Kangaroo Creek Dam Embankment Raising and Stabilisation – Balancing Competing Objectives

Peter Buchanan[1], Malcolm Barker[1], Paul Maisano[1], Marius Jonker[2]

1. GHD Pty Ltd Principal Engineers 2. Consultant

Kangaroo Creek Dam located on the Torrens River, approximately 22 km north east of Adelaide, is currently undergoing a major upgrade to address a number of deficiencies, including increasing flood capacity and reducing its vulnerability to major seismic loading.

Originally constructed in the 1960s and raised in 1983, recent reviews have indicated that the dam does not meet modern standards for an extreme consequence category dam.

The original dam was generally constructed from the rock won from the spillway excavation. This rock was quite variable in quality and strength and contained significant portions of low strength schist, which broke down when compacted by the rollers. The nature of this material in places is very fine with characteristics more akin to soil than rock. Review of this material suggests that large seepage flows (say following a major seismic event and rupture of the upstream face slab) could lead to extensive migration of the finer material and possible failure of the embankment. However, it is also envisaged that the zones of coarser material could behave as a rockfill and therefore transmit large seepage flows, which may result in unravelling of the downstream face leading to instability.

This paper addresses the design of the embankment raising and stabilising providing suitable protection against both these possible failure scenarios, which tend to lead to competing solutions. The final solution required the embankment to be considered both as a CFRD and a zoned earth and rockfill embankment

Keywords: *Concrete Face Rockfill Dam, Seismic Upgrade, Seepage Analysis.*

Introduction

History of the dam

Kangaroo Creek Dam is a concrete face rockfill dam (CFRD) located on the Torrens River about 22 km to the north east of the city of Adelaide. Owned and operated by the South Australian Water Corporation (SA Water) the dam forms part of the Adelaide Water Supply System and also performs a flood mitigation role for the Torrens River. It is an extreme consequence category dam with a large population at risk and significant potential loss of life depending on the failure mode.

The site was first recognised as a potential site for a dam in the early 20th century, but it wasn't until the late 1950s that detailed planning was undertaken. Original investigations for the dam directed the concept design to a concrete arch dam, however as drilling techniques improved and core losses were reinterpreted, the foundation was identified as being unsuitable for an arch dam and a CFRD was adopted as the most suitable and economic alternative.

The adopted arrangement was a CFRD with maximum height of about 59 m, a side channel spillway through the left abutment, an inclined dry intake tower on the right abutment and tunnel through the right abutment utilised as the diversion during construction and to contain the outlet pipes for the river outlet. The dam was constructed by the Engineering and Water Supply Department (E&WS) (the predecessor to SA Water) between January 1968 and October 1969.

1980s upgrade

In the early 1980s the dam was raised by about 4 m and the spillway modified to provide an additional function of flood mitigation to the original function of water supply. The raising was achieved by placing reinforced concrete U-shaped sections onto the crest of the existing dam and filling them with compacted fill. These units were not integrated with the original embankment, with the exception of a small shear key.

The spillway crest structure was modified by the inclusion of two rectangular ducts which provided a choke for floods up to the 1 in 200 AEP event, and raising the overflow crest by 2.75 m. This increased the spillway capacity to about 1,800 m^3/s, from 820 m^3/s, but required a loss of storage of about 5,400 ML.

Current upgrade

Following a number of dam safety reviews undertaken between 1995 and 2010, and the portfolio risk assessment undertaken in 2013, a number of deficiencies were identified including:-

- Inadequate spillway capacity;
- Potential for the 1980s crest raising U-shaped sections and embankment supporting them, to become unstable during a seismic event or extreme flood event; and

- Potential opening of the joints in the face slab, (exceeding the load capacity of the waterstops), leading to uncontrolled seepage and potential embankment failure.

SA Water are currently undertaking the “Kangaroo Creek Dam Safety Upgrade Project” to address these deficiencies. GHD were engaged in late 2014 to develop the preferred remedial options. The construction contract was awarded in late 2015, with construction commencing in early 2016.

The current upgrade includes:-

- Increasing the spillway capacity to safely pass the Probable Maximum Flood (PMF) by widening the existing chute by about 40 m and raising the embankment crest level by about 4 m. The spillway widening requires:
  - Excavation of about 280,000 m^3 of rock of varying quality;
  - Extending the ogee crest by about 40 m;
  - Post tensioned anchoring of the existing ogee crest;
  - Raising the spillway training walls by up to 10 m; and
  - Placement of about 30,000 m^3 of concrete.
- Increasing the post-seismic stability of the embankment by placing a weighting berm with a total volume of 242,000 m^3 on the downstream face, including the placement of about 180,000 m^3 of rockfill won from the spillway excavation;
- Retrofitting external waterstops to the face slab vertical and perimetric joints to provide a more robust arrangement that will accommodate large movements during a major seismic event; and
- Modifying the outlet works to allow for the greater embankment foot print.

This paper addresses the design of the weighting berm and the specific issues arising from using the poor quality rock won from the spillway excavation, both in the original construction and the remedial works.

Original construction and resulting legacy

Description of original embankment

The original embankment was constructed in four zones (refer Figure 1) nominally being:-

- Zone 1 (concrete face support and drain): Rock graded with 305 mm upper limit and 10 mm lower limit, compacted in 610 mm thick layers.
- Zone 2 (vertical drainage zone): Rockfill with 915 mm maximum size and not greater than 20% passing 25 mm sieve. Compacted in 915 mm thick layers.
- Zone 3 (bulk rockfill): Rockfill with 915 mm maximum size and not greater than 30% passing 25 mm sieve. Compacted in 915 mm thick layers.
- Zone 4 (horizontal drainage blanket): Rockfill with 1220 mm maximum size with not greater than 20% passing 25 mm sieve. Compacted in 1220 mm thick layers.

In addition, existing river gravels were left in the river valley with some level of compaction and this is shown as Zone 5.

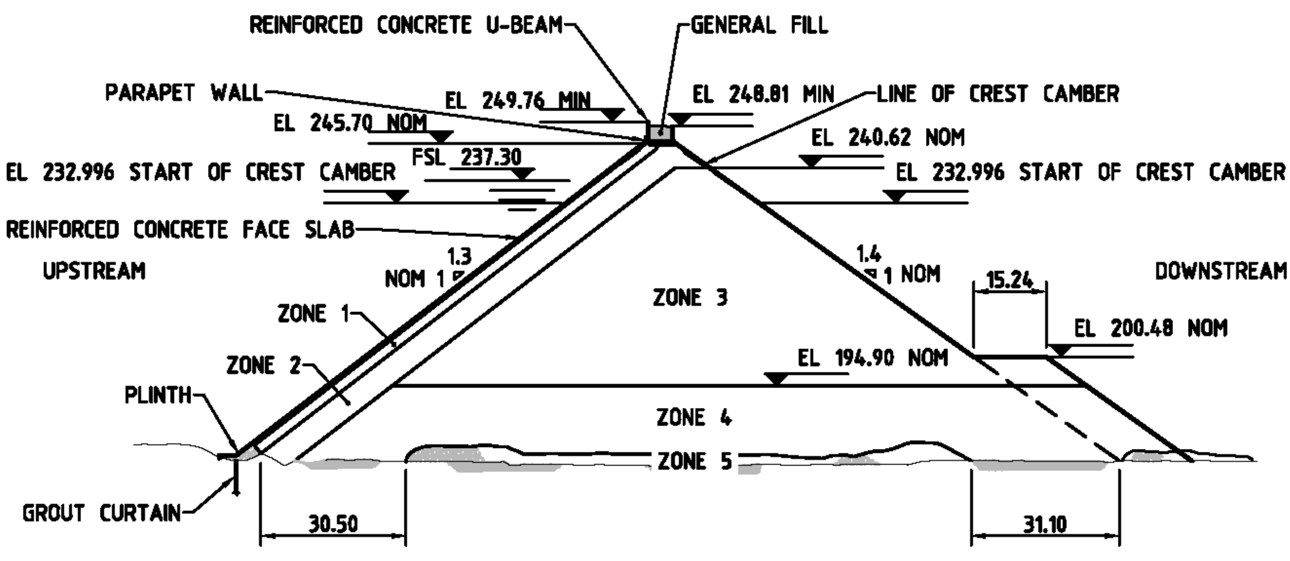

Figure 1 – Typical cross section of the original embankment showing zones (includes 1983 raising)

Each layer was sluiced at a rate of approximately 1,000 litres of water for each cubic metre of fill.

The construction of the concrete face was typical for the period, with a reinforced concrete plinth along the perimetric joint, below which a grout curtain was constructed. The reinforced concrete face slab was slip formed in 12.2 m wide panels constructed directly on Zone 1. The slab varies in thickness from 305 mm at the original crest level, increasing in thickness at a rate of 0.5% of the vertical distance below the original crest level, with a maximum thickness of about 710 mm. The reinforcement, waterstop and joint details are typical of the practice at the time. A vertical parapet wave wall was constructed monolithically with the slab to a height of 900 mm.

1983 raising

In 1983 the dam crest was raised to EL 249.77 m AHD by constructing reinforced concrete U-shaped wall sections on the crest of the dam. The upstream leg of the U-section was constructed against the downstream face of the original parapet wall, although it was not structurally connected. A 500 mm deep shear key was constructed into the original fill to provide some sliding resistance. The area between the upstream and downstream legs of the U-section was filled with about 2.5 m of compacted fill but the size of the U-section provided a crest raising of about 4 m.

Details of embankment fill materials

Rockfill for the embankment was sourced from seven different quarries, selected according to the suitability for each zone and the availability based on the construction program. Zone 1 material was won from a purpose selected quarry remote from the site to provide the correct grading to act as the bedding for the concrete face and for its durability to limit the breakdown of the material during handling and placing.

The materials for Zones 2, 3 and 4 were won on site, or from the access road construction. Zones 2 and 4 materials were selected to meet the required grading envelopes where possible, whereas Zone 3 material was generally won from the spillway excavation and was more random in its nature. The materials for each zone were generally selected by inspection at the quarry face.

Much of the spillway excavation consisted of low strength, moderately to slightly weathered schist and gneiss, which had a tendency to break down when worked. In all the zones, and in particular Zones 2 and 3, there was a tendency for the rockfill to break down during the sluicing and compacting, resulting in “skins” being formed on the top of each compacted layer (refer Figure 2 and Figure 3).

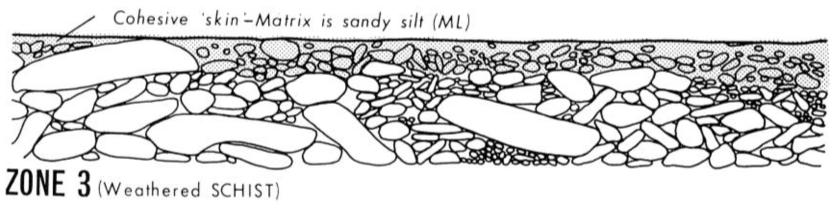

Figure 2 – Illustration of the breaking down of rockfill material during placement (reproduced from Trudinger 1972)

Figure 3 – Breaking down of the upper surface of a lift due to the trafficking and sluicing (photo courtesy of SA Water)

Table 1 provides a summary of the design and actual characteristics of each zone. It was recognised at the time of the original investigations of the dam site, that the available rockfill was not as durable as would have been preferred in an ideal situation, but if this rock could not be used then it was possible that the dam site would not be economically viable and would likely be abandoned. It was considered that provided the schist was well sluiced and compacted it would provide an adequate rockfill shoulder (Good et al. 1985).

A similar situation has arisen for the current raising and stabilising of the embankment. The widening of the spillway provides a large quantity of the same poor quality rockfill used in the original embankment construction. If this material was not used in the embankment works, a large volume of rockfill would have to be imported, while the material excavated in the spillway would have to be spoiled, both of which would have added a significant cost premium to the project. Therefore, the design progressed on the basis that the majority of the material to be placed in the raised and modified embankment would be similar in nature to the original Zone 3 material. The design of the modified embankment had to consider potential failure modes not considered by the original designers; these are associated with using the low quality material as a fill, which is not what is typically considered for modern CFRDs.

Table 1 – Embankment Zones and design requirements (Trudinger 1973)

Zone	**Layer Thickness (mm)**	**Maximum Fragment Size (mm)**	**Amount of sub 25 mm**		**Volume in dam (m^3)**	**Requirements**
			Specified	**Actual**		
1	600	300	0	12%-20%	19,000	• High permeability in vertical and horizontal directions so that all water leaking through the face and abutments is readily drained • Voids small to minimise loss of concrete
2	900	900	20% max	20%-50%	41,400	• Moderate permeability so that leakage water drains readily from Zone 1 to Zone 4
3	900	900	30% max	20%-50%	238,000	• Resistance to settlement • Some horizontal permeability to drain water from abutments and excess water from Zone 2
4	1200	1200	20% max	10%-30%	40,400	• Resistance to settlement • High permeability in all directions to ensure unimpeded drainage of water out of embankment
5		No Limit				• Freedom from all large areas of organic material • Moderate permeability

There were limited particle gradings taken of the material as placed at the time of the original construction, with only three recorded for Zone 3. These tests indicated that there is up to 10% fines (< 75 μm) in Zone 3.

Furthermore, the available construction records indicate that much of this fine material is concentrated on the top of each layer, as the material was broken down during the compaction process. This can be seen in Figures 2 and 3 where the combination of compaction loading and sluicing water have resulted in the rockfill breaking down to a "soil like" material forming a "skin" on the top of each lift. The extent to which these "skins" formed was dependent on the amount of finer material transported to the embankment, the proportion of weak rock in the material, and the amount and type of traffic over the fill. It is reported in construction records that these skins are typically 50 to 150 mm thick, although up to 500 mm thick in locations (Trudinger 1973). This has resulted in an anisotropic fill with the upper part of the layers consisting of finer material controlling the vertical permeability and the cleaner, less broken down in the lower part of the layer controlling the horizontal permeability of the layer. Furthermore, these "skins" are of such a depth that it is possible that there may not be rock to rock contact between lifts.

The post-seismic stability problem

It has been estimated that the joints in the face slab could open by as much as 200 mm towards the abutments and up to 100 mm toward the centre of the dam during a major seismic event. With the resulting high seepage flows the poor quality rockfill potentially leads to two issues which may contribute to embankment failure. Both these potential failure modes are not normally considered for a modern CFRD, which typically has zoning with increased permeability in the downstream direction to allow safe handling of high seepage flows. The two key concerns with the existing embankment are:-

1. The high fines content could lead to the potential scenario where internal erosion of the finer sized particles would cause their migration through the rockfill, resulting in a significant change in volume of the fill, particularly where there is no rock to rock contact, which in turn would cause further settlement and opening of the joints of the face slab and increased flows; and
2. The low permeability of the "skins" could lead to the potential scenario where the horizontal seepage flow is concentrated at the exit to a small area of the downstream face, which could cause unravelling of the face, and if it continued for long enough to gradually erode away the embankment crest, which would cause failure of the embankment.

Therefore, in order to modify the embankment to address both of these failure scenarios, it was necessary to consider the embankment both as an earthfill embankment and as rockfill dam.

Upgraded embankment

Design criteria

One of the key requirements for the design of the embankment upgrade was for the post-seismic stability factor of safety to be at least 1.5. Although this is in excess of the normal factor of safety for a post-seismic condition, this more stringent criterion was adopted to provide a higher level of confidence of failure not occurring given the extreme consequence category of the dam.

The following assumptions were made for the design of the upgraded embankment:

- The overall upgraded embankment configuration was based on a postulated post-earthquake scenario assuming that some of the joints and waterstops in the face slab have breached and the design leakage was flowing into the dam body;
- The reported “skins” in Zone 3 have reduced the permeability of the rockfill so that it is was no longer free draining, to the extent that the design leakage had saturated the existing embankment body. This could result in the downstream slope becoming unstable. A stabilising berm was thus required to target a post-earthquake stability factor of safety of 1.31.5;
- Conversely, a potentially high horizontal permeability of Zone 3 could result in erosion of the downstream face if the exiting hydraulic gradient of the leakage flows exceeded the critical gradient. An internal drainage system was required to control the exit flow rate, or the downstream zone modified (based on slope and flow rate) to prevent erosion (unravelling) on the downstream face; and
- As the expected extent of damage to the upstream face slab could not be clearly defined, it was conservatively assumed that flow into the embankment, and progression of erosion (unravelling) in Zone 3, will not be restricted by the upstream concrete slab.

Modified embankment arrangement

Description of modified embankment

To address the seismic load state failure modes (crest stability, the potential of internal erosion induced failure, and an unravelling induced failure), the following features are to be included in the modified embankment:

- The original embankment will remain as is with the exception of some trimming of the downstream batter;
- The U-shaped sections used for the 1980s raising will be removed due to concerns about their stability, and to allow the embankment to be raised to increase the flood capacity;
- A two-stage filter / drainage system will be placed against the existing downstream batter to reduce the potential for internal erosion induced failure;
- An additional weighting fill will be constructed downstream of the drainage system; and
- A high capacity rockfill zone will be placed at the toe of the embankment to reduce the potential for unravelling induced failure.

To address the flood load state failure modes, the embankment will be raised by 4 m to provide additional spillway flood capacity. Although not required for the dam safety upgrade, a permanent access ramp will be added to the downstream face. The Contractor identified the need for a temporary access on the face of the embankment and it was agreed between SA Water, the Contractor and the Designer to retain the ramp as part of the permanent works.

The typical cross section is shown in Figure 4.

Design of the drainage / filter system

Zones 6B and 6C (see Figure 4) form an internal filtration and drainage system for the original downstream shoulder material (Zone 3). The filters have been designed according to the no erosion criteria to prevent erosion of the fines from within the Zone 3 rockfill material into the respective filter zones (Fell et al. 2014). Zone 6C has been included to provide sufficient drainage capacity to transmit all seepage to the high capacity rockfill at the toe of the dam. The design procedure of filters is well documented and is not addressed in this paper.

The determination of the design seepage rate required assumptions as to the permeability of each of the zones and the extent to which the face slab could rupture.

As previously discussed the compacted rockfill layers within the embankment are nonhomogeneous and seepage through them will be highly random. Thus, an intensely layered seepage system has been created in the embankment. Ideally the layered system would have created independent, low-pressure, horizontal seepage flow paths in each rockfill lift, and virtually no vertical downward seepage. It is expected that the ratio of horizontal to vertical permeability is very high in Zones 3 and 4. It is expected that the ratio of vertical and horizontal permeabilities of Zones 1 and 2 are likely to be nearer to unity, however some construction records indicate that there are “skins” present also in some of the Zone 2 layers.

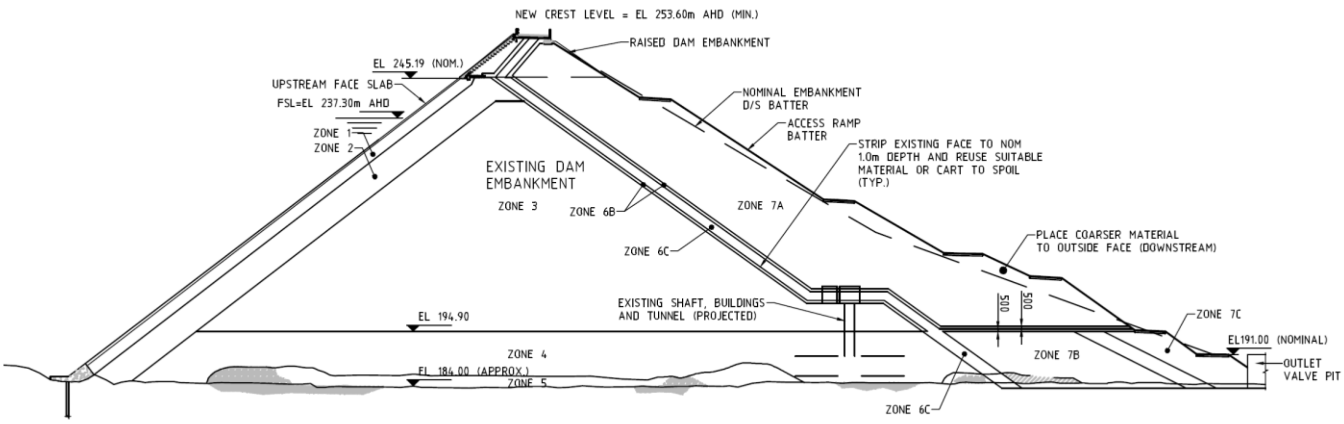

Figure 4 – Typical cross section of the upgraded embankment

Leakage would occur into the embankment in the case of (1) a rupture or breach in the upstream impermeable barrier. This could be the result of the failure of a waterstop in the perimetric or vertical joints; or (2) failure or damage of the concrete slabs, or both scenarios (1) and (2), resulting in leakage through the concrete face. Estimates of possible joint openings were made based on likely movements and settlements of the embankment, and thus the face slab, that would occur during an extreme seismic event. Analysis demonstrated that crest settlements of up to 1 m are possible during a major seismic event. If such settlements did occur, then openings in the joints of up to 200 mm are possible. It would be expected that these movements would be distributed over a number of the joints, particularly the perimetric joint and the first joints in from the abutments. Although external waterstops have been included on the face of the slab that are capable of accommodating these expected movements, the design of the embankment has assumed the failure of the waterstops.

The embankment seepage model initially included the estimated flows through the openings in the face slab. The seepage was estimated from first principles, and existing empirical models of flow through concrete joints and cracks; however, these do no accurately represent the actual conditions and the Zone 1 layer beneath the slab, particularly the three dimensional behaviour of the flow. Given the uncertainty with this approach, it was decided to assume that the face slab had failed completely (i.e. no longer in place). The calculated design seepage was therefore based on the permeability of the existing embankment zones (which could be estimated with more certainty), and is independent of the extent of the damage to the slab. This was considered to be a worst case scenario.

A number of different combinations of the assumed permeabilities, in particular the presences or otherwise of the skins on the top of each lift and their effect on the vertical permeability of the different materials, were considered. The various analyses indicated that the peak seepage flow rate that could reasonably be achieved through the dam was about 0.25 to 0.3 m^3/s per metre width of dam, based on a 2-dimensional analysis. The flow per metre width was adopted as the design case. Although the inflow would be relatively confined in the form of an open joint, and there would be some lateral spread of the seepage, no attempt was made to calculate what this spread would be. Given the calculated volume of seepage, a two-stage filter/drainage zone was required to provide both adequate filtering and drainage capacity.

Zone 7C, located at the toe of the dam below EL 195 m AHD, has been designed to prevent unravelling of the rockfill where the seepage flows would exit the embankment. The design seepage of 0.25 to 0.3 m^3/s per metre width of dam was arbitrarily doubled to provide a level of security against higher flows due to the concentration into the toe of the dam. This material was designed in principle according to the empirical equation presented by Ekström and Nilsson (2010), which provides a method for determining the rock size required to resist erosion due to through flow. This method was developed based on results from comprehensive laboratory and field testing undertaken in Norway in the early 2000s. It uses the unit discharge per metre and the external slope of the rockfill to determine to appropriate D_{50} size for the rock. It incorporates a load factor of at least 1.5 to give a margin to failure or collapse in the rockfill (damages are accepted but no failures) and a margin to the uncertainty in the estimation of the unit flow.

The required dimensions of the 7C material located at the toe of the dam are D_{50} between 400 and 700 mm and D_{max} of 1200 mm. It is likely that this material will need to be imported to both ensure that the correct grading is achieved and durable rock is placed to prevent breakdown.

Design of the weighting berm for post-seismic embankment stability

The inclusion of a fine filter in the upgrade works and the potential of fine particles migrating from the existing Zone 3, could lead to clogging of the filters placed against the existing downstream face. This leads to an additional design condition not present in the existing dam, being clogged filters during a major seepage episode. Given the high seepage flows that have been estimated and the possibly poorly graded rockfill materials, it is conceivable that sufficient fines could be transported so that the capacity of the drainage zones would be compromised with the finer particles sealing the Zone 6B filter.

For steady state conditions it has been assumed that if there is any seepage past the face slab, it is small enough that Zone 1 has more than enough capacity to drain all seepage to Zone 4, and therefore there is effectively no phreatic surface within the embankment. However, this may not be the case for higher seepage flows where the capacity of the Zone 1 material is exceeded and seepage flow reaches the Zone 6B filter.

As a worst case it was assumed that leakage occurs through the face slab into the existing embankment and transportation of the fines results in complete blockage of the new filter / drainage zone. Therefore, regardless of the leakage rate through the face slab, the leakage would eventually saturate the existing embankment fill and create the highest possible phreatic surface, with full reservoir head in the zones upstream of the filter / drainage zone.

As noted above, one of the key requirements for the embankment stability was for a post-seismic factor of safety to be at least 1.5. This clogged filters scenario was determined to be the critical load case and resulted in the final adopted embankment cross section having a nominal downstream batter 1V to 1.7H above EL 214.6 m AHD (the upper 39 m) and 1V to 2.25H below this level. However, in the final arrangement these batters vary locally across the embankment to accommodate the access ramp. This is a relatively flat downstream batter compared to typical CFRDs, but it is not the norm to consider the downstream shoulder as a saturated material in a typical CFRD. This is clearly a legacy of the quality of the material and the high fines content within the Zone 3 material.

Conclusions

Kangaroo Creek Dam was, like dams in general, constructed with local materials under specific physical conditions. The cost optimized design required that the original dam be constructed using locally won poor quality rock. The same rockfill will be used for a significant proportion of the embankment upgrade works. This rockfill, consisting of schist and gneiss, breaks down when worked, resulting in a rockfill with a very high fines content, atypical of CFRDs, and strong layering of the lifts with these excess fines tending to concentrate at the top of the lifts. This required the embankment remedial works to be designed to allow for this material to behave both as a rockfill and something more in line with an earthfill.

For dams constructed in very specific conditions, the design criteria may deserve a total review. In this case the cost optimized design of the embankment upgrade works required a tailored study for the specific conditions, and involved the dam owner, the design consultant, an expert review panel, and the contractor. Given the unusual embankment characteristics and associated uncertainties, the design was completed for conservative loads and conditions, and adopting acceptance criteria that are more stringent than normally adopted for CFRDs.

Traditional CFRD cross sections and typical design procedures for embankment dams did not meet the requirements for an optimum design under the specific local conditions. When comparing a number of solutions, it was necessary to challenge and review set ideas, avoid certain standardized solutions, and adapting the design and specifications to the real needs, in order to achieve a cost effective solution. In this case the resulting upgraded embankment will have downstream batters flatter than a typical CFRD, to provide adequate toe weighting should the fill become saturated, and internal filter and drainage layers to control the potential for downstream migration of fines leading to collapse of the embankment. The result is a robust solution that meets current practice for the various failure modes that tend to lead to competing solutions.

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