The British Dam Society

Design of Dam Safety Measures for three Dams in Zambia

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SYNOPSIS Arup was commissioned by the United Nations Office for Project Services (UNOPS) in 2022 to design measures for improvements to the safety of three dams in Zambia. The three dams were all homogeneous earthfill dams, but with differing geometries and spillway forms. All three dams share the common problems of inadequate flood capacity, lack of freeboard, scour damage and irregular dam construction with excessive seepage in places, but they differ in the hazard they pose to the downstream population.

The paper will describe the assessment of the downstream hazard posed by each dam, the identification of the proposed solutions and optimisation of the resulting designs.

INTRODUCTION

UNOPS was tasked to address the unresolved issues from the Zambia Water Resources Development Project funded by the World Bank. In 2022 Arup was appointed by UNOPS to design solutions for the dam safety issues on three of those dams: A, C and K. Arup visited the three sites in July 2022.

The dams were originally designed in 2014 and constructed in 2016 by local organisations and contractors. Others had carried out inspections of the dams and identified a number of deficiencies in common including the need to verify the construction quality of the dam and key design parameters.

Dam A

Dam A is an 11.5m high embankment dam situated in the Eastern Province of Zambia. The slopes are approximately 1:2.5 to 1:3 with a 4m to 5m wide crest. The overall storage capacity has been previously estimated at 710,000m^3, with surface area of the waterbody of 13.5ha and catchment of 516km^2. The reservoir is suffering from extensive siltation and is expected to have a limited useful life. The reservoir was designed for livestock watering, domestic water, recreation and fishing. However, it is understood that the reservoir is currently only used to provide water for livestock watering, with no supply offtake facilities included for other uses.

On the west side of the site there is an L-shaped concrete service spillway 110m long with a sloping concrete wall and masonry crest; the structure appears to be founded on rock. Low areas of dam crest were observed next to the weir abutment walls.

The recommendations in the interest of dam safety include mitigation measures to address the lack of sufficient overflow capacity and freeboard, leakage and erosion of the service spillway, leakage in the low level outlet, and extensive scour in the downstream channel. Seepage was noted as upwelling a few metres downstream of the dam toe in a single location.

Figure 1. Dam A Site arrangement (Google Earth, Arup elaboration)

Dam C

Dam C is an 8m high earth embankment dam in Laupula Province, Zambia. The overall storage capacity has been previously estimated at 3,000,000m^3, with surface area of the waterbody of 90.7ha and a catchment of 374km^2. The reservoir is to provide water for irrigation and livestock watering.

The dam alignment has a 90-degree bend to accommodate the service spillway approach channel and weir. The design is understood to have been changed during construction to locate the spillway weir on a rock outcrop. There is a secondary embankment to the right of the primary spillway weir with low areas in the crest adjacent to the primary spillway. For the main east-west dam the existing upstream and downstream slopes are typically at a gradient of 1:3. The embankments north and west of the spillway are steeper at around 1:2; visibly steeper than the main dam geometry and with a narrower crest.

The service spillway is a weir L-shaped in plan, 32m long, with stepped chute. There is a partially completed emergency spillway to the left-hand side of the embankment, however the crest level of this emergency spillway area is only marginally below the dam crest and concerns were raised about the lack of capacity and freeboard.

The site had been inspected by UNOPS in November 2021 following a 5m long crack being identified in the crest of the steep sided embankment to the left of the spillway approach, which had apparently been repaired prior to the arrival of UNOPS engineers. It also noted undercutting of stone pitching, abutments and training walls at the spillway and excessive leakage at a valve chamber. The dam was otherwise found to be in fair condition though some seepage was noted downstream of the dam toe.

Figure 2. Dam C Site arrangement (© UNOPS)

Dam K

Dam K is a 9m high earthfill embankment dam in the Copperbelt Province of Zambia. The overall dam storage capacity has previously been estimated to be 2,030,000m^3, with the surface area of the reservoir being approximately 63ha and a catchment of 287km^2. The main embankment length (running from west – east) up to the west side of the service spillway is approximately 210m. A marshy area was identified downstream of the dam and no toe drain could be identified.

There is a service spillway on the east side of the dam (left bank) with a three-stage unlined drop structure with three straight concrete weirs. The overall discharge structure length is approximately 105m, with weir lengths of 30m. There is scour undermining around the drop weirs and the training banks along with potential seepage. The spillway discharges into a vegetated channel and flows in the south-westerly direction towards a culvert that passes underneath a major highway. There is an incomplete emergency spillway on the west side of the site. The incomplete spillway is covered by vegetation and the current ground level has not been lowered, resulting in minimal freeboard between this channel and the dam crest.

The reservoir is to provide water for domestic supply, aquaculture and livestock watering though no water supply draw-offs are currently in use.

Figure 3. Dam K Site arrangement (Google Earth)

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DAM CLASSIFICATION

There is no specific dam classification system legislated for use in Zambia, so we considered a range of references. Zambian Dam Guidance (MAFF, 1998) makes reference to a system (Kabell, 1986) which classifies dams into four categories based on dam size (height, volume of reservoir) and hazard potential (loss of life, economic).

Dam A is located in a very remote area with no settlements observable downstream, so the risk to life and economic risk is therefore low to very low. It was debated whether to classify Dam A as class 2 or 3, depending on whether the risk to life could ever truly be "impossible". Ultimately a classification of 2 was chosen as economic losses were considered moderate and a smaller flow estimate had been obtained, so a higher classification combined with lower flow estimate was used.

Dam C would classify as a Medium size dam based on dam height and potentially large based on the large storage capacity, though this is expected to be slightly less than $3Mm^3$. The hazard potential would classify as Moderate to High as loss of life would be likely (possible to probable) and economic loss appreciable. The dam is therefore the highest Class 1.

Dam K would classify as a Medium size dam as loss of life would be unlikely (improbable) due to only sparse residential properties in the valley, therefore class 2 was selected.

An independent comparison has been made with ICOLD Bulletin 157 (ICOLD, 2016) which classifies small dams as follows:

> Dams 5m to 15m high and with a $H^2.V^{0.5} < 200$ (H height in metres, V storage in Mm^3).

It then classes the dam based on $H^2V^{0.5}$ and the downstream consequences. Dams A and K classify as small dams and with fewer than 10 lives expected to be at risk. Dam C has a $H^2V^{0.5}$ greater than 200 with potential loss of life greater than 10 and was therefore considered a higher risk large dam.

Table 1 Design and Safety Check Flood selection

Dam	Size	Hazard Potential	Kabell Classification	ICOLD 157	Design Flood	Safety Check Flood	
				$H^2V^{0.5}$	PHC		
A	Medium	Very Low to Low	2	121	2	1 in 500y	2000y
C	Medium to Large	High	1	293	3	1 in 2000y	1 in 10,000y
K	Medium	Low	2	118	2	1 in 500y	2000y

The definitions of design and safety check floods in Bulletin 82 (ICOLD, 1992), as also adopted in UK practice, were adopted. Kabell recommends design floods of between 100y for class 4 and 2,000y return periods for class 1 with dry freeboard allowance. For safety check floods it recommends 250y to 10,000y return period floods with no dry freeboard.

For comparison, ICOLD Bulletin 157 recommends safety check floods of 50y (PHC1) to 1,000y (PHC3) for rural areas and 1,000y (PHC1) to 10,000y (PHC2) for more populated areas. French guidance on small dams (CFGB, 2002) also recommends a minimum design flood of 5000y return period for $H^2V^{0.5}$ in the range of 100 to 700 and an absolute minimum of 1000y where there is a risk to life. The magnitude of safety check floods from Kabell is therefore comparable to the approach in many countries, however we additionally included freeboard in the safety check case.

A hydrological analysis was performed, initially using a regional flood method intended for Zimbabwe (Mitchell, 1998) as the closest available method. As rainfall data became available, a rainfall runoff analysis was performed. Rainfall data for a 30-40 year period was obtained from a wide range of rain gauges from different sources with at least three gauges in reasonable proximity to each site. No river flow gauge data could be obtained. Attempts were also made to use satellite rainfall data. However, this was found to not accurately identify peaks in rainfall.

Hydrological analysis had also been performed by a previous consultant using a South African form of modified rational method, which proved a useful comparison. The three estimates are shown in Table 2 below. The Arup and previous estimates generally showed good agreement and these estimates were typically lower than the Mitchell method. This was expected as most of Zambia experiences less significant rainfall than Zimbabwe where that regional flood equation was developed. The exception to this was at Dam C where the Arup estimates were greater. This is because more local rainfall data had been obtained and the site was found to be in a higher rainfall area proximate to a large inland lake. For Dam K Arup produced lower flow estimates from similar rainfall data, as the land cover was found to differ significantly from the earlier estimates.

Table 2 Flood estimates

	Peak flow (m^3/s)		
Method	**Dam C**	**Dam A**	**Dam K**
Safety Check	**10,000y**	**2,000y**	**2,000y**
Mitchell Method 1998	2072	1768	1265
Rational Method 2018	820	638	1175
Arup Type II	2083	686	877

DAM BREACH ASSESSMENT

Following the initial costing of the preferred option for each dam, the dam hazard categorisation was questioned by the funding agency. It was felt that the potential consequences of failure of Dams C and K were lower and that a smaller safety check flood should therefore have been selected. We therefore carried out a dam breach assessment with incremental consequence estimation to establish the hazard categorisation of dams C and K in a quantitative way. Dam A was not assessed as the consequence of failure could clearly be seen to be minimal due to the remoteness of the site.

The following flow scenarios (after ICOLD, 2005) under which failure of the dams can occur were considered in the analysis:

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- Sunny Day Failure, where the failure occurs under normal flow conditions (not associated with a flood event) and at the normal operating head water levels (water level at spill level).
- Rainy Day (Flood) Failure, where the failure of the dam is associated with the occurrence of a flood of a given return period and at head water levels above normal operating levels. For this scenario the potential range of safety check events was considered for each dam.
- Baseline Non-breach Scenario, where the flood inflow hydrograph is modelled passing through the river valley without routing through the reservoir. This scenario was run to enable performing incremental hazard assessment for the flood failure.

As the dam classification and therefore the return period of the safety check event was under examination, a range of possible safety check events was run for both dams.

The following dam breach models were applied for both sunny day and rainy-day scenarios: Xu and Zhang (Xu, 2009), USBR SEED (USBR, 1995) and Molinaro (ICOLD, 2005) to generate the breach hydrographs. For the sunny day failure for both Dam K and Dam C, the Xu and Zhang hydrograph was selected, as it was most applicable for low height dams. Xu and Zhang produced a credible result lying below USBR SEED, which is known to be conservative, but above Molinaro. Froehlich time to peak for Xu and Zhang is recommended by the UK's Environment Agency (Environment Agency, 2013) and produces a faster time to peak, so it was adopted in combination with Xu and Zhang peak flow. For consistency and using the same reasoning as for the Sunny day scenario, the Xu and Zhang hydrograph was selected for the Rainy Day scenario. The breach hydrographs were added to the peak of the inflow hydrographs for each return period.

The dam break analysis was carried out using a 2-dimensional model developed in the latest TUFLOW HPC (version 2020-10-AF). Topographic information was limited to freely available data. Following a review of potential topographic data sources, ALOS Global Digital Surface Model "ALOS World 3D" – 30m (AW3D30) was adopted. TUFLOW HPC's Quadtree Mesh capability was used to allow for variable cells sizes to be adopted within the model. The topography was manually modified to enforce the primary waterway channels for the full extent of the model and to manually smooth heavily vegetated areas (where vegetation captured in the AW3D30 data artificially interfered with the conveyance of flood flows). Manning's n roughness values were prepared initially using Open Street Map (OSM) data.

For the consequences assessment, the study focused on inundated buildings and potential fatality rates from the building occupancy. This was due to lack publicly available information about average road usage, as well property values. Flood extents and velocity rasters from the modelling were intersected with the building overlays form OSM. The buildings layer was reviewed against the aerial imagery within the maximum flood outlines and where additional buildings were visible, they were manually added to the overlay. Where building use can be suggested from the aerial photography, the building use was also recorded. As many buildings in the original layer did not have use indicated, it was assumed that buildings with unknown purpose with footprint of less than $100m^2$ are residential, and any buildings above that size are used for commercial/agricultural purposes.

The occupancy rate for the residential buildings was 5.1 people per household, as obtained from the 2015 Living Conditions Monitoring Survey (Zambia Central Statistics Office, 2016). As no information was found on occupancy rates for agricultural, commercial or industrial buildings, occupancy rate of 1 person per building, present at all times, was assumed. Due to lack of information of typical times spent outside of residential homes, the occupancy rate for the residential properties was not time averaged, i.e. it was assumed that 5.1 residents are always present. As the occupancy rate for the agricultural/commercial/industrial buildings was relatively low and irrespective of the building size, it was considered that there is little probability of double counting of the population. A 5m buffer was created around each building to compensate for the grid size around buildings. The maximum velocity and depth that were within the building envelope was used to calculate the exposure risk and fatality rate for each building (**Error! Reference source not found.**).

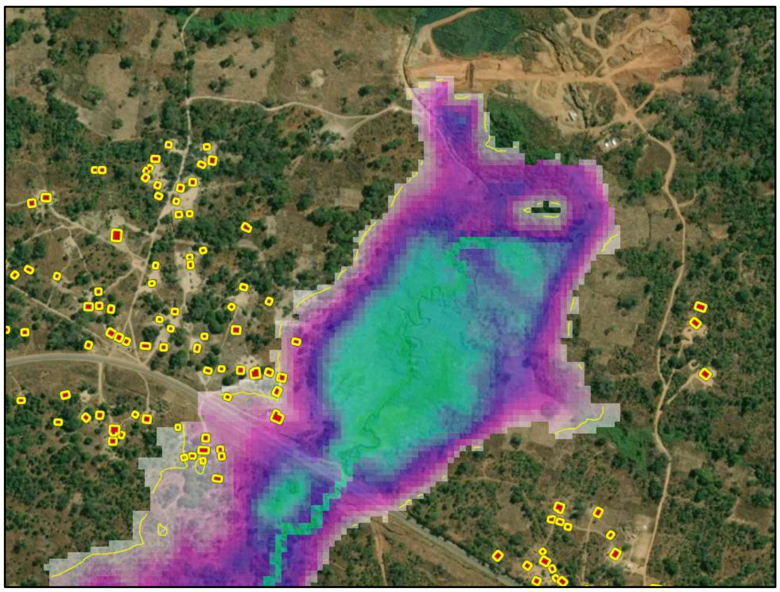

Figure 4. Dam K Rainy Day Flood Extents and Affected Properties with Buffer Zone

For the hazard assessment the Loss of Life is the critical measure at which the hazard rating of a dam is determined. Both the RARS methodology (Environment Agency, 2013) and Defra guidance (Defra, 2005) were used to produce a possible range of results. For the Sunny Day scenario the likely loss of life at Dams C and K was found to be little or none. For the Rainy Day Scenario, the Baseline No Breach Population at risk, loss of life and injured people were subtracted from the "Breach" measurements to calculate the incremental consequences considering with and without flood warning systems being introduced.

The consequences were plotted on an F-n Plot for both dams to assess the Hazard Category. Dam C showed high consequences and justified the selection of Class 1 by the Kabell system, as originally it was qualitatively assessed. Dam K was on the borderline between ALARP and unacceptable safety risk for a safety check event of 2000-year return period, and unacceptable for a return period of 500-year event. Combined with the damage on the major road downstream, it was decided to keep Dam K as Class 2.

EXISTING FLOOD CAPACITY

At Dam A the safety check flood estimate represented only a slight increase compared to the apparent design flood flow of 580m^3/s. Under free discharge conditions this flow was found to be a credible estimate of the spillway discharge capacity. However, an assessment of the tumble bay and downstream channel suggested submergence of the weir, reducing its capacity. This was supported by video evidence from the client, showing a past flood event where only shallow overtopping of the weir occurred but the downstream channel could be seen to be full.

Dam C was found to only be able to pass around 125m^3/s before overspilling of the low sections of dam crest adjacent to the primary spillway. This occurs at around the ground level of the emergency spillway, where negligible flow occurs.

At Dam K the existing spillway is expected to pass around 304m^3/s with a reservoir water level of 1195.75mEL (a head of 3.2m over the primary weir); this represents zero freeboard to the dam crest adjacent to the spillway weir with 281m^3/s on the primary spillway and the remaining 23m^3/s in the emergency spillway. However, the banks of the spillway were assessed as only being able to safely pass 152m^3/s, increasing to 174m^3/s if localised low points were filled.

OPTIONEERING

Multiple options were considered by Arup to address the safety measures for all three of the dams. The option of decommissioning or lowering of the full supply level was ruled out as unacceptable for all of the dams in a separate high-level optioneering exercise before Arup's commission. There were common problems to be addressed for all three dams, namely inadequate overflow capacity, excessive seepage, areas of over-steep embankment slopes and irregular crest elevation. The geometry of the dams, size of the reservoirs, required discharge capacity and the downstream consequences of failure, however, differed significantly.

For Dam A various options were considered, including (1) raising the dam with no new spillway works; (2) constructing a new spillway east of existing or (3) constructing a new spillway on the right abutment. The right bank was quickly discounted as it would be partially constructed on the dam, partially on the abutment and would require extensive works to returning the flow to the downstream river past the dam toe. It was therefore determined that there was no benefit to pursuing option (3) in preference to option (2).

A new spillway would need to be of similar length to the existing weir and so the same degree of dam raising. Different lengths were considered but even significantly longer spillways required significant dam raising and the downstream channel becomes a limiting factor.

Therefore raising the dam with use of the existing spillway was found to result in the need to raise the dam by approximately 3m and to reinforce the existing spillway and abutments with new structures to permit the raising of the dam crest and filling in the existing low areas of crest. It also required the widening of the downstream channel to prevent submergence of the weir. This option was selected as the most economic.

The options considered for Dams C and K included upgrading the existing service spillways and completing the emergency spillways or building new service spillways. For both existing spillways it was determined that large scale works would be required to allow them to safely pass significant flows, even in combination with an entirely new spillway elsewhere. There was also an advantage in being able to utilise the existing spillways to pass flows during construction of the new spillways. New spillways would therefore be designed for both dams with the original spillways decommissioned on completion. Two preferred options were identified, the first to construct a long channel spillway, which for Dam K would be located on the right bank on the line of the incomplete emergency spillway. For Dam C this would be similarly located at the left bank emergency spillway. The second option was to construct a spillway over the dam crest itself. The first option could be constructed on dry higher ground with the reservoir fully impounded, the disadvantage being that the long chute length would result in a high construction cost. The second option would be cheaper to construct but we considered it necessary to dewater the reservoir to safely construct it, as the existing flood capacity was so low we would want a cofferdam equal in height to the existing dam. Initially the client preferred the first option as it was not desirable to lose the reservoir storage. However, after some design development of the first option a cost estimate was produced. As a result the design was changed to the on-the-dam option which was estimated to be 60% cheaper.

For all three dams discrepancies were found between the original dam design drawings and what had been constructed. Some topographical surveys and ground investigations had been carried out, but some inconsistencies remained. As a result, we specified new topographical surveys and ground investigations to be carried out and worked closely with the local companies to ensure accuracy of the results. All three dams had similar geology with residual soils of varying proportions of dense silt, sand and stiff Clay with some areas of looser transported materials. At Dam C ground conditions below the dam were largely Medium to Very dense residual clayey Silt with some areas of transported sands and gravels and fill of sandy Clay. Dam K had similar residual soils but with areas of stiff sandy Clay and others of sandy Silt and clayey sand, fill material is sandy Gravel. At Dam A it was predominantly Medium dense clayey Sand with soft weathered sedimentary bedrock at a depth of around 4m, the fill material sandy Clay. In all cases the existing fill was found to have a permeability of between 10^{-7} and 10^{-8}m/s. In addition to testing of the dam fill the original borrow pits were identified and tested for obtaining additional fill.

The three dams were all found to be homogeneous despite drawings suggesting impermeable cores. Otherwise the dams were found to be well compacted. We carried out slope stability analysis and found that the dams achieved acceptable factors of safety in all load cases with the exception of areas where the slopes had been over-steepened. We also carried out seepage analysis based on both laboratory testing of the existing dams and back analysis of the seepage observed on site to calibrate the seepage rates used. The analysis was then repeated for the raised dam and flood cases. The planned raising of the dams allowed us to regularise the slopes at 1:3 (v:h) and incorporate a downstream filter layer and toe drain. The seepage would therefore remain, however the material had been found to be non-dispersive and with the filter layers the risk of migration of fines and internal erosion was mitigated. Additional works included providing a rock armour protection to the faces of the dam to provide protection from waves and cattle, which had been observed at all sites.

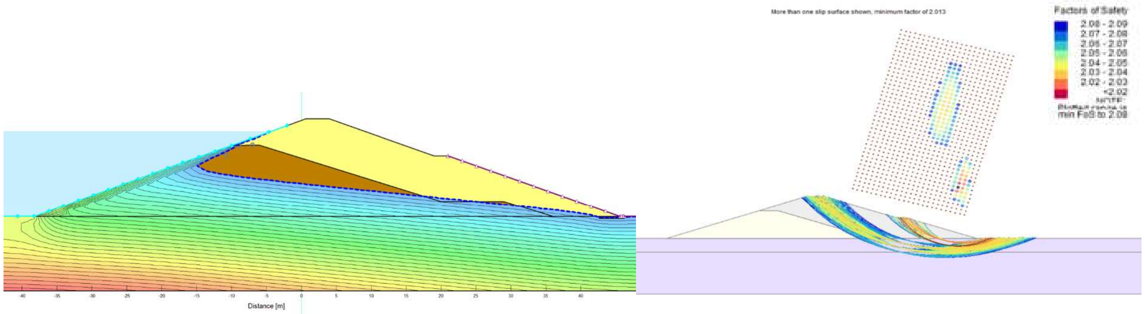

Figure 5. Dam C Seepage and stability analysis

DESIGN OF DAM A

The dam was raised to pass the design flow with free discharge over the existing weir; this then set the required height of dam raising. Raising the dam would also allow the overly steep section of dam to be corrected, the inclusion of a filter layer, rock toe and drainage to control the seepage and prevent internal erosion. Raising the dam meant the need to also raise the abutments walls of the weir which retained the end of the dam. The existing construction was unknown so we designed a new wall to line within the existing weir, doweled to the existing, to support and raise the wall.

To improve weir capacity we widened the downstream channel. This was analysed treating the downstream channel as a side channel weir to capture the water profile downstream of the L-shaped weir. Widening the channel also allowed us to re-line the channel bank protection to prevent scour. Given the importance of the downstream water level and preventing submergence of the weir and the complexity of the downstream geometry we carried out CFD modelling which allowed us to reduce channel excavation and identify the requirements for bank protection.

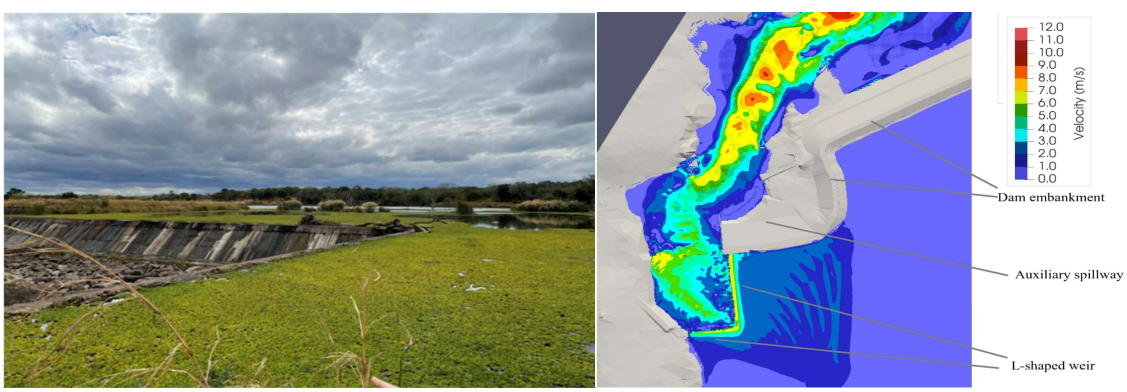

Figure 6. Dam A weir and CFD results

DESIGN OF DAM C AND DAM K

Spillway Weir type

A new over-the-dam spillway in combination with dam raising was selected as the preferred option for both Dams C and K. The cost of supplying concrete was a significant cost element, and reducing the size of the concrete spillway structure was a design priority. As the designs on the two dams were similar it was decided that the spillway weir design will be developed and modelled for Dam C and then scaled for Dam K.

Both ogee and labyrinth weir design options were developed for a hydraulic head of 5m at dam C in the safety check flood. Traditionally, ogee designs have been the more commonly adopted solution in scenarios with high hydraulic head due to their proven efficiency in handling such conditions. The primary concern with labyrinth weirs in high head cases is the potential for weir interference significantly reducing the efficiency. However, research by Crookston (Crookston, 2010), that extended the theoretical labyrinth weir efficiency coefficient curves up to ratio of H:P=2 was used to optimise the labyrinth weir design.

The analysis demonstrated that even with the severe reduction in the discharge coefficient, the labyrinth design demonstrated better efficiency in terms of overall spillway width when compared to the ogee design (Martin, 2024). The chosen labyrinth configuration provided a spillway width of 65m for Dam C, and a 22% potential reduction in concrete volume compared to the ogee. The labyrinth design was taken forward for CFD modelling to prove the concept and for further development into a detailed design.

Design and CFD Modelling

For Dam K the labyrinth was modified by reducing the number of cycles, but not changing the height, arm length and angle of the labyrinth. The head over the weir for the Dam K design flood was similar to the Dam C safety check; in this way the rating curve obtained from the CFD modelling of Dam C could be scaled. The labyrinth cycles were reduced to four (from seven in Dam C), and the overall width of the spillway was estimated to be 37.5m. The labyrinth weir design in combination with the updated hydrological assessment, allowed for the dam raising to be limited to approximately 1.5m above the existing dam crest.

This allowed us to undertake a single CFD model for Dam C and then use these results to verify the performance at Dam K. We found that the upstream reservoir water level was higher, at 1192.85mEL, than we had predicted in the calculations (1192.3mEL). We determined that this was likely due to the large head, around the maximum from the research studied, resulting in slightly greater interference over the labyrinth than predicted, but also due to the approach conditions to the weir. As a result, the wingwalls were flared to reduce the entry losses; the wall height had been set as a parapet height of 1.2m following the dam face profile, partly to provide edge protection but also to allow some overtopping to ease the approach to the weir but keeping the flow above the dam to reduce scour risk. This amendment was successful in reducing the peak safety check reservoir still water level to 1192.56mEL, however with reservoir attenuation this reduced to 1192.23mEL, within the original target.

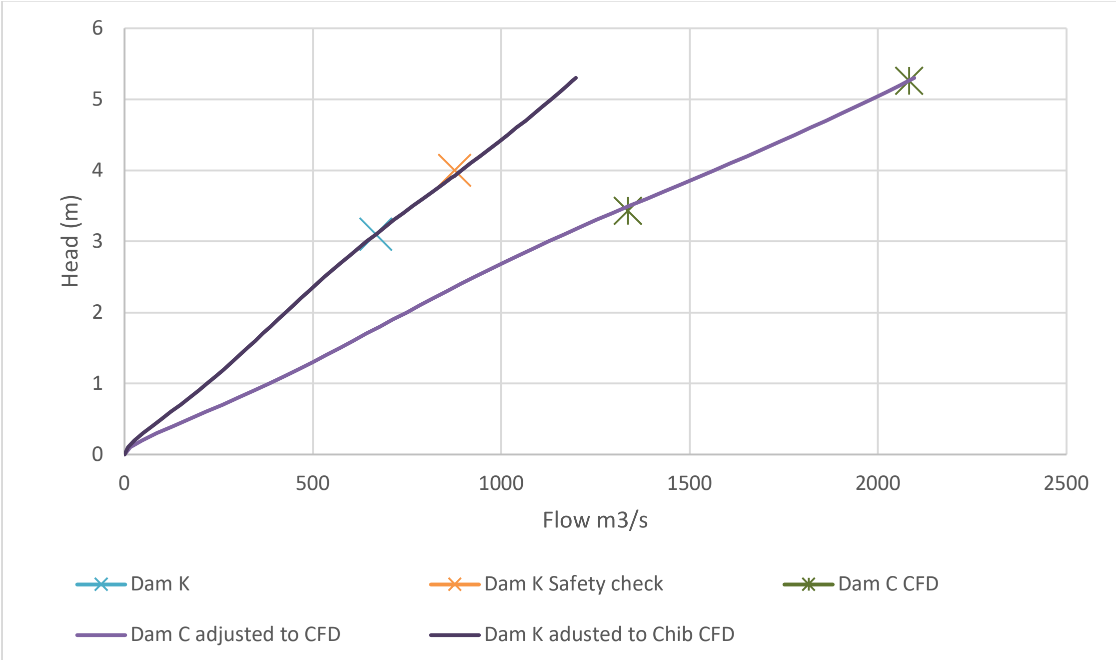

Figure 7. Rating curves with CFD results

CONCLUSIONS

Three different dams were all found to have largely the same dam safety problems which required resolution. A common approach was taken to the dam raising and seepage control measures due to the similarity of the dams and materials. After investigation differing approaches were taken to resolving the flood capacity shortfall, with Dam A making use of the existing structure with enlargement of the downstream channel and raising of the dam and abutments. For dams C and K entirely new spillways over the dam were designed. This was in part due to the hazard potential of the dams but also due to the site conditions, and the desire to reduce capital cost on the project. Overall, this paper highlights the importance of considering cost-effective design alternatives in dam safety projects, especially in remote locations, where access to materials, labour and data can be limited.

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