and within the decant pond) is estimated to be 5.4 Mm^3 which is approximately equivalent to a 0.9 m embankment raise over the 600 Ha dam surface area.

7.   Conclusions

This project allowed QAL to increase their water and mud storage capacity through increasing the efficiency and height of the spillway. This was achieved through designing a labyrinth spillway to be installed on the existing spillway crest together with a rigorous assessment of the impact of wind and wave action within the decant water pond to confirm the acceptability of the approach.

QAL have received a cost effective solution to a common problem within the alumina industry, implemented in fraction of the time and for a fraction of the cost of using a traditional embankment raise approach.

8.   Acknowledgements

References

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